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METHODOLOGIES FOR DEVELOPING DOWNSTREAM GAUGE RATINGS FOR OPERATING CANAL DISCHARGE REGULATING STRUCTURES

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TABLE OF CONTENTS

LIST OF ANNEXURES

LIST OF TABLES

LIST OF FIGURES

FOREWORD

The Netherlands Government has provided funding to IIMI-Pakistan for a research program on "Managing Irrigation for Environmentally Sustainable Agriculture in Pakistan". Under the Operational Management Component, there are two subcomponents on Decision Support Systems (DSS) that can be defined as a "set of tools and procedures that if properly used by the management of a canal command area would enhance the quality of decision-making processes in that system".

DSS activities are underway both in the Punjab and Sindh provinces, focused on Eastern Sidiqia Canal and Jamrao Canal, respectively. Many canal discharge regulating structures have been calibrated for discharge measurements in these two canal command areas. In addition, discharge ratings for canal structures in the Lower Swat Canal (North West Frontier Province) developed by the International Sedimentation Research Institute, Pakistan (ISRIP) have been used in this report.

The Provincial Irrigation Departments (PIDs) have used, for many decades, a vertical gauge located downstream from a canal discharge regulating structure, that is placed in the irrigation channel or along a bank. This downstream gauge is then calibrated so that a gate operator, knowing the required discharge rate and corresponding downstream gauge reading, can adjust the gate(s) until the water level in the irrigation channel is at the required gauge reading. The so-called KD-formula is used in developing these downstream gauge ratings.

The findings of this research disclose that the KD-formula can be employed in developing highly accurate downstream gauge ratings. However, in earthen canals, these ratings can significantly change in only two months. Thus, there is a need to frequently conduct current meter measurements in order to periodically adjust each downstream gauge rating. But, this is rarely down. As a consequence, discharge readings from these downstream gauges are frequently in error by 15-25 percent, and sometimes more. For such large canals, this is a tremendous amount of water that is not being managed.

Fortunately, the technology for downstream gauge ratings is standard operating procedure in the PIDs. Many of the experienced staff are quite familiar with this technology. However, most young field staff have not been exposed, nor are they equipped, to properly implement downstream gauge ratings. Certainly, implementation of this technology is not too difficult. The major requirement is a will to "make it happen"!

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1. USE OF DOWNSTREAM GAUGE RATINGS

1.1. BACKGROUND

The correct observation and recording of gauges and discharges of the supplies available in the rivers and those withdrawn by the various canals is of great importance, as this data forms the basis for the distribution of water among the provinces, allocation to each canal head regulator, discharge regulation along canals, and for the continual enhancement of agricultural productivity from the irrigation systems.

There are many ways and various practices of taking discharge measurements and then developing a discharge rating so that observing the water level on a gauge will yield the corresponding discharge from a rating table. These techniques and procedures all aim at being correct and meeting the system requirements for precision. The most commonly used methods of discharge observation are either based on calibrating the canal discharge regulating structure or the downstream channel by developing a stage-discharge relationship. Both methods have advantages and disadvantages and certain limitations.

The controlling factors for deciding about the method of discharge observations are the type of flow control structure, level of knowledge about hydraulics, and the required degree of precision. The trade-off is between accuracy and simplification. The comparison of the two commonly used methods for developing a discharge rating is given in Table 1.1.

Table 1.1. Comparison of advantages and disadvantages of Structure Calibration and Channel Stage-Discharge Rating methods for observing discharge.

