THE DESIGN OF COMPOUND CRITICAL SECTIONS FOR OPEN CHANNEL FLOW MEASUREMENT

THE DESIGN OF COMPOUND CRITICAL SECTIONS FOR OPEN CHANNEL FLOW MEASUREMENT

by

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SCOPE AND CONTENTS:

Weirs of the conventional shape are amenable to analysis based on an assumption of one-dimensional flow and a number of computational routines have been developed for this type of transition problem. When critical flow occurs in a highly non-uniform section, a more sophisticated approach is necessary.

In conjunction with laboratory tests on a typical compound control, a mathematical model was formulated for the development of the stage-discharge relation. It is felt that this model will allow an accurate prediction for water quantity from fluctuating sources.

ACKNOWLEDGEMENTS

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R. L. Smith
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CHAPTER 1

1.1 INTRODUCTION

In recent years, it has been fully understood that reliable records of water quantity are needed as a basis for developments that utilize water in any considerable amounts. For this reason, there has been an increasing demand for gauging structures, that measure a wide range of stream flow with high accuracy.

Although the sensitivity of the stage-discharge relation is important at all water levels, it is usually of primary importance at low stages. The principal reason for the construction of artificial controls is the desirability of improving discharge measurements at low flows, since this is the critical period for design consideration. In addition to the above stipulation, the following criteria for the design of an artificial control section must also be considered;

1. The constriction must be designed in such a manner that the control is functional over the entire range of minimum to maximum flows.

2. The crest profile should be designed in such a way that drift material will not lodge on the crest and cause increased backwater upstream.

3. Turbulent flow below the critical control may cause considerable erosion due to turbulence. In order to prevent turbulence the downstream face of the control should be designed so that the direction of flow will have a large horizontal component.

4. Field calibration should be unnecessary or at least minimal.

5. Both construction and maintenance costs should be held to a minimum.

6. The downstream face of the control should be constructed so that the nappe will cling to it at all times, in order to prevent discontinuities in the stage discharge relationship.

Although the sharp-crested weir and the sharp-edged notches of the trapezoidal, V-shape, or rectangular shape give accurate results at low flows, they are not practical for obtaining stream flow records for the following reasons;

1. The cost of construction is generally high.
2. Unless a combination of shapes is used a sharp-crested weir will suffer due to inaccuracies at low flows.
3. The sharp-edged metal blades require considerable maintenance, and in some cases may be damaged by floating debris.
4. A non-erodible bed is necessary downstream to prevent erosion due to the large vertical component of flow.

Except under very special conditions, when the discharge of a stream is so regulated that low flows do not occur, artificial broad-crested controls with horizontal crests generally will not be sufficiently sensitive, and for this reason they are seldom built. The combination of a[1] ninety degree V-notch plate in an artificial concrete control as shown in Figure 1.1 is equally unacceptable. The notch, though providing some increase in sensitivity for low stages would require continual maintenance and would also be very suspectible to drift material. In addition, this type of section would require extensive field calibration in the transitional range.

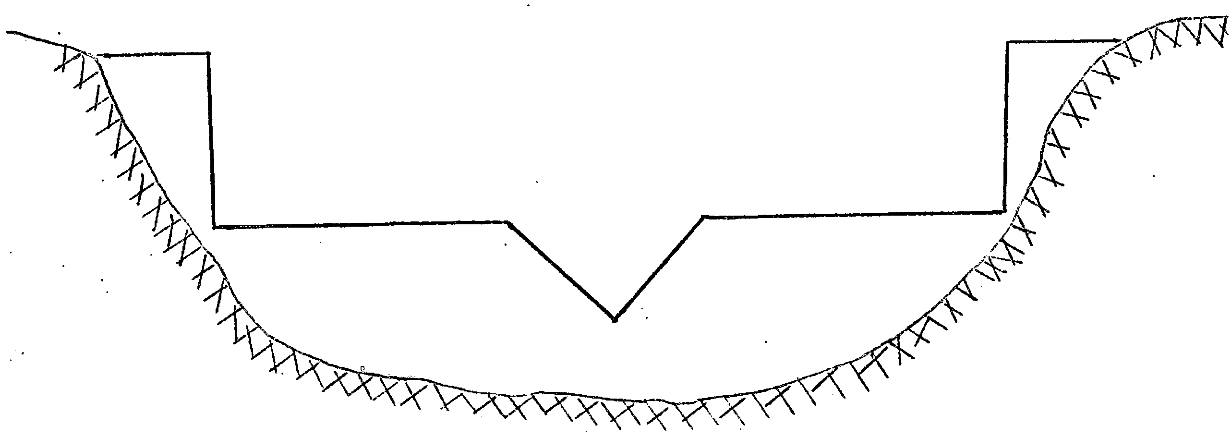


FIGURE 1.1 COMBINATION V-NOTCH AND STRAIGHT CREST

A design which has been found to be partially effective consists of a crest with a catenary or parabolic profile. The Trenton and Columbus[2] type deep notch control has a cross-section similar to that shown in Figure 2, thus avoiding some of the disadvantages of the sharp crested weirs. The section combines a large capacity with a reasonable sensitivity at lower discharges. The critical drawback of this section type is the necessity for calibration of the stage-discharge relationship. If a model is used for this purpose, both the model and prototype must be carefully constructed to similar specifications in order to ensure that the derived rating curve will apply in the field. With curved shapes such as

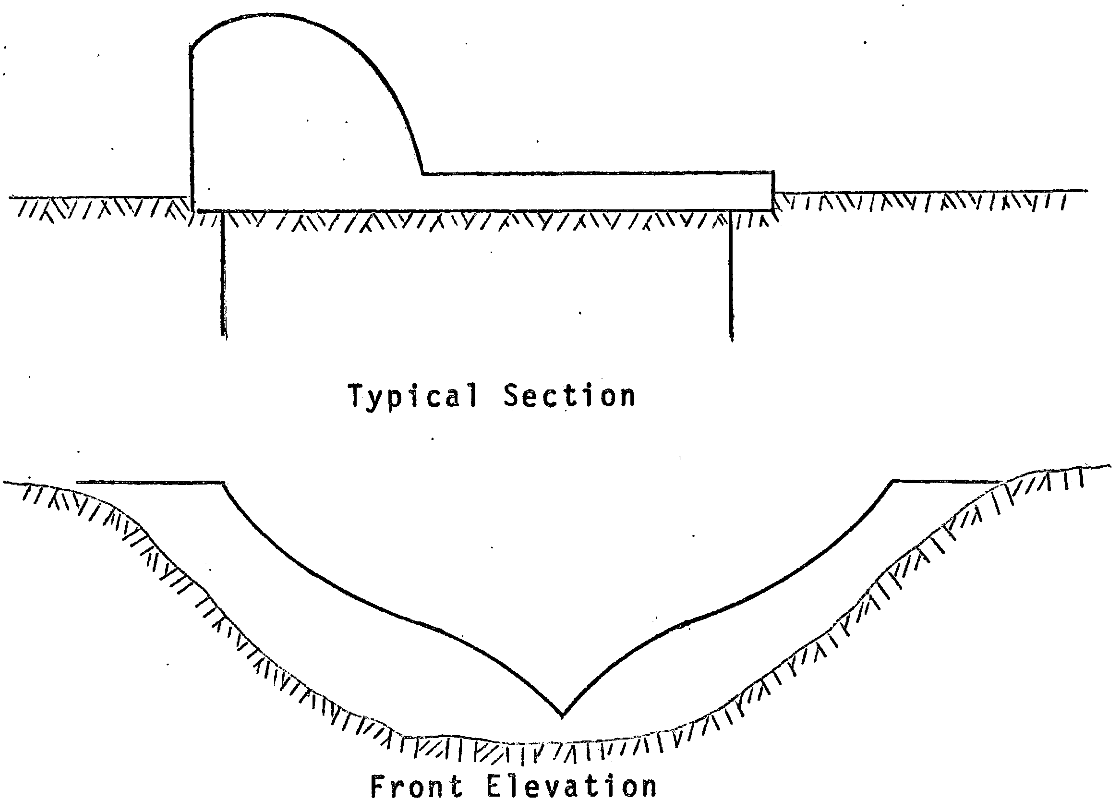


FIGURE 1.2 COLUMBUS CONTROL

those in Figure 1.2, duplication of geometry would be very difficult. This type of section may vary in geometric similitude for each river profile being gauged, and because of this a calibration model would be necessary for each prototype installation. This cost factor alone, limits extensive use of the Columbus and Trenton type control section as a flow measuring device.

In England, a modification of the triangular profile Crump[3] weir was developed for the purpose of measuring a wide range of flows. Illustration 1.3 shows the general layout for this type of control.

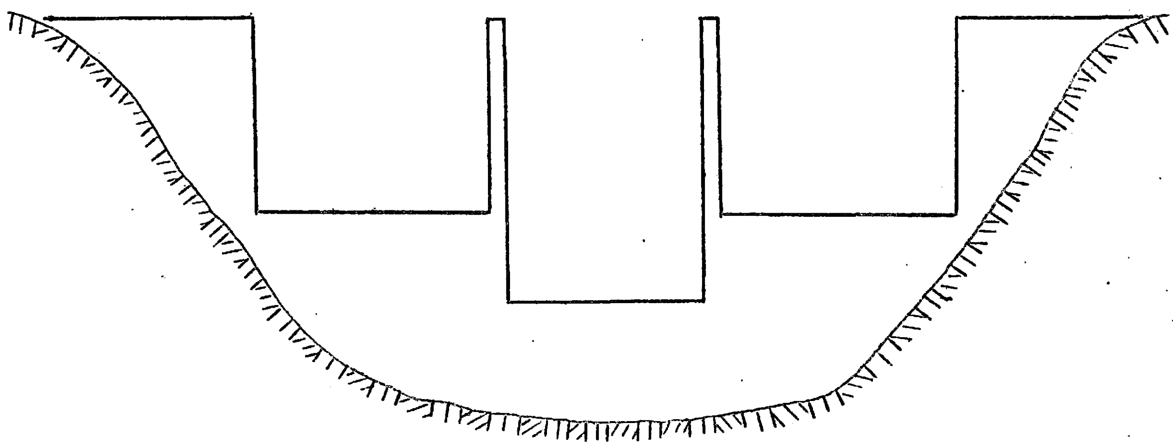


FIGURE 1.3 COMPOUND CRUMP WEIR

The basic philosophy of the design is based on the assumption that each section acts independently as a two-dimensional control. Low flows pass over the lower (lowest) sill only and therefore an increase in flow measurement sensitivity is achieved for small discharges. During periods of above average discharge all sections contribute to the capacity of the control section thereby allowing a measurement of reasonable accuracy and quantity for peak stage periods.

Due to the simple geometric shape specifications for the section, construction can easily be achieved. Laboratory calibration has become unnecessary for each individual prototype due to the similarity in shape between each prototype.

Although over thirty of these compound Crump weirs have been constructed in Britain, they are generally considered unacceptable in Canada. Because the divide piers are above the water level, ice sheets developed during winter months will lodge on these walls and, especially in the spring breakup period, they may cause structural failure of the divide piers or even the entire control section. Other secondary criticisms of the divider piers include construction costs, the possibility of river debris being caught between piers, and also their unsightly appearance.

The primary purpose of this research is, therefore, to develop suitable stage-discharge relationships for a compound control section similar to that shown in Figure 1.3, but with the removal of all interior dividing piers. This type of section could be used successfully with much less danger of debris entrapment or ice jamming. Obviously the cost would also be greatly reduced if the divide piers could be abandoned. With the use of a laboratory model, an attempt was made to develop a suitable scheme for any compound control of similar profile.

A secondary purpose of this study is an attempt to gain some insight concerning the properties of three-dimensional flow at a compound constriction. It is anticipated that the data collected from the series of laboratory experiments might some day be useful to those confronted by this problem.

Chapter II discusses the apparatus and instrumentation used for the model study. Laboratory tests and the development of a suitable mathematical model are described in Chapter III. A computerized stage-discharge relation is developed for any compound prototype section and conclusions regarding performance and preference are discussed in Chapter IV.

CHAPTER II

EQUIPMENT AND INSTRUMENTATION

2.1 FLUME INSTALLATION

Due to the lack of available facilities, a recirculating flume was designed by the author for the purposes of this study. Figure 2.1 shows both plan and elevation of the apparatus designed.

Recirculation was achieved using a 2.5 cusec centrifugal pump, while quantity regulation was achieved using a six-inch gate valve, installed in the discharge line downstream of the metering orifice.

The delivery tank is wider than the test channel to allow the introduction of a streamlined convergence between the tank and the working section of the channel. This convergence assists in the development of smooth flow in the test channel. In conjunction with the convergence, two smoothing screens as shown in Figure 2.1, were also added for the purpose of promoting a uniform distribution of velocity. The downstream tank supplies the necessary reservoir for the one stage centrifugal pump.

The test channel is 10 feet in length, with a smooth painted bottom (30 inches wide) and plexiglass sides (18 inches high). The control sections are positioned with the crest six feet downstream from the delivery tank in order to ensure that a suitable reach of established flow for measuring the upstream stage is available. A hydraulic jump was induced immediately downstream of the control in order to sustain the maximum possible sump depth. This depth ensured that air entrainment due to pump suction would be kept to a minimum.

2.2 DISCHARGE MEASUREMENT

A 900 USGM (2.005 cfs) mercury manometer was used in conjunction with an orifice plate installed in the six-inch return line upstream of the control valve. A series of volumetric flow measurements were carried out in order to check the calibration of the meter. A comparison of results proved the meter accurate to within the readable divisions of the static scale (± 2.5 USGM or ± 0.0056 cfs). Normally the meter remained untouched for several minutes before a reading was taken to ensure stabilization.

At low flow (Q = 2000 USGM)

E_q (% error) = ± 2.5*100/200 = 1.25%

At high flows (Q = 900 USGM)

E_q (% error) = ± 2.5*100/900 = 0.28%

2.3 WATER LEVEL MEASUREMENT

The upstream water level was obtained using a point depth gauge with an electronic sensing device, that indicated

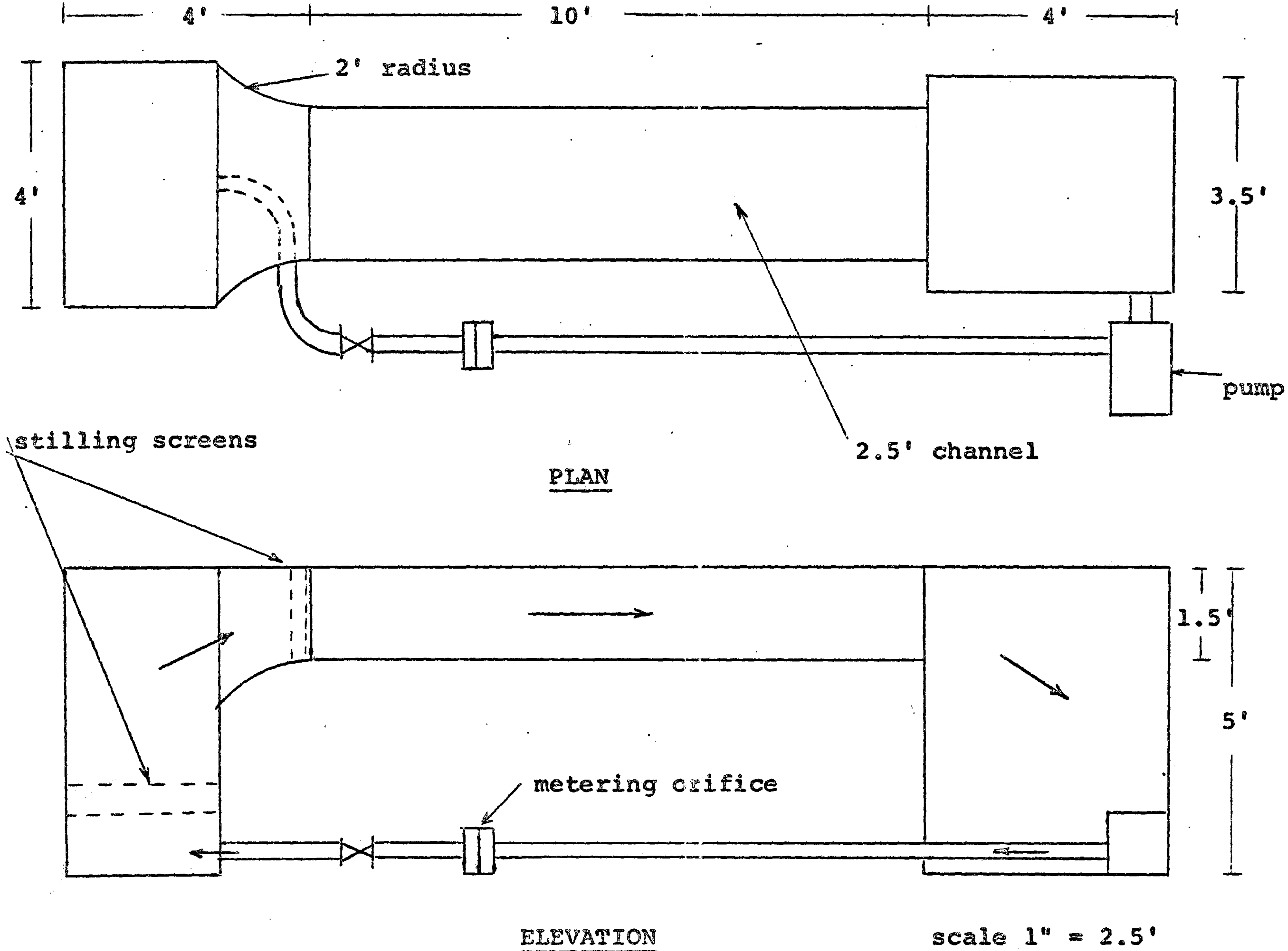


Fig. 2-1 GENERAL FLUME LAYOUT

when contact with the water surface had been made. The gauge was found to read consistently to ± 0.01 inches. Since the water depths were generally less than 12 inches, the maximum percentage error is equal to $E_{wl} = \frac{0.01}{12}*100 = 0.09\%$. The water levels readings were taken at three equal intervals across the channel to ensure accuracy in the measurement.

Because these water level measurements are used for the computation of total head, there are two factors which must be considered when determining the distance from the weir crest to the point upstream where gauging is to take place. The distance must be sufficiently short so that the head losses due to friction may be neglected but must however, extend far enough upstream to be free of local drawdown effects. During the course of the experiments it was found that a suitable upstream distance is approximately ten times the height of the upstream water level exceeding the crest.

2.4 VELOCITY MEASUREMENT

A propeller type velocity meter was used to find velocity distribution in the channel upstream of the constriction. This measurement was used for calculation of the Coriolis coefficient. The range of the instrument was from 2.5 to 150 centimeters per second. In the 2.5 to 30 range, the device was accurate to 0.2 centimeters per second and in the 30.0 to 150 range the accuracy was 1 centimeter per second.

2.5 CHANNEL GEOMETRY

The crest and channel width was measured to within ± 0.03 inches in 30.0 inches in several positions above the channel floor giving a maximum error of 0.17%.

In elevation, the section crests were accurate to 0.03 inches in 4 inches with a 1.25% maximum error. Therefore, E_{cl} (error in crest level) = ± 0.03 inches,

E_{cw} (error in crest width) = ± 0.03 inches.

2.6 TOTAL HEAD CALCULATION

The energy level above the crest calculated at the upstream measuring section is given by the following equation.

$$H = (WL - Z) + Q^2/2GA^2 \qquad 2.1$$

where Z = crest height,

WL = water level,

Q = discharge,

A = cross-sectional area at the section,

G = acceleration due to gravity.

Using the maximum errors for the variables in Equation 2.1, the maximum error for energy level may be calculated as shown below.

$$E_h = \pm (E_{wl} + E_{cl}) \pm - \frac{\sqrt{2(E_q)^2 + 2(E_{cw})^2 + 2(E_{wl})^2}}{2G} \qquad 2.2$$

$$E_h = \pm 0.0036 \text{ ft} \qquad 2.3$$

The error in discharge due to the relative error in head may now be calculated. It is generally accepted that the discharge formula may be written as

$$Q = CBH^{1.5} \tag{2.4}$$

where C = coefficient of discharge,

B = crest width.

Equation 2.4 expressed in differential form becomes

$$dQ = 1.5CBH^{1/2}dH \tag{2.5}$$

Substitution of Equation 2.4 gives

$$\frac{dQ}{Q} = \frac{3dH}{2H} \tag{2.6}$$

For small, finite increments, this may be expressed as follows

$$\frac{\Delta Q}{Q} = 1.5\ \frac{\Delta H}{H} \tag{2.7}$$

Therefore, the percentage error in the computed discharge is 1.5 times the percentage error in the observed head. For the error in energy level as computed above (Equation 2.3), the percentage error in discharge may be calculated for any head as shown in Figure 2.2.

2.7 WEIR PROFILE

In order to easily form a series of compound sections, the control profile under investigation was composed of sections. Each section was 6 inches wide - five sections thus filling the breadth of the channel - and formed with an

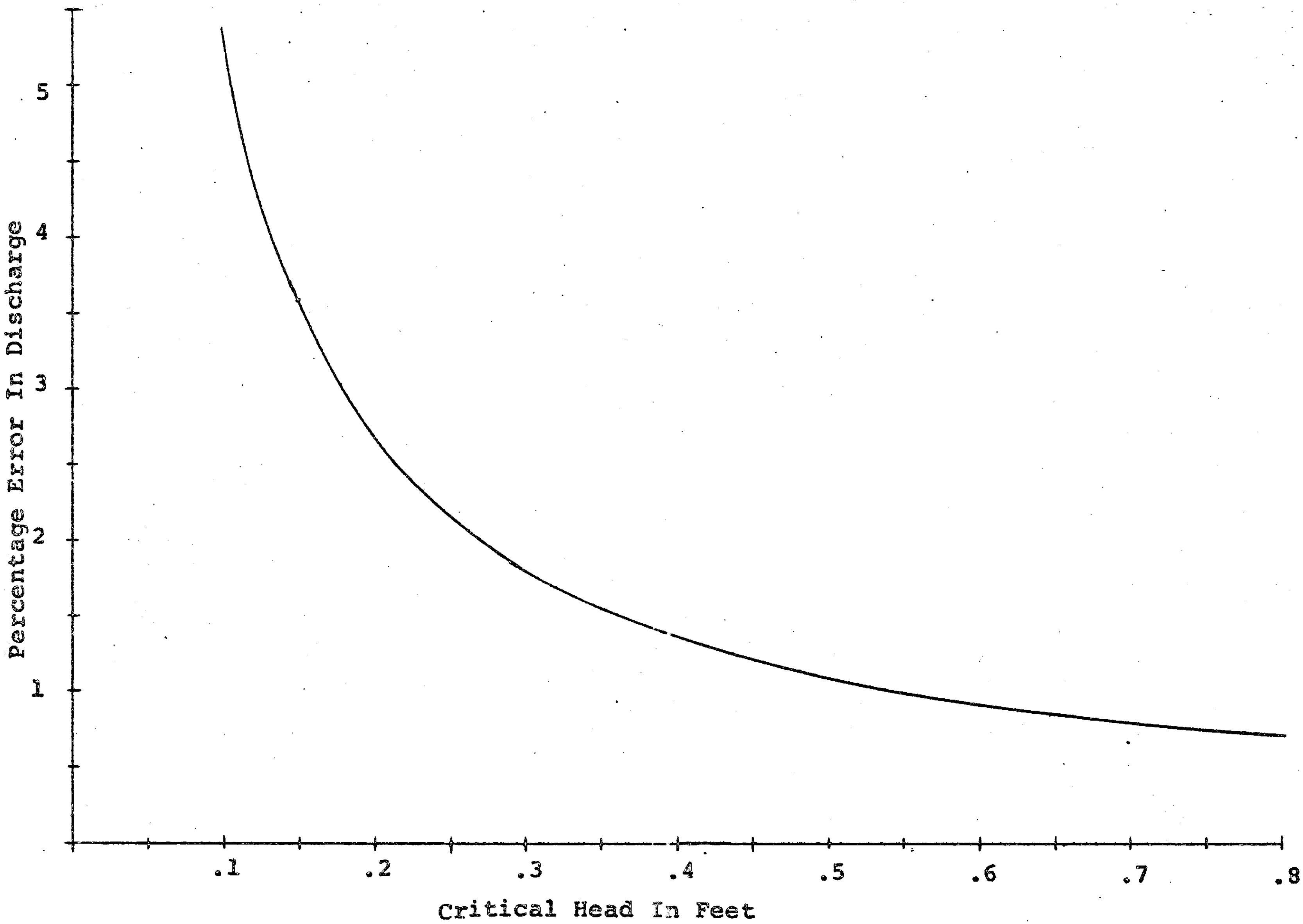


Fig. 2-2 RELATIVE ERROR IN DISCHARGE AS A FUNCTION OF HEAD

upstream slope of 1:3.5, a downstream slope of 1:4.0, and a height at the crest of 4.0 inches. In order to achieve compound crests the individual weir segments were raised on rectangular blocks as shown in Figure 2.3.

As discussed in Section 3.1, the truncation effect due to the rectangular blocks does not significantly effect the discharge section properties. As discussed by Burgess and White[4], an acceptable truncation point was 2.0H for a 1:5 slope downstream, and 1.0H for a 1:2 slope upstream. Since H was never greater than 0.5 of a foot over the truncated sections, it would seem reasonable to assume that the 1:3.5 upstream slope and the 1:4 downstream slope which are both 14 inches in length will not be adversely effected by truncation.

The sections were constructed of concrete with the upper surface sanded smooth to minimize rough turbulence. The crests were accurate to 0.03 inches in the vertical and horizontal directions. All cracks were sealed to prevent leakage with modeling clay.

Since the section slopes effect the discharge relation, the profile chosen allows a comparison with the experiments performed by Burgess and White[4] on the Crump weir (upstream slope 1:2, downstream 1.5).

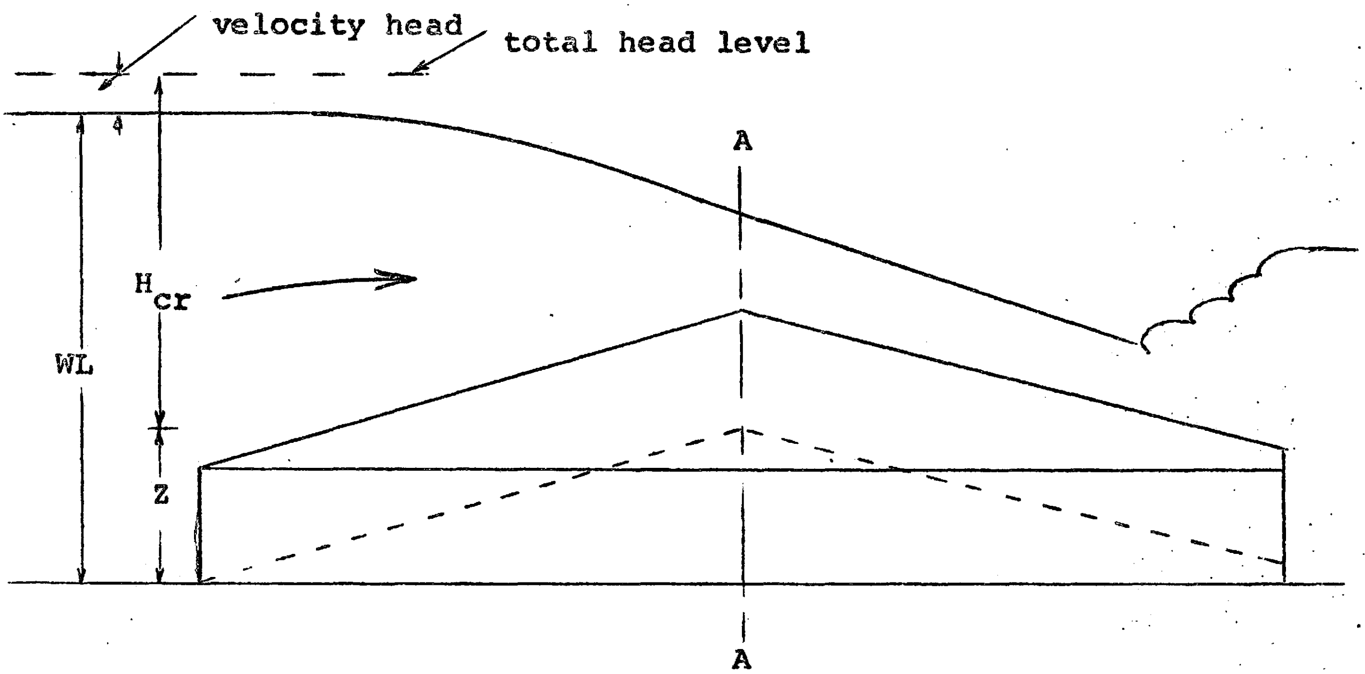
It was felt that this slope may be superior to the Crump type for the following reasons;

1. Since the change in angle at the crest is substan-

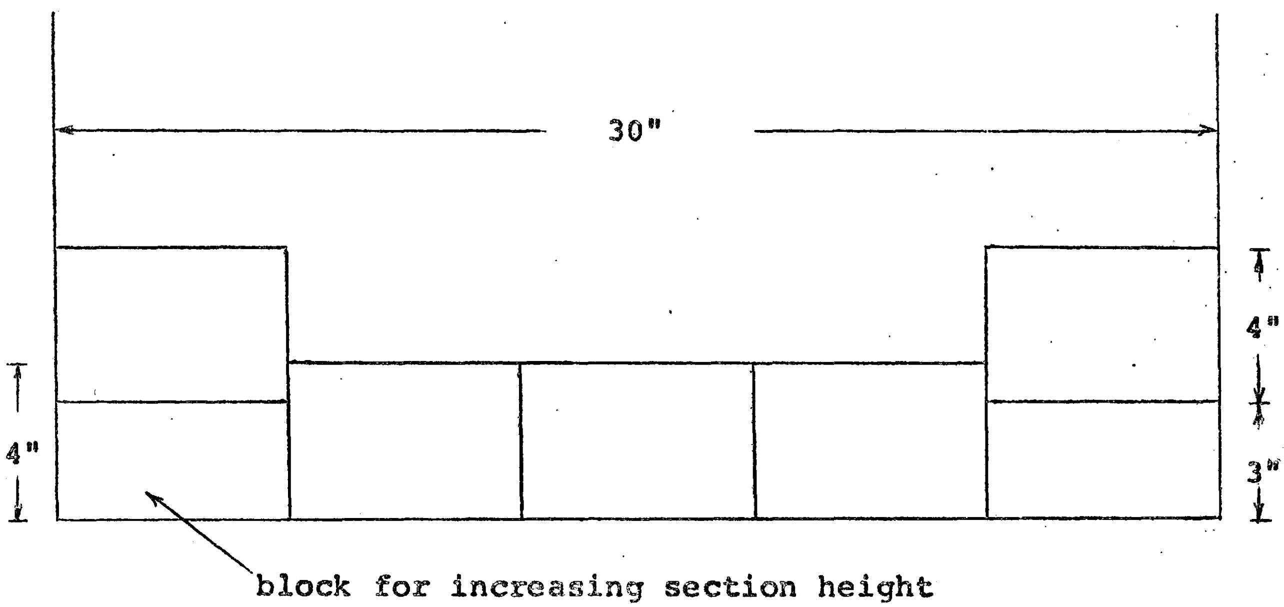
tially reduced, it is more probable that the nappe will cling to the downstream face of the control at all times.

2. The decrease in upstream slope will cause a more gradual transition to take place.

3. The decrease in upstream slope will make deposition of sediment materials less likely, since the upstream velocity should change less rapidly.



PROFILE OF COMPOUND SECTION



CROSS-SECTION FOR THE PROFILE ABOVE (A-A)

Fig. 2-3 TYPICAL TEST SECTION

CHAPTER III

THEORETICAL AND TEST RESULTS

3.1 THE CONTINUOUS PROFILE

A logical starting point in developing a suitable stage-discharge relation for a compound section is a two-dimensional study of the discharge characteristics for the continuous section shown in Figure 3.1.

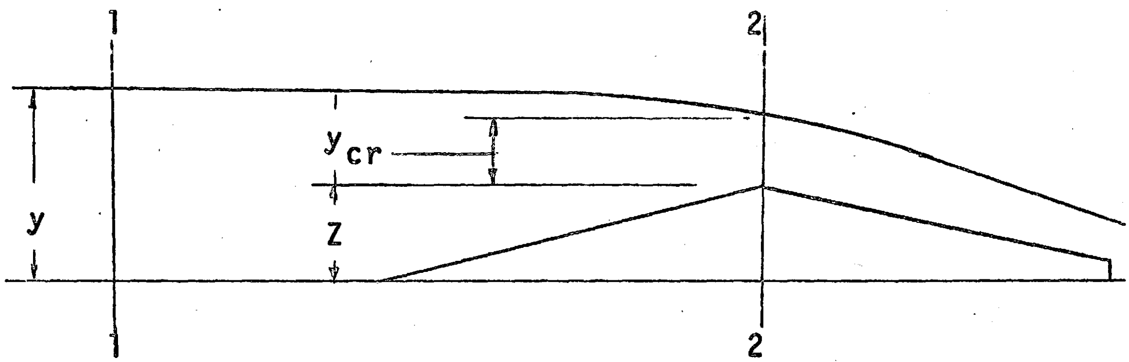


FIGURE 3.1 SIMPLE CONTINUOUS SECTION

The total head[5] over a specified vertical section is given by the expression

$$E = Z + \beta y + \alpha \frac{\bar{V}^2}{2G} \qquad 3.1$$

where E = total head at the section,

y = water depth,

V = mean velocity,

G = acceleration due to gravity,

α = Coriolis coefficient,

β = pressure coefficient,

Z = channel bottom measurement relative to a fixed datum.

The Coriolis coefficient is a correction coefficient for the velocity head $\frac{\bar{V}^2}{2G}$ as a result of nonuniform distribution of velocities over the channel section.

In parallel flow the pressure is hydrostatic, and the pressure head may be represented by y. The pressure head for curvilinear flow may be represented by βy where β is a correction coefficient for a nonhydrostatic pressure distribution resulting from the accelerative forces on the curvilinear flow.

The elevation of the energy line above the lower boundary may be written simply as the sum of the velocity head and depth.

$$H = E - Z = \beta y + \alpha \frac{V^2}{2G} \qquad 3.2$$

This quantity is commonly known as the specific energy H, or the energy referred to a datum which is coincident with the lowermost streamline.

In two-dimensional flow, the rate of discharge (q) per unit width of section is the product of the average velocity and depth. Equation 3.2 thus becomes

$$H = \alpha \frac{Q^2}{2G(yB)^2} + \beta y \qquad 3.3$$

where Q = total discharge for the section,

B = channel breath.

At section 1-1 in Figure 3.1, the flow is parallel, therefore the coefficient β is equal to unity. The velocity coefficient generally may be shown to be only slightly greater than unity for a uniform channel as discussed in Section 3.2, and for this reason may be reasonably assumed equal to unity (i.e. $\alpha = 1$).

Equation 3.3 at section 1-1 thus becomes

$$H = y_1 + \frac{Q^2}{2G(y_1 B_1)^2} \qquad 3.4$$

If it is assumed that a critical section occurs at section 2-2 (i.e. the point minimum specific energy for constant discharge) the first derivative of Equation 3.3 must here be equal to zero.

$$H_{cr} = \beta_{cr} y_{cr} + \frac{\alpha_{cr} Q^2}{2G(y_{cr} B)^2} \qquad 3.5$$

$$\frac{dH}{dy} = \beta_{cr} - \frac{\alpha_{cr} Q^2}{GB^2 y_{cr}^3} \qquad 3.6$$

$$Q = \sqrt{\frac{\beta_{cr}}{\alpha_{cr}}} \, GBy_{cr}^{1.5} \qquad 3.7$$

Using Equations 3.5 and 3.6, the following relationship between critical energy and critical depth may be found

$$H_{cr} = 1.5\beta_{cr}y_{cr} \tag{3.8}$$

Substitution of H_{cr} for y_{cr} in Equation 3.7 yields the discharge relationship as given below

$$Q = \frac{0.544\sqrt{G}\ B\ H_{cr}^{\ 1.5}}{\beta_{cr}\sqrt{\alpha_{cr}}} \tag{3.9}$$

Since both α and β vary with the quantity of flow over the section, Equation 3.9 may be written as follows

$$Q = 0.544C_f\sqrt{G}\,B\,H_{cr}^{\ 1.5} \tag{3.10a}$$

where $$C_f = \frac{1}{\beta_{cr}\sqrt{\alpha_{cr}}} \tag{3.10b}$$

The horizontal distance between section 1-1 and 2-2 in Figure 3.1 is small and the energy at the two sections should be equal. Thus

$$H_1 = H_{cr} + Z \tag{3.11}$$

If the value of C_f is known for any given upstream water level and critical section geometry, the quantity of flow may be calculated iteratively using Equations 3.1, 3.10 and 3.11.

It should be noted that Equation 3.10 is based on the premise that a critical section occurs at the crest. Many writers have described the location of the critical depth as being a short distance upstream of the brink or overfall. This is the point in the drawdown profile where the depth is equal to the parallel-flow critical depth, i.e. the section where, assuming $\alpha = \beta = 1.0$, the Froude No. $V/\sqrt{G}.y_{cr} = 1.0$. If the critical specific energy is properly defined using both α and β the Froude number becomes $\sqrt{\frac{\alpha_{cr}}{\beta_{cr}}} \quad \frac{V}{\sqrt{Gy_{cr}}}$. Obviously if the values of α and β are not unity the true critical section will not be coincident with the parallel flow location for critical depth.

A series of experiments was performed in order to determine the value of the discharge coefficient given in Equation 3.10. Both truncated and simple sections were tested in order to check the effect of both truncation and an increase in weir height on the discharge coefficient. The results of the test are shown in Appendix C, Tables 1 to 3.

Within the range of the experimental results it was found that a log-log plot of discharge versus critical energy[6] (Figure 3.2) was a straight line thus yielding a relationship of the general form

$$Q = aH_{cr}^{b} \qquad 3.12$$

where a and b are constants dependent on the Q versus H_{cr} curve. Solving for the two constants for the line drawn in Figure 3.1, Equation 3.12 becomes

$$Q = 10.02\ H_{cr}^{1.5825} \qquad 3.13$$

Substituting Equation 3.13 into Equation 3.10 yields the following relationship for the discharge coefficient

$$C_f = 1.2930\ H_{cr}^{\ 0.0825} \qquad 3.14$$

This function is shown graphically in Figure 3.3. The illustration also shows the dimensionless discharge coefficient as found by Burgess and White for the Crump weir.

The value of the coefficient is greater than unity for any value of H_{cr}. This is to be expected from an examination of Equation 3.10a. The value of β is less than one for the case in question, and the square root of the velocity coefficient is very close to unity. Therefore, the inverse of the product of the above mentioned quantities - i.e. C_f - will be in all probability be greater than unity.

Since Equation 3.10 is implicit, the solution for discharge must be accomplished indirectly. The H_{cr} term is calculated using Equation 3.2, where it is necessary that the discharge Q be known. The indirect solution for discharge may be calculated using the method outlined in the flow chart below.

Equation 3.10 is of course only suitable for uniform crested sections, and a refinement of this discharge relation is necessary for compound weir sections.

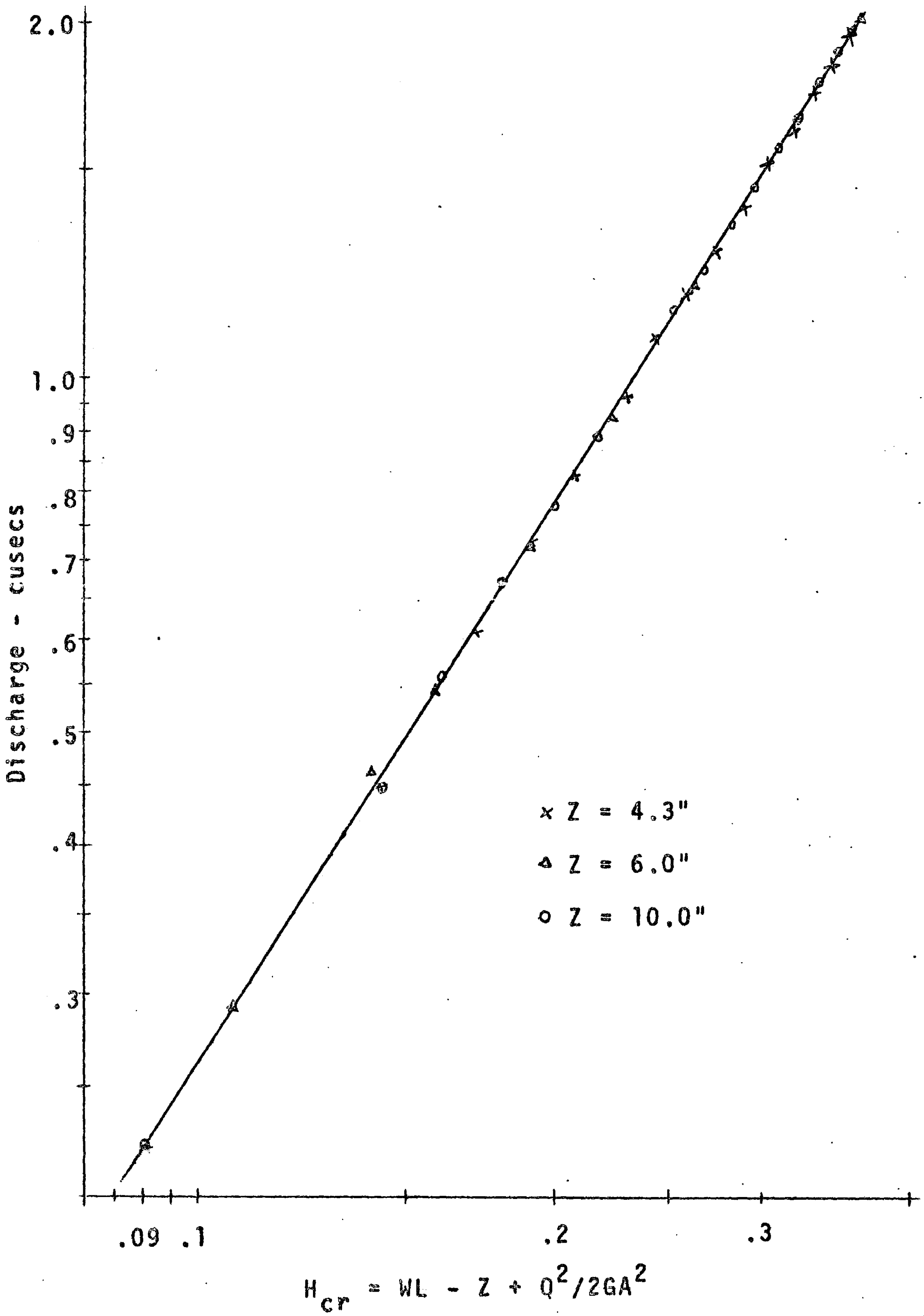


Figure 3.2 DISCHARGE VERSUS CRITICAL HEAD

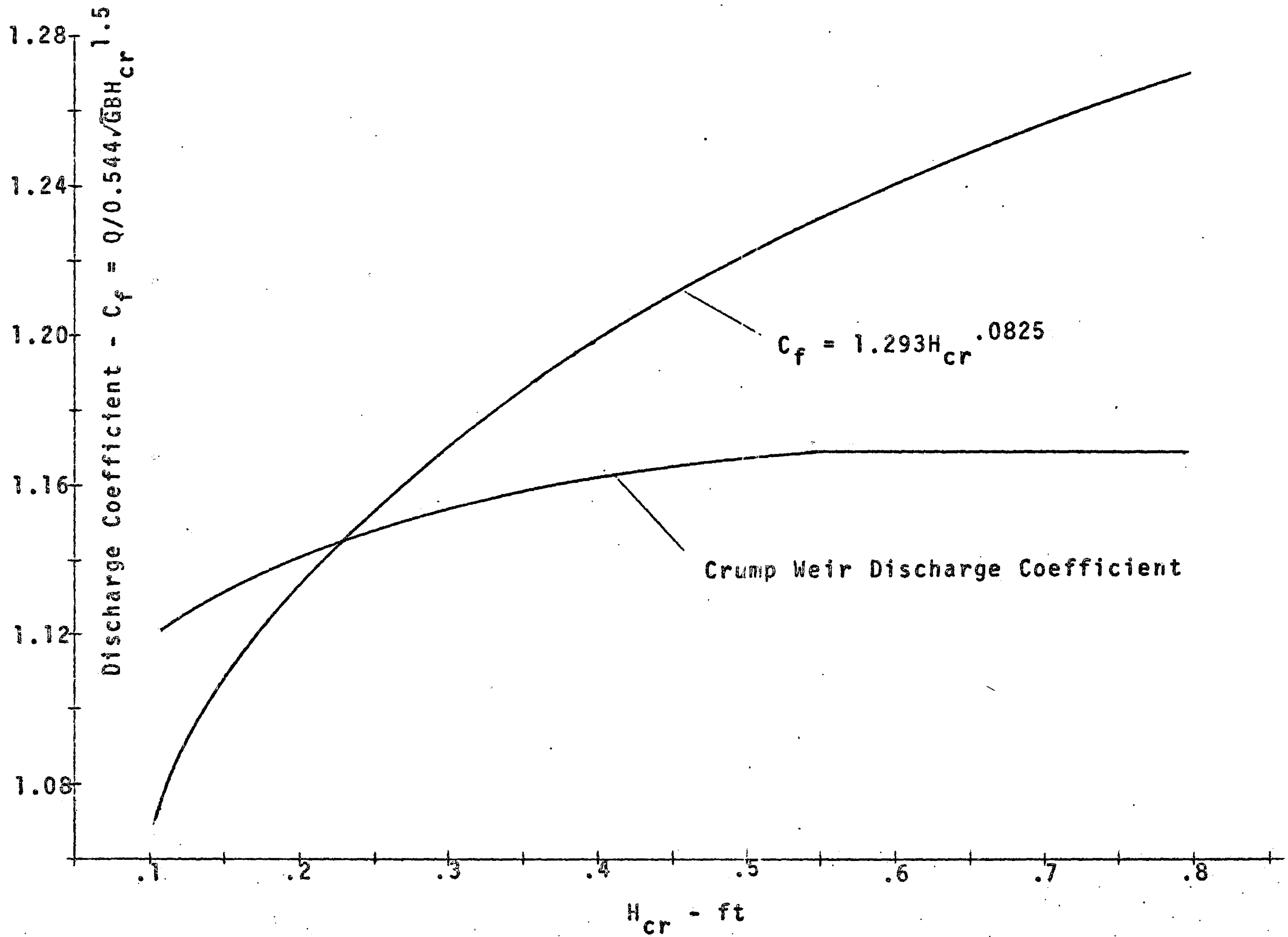
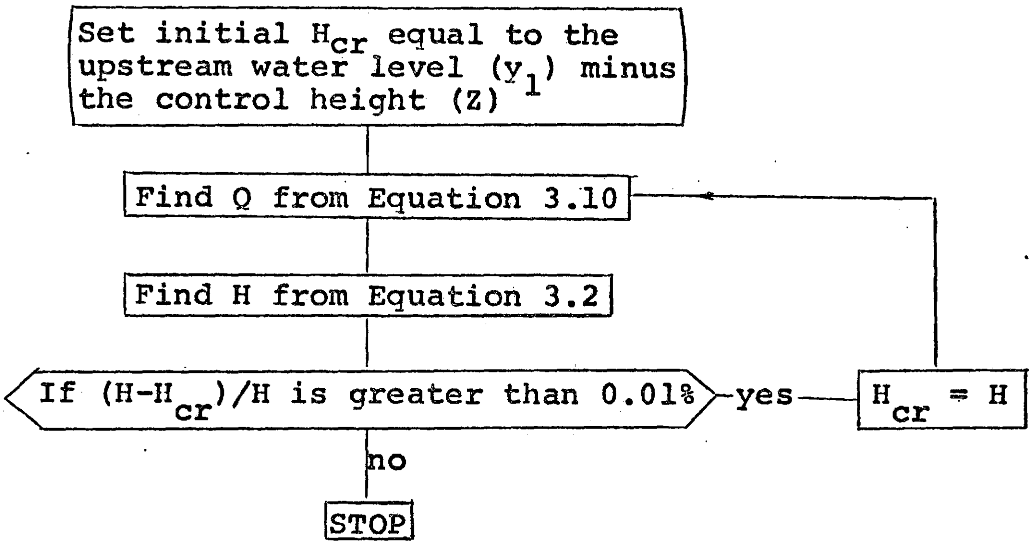


Fig. 3. 3 DISCHARGE COEFFICIENT VERSUS CRITICAL HEAD



3.2 CROSS-SECTIONAL VELOCITY DISTRIBUTION

As a result of non-uniform velocity distribution in the approach channel, the velocity head is always greater than the value given by $\bar{V}^2/2G$, where $\bar{V}$ is the mean velocity for the section. The true velocity head may be expressed as in Equation 3.2 ($\alpha V^2/2G$) where α is defined as the kinetic energy or Coriolis coefficient in honour of G. Coriolis[7] who first proposed it. The magnitude of the coefficient is dependent on velocity distribution, which in turn is affected by channel geometry and roughness, rate of discharge, and the depth of flow.