Structure Calibration	Channel Stage-Discharge Rating
➢ Data intensive approach requiring gate opening plus one or two gauges	➢ Only one gauge reading is required
➢ Long-term stability of discharge rating	➢ Will likely need adjustment after every season
➢ Knowledge about flow conditions through structure is required	➢ Independent of the structure flow condition
➢ Accurate only for rigid boundary structures not irregular wooden planks	➢ Depends only on the cross-section, not type of structure, but affected by backwater, as well as sediment deposition and removal
➢ Four-six discharge measurements are required for one flow condition to develop an accurate rating	➢ Four discharge observations are required for preparing a reliable rating curve

1.2. STRUCTURE CALIBRATION

Canal discharge rating structures commonly consist of regulating gates. There are two flow conditions; namely, free orifice flow that requires observing a gauge upstream of the structure and submerged orifice flow that requires reading both an upstream gauge and a downstream gauge. Both flows conditions are presented more fully in Section 8.

The structure calibration basically involves measuring the actual discharge over the widest possible range of flows. Each discharge measurement will correspond with a different gate opening. A coefficient of discharge, C_d, is calculated for each set of measurements consisting of discharge (usually measured with a current meter), gate opening, and gauge reading(s). Then, a graph is prepared on rectangular coordinates with gate opening, G_o, plotted on the ordinate and C_d along the abscissa.

For designing these gates structures, an appropriate coefficient is selected from the available literature. Often, the theoretical value of 0.61 is used for purposes of design, while the literature contains values usually ranging from 0.5 – 0.7 depending on gate geometry. However, many factors affect the coefficient of discharge, with the most prominent being the fabrication geometry of the gate(s) and the gate frame(s). Also, the geometry of the structure containing the gate(s) will influence C_d.

Based on experiences in field calibration of gate structures, the variation of the actual discharge rating from the design discharge equation is usually 10 -30 percent, while on rare occasions this difference will be within five percent. Unfortunately, the majority of gate structures are operated using the design discharge equation as the rating.

There are two primary advantages in undertaking a structure calibration. First of all, the actual discharge rate can be obtained at anytime, usually within an accuracy of five percent, that allows fairly precise canal operations. Secondly, the calibration should remain stable for a number of years, but can slowly change with time due to a lack of maintenance (e.g. if the leakage through a closed gate increases over time, the discharge calibration will be affected). Although changing downstream backwater conditions will alter the discharge rate passing through a gate structure, the discharge calibration will not change, so that the actual discharge passing through the canal regulating structure would be measured and recorded. This would be the situation for submerged orifice flow, where the structure calibration would only become altered if there are some physical changes occurring between the upstream and downstream gauges that affect the hydraulics between these gauges. For free orifice flow, downstream backwater effects have no influence on the discharge passing through the gate structure, so that the structure calibration does not change unless there are physical changes between the upstream gauge and the gate(s), including the gate(s), that would alter the hydraulics between these two points.

1.3. CHANNEL STAGE-DISCHARGE RATING

1.3.1. Stage-Discharge Relationship

A stage-discharge relationship is basically an empirical relationship. According to the relationship, the discharge passing through the section is directly proportional to the flow depth, D, but expressed exponentially as D^n , where n is an exponent which primarily depends upon the geometric shape of the cross section.

A stage-discharge relationship can be expressed mathematically as

$$Q = f\ (D^n) \tag{1.1}$$

The commonly used form is

$$Q = K\,D^n \tag{1.2}$$

which is often referred to as "the KD formula". A gauge is installed, usually along the channel bank, but sometimes in the center of the channel. The flow depth, D, is obtained by reading the gauge, G, but applying a gauge correction, ΔG, which can be expressed as