The velocity[8] coefficient may be expressed in the form

$$\alpha = \frac{\int V^3 dA}{\bar{V}^3 A} = \frac{\Sigma V_i^3 \Delta A}{\bar{V}^3 A} \qquad 3.15$$

where A = total cross-sectional area,

$\bar{V}$ = average velocity,

ΔA = elemental area,

V_i = velocity of flow through each elemental area ΔA.

For channels of regular cross-section and straight alignment (i.e. flumes and spillways) the effect of non-uniform velocity distribution on computed velocity head is generally very small. It is expected to be at least less than 1.10. Kolupaila[9] proposed that values of the coefficient which may reach as high as 1.50 for natural streams and channels. Because of the possibility of these extreme differences in the velocity coefficient between a laboratory model and an in situ prototype, the sensitivity of the discharge relationship might be significantly effected. If the Coriolis coefficient is significantly greater than unity the discharge coefficient should effectively be reduced to ensure an accurate discharge measurement. Since the coefficient does depend both on the discharge rate (increases with increasing velocity) and the channel depth (decreases with increasing depth, assuming the discharge is constant), it is impossible to assume a single value for any given channel. However, because this coefficient can significantly effect the discharge accuracy it was considered necessary to calculate the Coriolis coefficient for an extreme case, that is one for which α should be approaching a maximum. In order to achieve a maximum value of α, a grid of velocity measurements were taken for a case of low weir height (Z = 4.0") and maximum flow (Q = 2.0 cusecs). The readings were taken using the velocity meter discussed in Chapter II. Contours of equal velocity are shown in Figure 3.4.

3.3 CALCULATION OF CORIOLIS COEFFICIENT

Using Equation 3.15, a computer subroutine may be developed to carry out the necessary calculations. The routine CORLIS was designed for this purpose.

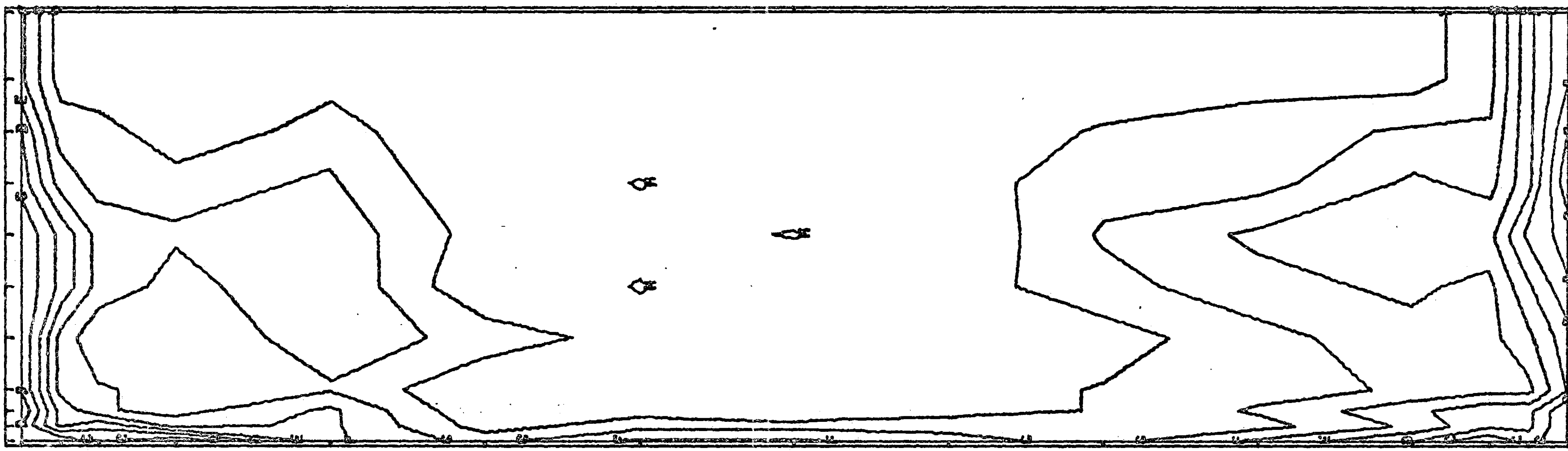
> CORLIS(V,XC,YC,II,JJ,VELCOF,AVEL,TAREA)
> The routine finds the Coriolis coefficient VELCOF, average velocity AVEL, and total area of flow TAREA, for any rectangular channel. V is a two-dimensional array of size II by JJ which contains the measured velocities of flow. The distance between each column and row of the velocity grid are stored in the respective arrays XC(II) and YC(JJ).

The flow diagram of Figure 3.5 shows the sequence of operations carried out by the routine.

3.4 EFFECT ON DISCHARGE

Using the aforementioned test section and the routine CORLIS, it was found that the Coriolis coefficient was equal to 1.02. This value is relatively small and has little effect on the discharge coefficient as can be seen in Figure 3.6.

It should be made clear at this time that although the velocity coefficient is insignificant in the laboratory approach channel, it will in all probability be significant



Velocity contours range from 9 to 16 inches per second by increments of unity.

A - 9
B - 10
C - 11
D - 12
E - 13
F - 14
G - 15
H - 16

Figure 3.4 VELOCITY DISTRIBUTION IN APPROACH CHANNEL

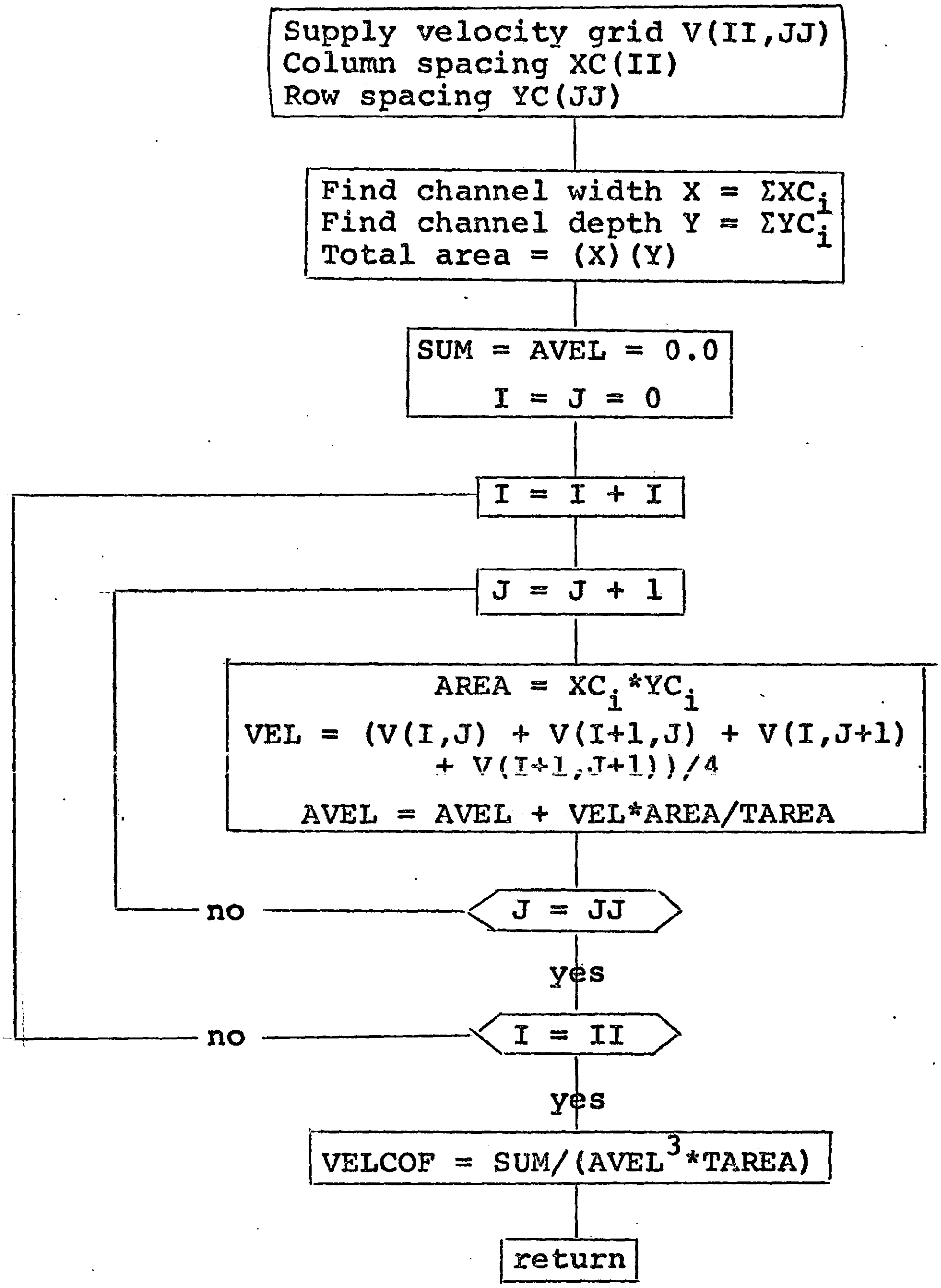


FIGURE 3.5 ROUTINE TO CALCULATE CORIOLIS COEFFICIENT

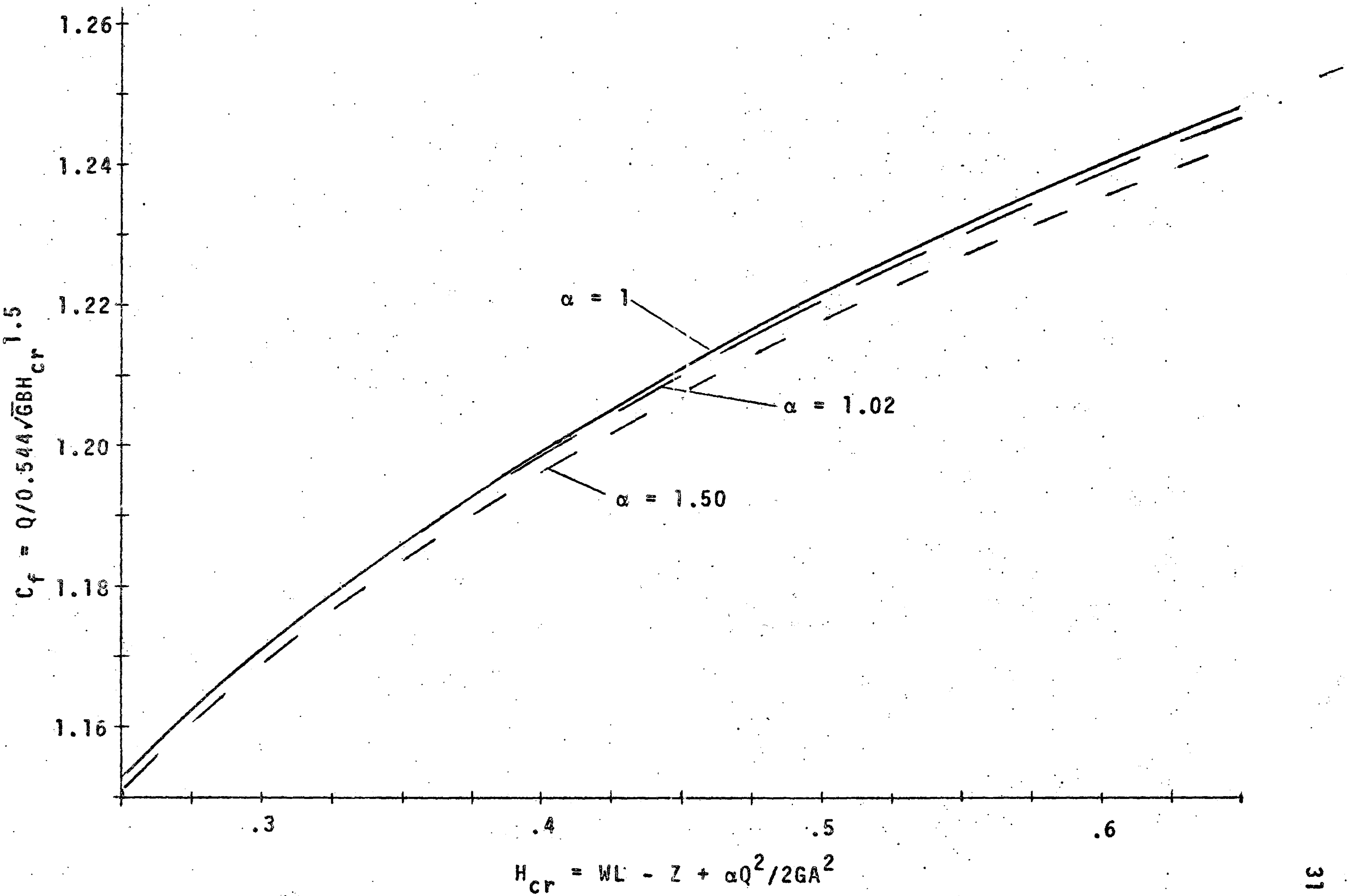


Figure 3.6 EFFECT OF THE CORIOLIS COEFFICIENT ON THE DISCHARGE FUNCTION

in a natural channel approach. It must be realized, therefore, that especially approach channels of shallow depth may require an in situ correction factor for the discharge coefficient in order to compensate for the difference in approach channel properties. This effect will give calculated discharge rates that will be slightly lower than the correct discharge.

3.5 THEORETICAL DISCHARGE FUNCTION FOR COMPOUND SECTIONS

For the development of a discharge relation for a compound section such as that shown in Figure 3.7, the characteristics of discharge may no longer be considered two-dimensional.

In order for the control section and the upstream channel to retain two-dimensional steady state flow conditions the discharge per unit width must be constant over the entire section (i.e. over both subsections A and B in Figure 3.7). This is obviously not the case since the lower section most definitely will carry a greater quantity of discharge per unit width than the higher one. If it is assumed that two-dimensional flow conditions are valid for each individual crest one might hypothesize that the total head is constant over the entire section. The specific energy does not remain constant since it depends on crest level. For the assumption of fixed total head, the discharge per unit breadth and critical depth for each section are not constant (i.e. different for each crest elevation) as implied by Equations

3.7 and 3.8.

The total discharge for any compound section may be easily found as shown below, assuming a constant total head equal to that measured at an upstream section. Thus, using Equation 3.10, the increment of discharge on each segment of the crest may be calculated and summed to give

$$Q_{total} = Q_a + Q_b + Q_c + \ldots.$$
$$= 0.544\sqrt{G} \sum_{i=1}^{N} C_{f_i} B_i \, H_{cr_i}^{\,1.5} \qquad 3.16$$

where N represents the number of subsections or segment in the compound weir section. Equation 3.16 may only be used if a suitable measuring point is available upstream were uniform flow conditions occur.

3.6 TESTS FOR CONSTANT HEAD HYPOTHESIS

For the purpose of studying the distribution of total head across a compound section, a large number of horizontal velocity and water level measurements were made upstream on the crest shown in Figure 3.7 for flows of approximately 1 and 2 curves. The tests were performed both with and without a thin plexiglass divider between sections A and B. The object of incorporating the divider was to more realistically obtain two-dimensional flow. A comparison of the results obtained with and without the flow divider thus provided valuable insight into the validity of the assumptions of two-

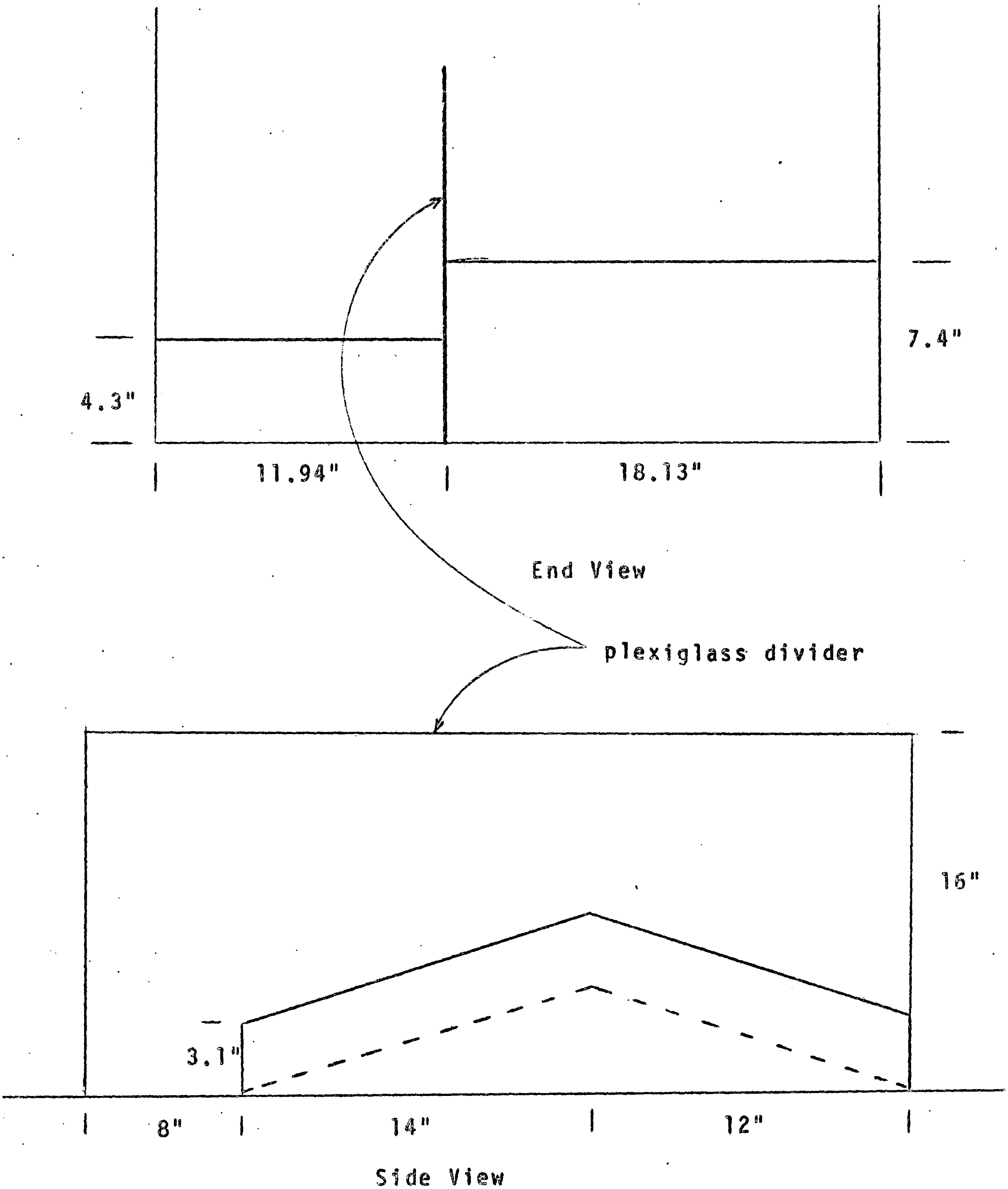


Figure 3.7 TWO SECTION COMPOUND CONTROL

dimensional behaviour.

For each weir configuration and flow rate, the total energy level was obtained from the upstream water surface elevation plus the velocity head, the latter being calculated using the average velocity over a vertical traverse where the velocity probe was used. Although the velocity was not uniform along a vertical, the Coriolis coefficient was taken as equal to unity. Both the channel invert and the water level were measured using the depth gauge.

Table 3.1 gives a summary of the results for the test. For the tests with the divider plate, there was a very noticeable eddy at the upstream edge of the divider when a large proportion of the total flow was routed over the lower crest. This three-dimensional eddy effect can not be modelled by a two-dimensional flow theory, since it will cause an upstream energy level greater than that predicted by a two-dimensional model. This condition implies that the two-dimensional model used for the Crump weir may not be suitable for the high flow ranges.

In the immediate vicinity of the junction of the two sections with no divider, there was evidence of irregular flow. No velocity or depth readings were attempted in this narrow region of disturbance at the intersection of the two subsections.

The energy levels, water levels and discharge per unit breadth are almost identical for the section both before

TABLE 3-1A

TEST #1					Q = 1.10 cfs	
Splitter Plate	Side of Section	WL (inches)	Velocity ft./sec.	EL (inches)	Distance From Peak	Flow Per Foot Width
YES	LOW	8.84	0.621	8.91	42	.456
		8.78	0.776	8.89	24	.567
		8.71	1.02	8.90	18	.739
		8.49	1.42	8.87	12	.803
		8.13	2.08	8.94	6	.870
		7.31	3.34	9.39	0	.849
	HIGH	8.84	.662	8.92	42	.485
		8.87	.538	8.92	24	.398
		8.86	.454	8.91	18	.336
		8.86	.461	8.89	12	.192
		8.83	.674	8.91	6	.178
		8.46	1.96	9.18	0	.177
NO	LOW	8.85	.643	8.93	42	.462
		8.83	.747	8.92	24	.548
		8.80	.863	8.94	18	.634
		8.70	1.08	8.92	12	.707
		8.45	1.64	8.95	6	.805
		7.65	3.07	9.40	0	.865
	HIGH	8.86	.693	8.95	42	.517
		8.87	.604	8.94	24	.446
		8.85	.579	8.91	18	.426
		8.83	.686	8.92	12	.286
		8.78	.862	8.92	6	.225
		8.40	1.83	9.02	0	.157

TABLE 3-1B

TEST #2 Q = 2.01 cfs						
Splitter Plate	Side of Section	WL (inches)	Velocity ft./sec.	EL (inches)	Distance From Peak	Flow Per Foot Width
YES	LOW	10.23	.93	10.39	42	.792
		10.16	1.09	10.38	24	.923
		10.05	1.32	10.38	18	1.105
		9.77	1.80	10.37	12	1.340
		9.19	2.53	10.38	6	1.395
		9.48	3.76	11.11	0	1.315
	HIGH	10.24	.98	10.42	42	.833
		10.26	.96	10.43	24	.820
		10.29	.88	10.43	18	.755
		10.27	1.00	10.44	12	.534
		10.14	1.35	10.48	6	.505
		9.46	2.72	10.84	0	.472
NO	LOW	10.26	0.95	10.43	42	.810
		10.18	1.13	10.42	24	.960
		10.15	1.17	10.41	18	.990
		10.01	1.42	10.39	12	1.090
		9.65	2.01	10.40	6	1.182
		8.80	3.38	10.92	0	1.285
	HIGH	10.27	1.07	10.48	42	.913
		10.27	1.04	10.47	24	.888
		10.24	.93	10.44	18	.793
		10.16	1.25	10.45	12	.657
		10.02	1.49	10.43	6	.542
		9.37	2.47	10.77	0	.457

and after the removal of the plexiglass sheet. A slight increase in discharge over the lower section was noted when the divider was removed. This may be due to a small amount of discharge cascading over the section incline.

The results of the test show that the total head is generally constant over the entire section no matter what the constriction height, either with or without the implementation of the divider plate. For this reason it is felt that the discharge function for compound sections might be based on two-dimensional flow characteristics. At the crest, the calculated total head is greater than that for the remainder of the channel. The reason for this apparent increase in energy is easily explained by considering Equation 3.2. The empirical coefficient β, is dependent on the streamline curvature in the channel section. For convex flow (the type of flow occurring at the crest) β is always less than unity. If the pressure distribution coefficient is applied to the potential head calculation, it would most probably reduce the total head to a value similar to that obtained in the upstream part of the channel.

The addition of a greater number of sections to the control would in no foreseeable way effect the concept of constant head for the section. It is, therefore, assumed that the above hypothesis remains valid for all compound sections.

3.7 DISCHARGE CALCULATION FOR A COMPOUND SECTION ASSUMING TWO-DIMENSIONAL FLOW CHARACTERISTICS

Assuming a constant head, the discharge over each segment of a compound weir may be calculated independently. For a known constant energy level, the discharge over any section is given by Equation 3.16. Since only upstream water level can be measured independently of discharge, the total head at the critical section must be solved iteratively making successive approximations to the flow rate. The routine DISCHAR was developed for this purpose.

> DISCHAR(B,H,NPTS,WL1, B1,QCR,ACR,ECR)
> The geometry of the downstream critical section is given by the arrays B and H(1,NPTS). The water level WL1 and the channel breadth B2 is given for the upstream section. Assuming two-dimensional flow, characteristics the routine calculates discharge QCR, area ACR and total critical head ECR for the profile shown in Figure 2.3. H(1,NPTS) refers to the upstream invert as water level.

The process is illustrated in the flow diagram of Figure 3.8 and the completed routine is given in Appendix II.

If the upstream section is irregular, the upstream area can be introduced directly as a function of water level. Smith[10] has developed suitable subroutines by which this can easily be accomplished.

3.8 TEST RESULTS FOR COMPOUND SECTIONS

In order to verify and refine where necessary the method described in the routine DISCHAR, a series of laboratory tests were performed on compound sections. Four main

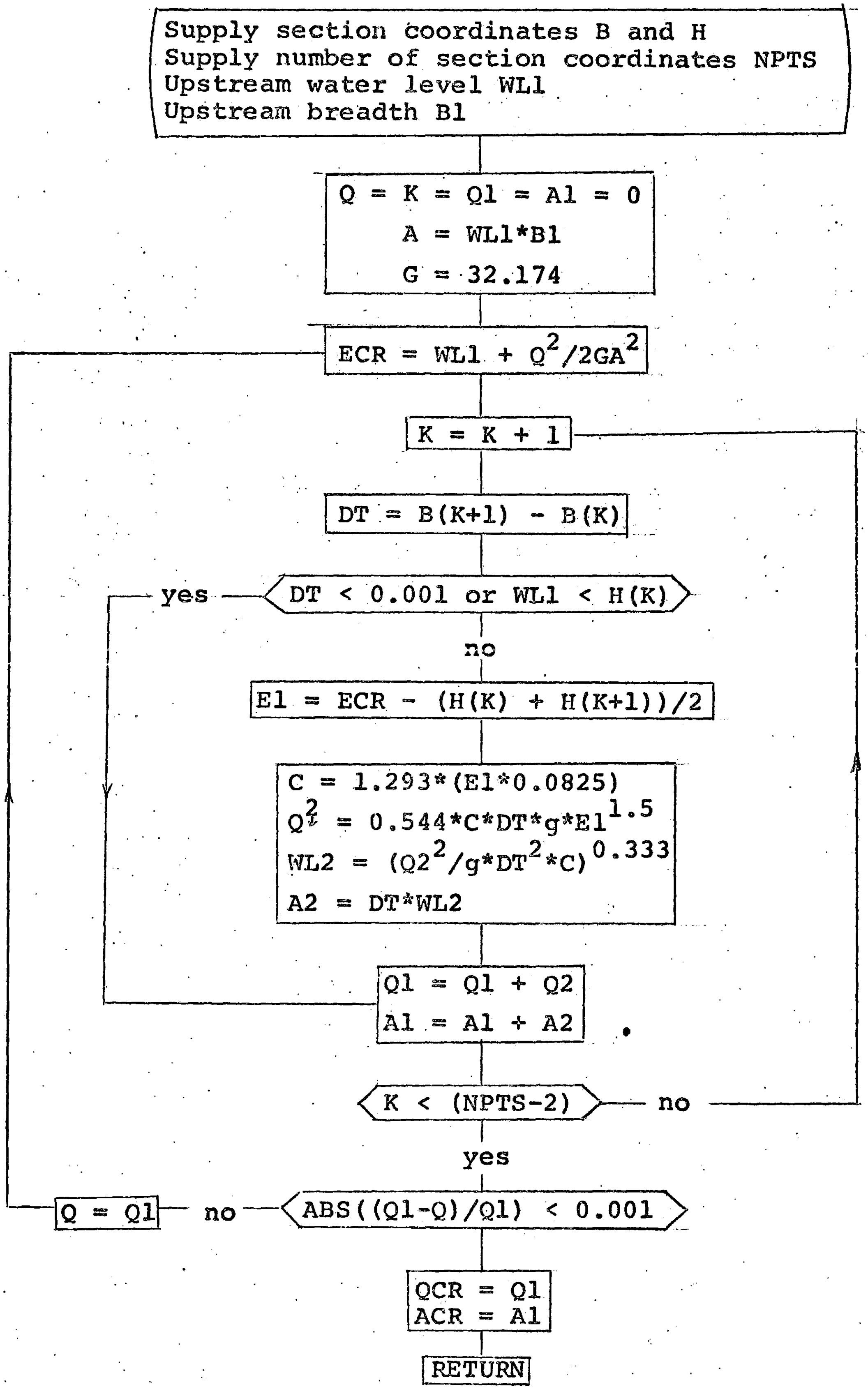


FIGURE 3.8 FLOW DIAGRAM FOR ROUTINE DISCHAR

shapes were used for these tests, with the number of subsections or segments varying from two to five as shown in Figure 3.9. The test results are given in Appendix C, Table 4 to 22 inclusive.

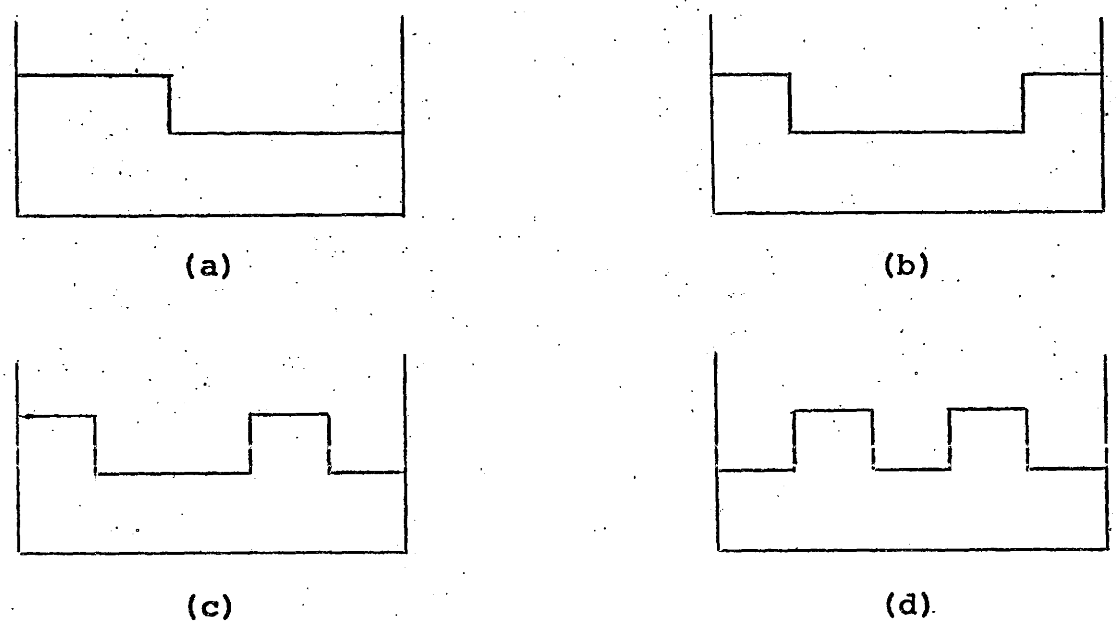


FIGURE 3.9 TYPICAL COMPOUND SECTIONS SHAPES

For each geometrical arrangement, the upstream water level was measured for a range of known flow rates. By means of the routine DISCHAR the total energy level ECR and the predicted flow rate Q1 were computed as functions of the upstream water level. Finally the difference between the discharge computed (i.e. equation 3.16) and the actual (QGIVEN) was obtained. Quantities are tabulated in Tables C-4 to C-22 as follows.

Column #1 QGIVEN = Measured flow rate,

Column #2 Q1 = Flow rate by Equation 3.16,

Column #3 WL = Measured upstream water level,

Column #4 ECR = Computed energy level,

Column #5 QLOST = Q1 - QGIVEN = Error in flow rate.

This difference is a measure of the inadequacy of the model based on a two-dimensional premise, to properly describe the three-dimensional conditions of flow over a compound weir. It is notable that this shortcoming is appreciable. The calculated value for discharge becomes increasingly greater than the actual discharge as the water level increases above the lower part of the section. Although it is apparent from the previous results that the total head does not fluctuate over the compound section, a two-dimensional head-discharge relation is not totally valid.

3.9 DISCHARGE REDUCTION EFFECT

With the above discussion serving as an introduction, the discharge loss effects may now be considered. The typical section shapes shown in Figure 3.9 (a) - (d) represent a series of possible compound shapes. In order to introduce an element of conformity for the purpose of comparing results of various sections, each vertical discontinuity in the section geometry will be considered as a potential cause of "discharge loss" or reduction in the effective breadth of the lower adjacent segment. Therefore, the discharge reduction can be given

$$Q_{LT} = \frac{Q_1 - Q}{n - 1} \qquad 3.17$$

where $Q1$ = discharge calculated by the two-dimensional model,

Q = actual discharge,

n = number of discontinuities all with an equal vertical distant C.

Thus, the five part section in Figure 3.9 (d) will have twice the discharge reduction effect of the three part section - Figure 3.9 (b) - other things being equal. This implies that the reason for the difference between calculated and measured discharge is due to local loss effects or end contractions at the transverse discontinuities in the control section.

There are two distinct regions of discharge loss for each discontinuity, each of which appears to function independently.

1. The water level is less than the higher subsection,
2. The water level is greater than the higher subsection.

A plot of discharge reduction factor versus water depth (where water depth is measured from the crest of the lower subsection) is shown in Figure 3.10. The graph shows plainly the two distinct regions as discussed earlier.

When the water level overtops the higher section the discharge reduction essentially remains constant, while the

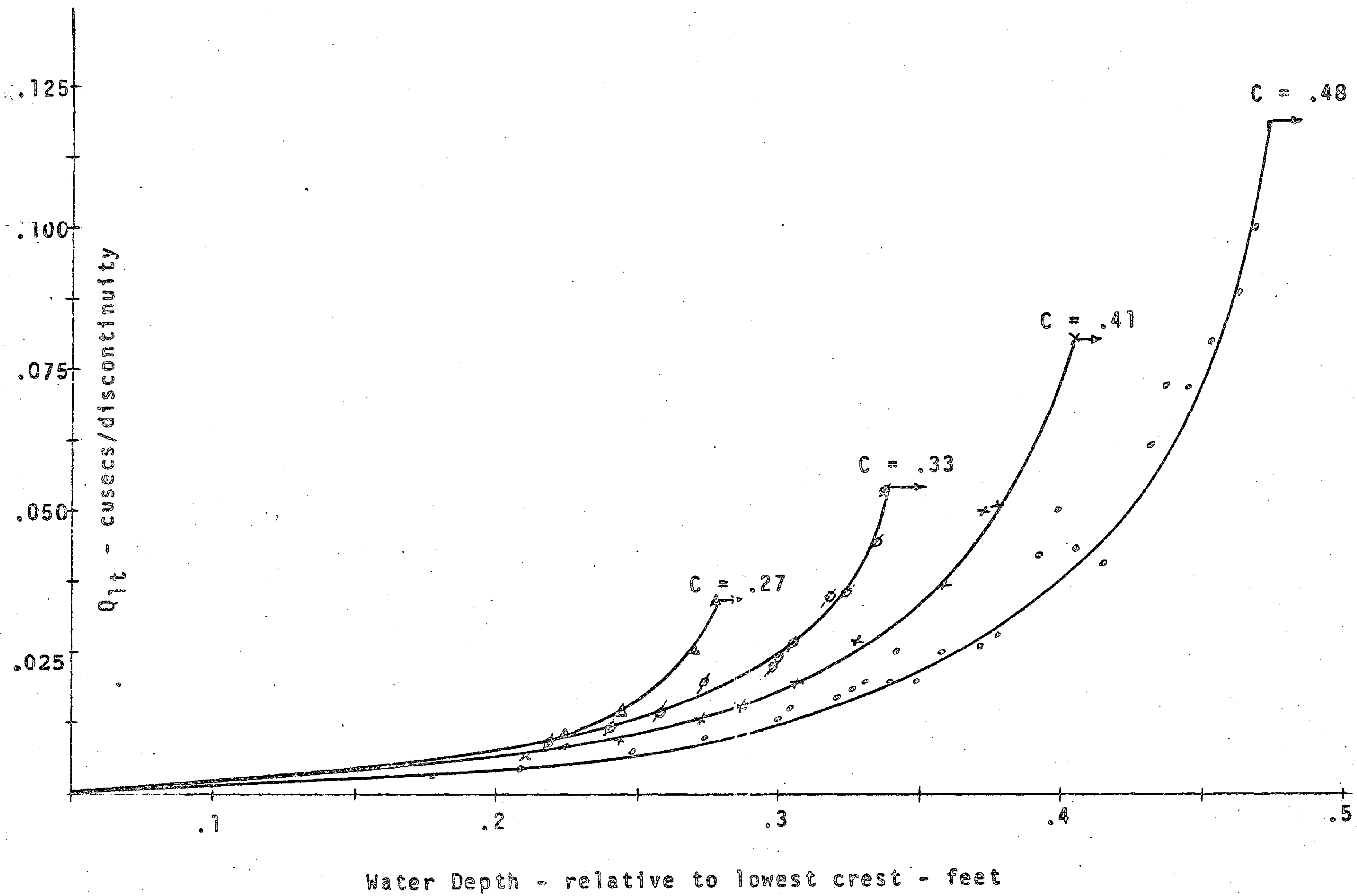


Figure 3.10 WATER DEPTH VERSUS DISCHARGE LOSS

loss effect is dependent on water level when the water level is below the higher section. A plot of water level upstream versus $\log_{10}$ (discharge reduction factor) gives a convenient straight line representation for the series of curves, as can be seen in Figure 3.11. The general equation for a straight line semi-log fit is shown below

$$WL = a + b \log_{10}(Q_1) \qquad 3.18$$

where WL = upstream water level,

Q_1 = effective discharge reduction,

a = constant depending on the intersection of the line,

b = constant depending on the slope of the line.

Curve A represents the condition for WL greater than the vertical discontinuity distance given by C. The remaining curves represent the condition where water level is below the higher crest for various values of vertical discontinuity height given by C. The intersection of these curves with the A curve represents the point of over-topping of the higher crest.

The results obtained for the two part sections (i.e. a single discontinuity) were not used in plotting Figures 3.10 and 3.11, since it was found that these results were not compatible with the other section types used for the experiments. These results are discussed more fully later in this Chapter.

In order to simplify the series of curves developed in Figure 3.11, it would be necessary to consider each of the

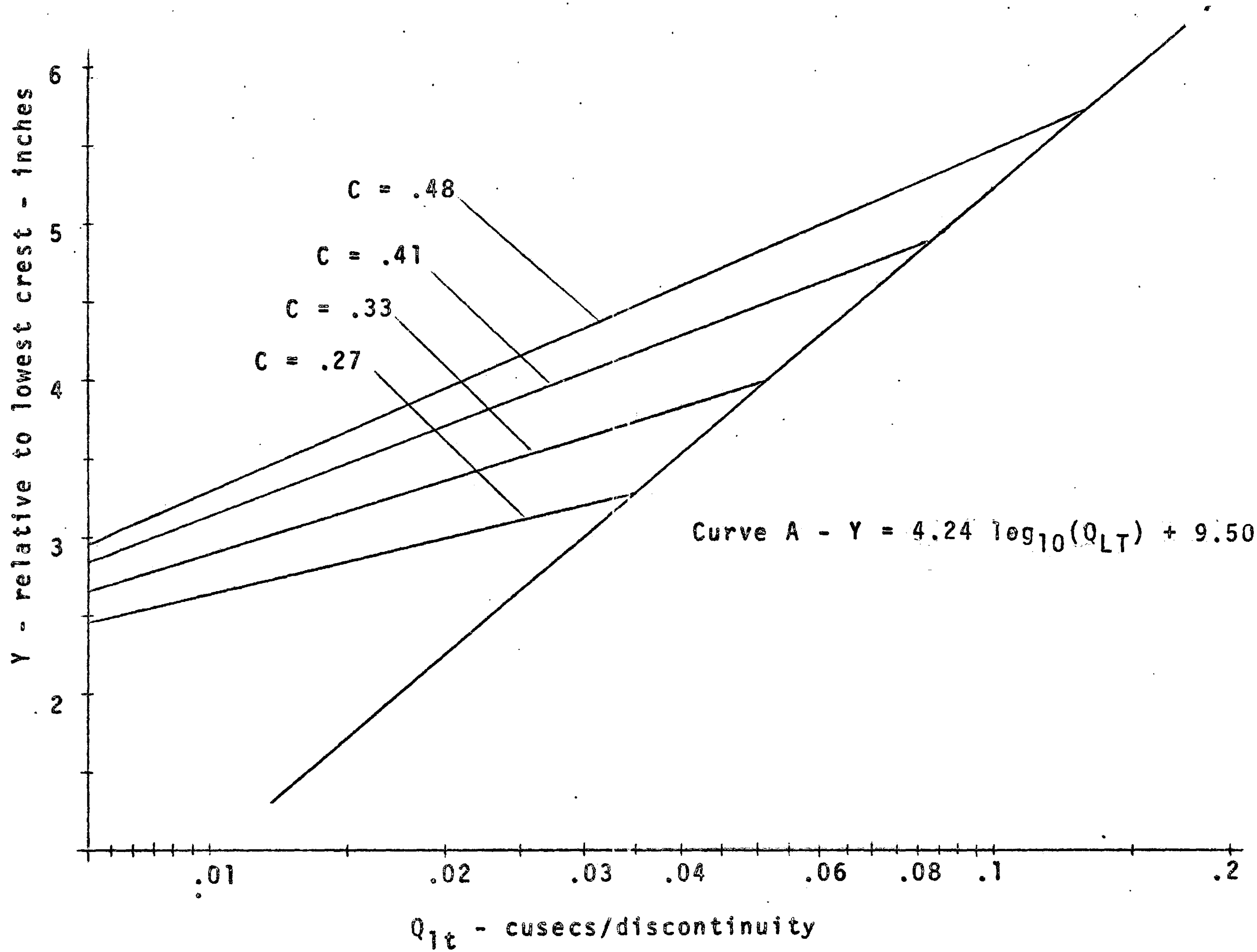


Figure 3.11 LOG_{10}(DISCHARGE LOSS) VERSUS WATER DEPTH

variables discussed below and shown in Figure 3.12.

1. Upstream water depth Y: As the upstream water level increases so does the end contraction or reduction effect, provided the vertical difference in weir sections (C) is constant.

2. Difference in Subsection Elevation: (C) As this difference increases the effective reduction decreases given constant water level.

3. Distance of the point of contact of the upstream water level from the extreme peak (X). The distance of the upstream water level from the extreme peak depends both on C and Y.

Since these variables are interdependent, it is not possible to develop a single curve or function that will satisfactorily represent the series of curves.

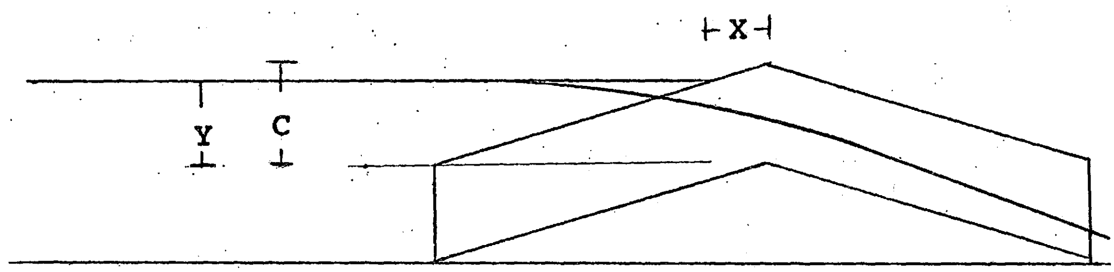


FIGURE 3.12 CRITICAL DIMENSIONS FOR EFFECTIVE DISCHARGE REDUCTION

A routine named COEF was therefore developed to calculate the reduction in discharge for the given water level on the basis of empirical data and relationships.

> COEF(H,NPTS,WL,QCR,QLOST,QCORR)
> Given the elevation coordinates H(1,NPTS) for a critical section, the routine calculates the effective reduction in discharge QLOST, necessary to adjust the given input value of critical discharge ACR calculated by the routine DISCHAR. The adjusted value of flow quantity is named QCORR.

For the case where water depth is greater than C, the single Curve A is represented by Equation 3.18 allowing a direct calculation of discharge reduction per discontinuity for a given value of C. Since the series of curves for water level less than C converge, two sets of coordinates are known on each of these curves, one from the convergence point and the other from Equation 3.19.

$$C = 4.25 \log_{10}(Q_1) + 9.50 \qquad 3.19$$

The constants a and b in Equation 3.17 can thus be calculated, and the discharge reduction calculated for any given water level and value for C. The flow diagram for the routine is given in Figure 3.13, and the corresponding Fortran listing is included in Appendix D.

3.10 TOTAL DISCHARGE FUNCTION

Using the aforementioned routines (COEF and DISCHAR) a suitable driving program (see Appendix D) was developed to predict the discharge as a function of upstream water level.