$$D = G - \Delta G \tag{1.3}$$

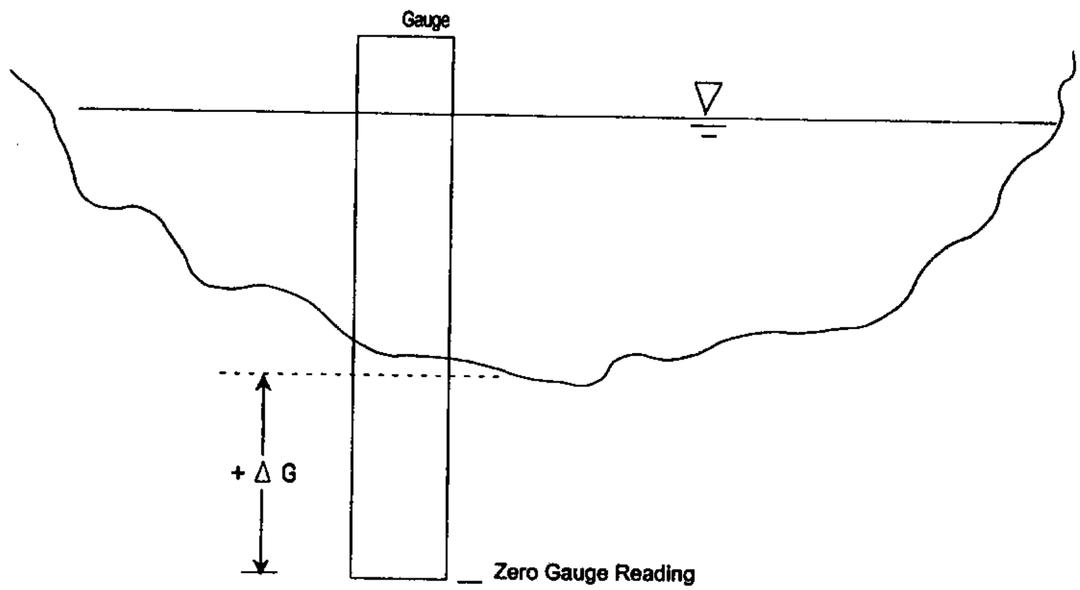
Usually, the zero gauge reading will be below the channel bed as illustrated in Figure 1.1a. For this case, the gauge correction, ΔG, will correspond with a positive gauge reading representing the channel bed. Thus, when reading the water level on the gauge, G, then the gauge correction, ΔG, must be subtracted from G to obtain the depth of flow, D, as indicated in Equation 1.3.

Figure 1.1b illustrates the case where the zero gauge reading is located above the channel bed. In this case, the vertical elevation difference between the zero gauge reading and the representative channel bed would be the gauge correction, ΔG, which would be a negative number. Thus, using Equation 1.3,

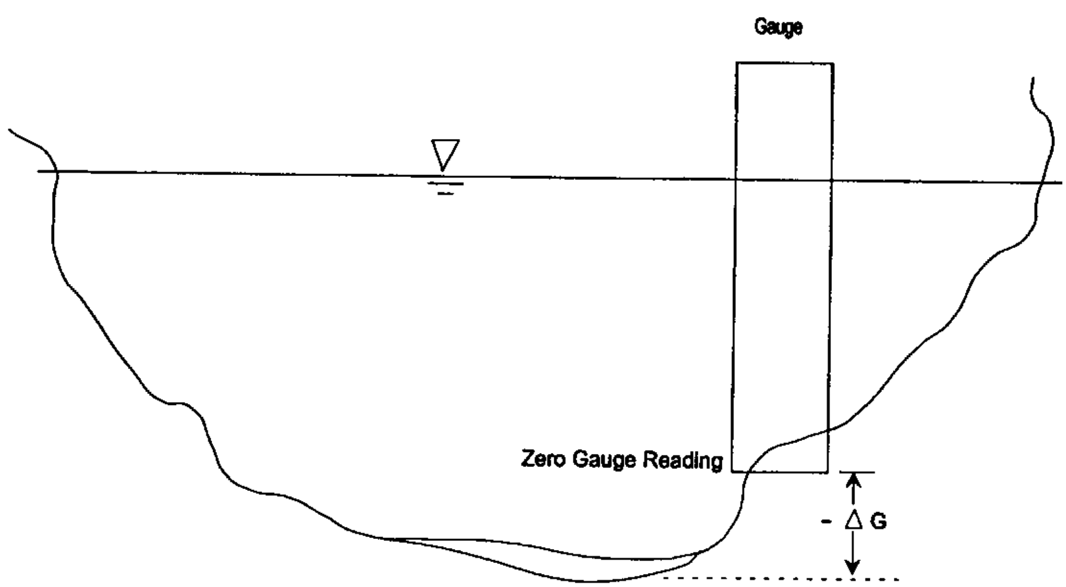
$$D = G - (-\Delta G) = G + \Delta G \tag{1.4}$$