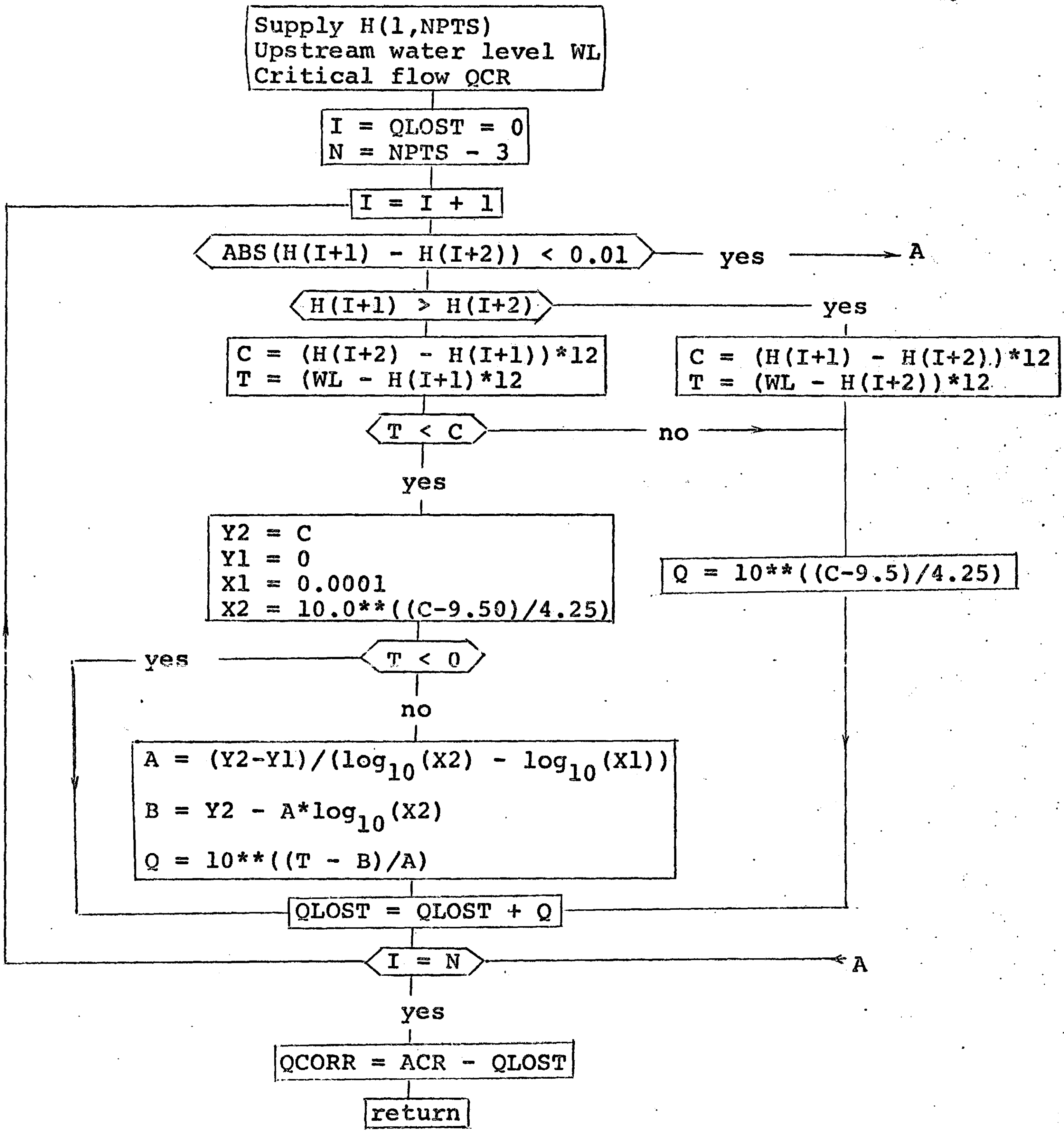


FIGURE 3.13 FLOW DIAGRAM FOR SUBROUTINE COEF

For the series of tests used to develop Figure 3.10, the predicted discharge for the method is given in Appendix C (Tables 11 - 22, Column #7). The percentage error relative to the measured discharge is given in Column 8 of the same Tables. These values are well within the limits stipulated by the error graph in Figure 2.2. In addition to these sections, six additional sections were tested solely for the purpose of independently checking the discharge relation as derived from the previous data. These sections as well are within the acceptable range of accuracy.

Although the discharge relation functions quite adequately, there are two points in conjunction with the development that should be discussed at greater length. These are (i) the incompatibility of the two part compound section and (ii) the empirical nature of the discharge relation.

The discharge reduction effect for the single discontinuity is approximately equal to that of the three part section. Therefore, this data could not be used for the computation of the discharge relation. Although no definite solution was found to explain this problem, possible reasons are discussed below.

With reference to Figure 3.8, a possible cause for the difference in the two part section could be a boundary layer effect along the plexiglass wall at the lower side of the constriction. This boundary layer would tend to make the critical section narrower which in turn would make the discharge reduction greater. It is felt, however, that this argument is not valid for the following reasons.

1. A similar boundary layer would exist for the uniform section calibration. Therefore, the compound section would have no change in discharge when compared with the uniform section.

2. Other typical sections in Figure 3.8 have full water contact with the plexiglass sheeting and their effective discharge reductions were compatible.

3. The boundary layer developed on the channel sides was calculated using the method described by Harrison[10]. It was found that the thickness of the layer thus determined, is so small that it has little or no effect on the capable discharge of the section.

Although these results do not coincide with those of greater subsection numbers, the discharge relation is still considered valid for the following reasons.

1. As can be seen by the results in Appendix C (Tables 4 - 10) the percentage error between given and calculated discharge varies from zero to about six percent. This difference although significant does not justify total rejection of the section. Several control sections now in use have discrepancies of at least 5 percent or greater for the low flow ranges.

2. In prototype applications this type of section would not be generally implemented. Because channel profiles

are usually symmetrical, the constriction would likewise tend to be of symmetrical shape as shown in Figure 3.14. Even for rectangular sections, it would be assumed that the low flow section would be placed in mid-section. This would tend to give the control a more pleasing appearance.

It can, therefore, be seen that although this type of section does have significant academic interest it has little or no practical application.

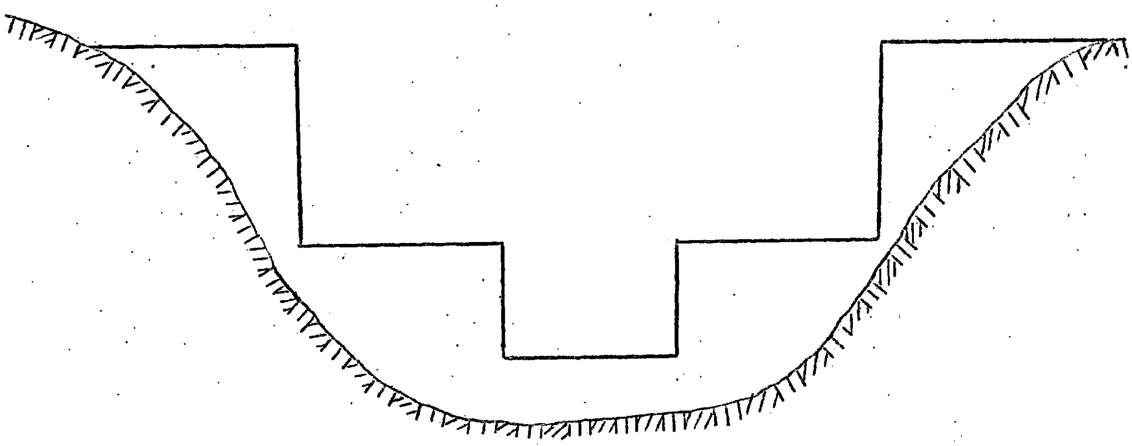


FIGURE 3.14 SYMMETRICAL CONTROL SECTION

It was felt that an empirical solution for the discharge relation was the only possible one that could be used at this time. As discussed earlier the discharge loss effects for two-dimension flow are felt to be local effects occurring at the vertical discontinuity. There are two independent cases to be considered at this point. The first being when the water level is less than the height of the discontinuity and the other when the water level has overtopped the higher subsection. Harrison[11] and Hall[12] have done considerable work on the loss in effective discharge for flow around sharp 90° corners and bends. Because the flow in this case is over a sloping profile neither of these methods seem applicable to this specific problem. Since the main purpose of this study is the development of a stage-discharge relation for prototype installations it was decided that the empirical solution is satisfactory at this time. A theoretical solution may, however, incorporate the incompatible results obtained.

3.11 DROWNED FLOW CONDITION

In the drowned flow range the discharge is dependent on both the upstream and downstream water levels. It is therefore, a function of two variables. When the weir is controlling the regimen, the tail water elevation can be changed without altering the depth of flow over the weir or the upstream water level, with discharge remaining constant. However, when the weir is in the drowned condition each change in tail-water elevation produces a corresponding change in the depth of flow over the weir crest and the headwater elevation. An extensive survey of existing prototype structures of triangular shape was carried out by Burgess and White[4] in order to find the ratio between critical energy level and control section height for the modular limit. They found that the average value for H_{cr}/ Z (see Figure 3.15) at the modular limit to be equal to 2.15. It must be realized that the value of H_{cr}/ Z depends upon the nature of the channel properties. When the normal depth downstream is such that the total energy downstream is greater than $H_{cr} + Z$ the control section will be drowned assuming no energy loss at the section. The value of H_{cr}/ Z therefore, is not a constant

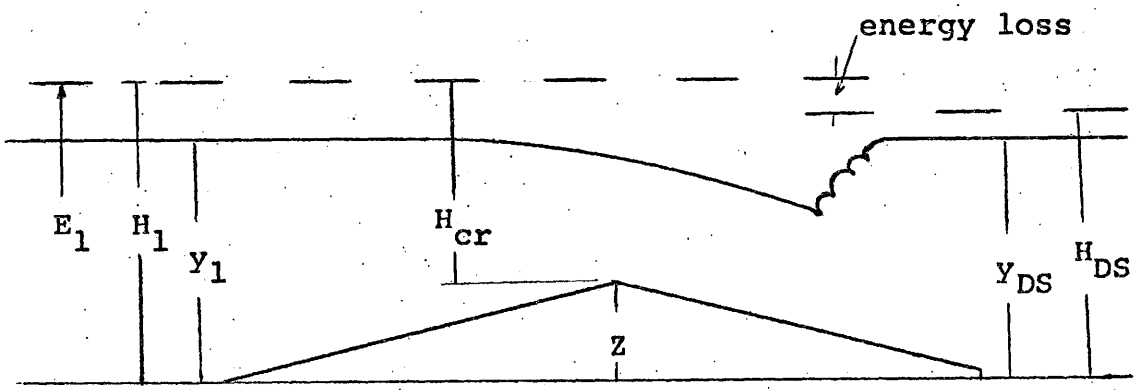


Figure 3.15 APPROACH TO MODULAR FLOW LIMIT

but varies with the type of channel. A reasonable estimate for the value of the control height to prevent the drowned condition for a given maximum discharge could, therefore, be estimated by

$$Z = H_o - H_{cr} \tag{3.20}$$

where H_{cr} = critical energy for a given discharge,

H_o = total energy level for normal flow at maximum discharge.

The value of Z here refers to the lowest section level in the compound control. It must be emphasized that Equation 3.20 gives only an estimate for the minimum section height since there is an assumption of no energy loss at the control. Since the estimate of maximum discharge for gauging so greatly depends upon the designer, a value of Z slightly greater than that given by Equation 3.20 would be more acceptable.

The development of a stage-discharge relation for the drowned condition might also be possible for compound sections as has already been done for uniform crests. A more detailed consideration of this problem is given in Chapter IV.

3.12 STANDING WAVES IN THE APPROACH CHANNEL

If the flow properties in the upstream channel are such that the Froude number is relatively high ($\frac{V}{\sqrt{Gy}}$ greater than 0.5) stationary waves may be formed in this region.

These waves will influence the head measurement upstream in two ways.

1. The direct measurement of water level will be affected since the waves make it difficult to estimate the free water surface accurately.

2. The waves indicate a non-hydrostatic pressure distribution in the channel. This will cause the value of β for Equation 3.1 to be less than unity. Therefore, the total head calculation for $\beta = 1$ is an overestimation. It was decided to limit the range of the tests in order to eliminate this problem. Therefore, the Froude number in the upstream channel always was kept less than 0.6.

CHAPTER IV

THE COMPLETE DISCHARGE MODEL

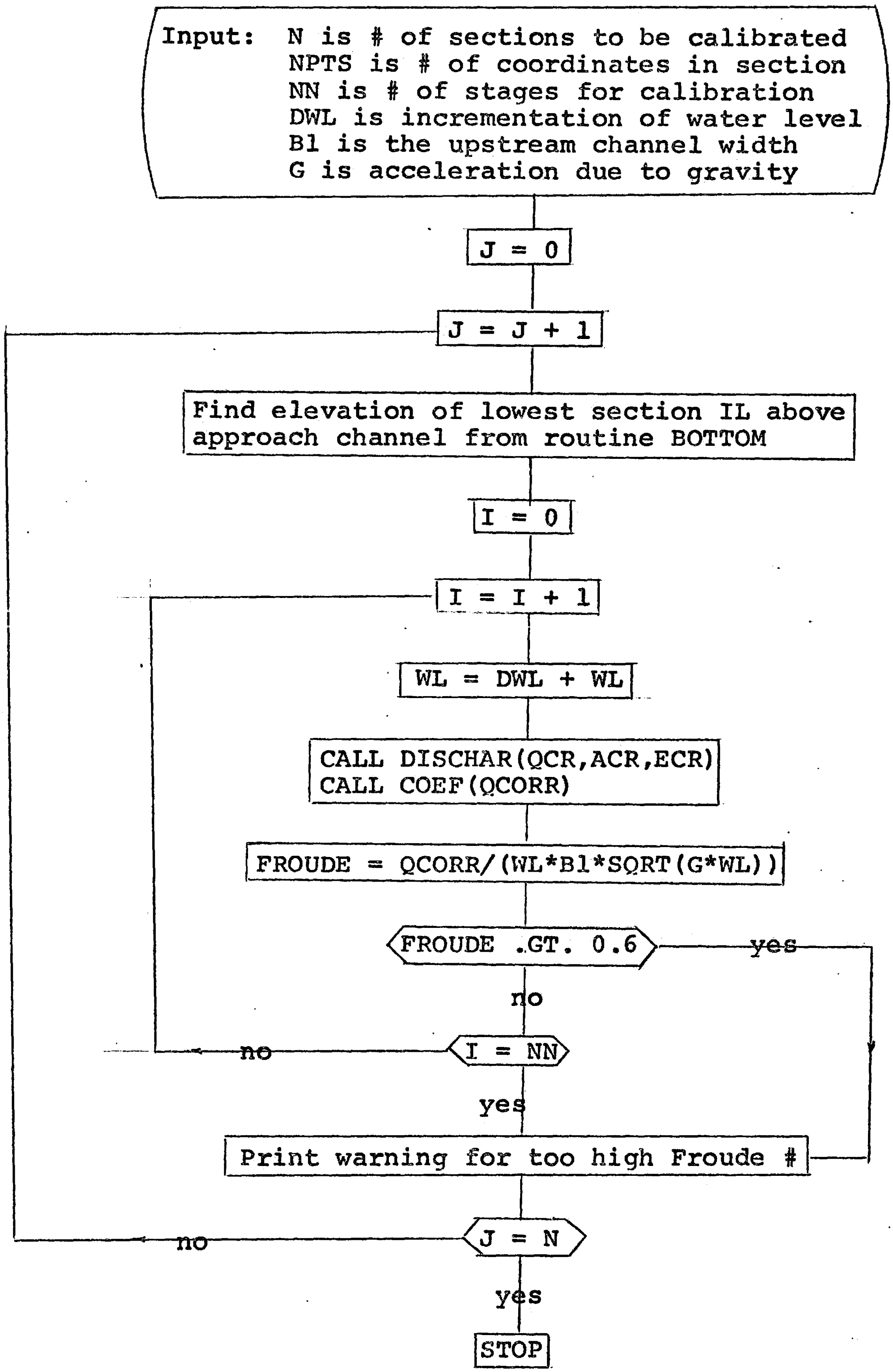
4.1 DEVELOPMENT OF RATING CURVE

In order to easily utilize the subroutines developed in Chapter III, a driving program was written for computing the stage-discharge relation for any compound section. When in the process of designing a compound section, this routine supplies a quick and easy way of checking the adequacy of a particular section.

The only input required is the physical geometry of the control and gauging section, and the incremental water levels desired for the stage-discharge relation. The driving routine then makes use of the subroutines BOTTOM[10], DISCHAR, and COEF in that order to calculate the discharge and specific energy for a given water level. The routine terminates if the following condition becomes critical. The Froude number in the approach channel becomes significantly large so that standing waves might occur.

The specifications for the routine are given in the figure below, while a Fortran printout is given in Appendix D.

```
Driving Routine (Input,N,npts,NN,DWL,B1,G:
                  Output,WL,ECR,DISCHAR)
The routine calculates the rating curve for
a compound control section given the critical
section coordinates B and H(1,NPTS) and the
upstream channel width B1.
```


Input: N is # of sections to be calibrated
NPTS is # of coordinates in section
NN is # of stages for calibration
DWL is incrementation of water level
B1 is the upstream channel width
G is acceleration due to gravity
J = 0
J = J + 1
Find elevation of lowest section IL above approach channel from routine BOTTOM
I = 0
I = I + 1
WL = DWL + WL
CALL DISCHAR(QCR,ACR,ECR)
CALL COEF(QCORR)
FROUDE = QCORR/(WL*B1*SQRT(G*WL))
FROUDE .GT. 0.6
yes
no
I = NN
no
yes
Print warning for too high Froude #
J = N
no
yes
STOP

A typical rating curve for a given compound section is shown in Figure 4.1. The three stage-discharge curves shown represent the following models.

Model 1

This is a one-dimensional model assuming a uniform velocity distribution at the critical section. The routine WIERFL[10] was used for the purposes of this calculation.

Model 2

This two-dimensional model assumes that the total energy line is horizontal over the crest and equal to the total energy measured upstream. The routine DISCHAR is used for calculating discharge from a given water level on the following basic assumptions. The dimensionless discharge coefficient over each section is given by $C_f = 1.293 H_{cr}^{0825}$ and the total discharge is equal to the sum of the discharges over each section as given by Equation 3.16.

Model 3

This curve represents a three-dimensional model since it considers the local loss effects occurring at the vertical discontinuities. Using the discharge calculated by Model 2 the routine COEF subtracts a suitable correcting discharge for the given section.

The actual discharge is represented by experimental points on the rating curve. As can be seen by Figure 4.1, the three-dimensional model is far superior to the other two for predicting the discharge accurately.

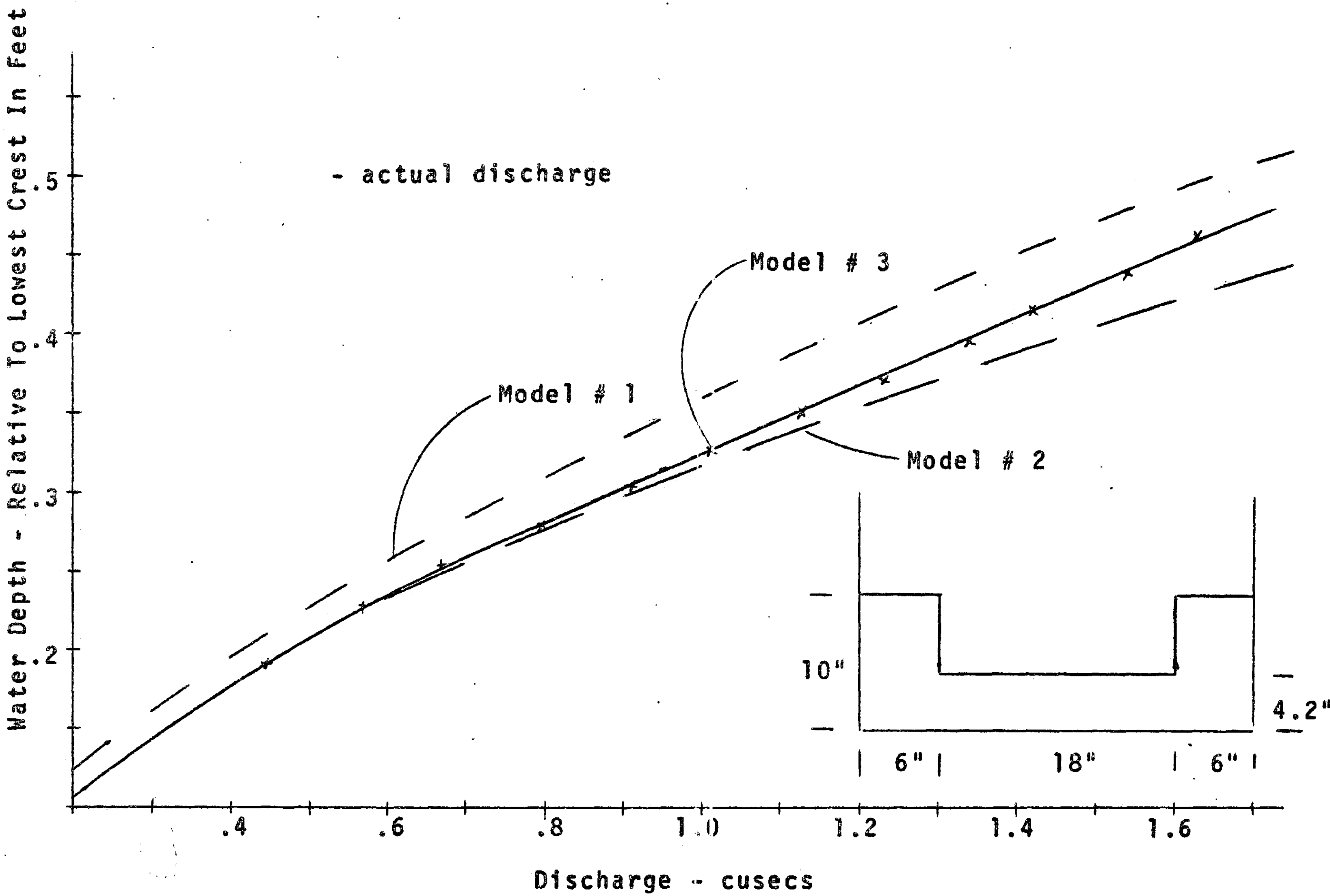


Fig. 4.1 COMPARISON OF MODELS TO ACTUAL DISCHARGE

The curve should not be drawn for water levels of less than 0.1 feet, due to the explosive nature of errors below this level. Therefore if a minimum discharge for gauging is given the section can be altered by the following means to meet this specification.

1. If the minimum discharge is below the rating curve, the width of the lower section will have to be decreased to achieve this minimum.

If a maximum discharge specification is given for the section, one of the following alterations to the section will become necessary.

1. If maximum discharge is not obtained for the section being calibrated, the section may either by made wider or higher relative to the upstream channel base, thus increasing the discharge capacity.

2. If the maximum discharge is below the maximum point on the rating curve the section might either be made narrower or lower, thus decreasing discharge as well as cost.

The curve should also be checked to ensure that a reasonable flow of water is passing through the lower sub-section independent of the remaining sections. Since it is desirable that this section take the entire flow for the majority of time, approximately five percent of the maximum discharge would seem to be a reasonable amount.

4.2 ADVANTAGES OF THE SYSTEM

This type of measuring device has several advantages over the conventional systems now being used for this purpose.

1. The range of discharge that can be measured to a greater degree of accuracy is significantly improved. With a segmental arrangement, the initial change in water level for a given change in discharge may be increased thus allowing measurement of low flows with a higher degree of accuracy.

2. Since the placement of subsections is relatively arbitrary the control can easily be designed to accommodate irregular channel geometry, or unusual downstream stage-discharge relations.

3. The removal of interior dividing piers almost totally eliminates the possibility of drift material being collected at the control section. This is especially important during the period of spring breakup, as ice flows could be passed by this type of section relatively easily. Since the water level is usually high during this period (well above the lower crest) there is very little possibility of ice being lodged across the narrower, and lower section.

4. The gentle approach slope will encourage the passage of sediment over the control more freely as opposed to the abrupt broadcrested sections which tend to allow extensive sedimentation in the approach channel.

5. The mild downstream face directs a large component of the discharge momentum in the direction of the downstream

channel flow. This will help to decrease downstream channel erosion and therefore decrease the necessary cost of bed protection.

5. Because of the nature of the section geometry, insitu construction is made easier and therefore less costly for the following reasons:

a) no internal dividing piers are necessary,

b) no curved sections are used thus decreasing the cost of formwork,

c) the subsections could be made from independent precast sections, thus reducing the necessary formwork.

6. For the tests performed the small angle at the crest did not allow the formation of a nappe. Since no free jet is formed there will be no discontinuity in the rating curve because of this effect.

7. Once this type of measuring device has been proven on the prototype scale there will be no necessity for field calibrations. Unlike sections with curved cross-sections the coefficients for discharge do not change when the cross-sectional shape is altered. Therefore, no matter what the variation in cross-sectional shapes, the discharge model developed in Chapter III is still valid. A laboratory model is not necessary for each prototype installation used.

8. The computer model for the compound control allows the study of a variety of cross-sectional shapes for any field

design to be carried out both quickly and efficiently.

4.3 DISADVANTAGES OF THE SYSTEM

Although the compound control has several advantages that make its practical application look both promising and desirable, there are also some disadvantages to its application.

1. The non-conformity of some of the (single discontinuity) data obtained is rather unsatisfactory but as discussed earlier, these results will not in all probability affect the suitability of the method for practical installations. Some explanation of this incompatibility would, however, increase confidence in the model.

2. Because of the lower compound section, the overall height of the weir must be increased to prevent the possibility of drowned flow conditions occurring.

3. Surface wear at the well defined corners will tend to decrease the sensitivity of the rating curves. Careful consideration must be used in construction in order to make the corners as durable as possible.