The form of the KD formula that can be used for establishing a channel stage-discharge rating is obtained by substituting Equation 1.3 into Equation 1.2, so that

$$Q = K\ (G - \Delta G)^n \tag{1.5}$$



(a) Gauge embedded in channel bed.



(b) Gauge installed above channel bed.

Figure 1.1. Possible cases for applying a gauge correction.

where

K	=	Coefficient;
D	=	Depth of Flow;
G	=	Gauge Reading;
ΔG	=	Gauge Correction; and
n	=	Exponent.

1.3.2. Disadvantage of Channel Stage-Discharge Rating

The stage-discharge relationship depends on: (I) the depth of flow, D, which requires a defining of the zero depth of flow; (2) an exponent, n, that is largely dependent on the geometric shape of the cross-section; and (3) a coefficient, K, that is mostly dependent on the cross-sectional area that is influenced by backwater from downstream regulating structures, as well as to some extent, downstream vegetative growth, sediment deposition, and in the case of earthen channels, scouring.

Most irrigation channels are strongly influenced by backwater from downstream regulating structures. These backwater effects extend upstream from 1 - 40 kilometers (km) or 0.6 - 25 miles, depending upon hydraulic conditions, with the larger values corresponding with large discharges and large structures, say greater than 1,000 cubic feet per second (cusecs) or 30 cubic meters per second (cumecs).

For irrigation channels transporting clear water, vegetative growth becomes more and more of a problem as the irrigation season progresses. This is often the case for lined channels, as well, because of sediment deposition that allows roots to be established. The degree of backwater increases as the vegetation continues to grow.

For irrigation channels transporting water heavily laden with sediment, there are usually no problems with vegetative growth, but there are significant difficulties associated with sediment deposition. As more and more sediment is deposited in a channel reach, the water level increases in the reach with associated backwater effects extending upstream. Likewise, when these sediment deposits are removed from a channel, then the water levels will fall if the same discharge rate is being conveyed.

Thus, the primary disadvantage of a channel stage-discharge rating is that this rating is not hydraulically stable. Instead, this rating is usually changing over time. As a minimum, this rating should be adjusted after every irrigation season. An argument could be made that the rating should be checked every month during operating, then the rating adjusted if necessary. After a few irrigation seasons of using a particular channel stage-discharge ratings, the frequency required for checking the ratings would be known.

1.3.3. Advantages of Channel Stage-Discharge Rating

The stage-discharge relationship has some advantages over the structure calibration because of its versatility and flexibility as it is dependent only on the depth of flow and the shape of the channel cross-section. In some countries (e.g. Pakistan and India) where the structures are operated by using the karries (wooden battens or planks) with varying crest levels, as well as not being well formed so there is considerable leakage of water through the karries, so that using a structure calibration would be extremely cumbersome. Also, recognizing that the karries top behaves as a crest, which keeps on changing every time that the karries are reinserted in the structure, would discourage the development of a structure calibration, not only because so many physical conditions would have to be calibrated, but their accuracy would also suffer.

When the objective is simplification in the method of observing the discharge, then the stage-discharge relationship is the best choice as only the gauge at the cross-section is required. For the structure calibration, the flow condition has to be identified and the gate opening(s) measured as well as the upstream, or upstream and downstream, gauge(s).

However, the principal advantage in using the Channel Stage-Discharge Rating Method for developing a downstream gauge rating below a canal discharge regulating structure is ease of operation. The gate operator, knowing the required downstream discharge and the corresponding downstream gauge reading, can open or close the gate(s) until the required downstream water level is achieved.