4. Although the problem of a nonuniform velocity distribution in the upstream section is not solely applicable to compound sections, it must be considered as a possible source of inaccuracy in the discharge relation. As discussed in Chapter III, it might be necessary to vary the discharge relation if the effect seems significant.

4.4 COMPARISON TO PREVIOUS RESEARCH

The results of the laboratory tests performed by the author agree to a reasonable extent with the tests performed by Crump[3], Burgess and White[4], and Smith[6].

The dimensionless coefficient of discharge for a straight line cross-section varies only slightly with work done by the authors mentioned above. This can easily be explained due to the control section profile. As discussed earlier both the downstream and upstream slopes of the section effect the discharge relation.

Crump assumed that the energy level remains constant for a compound Crump weir, a property that has been duplicated by the section used in this report. He also introduced an effective discharge reduction due to the dividing piers introduced in his model. These piers caused an increase in upstream water level of a similar nature to the vertical discontinuities.

4.5 CONSIDERATIONS OF FURTHER RESEARCH

The design and construction of a prototype section would give invaluable information with regards to the practical applications of the model. Using other acceptable methods for measuring discharge (velocity-area or chemical dilution) a check on the validity of the model could easily be made. Considerable information could also be gained on the possibility of harmful effects occurring in the channel due to the introduction of the control section.

The discharge relation for the compound sections might be extended to include drowned flow conditions. If a successful extension of the discharge relation for drowned flow could be made, the overall height of the control section could be reduced. This not only decreases the cost of construction but also the upstream water level and thus the distance upstream affected by the backwater curve.

4.6 PRACTICAL APPLICATIONS

The control would be useful in any stream or channel that due to natural conditions has a fluctuating discharge rate. The increased range of accuracy would make the results of the control more beneficial for any project that requires high accuracy water quantity measurements.

The section might also be used in the design of small dams or spillways. As well as a measuring device, the step-like nature of the control would act as an automatic regulation gate. During periods of low flow, the lower section alone would pass the excess discharge of the reservoir. At flood peaks, the entire section could be used to allow a sufficient flow of water to pass to prevent overtopping of the dam.

4.7 SUMMARY

With the results obtained using a compound control section of triangular profile an empirical model was developed which will establish the stage-discharge relationship for any similar section. The sensitivity obtained for the relation is well within the limits of the instrumentation used. Although some non-conformity was noted for a particular section shape, it was considered that the elimination of these results did not significantly affect the validity of the model. The computer routines developed for the discharge model make the iterative calculation of rating curves an efficient and highly accurate process.

It is felt that the implementation of this model to prototype situations will give a high degree of sensitivity for discharge, especially in situations where the discharge fluctuation is extreme.

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APPENDIX A

DEFINITION OF SYMBOLS AND TERMINOLOGY

Artificial Control Section - A constriction section, which for a given range of flows renders the upstream elevation of the water surface independent of the water surface downstream. Under these conditions the discharge is a function only of upstream water level.

Backwater - The water profile backed up behind a control when it is introduced to a channel.

Crest - The line or area defining the upper surface of that portion of a weir over which the water flows.

Critical Flow - The flow condition in which the total energy is a minimum for a given discharge. Under these conditions the Froude number is equal to unity.

Crump Weir - A Crump weir is a triangular profile weir with an upstream slope of 1.2 and a downstream slope of 1.5.

Discharge - Volume of water flowing through a given

		cross-section in a unit of time.
Drowned Flow	-	Flow over a control section is drowned when it is affected by changes in the downstream energy level.
Mean Depth	-	Depth obtained by dividing the cross-sectional area of flow by the free surface breadth.
Froude Number	-	A dimensionless number obtained by dividing the mean velocity by the square root of the product of the mean depth and the acceleration due to gravity.
Modular Flow	-	Flow over a control section when the upstream level is independent of the downstream level.
Nappe	-	Free jet formed by the flow over a weir.
One-Dimensional Flow	-	Flow of a fluid for which the velocity is constant for a given discharge and section area (i.e. $V = Q/A$).
Rating Curve	-	A plot of discharge versus upstream water level for a given weir section.
Stage	-	Water level measurement.
Two-Dimensional Flow	-	Flow of a fluid for which the velocity varies as a function of the water depth (i.e. $V = V(y)$).
Three-Dimensional Flow	-	Flow of a fluid for which the velocity varies both as a function of water

depth and breadth (i.e. $V = V(b,y)$).

Uniform Flow - Flow in which the energy line, the longitudinal water surface profile and the channel bed are all parallel.

Velocity Head - The head measured in terms of the liquid flowing and equal to the square of the mean velocity divided by twice the acceleration due to gravity.

Weir - An overflow structure which may be used for controlling upstream surface level or measuring discharge.

SYMBOLS

A - cross-sectional area at a given section,

B - channel breadth at the water's surface,

C - a measure of the vertical discontinuity in a compound section,

C_f - dimensionless coefficient of discharge,

E - the total head at any point measured relative to some fixed datum,

G - acceleration due to gravity,

H - total head at the upstream section measured relative to the channel invert,

H_{cr} - total energy at the upstream section measured relative to the lowest crest,

Q - measured discharge from a control section,

$\bar{V}$	- mean velocity of flow for a given discharge and cross-section,
WL	- water level at the gauging point upstream from the control,
y	- measured water depth,
Z	- channel bottom measurement relative to a fixed datum,
α	- Coriolis coefficient,
β	- coefficient of curvature,
z	- height of the lowest weir crest above the channel invert.

APPENDIX B

WEIR SPECIFICATIONS FOR FIELD APPLICATION

The following recommended weir specifications are based on the results obtained during the course of the experimental work plus the information obtained from various literary sources as discussed earlier. Since this type of section has never been used in practice the recommendations are necessarily on the conservative side.

WEIR POSITIONING

The weir block should be located in a straight reach of channel which contains no local obstructions with an even bedslope. The channel should be reasonably straight for a sufficient distance upstream in order that a normal velocity distribution be developed at all discharges. It is essential that the channel section be as uniform as possible with a regular and preferably small roughness coefficient. This will keep the value of the velocity distribution coefficient close to unity and may therefore make any correction unnecessary.

The weir should be symmetrical with respect to the approach channel with the lowest subsection in the center of the control. It would also be advantageous to choose a symmetrical channel reach.

WEIR GEOMETRY

For the results of the laboratory research to be applicable the control should have an upstream slope of 1 (vertical) to 3.5 (horizontal) and a downstream slope of 1 (vertical) to 4 (horizontal). The intersection of these slopes should form a straight line crest for each subsection. The crest of each subsection must also form a horizontal straight line across the channel constriction. The vertical distance between subsections and horizontal crests intersects at 90 degrees.

In order to maintain the discharge-relation characteristics as developed in the report, the control section must not be truncated beyond the acceptable limits. Upstream and downstream lengths should be not less $1.5H_{cr}$ for the 1 to 3.5 slope and $2.0H_{cr}$ for the 1 to 4 slope respectively, where H_{cr} is the maximum head over the lowest crest for modular flow. The elevation of upstream truncation must not be greater than the crest height of the adjacent lower section.

For operation in the modular flow range it is necessary to set the lowest crest level in order that the normal depth downstream will not cause a drowned flow condition to occur at the lowest section.

CONSTRUCTION SPECIFICATIONS

Concrete is probably the only acceptable material which can be used for the control block construction. It should have a smooth cement finish or be covered with a smooth non-corrodible material. The crest and the sloping crests should possess a durable well defined corner to prevent undue wear because of the abrasive nature of critical flow. The crests could possibly be made of precast concrete however, care must be taken to achieve proper alignment and a suitable sealant applied to prevent leakage between subsections.

The choices for the number of subsections and the size of each are obviously two design variables that must essentially be left to the discretion of the design engineer in charge of implementation. These variables obviously depend on the variability and range of discharge to be gauged and the physical dimensions of the proposed channel. It is felt however, that a three part section as shown in Figure 2.3 with equal height outer subsections would be applicable for most cases. The width and elevation of each subsection will of course depend on the discharge. The lower section should have suitable capacity to take the total stream flow independently for low discharge periods. It is suggested that approximately five percent of the maximum discharge be carried by this section. This would of course imply that this center section would carry the entire flow for the majority of the time.

FIELD MEASUREMENTS AND THEIR ACCURACY

As discussed earlier the stage should be measured far enough upstream to be free from drawdown effects caused by the control section and yet close enough to the control that any losses due to friction along the channel may be neglected. A reasonable distance for this measurement upstream from the control is $10H_{cr}$. This reading would most probably be taken using a continual recorder since the variation in stage relates directly to the variation in discharge thus giving continual discharge records.

High Froude numbers in the approach channel may indicate the presence of surface waves that will affect the head level reading due to the non-hydrostatic pressure distribution in the flow. It is, therefore, recommended that the channel approach Froude number be less than 0.5.

In laboratory tests the Coriolis coefficient was taken as unity. If this is not the case for the prototype model the upstream head will be underestimated in turn making the discharge less than the actual. Since this coefficient depends on both channel properties and discharge, no single value can be assigned to it at this time.

The section geometry at the control should be measured within tolerable limits and the upstream water level must be referenced with the lowest crest of the control.

With the estimated errors for the above measurements a similar plot to that shown in Figure 2.2 can be drawn for any section thus giving the error in discharge expected for the calculated H_{cr}. In practice it would not be reasonable to use flow heads of less than 0.1 foot since the errors in head measurements are critical at this low flow.

DISCHARGE EQUATION

The basic discharge equation for modular flow for the compound weir is given by the following equation

$$Q = \sum_{i=1}^{n} (0.544\, GC_{f_i}\, B_i H_{cr_i}^{1.5}) - \sum_{j=1}^{n-1} Q_{lt_j}$$

where n = number of subsections, thus (n-1) discontinuities,

Q = total measured discharge,

B = individual section breadths,

H_{cr} = upstream total head minus the section height,

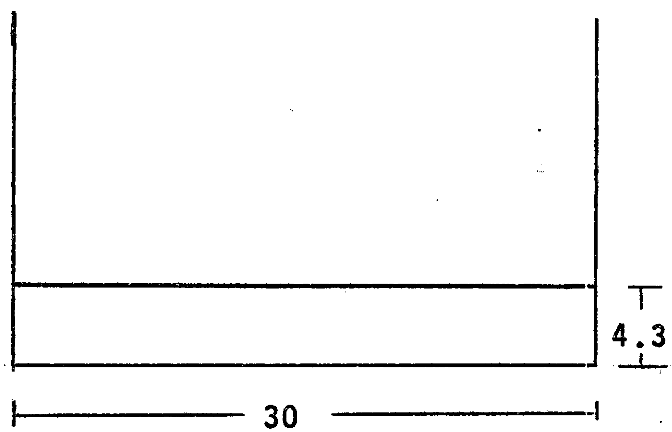
C_f = dimensionless discharge coefficient,

Q_{lt} = lost in discharge due to section elevation changes.

Since the equation is iterative (i.e. discharge must be known to find H_{cr}) a computer solution can best be used to develop a stage versus discharge plot (see Figure 4.1) which can be used directly to find discharge. The use of a computer solution also yields the added advantage of rapid calculation, therefore allowing several different shapes to be studied before a design is finalized. The dimensionless discharge coefficient is given by $C_f = 1.293\, H_{cr}^{0.0825}$ while the specifications for Q_{lt} are represented graphically in Figure 3.10.

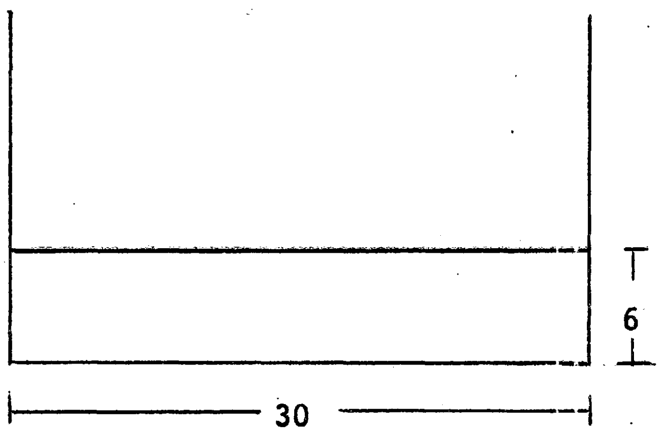
APPENDIX C - TEST RESULTS

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 PERCENTAGE ERROR IN Q1
.435	.432	.135	.137	.810
.542	.531	.153	.156	2.123
.609	.598	.165	.168	1.819
.658	.648	.173	.177	1.524
.719	.721	.185	.189	.378
.795	.798	.197	.202	.388
.853	.854	.205	.211	.184
.926	.936	.217	.223	1.049
.973	.990	.224	.231	1.760
1.025	1.033	.230	.238	.871
1.094	1.096	.238	.247	.250
1.190	1.201	.252	.261	.919
1.266	1.288	.262	.273	1.797
1.393	1.407	.277	.289	1.028
1.475	1.487	.286	.299	.753
1.567	1.560	.294	.308	.419
1.897	1.897	.330	.349	.020
2.076	2.089	.349	.371	.645



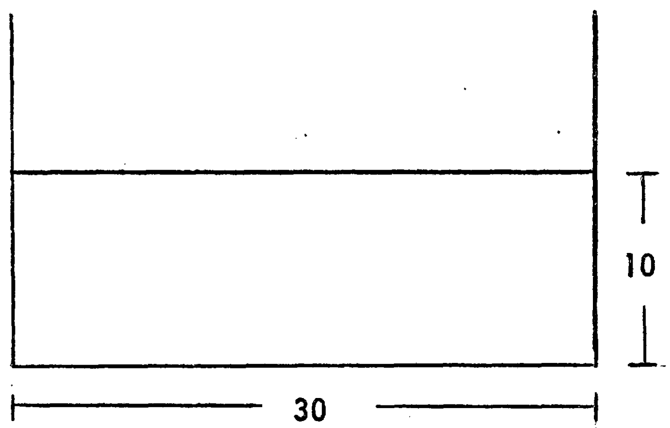
C-1

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 PERCENTAGE ERROR IN Q1
.223	.221	.089	.090	.770
.460	.459	.140	.142	.321
.670	.659	.175	.179	1.526
.893	.902	.212	.218	1.019
1.116	1.105	.239	.248	1.010
1.339	1.339	.268	.280	.009
1.562	1.564	.294	.309	.104
1.786	1.789	.318	.336	.158
2.042	2.044	.344	.366	.066



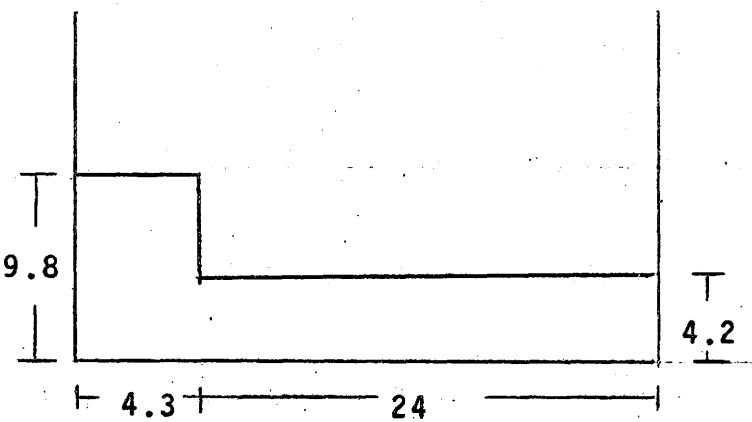
C-2

1 Q GIVEN	2. Q1 DISCHAR	3 WL	4 ECR	5 PERCENTAGE ERROR IN Q1
.357	.369	.122	.124	3.362
.558	.560	.158	.161	.320
.625	.624	.169	.173	.206
.725	.738	.188	.192	1.770
.848	.844	.203	.209	.523
1.004	.998	.225	.232	.669
1.071	1.066	.234	.242	.486
1.172	1.176	.248	.258	.347
1.272	1.290	.262	.273	1.419
1.417	1.424	.278	.291	.453
1.518	1.526	.290	.304	.532
1.634	1.655	.304	.320	1.260
1.741	1.764	.316	.333	1.305
1.837	1.868	.327	.346	1.693
1.953	1.967	.337	.357	.709
2.031	2.043	.344	.366	.561
2.411	2.395	.378	.404	.639



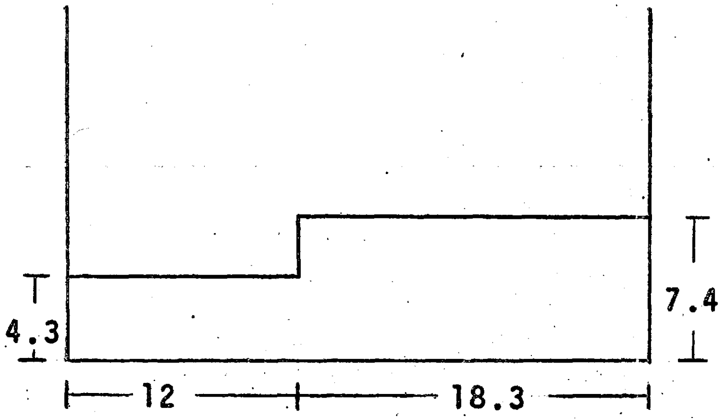
C-3

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.513	.528	.177	.180	.015	.002	.526	2.422
.690	.703	.212	.215	.013	.004	.699	1.334
.770	.771	.224	.228	.001	.004	.767	-.406
.875	.897	.246	.251	.022	.006	.891	1.794
.991	1.077	.275	.282	.086	.009	1.068	7.810
1.103	1.170	.289	.297	.067	.011	1.159	5.127
1.243	1.306	.309	.319	.063	.014	1.292	3.913
1.317	1.388	.321	.331	.071	.016	1.372	4.163
1.458	1.528	.340	.352	.070	.021	1.507	3.358
1.589	1.660	.357	.371	.071	.027	1.633	2.761
1.714	1.811	.377	.392	.096	.035	1.775	3.562
1.808	1.925	.391	.407	.117	.043	1.882	4.116
1.960	2.092	.411	.429	.132	.056	2.036	3.881
2.045	2.149	.417	.436	.104	.062	2.087	2.093



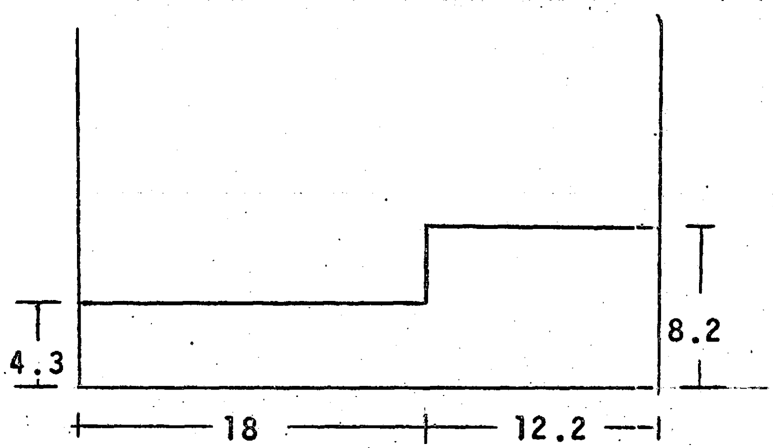
C-4

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.335	.366	.223	.224	.031	.014	.352	5.261
.446	.465	.258	.260	.019	.027	.439	1.717
.558	.586	.292	.294	.028	.034	.552	1.077
.681	.726	.321	.324	.045	.034	.692	1.648
.786	.836	.341	.344	.050	.034	.802	2.064
.904	.966	.363	.367	.062	.034	.933	3.155
1.007	1.075	.379	.384	.068	.034	1.041	3.382
1.132	1.189	.396	.402	.058	.034	1.155	2.098
1.339	1.412	.426	.434	.072	.034	1.378	2.883
1.460	1.530	.441	.450	.071	.034	1.497	2.518
1.681	1.753	.468	.479	.073	.034	1.720	2.312
1.797	1.871	.481	.493	.074	.034	1.837	2.229
1.908	2.000	.495	.509	.091	.034	1.966	2.998
2.031	2.117	.508	.522	.085	.034	2.083	2.541



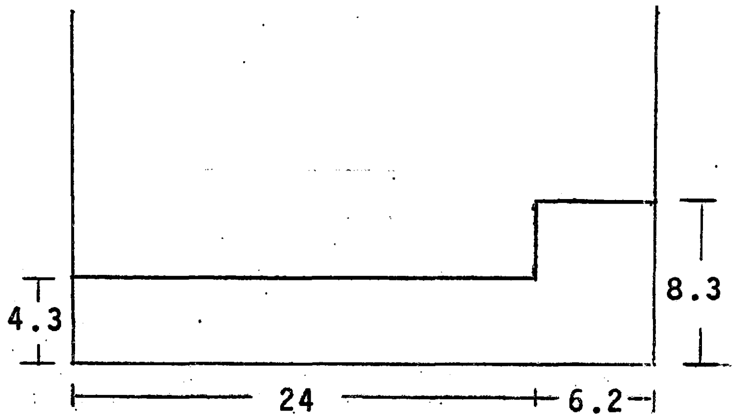
C-5

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.223	.228	.127	.128	.005	.002	.226	1.349
.335	.356	.168	.169	.022	.003	.353	5.483
.469	.480	.202	.204	.011	.006	.474	1.202
.558	.566	.224	.227	.008	.008	.557	.098
.665	.675	.250	.253	.010	.013	.662	.466
.781	.799	.277	.281	.018	.020	.779	.315
.911	.944	.307	.313	.033	.034	.910	.050
1.002	1.063	.331	.337	.061	.050	1.013	1.101
1.118	1.193	.353	.360	.075	.050	1.143	2.171
1.219	1.297	.368	.376	.078	.050	1.246	2.269
1.339	1.438	.388	.398	.098	.050	1.387	3.586
1.451	1.563	.405	.415	.112	.050	1.513	4.247
1.578	1.681	.420	.432	.103	.050	1.631	3.348
1.672	1.784	.433	.445	.112	.050	1.734	3.700
1.795	1.905	.447	.461	.110	.050	1.855	3.340
1.895	2.015	.459	.474	.120	.050	1.965	3.690
2.031	2.145	.473	.490	.113	.050	2.094	3.111

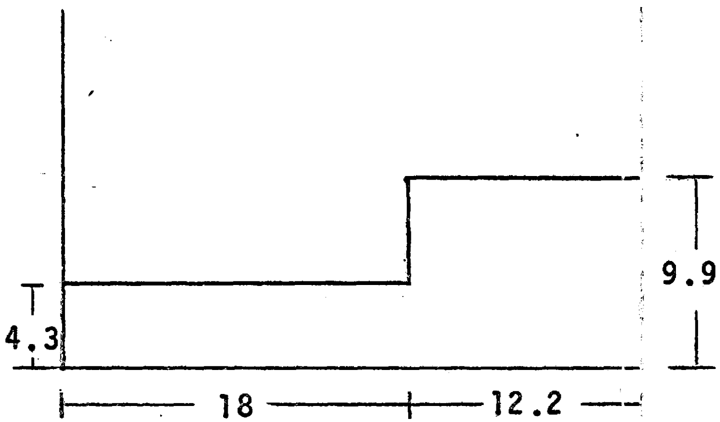


C-6

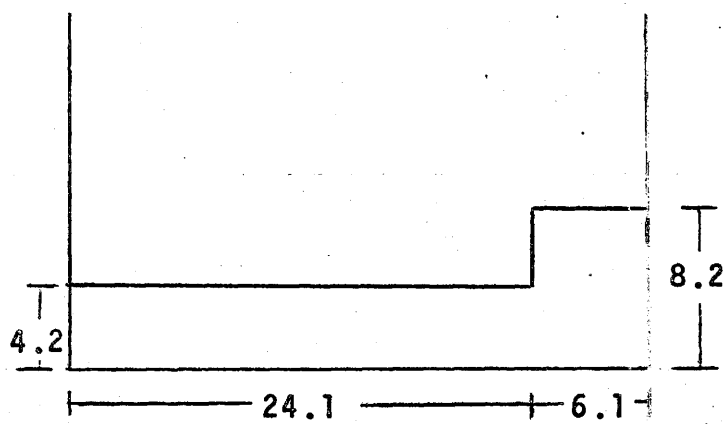
1 GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.246	.244	.110	.110	-.002	.001	.242	-1.303
.446	.460	.163	.165	.014	.003	.457	2.450
.562	.567	.185	.188	.005	.004	.563	-.085
.658	.674	.206	.210	.016	.006	.668	-1.447
.781	.793	.228	.232	.012	.009	.784	.376
.904	.920	.250	.255	.016	.013	.907	.319
1.009	1.033	.268	.275	.024	.017	1.015	.649
1.123	1.151	.286	.294	.028	.023	1.128	.442
1.232	1.263	.303	.312	.031	.031	1.232	.028
1.344	1.392	.321	.332	.048	.042	1.350	.460
1.451	1.523	.339	.351	.072	.051	1.472	-1.474
1.565	1.636	.353	.366	.071	.051	1.585	1.326
1.685	1.783	.370	.385	.098	.051	1.733	2.813
1.786	1.893	.383	.399	.107	.051	1.843	3.181
1.904	2.023	.397	.415	.119	.051	1.972	3.566
2.031	2.149	.410	.430	.117	.051	2.098	3.281