1.4. COMBINED STRUCTURE CALIBRATION AND DOWNSTREAM GAUGE RATING

For a canal discharge regulating structure using metal gates, an accurate structure calibration can be accomplished. This is usually done by establishing a gauge upstream of the gates if free orifice flow is the flow condition through the gate(s), or gauges both upstream and downstream of the gate(s) for submerged orifice flow. The zero reading for these gauges is established using a Surveyor's level with reference to the crest level for the gate(s). The gate openings are also measured with reference to this same crest level. Then, the gate opening is set, the flow is allowed to stabilize so that steady-state flow conditions exist, then a current meter measurement can be undertaken and the gauge(s) read. The gate opening is changed and the procedure repeated until four to six sets of measurements have been completed. Now, the structure calibration can be calculated.

When a gauge is installed in the channel downstream from a regulating structure, then the procedure is to take a series of current meter measurements when the channel is experiencing different water levels. Then Equation 1.5 is used to develop a discharge rating.

When compared with doing a structure calibration, the only added work in developing a downstream gauge rating is reading this downstream gauge each time a current meter measurement is made, plus the time to calculate the downstream gauge rating. This combination has two benefits. First of all, the downstream gauge rating facilitates the gate operators in their work. But more importantly, the structure calibration can be readily used to periodically adjust the downstream gauge rating every month or season without having to make a current meter measurement to accomplish this task.

1.5 ORGANIZATION OF THE REPORT

The purpose of this report is to analyze various approaches for developing downstream gauge ratings, along with providing some sensitivity about the degree of accuracy associated with the different approaches. Following this report, a manual will be prepared to provide guidelines for irrigation field managers.

The next section in this report relates the KD-formula to the Manning–Strickler equations. The analytical part of this report begins with three parts (Sections 3, 4, and 5) where different approaches have been used to establish the Channel Stage-Discharge Rating by analyzing the degree of accuracy achieved by the different approaches. For this purpose, examples from the provinces of North West Frontier (NWFP), Sindh and Punjab in Pakistan have been used. In the first part (Section 3), out of the three factors used in Equation 1.5, only K has been taken as a variable, while the value of n is fixed as 5/3 and two approaches for finding the depth of flow are also evaluated; the resulting values of K are first averaged for developing the stage-discharge curve, which is followed by a technique employing a graphical relationship between gauge reading on the ordinate and K along the abscissa. In the second part (Section 4), D has been calculated by using the Hydraulic Mean Depth, while n and K are considered variables, which are calculated using regression analysis; then the analysis is repeated by treating n as a constant of 5/3, while K and D are variables. In the third part (Section 5), referred to as the three variables approach, all of the three factors -- K, D and n -- are assumed to be variables, which are calculated by the method of least squares. The above discussion is summarized in Table 1.2. In all cases, D is calculated using Equation 1.3, where the gauge reading has been measured, so the problem is determining the gauge correction.

Table 1.2. Approaches used for evaluating Channel Cross-Section Ratings.

Approach	Factors: K	n	D
One Variable Approach	Variable	Constant	Constant
Two Variables Approaches	Variable	Variable	Constant
	Variable	Constant	Variable
Three Variables Approach	Variable	Variable	Variable

Following the analysis of the three approaches, some lessons will be derived regarding gauge corrections and discharge accuracy. Then, a procedure and example will be presented in Section 7 on using periodic current meter measurements to adjust a Downstream Gauge Rating, either monthly or during the start of each irrigation season.

This will be followed by Section 8 on combining a Structure Calibration with a Downstream Gauge Rating (see Section 1.4 above).

For undertaking the analyses described above, flow control structures located in three major canal commands of the Indus Basin Irrigation System have been selected, with each canal command located in a different province as shown in Figure 1.2. Some of the general characteristics of these canal command areas are listed in Table 1.3.