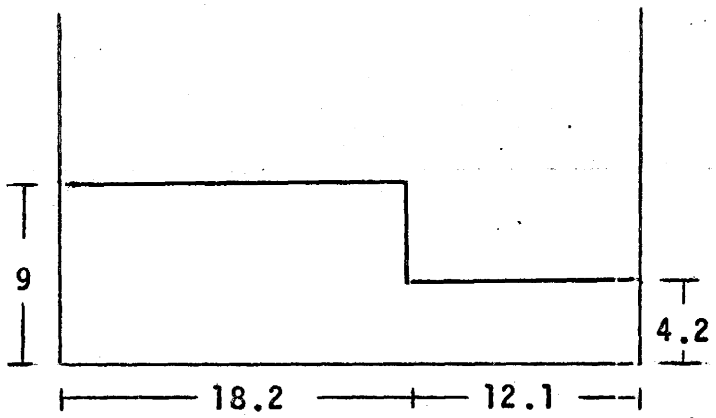
1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.335	.351	.166	.167	.017	.002	.349	4.368
.460	.471	.199	.201	.012	.003	.468	1.838
.558	.567	.223	.225	.009	.004	.562	.764
.687	.701	.255	.258	.014	.007	.695	1.062
.783	.808	.278	.282	.024	.009	.799	1.940
.900	.932	.304	.309	.033	.013	.919	2.205
1.020	1.072	.331	.337	.052	.019	1.054	3.288
1.127	1.184	.352	.359	.056	.025	1.159	2.789
1.223	1.285	.371	.378	.062	.032	1.253	2.433
1.344	1.419	.394	.403	.075	.044	1.375	2.312
1.467	1.573	.420	.430	.107	.063	1.510	2.987
1.554	1.681	.437	.448	.128	.080	1.601	3.073
1.690	1.846	.463	.476	.157	.114	1.732	2.501
1.806	2.001	.484	.498	.195	.122	1.879	4.080
1.904	2.066	.492	.507	.162	.122	1.944	2.125
2.020	2.209	.510	.526	.189	.122	2.087	3.329



1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.491	.491	.169	.171	-.000	.003	.487	.777
.652	.656	.203	.206	.005	.006	.651	.194
.725	.737	.218	.222	.012	.007	.730	.579
.833	.846	.237	.242	.013	.010	.836	.351
.949	.976	.258	.265	.027	.015	.961	1.316
1.074	1.097	.277	.285	.024	.020	1.077	.322
1.170	1.185	.291	.299	.015	.025	1.160	.823
1.281	1.322	.311	.321	.041	.035	1.287	.464
1.406	1.459	.330	.341	.053	.048	1.411	.357
1.511	1.590	.347	.359	.079	.051	1.540	1.877
1.634	1.707	.361	.375	.073	.051	1.656	1.354
1.746	1.821	.374	.390	.076	.051	1.770	1.427
1.846	1.970	.391	.408	.124	.051	1.920	3.990
1.951	2.064	.401	.419	.113	.051	2.013	3.169
2.042	2.159	.411	.430	.116	.051	2.108	3.208

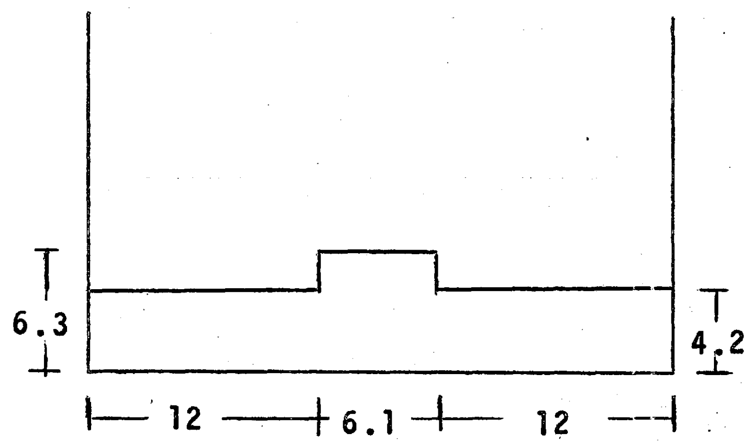


1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.364	.387	.227	.228	.024	.006	.382	4.882
.587	.624	.306	.308	.037	.019	.606	3.143
.663	.710	.332	.335	.047	.027	.683	3.023
.712	.762	.347	.350	.050	.034	.728	2.259
.759	.816	.362	.365	.057	.043	.773	1.851
.835	.911	.388	.392	.076	.063	.848	1.594
.891	1.008	.412	.416	.118	.081	.927	4.081
1.025	1.169	.443	.448	.144	.081	1.088	6.161
1.116	1.268	.460	.466	.152	.081	1.187	6.352
1.228	1.375	.476	.483	.147	.081	1.294	5.382
1.350	1.512	.496	.504	.162	.081	1.431	5.984
1.462	1.641	.514	.523	.178	.081	1.559	6.657
1.585	1.756	.529	.539	.171	.081	1.675	5.679
1.687	1.876	.544	.555	.189	.081	1.795	6.389
1.781	1.988	.557	.569	.207	.081	1.907	7.037

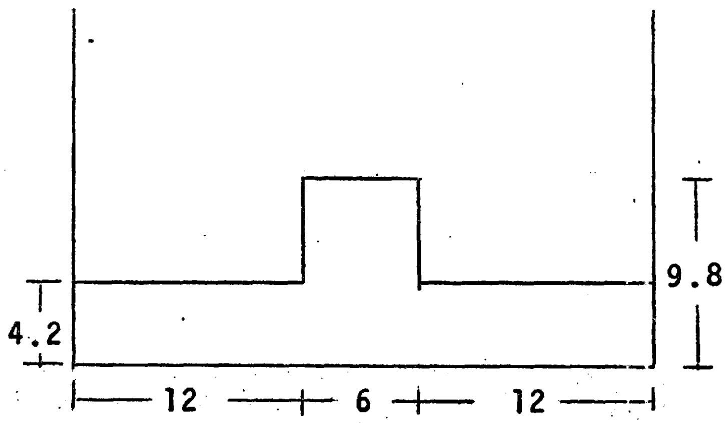


C-10

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.319	.329	.133	.134	.009	.012	.317	.736
.424	.450	.162	.164	.026	.025	.425	.246
.513	.562	.185	.188	.049	.036	.526	2.423
.670	.701	.210	.214	.031	.036	.665	.726
.781	.819	.229	.234	.037	.036	.783	.264
.911	.963	.251	.257	.052	.036	.927	1.755
1.016	1.069	.266	.273	.054	.036	1.033	1.721
1.116	1.175	.280	.289	.059	.036	1.139	2.010
1.232	1.278	.293	.303	.046	.036	1.242	.799
1.339	1.400	.308	.320	.060	.036	1.363	1.797
1.451	1.519	.322	.335	.068	.036	1.483	2.195
1.578	1.628	.335	.349	.050	.036	1.592	.874
1.679	1.741	.348	.363	.062	.036	1.705	1.555
1.797	1.834	.357	.374	.037	.036	1.798	.050
1.904	1.961	.371	.389	.057	.036	1.925	1.110
2.009	2.060	.381	.401	.051	.036	2.023	.720

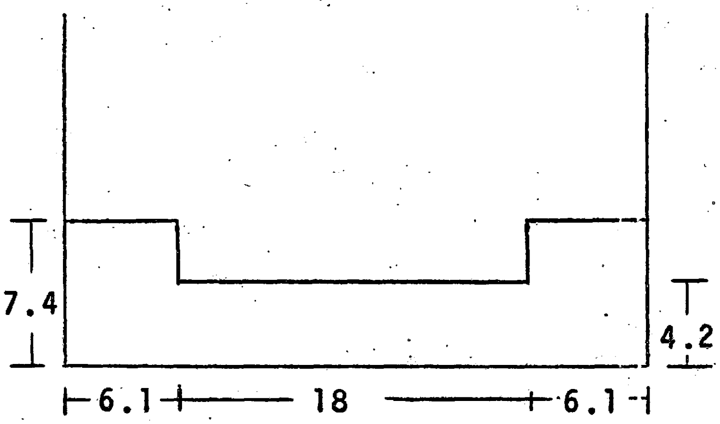


1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.446	.450	.161	.163	.004	.004	.447	.060
.547	.553	.183	.185	.006	.005	.548	.170
.681	.681	.207	.211	.000	.007	.674	.987
.781	.786	.227	.231	.005	.009	.778	.475
.902	.913	.248	.254	.011	.012	.901	.081
1.009	1.037	.268	.275	.028	.016	1.021	-1.195
1.125	1.150	.286	.294	.025	.020	1.130	.461
1.239	1.274	.304	.313	.036	.026	1.249	.782
1.342	1.380	.319	.332	.038	.032	1.348	.488
1.462	1.495	.335	.347	.033	.039	1.455	.452
1.571	1.601	.349	.362	.030	.048	1.553	-1.159
1.685	1.737	.367	.381	.051	.061	1.676	.569
1.788	1.856	.382	.397	.069	.075	1.782	.357
1.893	1.994	.398	.416	.101	.094	1.899	.346
2.036	2.164	.418	.438	.128	.124	2.040	.194

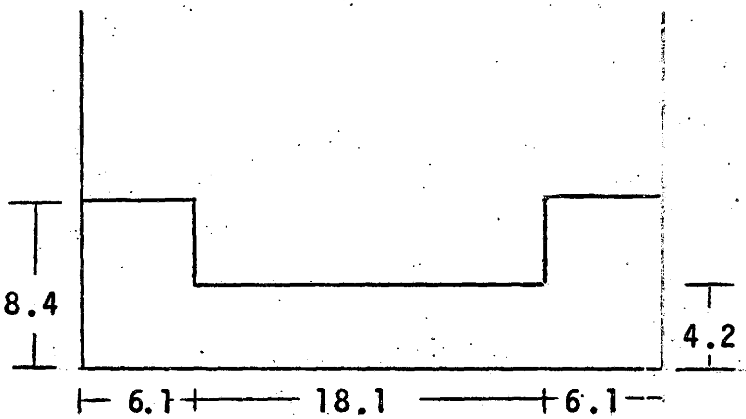


C-12

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.335	.342	.163	.164	.007	.009	.333	-.544
.435	.442	.191	.193	.006	.015	.427	-1.890
.547	.562	.222	.224	.015	.026	.535	2.111
.625	.663	.246	.249	.038	.042	.622	-.524
.770	.831	.282	.286	.061	.068	.762	1.025
.893	.953	.303	.309	.060	.068	.885	.897
1.000	1.073	.323	.329	.073	.068	1.005	.474
1.121	1.180	.338	.346	.059	.068	1.111	-.835
1.228	1.280	.353	.361	.052	.068	1.212	1.304
1.346	1.405	.369	.379	.059	.068	1.336	.735
1.458	1.522	.384	.395	.065	.068	1.454	.269
1.567	1.638	.399	.410	.071	.068	1.569	.162
1.685	1.751	.412	.425	.066	.068	1.683	.153
1.786	1.861	.424	.439	.075	.068	1.792	.378
1.897	1.967	.436	.451	.069	.068	1.898	.041
2.009	2.083	.448	.465	.074	.068	2.015	.284

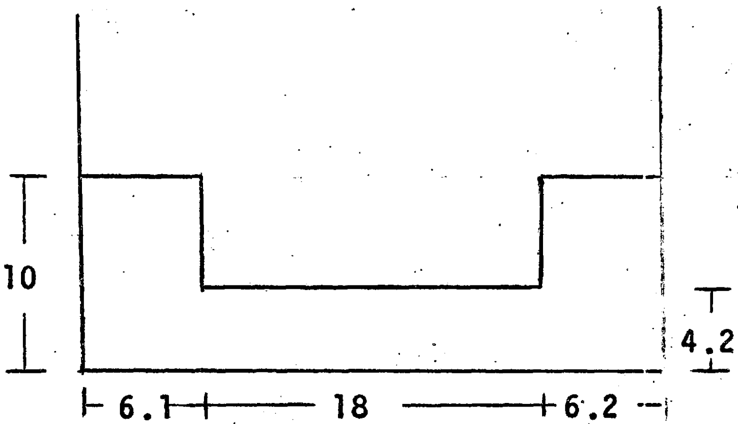


1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST G1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.446	.450	.192	.194	.004	.009	.441	-1.210
.580	.585	.227	.229	.005	.016	.569	1.951
.658	.678	.248	.251	.020	.023	.655	.584
.759	.811	.277	.282	.052	.038	.773	-1.868
.893	.945	.305	.310	.052	.059	.885	-.851
1.004	1.082	.332	.338	.077	.092	.990	1.453
1.116	1.218	.355	.362	.102	.107	1.111	.453
1.239	1.338	.373	.382	.099	.107	1.231	.598
1.344	1.450	.389	.399	.106	.107	1.343	.049
1.446	1.555	.403	.414	.109	.107	1.449	.148
1.562	1.666	.418	.429	.103	.107	1.559	.219
1.674	1.788	.433	.445	.114	.107	1.681	-.434
1.779	1.908	.447	.461	.129	.107	1.802	1.278
1.902	2.019	.459	.474	.117	.107	1.912	.529
2.013	2.132	.472	.488	.119	.107	2.026	.602



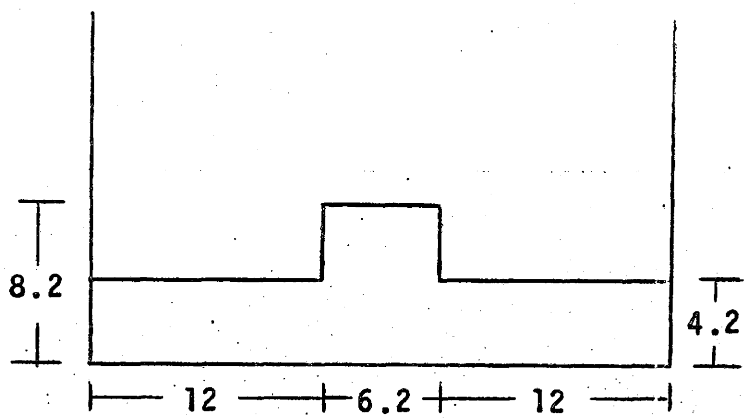
C-14

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.446	.450	.193	.195	.004	.006	.445	-.366
.569	.588	.228	.231	.019	.009	.579	1.768
.670	.684	.251	.254	.015	.012	.672	.359
.790	.809	.278	.282	.019	.018	.791	.150
.904	.938	.305	.310	.034	.025	.912	.931
1.013	1.057	.328	.334	.043	.035	1.022	.815
1.132	1.176	.351	.358	.044	.047	1.129	.252
1.232	1.287	.371	.379	.055	.062	1.225	.610
1.348	1.421	.394	.403	.073	.085	1.335	.946
1.429	1.550	.416	.426	.121	.115	1.435	.468
1.540	1.704	.441	.452	.164	.161	1.543	.198
1.6 9	1.854	.464	.477	.224	.221	1.632	.178
1.763	2.004	.485	.499	.241	.255	1.750	.778
1.906	2.148	.503	.519	.242	.255	1.894	.663
2.047	2.288	.520	.537	.241	.255	2.033	.692



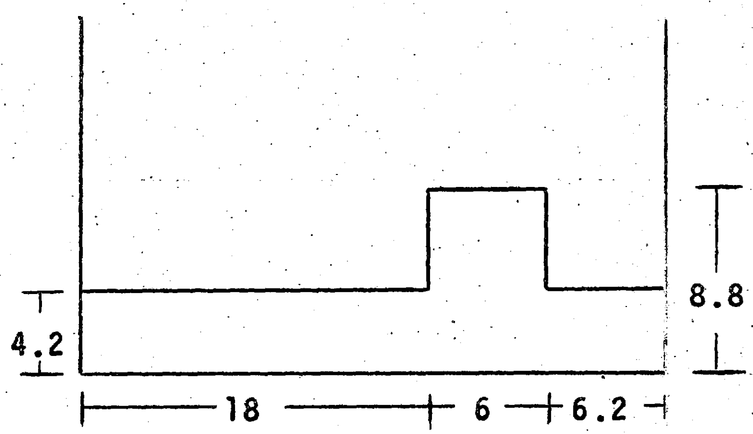
C-15

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.357	.364	.141	.142	.007	.004	.360	.671
.458	.466	.164	.166	.009	.006	.460	.528
.556	.562	.184	.187	.006	.009	.553	.467
.681	.691	.209	.213	.013	.013	.678	.393
.781	.793	.227	.232	.011	.018	.775	.788
.913	.935	.252	.258	.022	.026	.909	.453
1.022	1.066	.273	.280	.043	.037	1.028	.597
1.125	1.164	.287	.296	.039	.048	1.116	.790
1.241	1.289	.306	.315	.048	.065	1.224	-1.379
1.348	1.425	.325	.336	.077	.089	1.336	.923
1.440	1.528	.338	.350	.088	.100	1.428	.847
1.569	1.648	.353	.367	.079	.100	1.548	-1.326
1.679	1.782	.369	.384	.104	.100	1.682	.226
1.799	1.908	.383	.400	.108	.100	1.808	.470
1.897	2.007	.394	.412	.114	.100	1.907	.505
2.018	2.109	.405	.424	.091	.100	2.009	.447

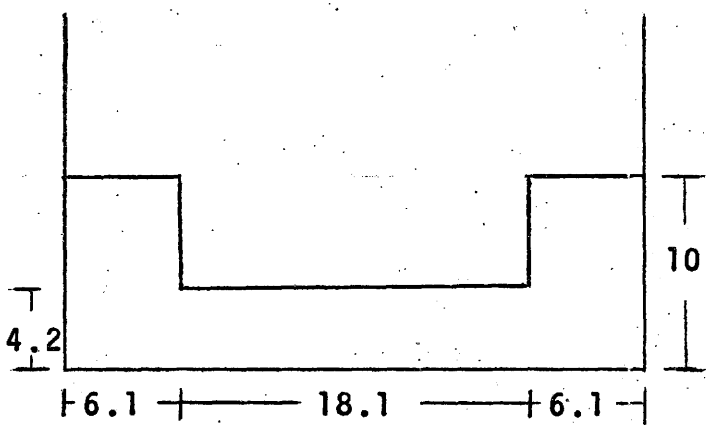


C-16

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.469	.478	.168	.170	.010	.004	.474	1.216
.565	.575	.188	.191	.010	.005	.570	-.907
.696	.715	.215	.219	.018	.007	.707	1.553
.792	.818	.233	.238	.025	.010	.808	1.987
.920	.947	.255	.261	.027	.013	.934	1.538
1.022	1.057	.272	.280	.034	.016	1.040	1.742
1.125	1.160	.288	.296	.035	.020	1.140	1.310
1.234	1.273	.305	.314	.039	.026	1.248	1.092
1.348	1.397	.322	.333	.049	.033	1.365	1.218
1.455	1.513	.338	.350	.058	.040	1.473	1.199
1.565	1.608	.351	.364	.043	.048	1.560	.326
1.687	1.750	.369	.384	.063	.062	1.689	.070
1.795	1.878	.385	.401	.083	.077	1.801	-.363
1.906	2.030	.403	.421	.124	.099	1.932	1.327
2.047	2.187	.422	.441	.141	.127	2.061	.677

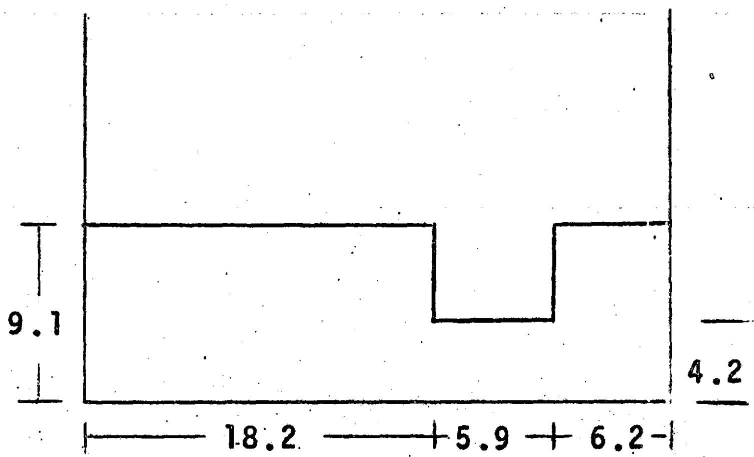


1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECP	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.487	.503	.207	.209	.016	.007	.496	1.994
.609	.625	.237	.239	.016	.010	.615	.972
.723	.750	.265	.269	.027	.015	.735	1.687
.871	.892	.295	.300	.021	.022	.870	.106
.982	1.009	.318	.324	.027	.031	.978	.422
1.054	1.104	.337	.343	.051	.039	1.065	1.077
1.161	1.222	.358	.366	.061	.053	1.169	.694
1.277	1.363	.383	.392	.086	.075	1.289	.921
1.411	1.496	.406	.415	.085	.101	1.394	1.179
1.518	1.643	.430	.441	.125	.141	1.502	1.046
1.618	1.780	.452	.464	.162	.190	1.590	1.726
1.737	1.945	.476	.489	.208	.241	1.703	1.916
1.833	2.099	.496	.511	.266	.252	1.847	.807
1.958	2.222	.511	.527	.264	.252	1.970	.636
2.054	2.300	.520	.537	.246	.252	2.048	.271



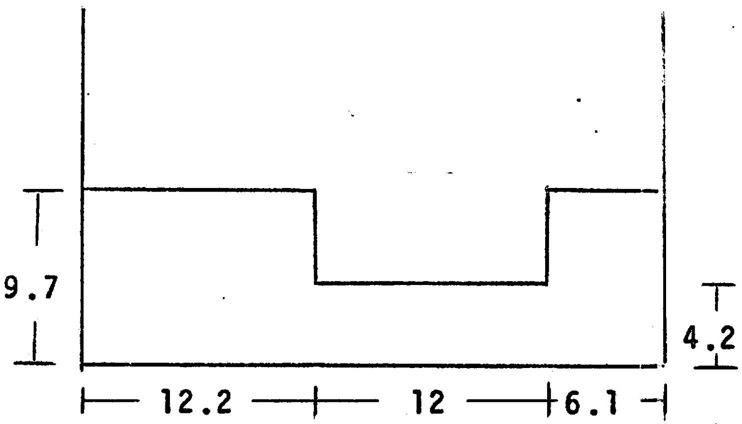
C-18

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.279	.345	.336	.336	.066	.057	.287	2.966
.391	.551	.435	.436	.160	.164	.387	.911
.502	.666	.463	.465	.163	.164	.502	.045
.6[illegible]4	.774	.486	.488	.160	.164	.610	.604
.692	.844	.499	.502	.152	.164	.680	1.659
.8[illegible]7	.998	.526	.529	.161	.164	.834	.340
.949	1.108	.543	.547	.159	.164	.944	.484
1.083	1.242	.563	.568	.159	.164	1.078	.435
1.194	1.360	.580	.585	.165	.164	1.196	.149
1.295	1.440	.591	.597	.145	.164	1.276	1.450
1.429	1.574	.608	.615	.145	.164	1.410	1.297
1.518	1.673	.621	.628	.156	.164	1.510	.537
1.741	1.911	.649	.658	.170	.164	1.747	.364
1.897	2.126	.673	.684	.229	.164	1.963	3.441