Table 1.3. General characteristics of canal commands in Pakistan for analyzing downstream gauge ratings at selected discharge regulating structures.

Province	Name of Canal Command	Gross Command Area, acres	Culturable Command Area, acres	Discharge at Head, cusecs
North West Frontier	Lower Swat Canal	182,000	134,500	1940
Sindh	Jamrao Canal	943,422	888,354	3400
Punjab	Eastern Sadiqia Canal	1,166,300	972,700	4900

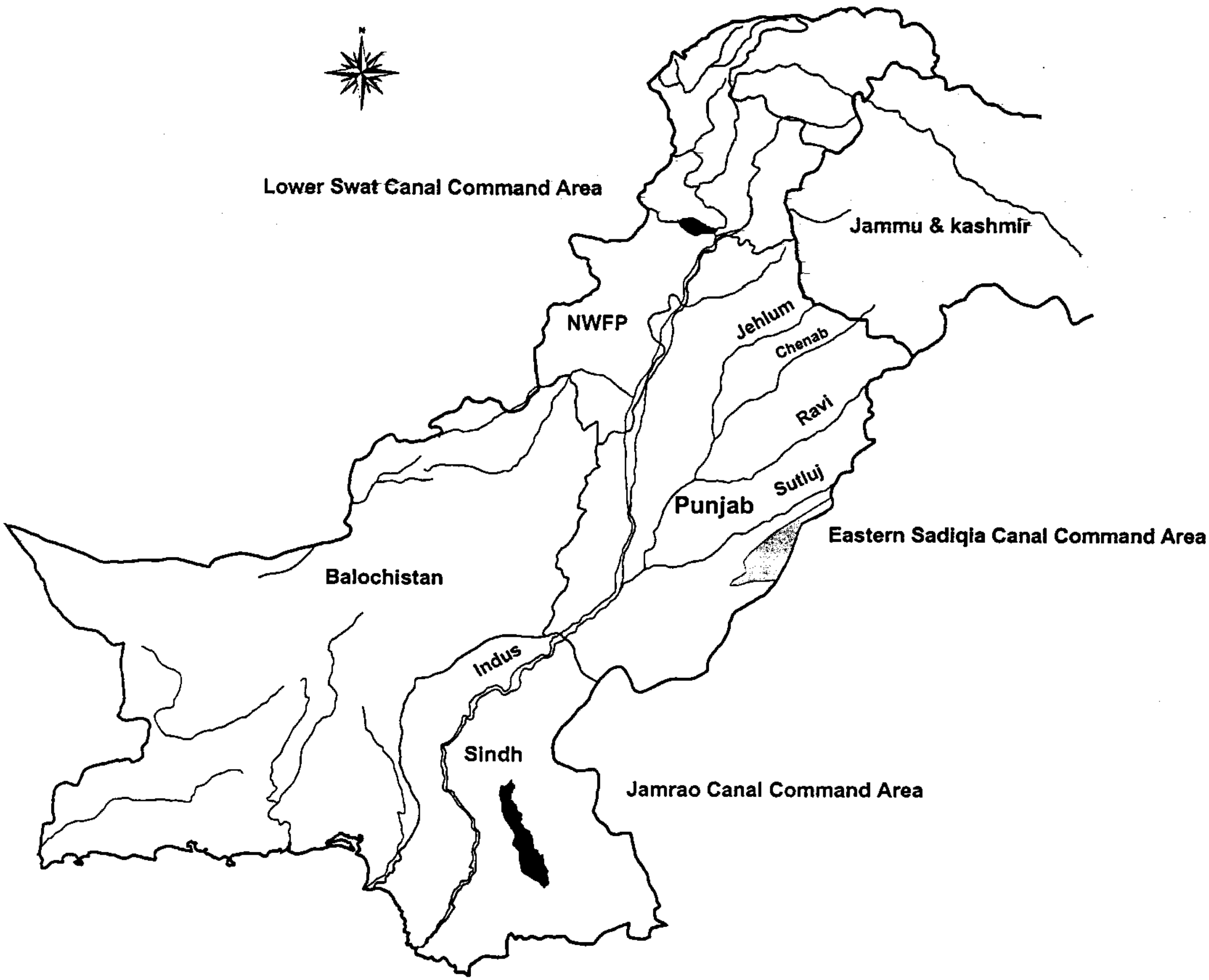


Figure 1.2. Location of canal commands in Pakistan for analyzing downstream gauge ratings at selected discharge regulating structures.

2. BACKGROUND OF THE KD-FORMULA

2.1. DERIVING KD-FORMULA FROM MANNING-STRICKLER EQUATION

In the previous section, the advantages and disadvantages of using the KD-formula were highlighted. In this section, the background of the KD-formula is discussed. This will show the way to obtain insights on whether or not collected discharge measurements are suitable for making a downstream gauge rating.

The KD-formula is derived from the equation of Manning-Strickler. This equation describes steady and uniform flow in a canal. This means that, in principle, it is only applicable when the flow in a canal is constant both in place along its length and in time. The Manning-Strickler equation is written as:

$$Q = k_s A i^{1/2} R^{2/3} \tag{2.1}$$

$$k_s = 1/n_m \tag{2.2}$$

where

Q	=	Discharge (m^3/s);
k_s	=	Strickler's roughness coefficient ($m^{1/3}/s$);
n_m	=	Manning's roughness coefficient ($s/m^{1/3}$);
A	=	Cross section area (m^2);
i	=	Bed and water surface gradient (-); and
R	=	Hydraulic radius (m).

If all variables are expressed in English units and the Strickler and Manning coefficients remain in SI-units, the formula changes to:

$$Q = 1.49 k_s A i^{1/2} R^{2/3} \tag{2.3}$$