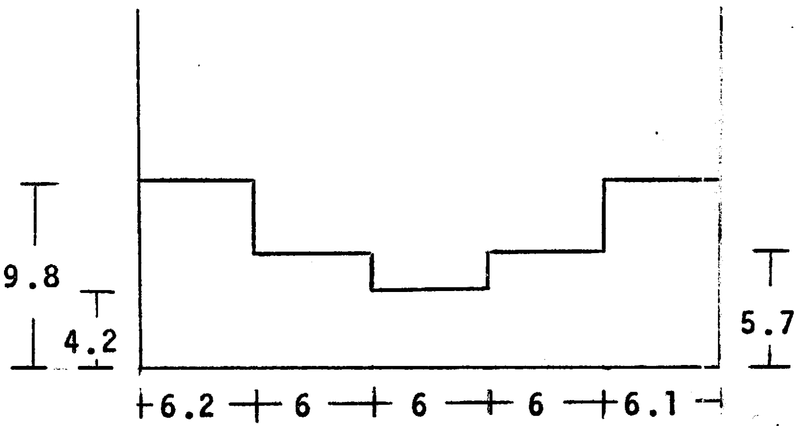
C-19

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.458	.465	.258	.260	.007	.018	.447	-2.414
.513	.536	.282	.284	.023	.026	.510	.662
.558	.592	.301	.303	.034	.034	.558	.083
.625	.678	.328	.330	.053	.050	.628	.503
.670	.729	.343	.345	.059	.063	.666	.547
.759	.857	.379	.383	.098	.108	.749	-1.245
.804	.966	.408	.412	.163	.165	.801	.270
.982	1.140	.443	.448	.158	.165	.975	.690
1.103	1.248	.462	.467	.145	.165	1.083	-1.829
1.248	1.386	.483	.490	.138	.165	1.221	2.160
1.357	1.506	.501	.509	.149	.165	1.341	1.174
1.475	1.634	.518	.527	.158	.165	1.469	.451
1.609	1.742	.533	.542	.133	.165	1.577	-1.997
1.797	1.958	.559	.571	.162	.165	1.793	.191
2.020	2.168	.583	.597	.148	.165	2.003	.862

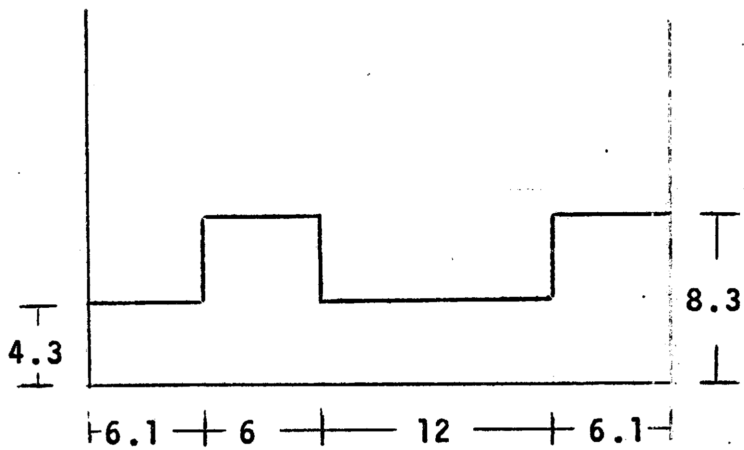


C-20

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.424	.448	.273	.274	.024	.031	.418	1.540
.558	.582	.308	.309	.024	.034	.547	1.950
.670	.703	.337	.339	.033	.039	.664	.880
.770	.811	.361	.364	.041	.046	.765	.647
.908	.949	.390	.394	.040	.058	.891	1.893
1.004	1.057	.412	.416	.053	.071	.986	1.829
1.125	1.188	.437	.442	.063	.094	1.094	2.764
1.239	1.346	.465	.472	.107	.130	1.216	1.846
1.344	1.481	.486	.493	.137	.130	1.351	.502
1.462	1.610	.504	.513	.147	.130	1.480	1.197
1.578	1.690	.515	.524	.112	.130	1.560	1.151
1.674	1.813	.531	.541	.139	.130	1.683	.515
1.875	2.004	.554	.566	.129	.130	1.874	.031
1.891	2.054	.560	.572	.164	.130	1.924	1.781
2.042	2.186	.575	.589	.144	.130	2.056	.666

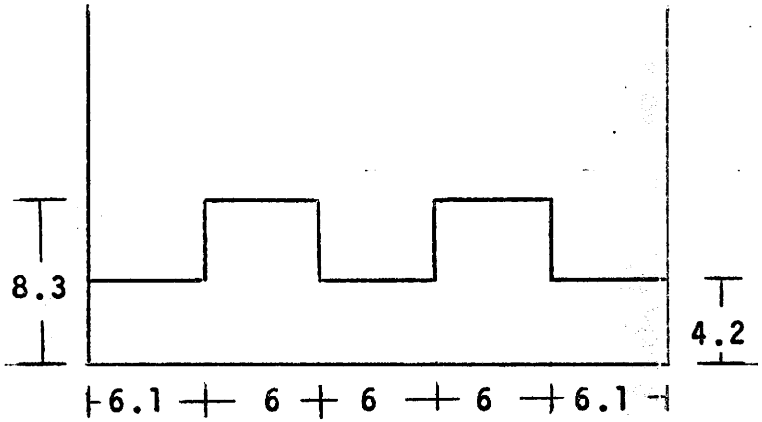


1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.478	.488	.205	.207	.010	.017	.470	1.507
.587	.612	.236	.239	.025	.028	.584	.564
.670	.695	.255	.258	.025	.039	.656	2.029
.759	.813	.281	.285	.054	.060	.753	.729
.893	.980	.315	.320	.088	.105	.875	1.955
.993	1.122	.342	.348	.129	.155	.968	2.598
1.116	1.235	.360	.367	.119	.155	1.080	3.189
1.234	1.359	.378	.387	.124	.155	1.204	2.490
1.348	1.472	.394	.404	.124	.155	1.317	2.293
1.458	1.592	.410	.421	.135	.155	1.437	1.392
1.576	1.691	.422	.434	.115	.155	1.536	2.509
1.681	1.822	.438	.451	.141	.155	1.667	.814
1.799	1.966	.455	.470	.167	.155	1.811	.665
1.946	2.086	.468	.484	.139	.155	1.931	.804
2.062	2.217	.483	.500	.155	.155	2.062	.007

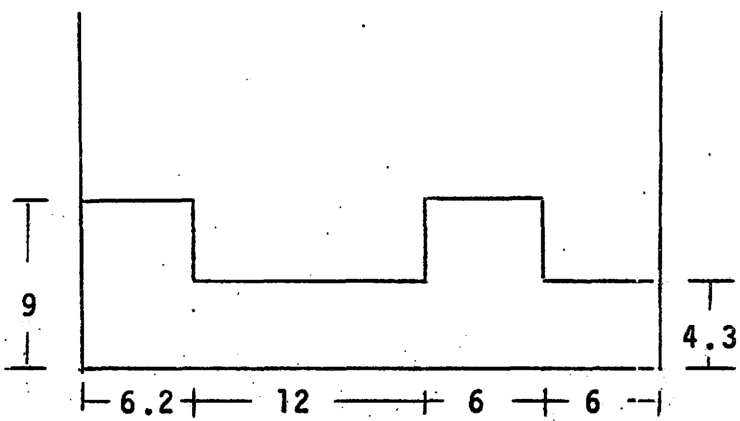


C-22

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.525	.540	.217	.219	.015	.028	.511	2.553
.6[illegible]3	.640	.241	.244	.038	.042	.598	-.764
.714	.771	.270	.274	.057	.069	.702	1.676
.799	.894	.296	.301	.095	.105	.789	1.269
.893	1.082	.332	.339	.189	.194	.888	.540
.971	1.175	.348	.355	.204	.205	.971	.043
1.116	1.316	.370	.378	.200	.205	1.112	-.394
1.257	1.434	.387	.396	.177	.205	1.229	2.194
1.388	1.565	.404	.415	.176	.205	1.360	2.035
1.473	1.683	.419	.431	.210	.205	1.478	-.354
1.627	1.793	.433	.445	.165	.205	1.588	2.406
1.734	1.914	.447	.461	.179	.205	1.709	1.455
1.848	2.054	.462	.478	.206	.205	1.850	.090
1.908	2.108	.468	.485	.199	.205	1.903	-.279
2.045	2.217	.480	.497	.172	.205	2.012	1.595

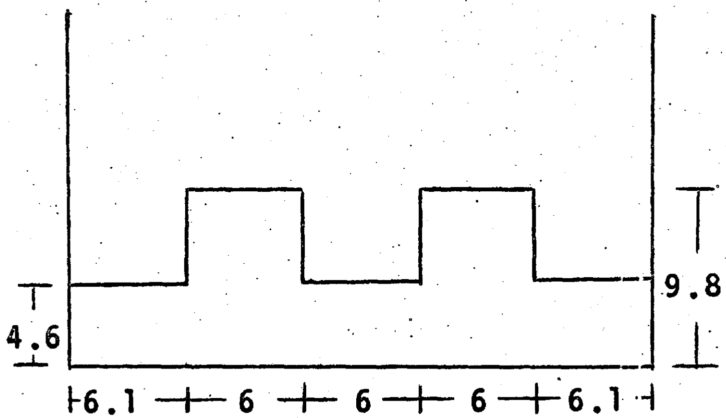


1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.536	.543	.220	.222	.007	.014	.529	-1.271
.725	.769	.273	.276	.044	.029	.740	1.998
1.004	1.047	.329	.335	.043	.067	.981	2.380
1.172	1.296	.375	.383	.124	.130	1.166	.481
1.406	1.619	.426	.436	.213	.203	1.416	.685
1.674	1.948	.471	.485	.274	.302	1.646	-1.650
1.786	2.106	.490	.505	.321	.302	1.804	1.046
1.931	2.266	.508	.525	.335	.302	1.964	1.735
2.076	2.380	.521	.539	.304	.302	2.078	.104



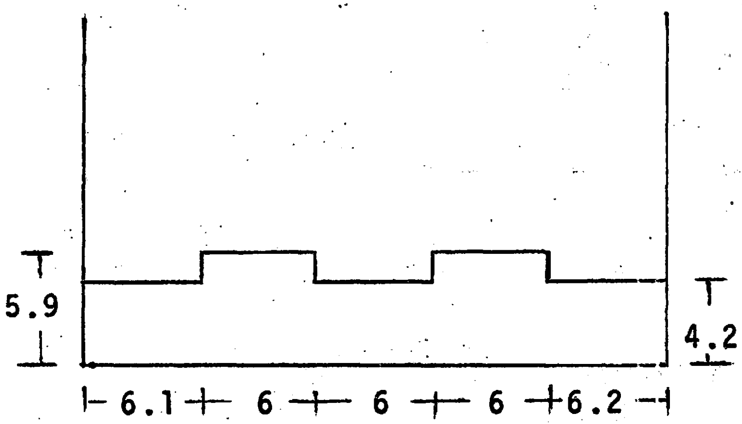
C-24

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.480	.497	.207	.209	.017	.016	.481	.211
.665	.703	.257	.260	.038	.033	.670	.692
.893	.986	.316	.321	.093	.079	.907	1.551
1.080	1.217	.359	.366	.154	.151	1.066	.516
1.286	1.529	.411	.421	.243	.246	1.283	.177
1.496	1.814	.452	.464	.318	.330	1.484	.799
1.786	2.156	.494	.510	.370	.364	1.792	.354
1.853	2.222	.502	.518	.369	.364	1.858	.261
1.953	2.319	.512	.530	.366	.364	1.955	.071
2.009	2.395	.521	.539	.386	.364	2.031	1.101



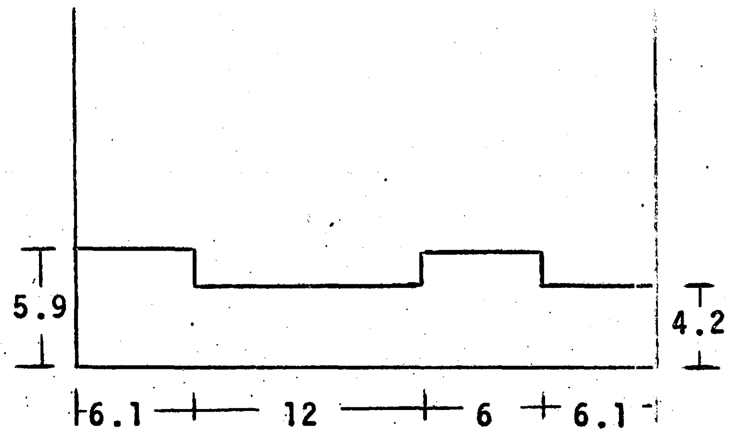
C-25

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.480	.533	.201	.203	.053	.056	.476	.766
.670	.731	.236	.240	.061	.056	.674	.716
.911	.953	.270	.276	.042	.056	.896	-1.595
1.156	1.182	.302	.310	.026	.056	1.126	2.621
1.357	1.386	.328	.338	.029	.056	1.330	1.991
1.585	1.606	.353	.366	.021	.056	1.549	2.246
1.786	1.832	.378	.394	.046	.056	1.775	.579
2.031	2.064	.403	.421	.033	.056	2.008	-1.166



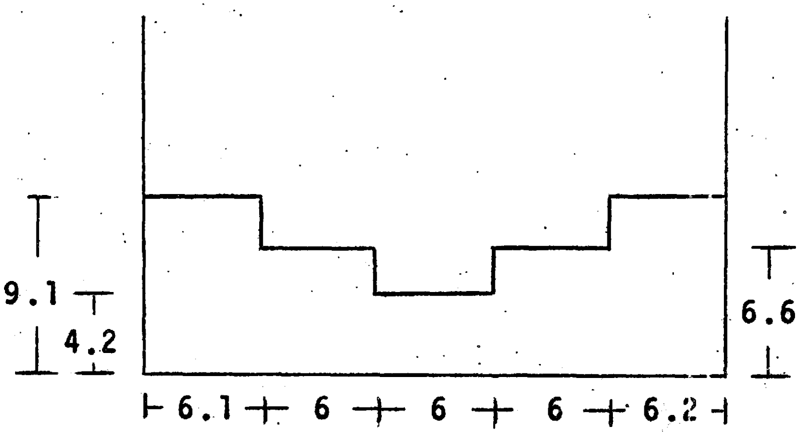
C-26

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
2.020	2.050	.403	.421	.030	.043	2.007	.644
1.770	1.826	.379	.394	.056	.043	1.783	.756
1.507	1.557	.349	.361	.050	.043	1.514	.499
1.315	1.348	.324	.334	.033	.043	1.305	.728
1.080	1.110	.293	.301	.029	.043	1.067	1.269
.871	.904	.264	.269	.033	.043	.861	1.106
.670	.703	.233	.236	.033	.043	.660	1.426
.469	.501	.196	.198	.032	.043	.458	2.277



C-27

1 Q GIVEN	2 Q1 DISCHAR	3 WL	4 ECR	5 QLOST Q1 - Q	6 QLOST CALCULATED	7 Q2 Q1 - QLOST	8 PERCENTAGE ERROR IN Q2
.346	.388	.300	.301	.042	.048	.340	1.707
.469	.514	.337	.338	.045	.052	.461	1.568
.558	.612	.363	.364	.054	.058	.554	.738
.636	.704	.385	.387	.068	.067	.637	.148
.781	.860	.420	.423	.079	.090	.770	1.405
.897	.991	.444	.448	.093	.090	.900	.331
1.016	1.111	.464	.469	.096	.090	1.021	.557
1.121	1.209	.479	.484	.088	.090	1.119	.156
1.239	1.336	.497	.504	.097	.090	1.245	.523
1.357	1.439	.512	.519	.081	.090	1.348	.644
1.458	1.566	.528	.536	.108	.090	1.476	1.233
1.578	1.665	.541	.549	.087	.090	1.575	.208
1.681	1.775	.554	.564	.094	.090	1.684	.217
1.797	1.902	.569	.580	.106	.090	1.812	.856
1.897	2.005	.581	.592	.108	.090	1.915	.928
2.020	2.126	.594	.607	.106	.090	2.036	.774



APPENDIX D - FORTRAN LISTING OF SUBROUTINES

```
      SUBROUTINE CORLIS(V,XC,YC,II,JJ,VELCOF,AVEL,TAREA)
C
C       THE ROUTINE FINDS THE CORIOLIS COEFFICIENT VELCOF, AVERAGE CHANNEL
C     VELOCITY AVEL, AND TOTAL AREA OF FLOW TAREA, FOR ANY RECTANGULAR SECTION.
C     V IS A TWO DIMENSIONAL ARRAY OF SIZE II BY JJ, WHICH CONTAINS THE
C     MEASURED VELOCITIES OF FLOW. THE DISTANCE BETWEEN EACH ROW
C     AND COLUMN OF THE VELOCITY GRID ARE STORED IN THE RESPEVTIVE ARRAYS YC(JJ)
C     AND XC(II).
C
      DIMENSION V(II,JJ),XC(II),YC(JJ)
      III=II-1
      JJJ=JJ-1
      X=0.0
      DO 10 I=1,III
      X=X+XC(I)
   10 CONTINUE
      Y=0.0
      DO 20 J=1,JJJ
      Y= Y+ YC(J)
   20 CONTINUE
      TAREA=X*Y
      SUM=AVEL=0.0
      DO 30 I=1,III
      DO 30 J=1,JJJ
      X=XC(I)
      Y=YC(J)
      AREA=X*Y
      VEL=(V(I,J) + V(I+1,J) + V(I,J+1) + V(I+1,J+1))/4.0
      SUM=(VEL**3)*(AREA) + SUM
      AVEL=AVEL+VEL*(AREA/TAREA)
   30 CONTINUE
      TSUM=((AVEL)**3)*TAREA
      VELCOF=SUM/TSUM
      WRITE(6,44) AVEL,TAREA,VELCOF
   44 FORMAT(///,36H FOR SECTION WITH AVERAGE VELOCITY =,F10.2,3X,
     1    11H AND AREA =,F10.2,///3X,23H VELOCITY COEFFICIENT =,F10.4)
      RETURN
      END
```

```
      SUBROUTINE DISCHAR(B,H,NPTS,WL1,B1,QCR,ACR,ECR)
C
C       THE GEOMETRY OF THE DOWNSTREAM CRITICAL SECTION IS GIVEN BY THE ARRAYS
C     B,H(1,NPTS). THE WATER LEVEL WL1 AND THE CHANNEL BREADTH B1 IS GIVEN
C     FOR THE UPSTREAM SECTION. ASSUMING TWO-DIMENSIONAL FLOW CHARACTERISTICS
C     THE ROUTINE CALCULATES DISCHARGE QCR, AREA ACR, AND THE TOTAL HEAD ECR FOR
C     TYPE OF PROFILE SHOWN IN FIGURE 2.3.
C
      DIMENSION B(12),H(12)
      G=32.174
      Q=0.0
      Q=WL1*B1*0.5
      A=WL1*B1
   30 ECR=WL1+(Q**2)/(2.0*G*(A**2))
      Q1=A1=0.0
      K=0
   10 K=K+1
      A2=Q2=0.0
      IF(ECR.GT.H(K).AND.ECR.GT.H(K+1)) GO TO      5
      GO TO 15
    5 DT=B(K+1)-B(K)
      IF(ABS(DT).LT.0.001) GO TO 15
      IF(WL1.LT.H(K)) GO TO 15
      E1=ECR-(H(K)+H(K+1))/2.0
      C=1.293*(E1**0.0825)
      Q2=0.544*C*DT*SQRT(G)*(E1**1.5)
      WL2=((Q2**2)/(G*(DT**2)*C))**0.33333
      A2=DT*WL2
   15 Q1=Q1+Q2
      A1=A1+A2
      IF(K.LT.(NPTS-2)) GO TO 10
      IF(ABS((Q1-Q)/Q1).LT.0.001) GO TO 20
      Q=Q1
      GO TO 30
   20 QCR=Q1
      ACR=A1
      RETURN
      END
```

```
      SUBROUTINE COEF(H,NPTS,WL,QCR,QLOST,QCORR)
C
C       GIVEN THE ELEVATION COORDINATES H(1,NPTS) FOR A CRITICAL SECTION THE
C     ROUTINE CALCULATES THE EFFECTIVE REDUCTION IN DISCHARGE QLOST, AND THE
C     REVISED VALUE OF DISCHARGE QCORR FROM THE GIVEN INPUT VALUE OF CRITICAL
C     DISCHARGE QCR.
C
      DIMENSION H(NPTS)
      QLOST=0.0
      N=NPTS-3
      DO 20 I=1,N
      Q=0.0
      IF(ABS(H(I+1)-H(I+2)).LT.0.1) GO TO 100
   10 IF(H(I+1).GT.H(I+2)) GO TO 50
      C=(H(I+2)-H(I+1))*12.0
      T=(WL-H(I+1))*12.0
      GO TO 75
   50 C=(H(I+1)-H(I+2))*12.0
      T=(WL-H(I+2))*12.0
   75 IF(T.LT.C) GO TO 750
      Q=10.0**((C-9.50)/4.25)
      GO TO 100
  750 Y2=C
      IF(T.LT.0.0) GO TO 100
      Y1=0.0
      X1=0.0002
      X2=10.0**((C-9.50)/4.25)
      A=(Y2-Y1)/(ALOG10(X2)-ALOG10(X1))
      B= Y2 - A*ALOG10(X2)
      Q=10.0**((T-B)/A)
  100 QLOST=QLOST+Q
   20 CONTINUE
      QCORR=QCR-QLOST
      RETURN
      END
```

```
      PROGRAM TST (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C
C     THIS IS A DRIVING ROUTINE TO CALCULATE THE RATING CURVE FOR
C    A COMPOUND CONTROL SECTION, GIVEN THE CRITICAL SECTION COORDINATES
C    B AND H(1,NPTS) AND THE UPSTREAM CHANNEL WIDTH B1.
C     N IS THE NUMBER OF SECTIONS TO BE STUDIED.
C     NPTS IS THE NUMBER OF COORDINATES FOR A GIVEN SECTION.
C     NN IS THE NUMBER OF WATER LEVELS TO BE USED FOR THE RATING CURVE.
C     DWL IS THE INCREMENTAL VALUE USED FOR WATER LEVEL.
C     B1 IS THE APPROACH CHANNEL WIDTH.
C     G IS THE ACCELERATION DUE TO GRAVITY.
C     B AND H(1,NPTS) IS A ONE DIMENSIONAL ARRAY CONTAINING WATER LEVELS.
C
      DIMENSION B(12),H(12)
      REAL IL
      READ 5,N
    5 FORMAT(I5)
      DO 15 J=1,N
      READ 10,NPTS,NN,DWL,B1,G
   10 FORMAT(2I5,3F10.3)
      READ 20,(B(M),H(M),M=1,NPTS)
   20 FORMAT(2F10.2)
      PRINT 200
  200 FORMAT(*                DISCHARGE                  WL                    HCR
     1 *,//)
      CALL BOTTOM(H,NPTS,IL,WLMAX)
      WL = IL
      DO 30 I=1,NN
      WL=WL + DWL
      CALL DISCHAR(B,H,NPTS,WL,B1,QCR,ACR,ECR)
      CALL COEF(H,NPTS,WL,QCR,QLOST,QCORR)
      FROUDE=QCORR/(WL*B1*SQRT(G*WL))
      IF(FROUDE.GT.0.6) GO TO 88
      PRINT 80,QCORR,WL,ECR
   80 FORMAT(3F20.3)
   30 CONTINUE
   88 CONTINUE
      PRINT 90
   90 FORMAT(* FROUDE NUMBER IN APPROACH CHANNEL GREATER THAN 0.6 *)
      PRINT 33
   33 FORMAT(1H1)
   15 CONTINUE
      STOP
      END
```