and

$$k_s = 1/n_m \tag{2.2}$$

in which 1.49 is derived from the conversion of $m^{1/3}/s$ into $ft^{1/3}/s$. Thus, it has the dimension $ft^{1/3}/m^{1/3}$. So,

Q	=	Discharge (ft^3/s);
k_s	=	Strickler's roughness coefficient ($m^{1/3}/s$);
n_m	=	Manning's roughness coefficient ($s/m^{1/3}$);
A	=	Cross-section area (ft^2);
i	=	Bed and water surface gradient (-); and
R	=	Hydraulic radius (ft).

The hydraulic radius R of a canal at a certain discharge can be obtained by dividing the cross-sectional area A by the wetted perimeter P:

$$R = A / P \tag{2.4a}$$

where P is the wetted perimeter (ft).

Thus, area, A, is a product of the wetted perimeter P and the hydraulic radius, R:

$$A = PR \tag{2.4b}$$

If this last equation is incorporated in the Strickler-Manning formula, the result is as follows:

$$Q = 1.49 k_s i^{1/2} P R^{5/3} \tag{2.5}$$

For a specific canal reach, the value of the Strickler coefficient and the gradient of the bed are constant. Additionally, for the case of wide canals, it can be assumed for different discharges that the wetted perimeter does not change very much in comparison with the hydraulic radius. Thus, the discharge Q can be written as a product of a constant coefficient, K, and the hydraulic radius, R. First of all, K can be written as:

$$K = 1.49 k_s i^{1/2} P \tag{2.6}$$

Then, Equation 2.6 can be substituted into Equation 2.5, so that:

$$Q = K R^{5/3} \tag{2.7}$$

This last equation can be seen as the basis for the KD-formula. When using the KD formula, a relation is developed between the depth, D, and the discharge, Q:

$$Q = K D^{n} \tag{1.2}$$

where

K	=	Coefficient related to constant bed parameters (ft^{3-n}/s)
D	=	Depth parameter related to the cross-section (feet)
n	=	Exponent

If the hydraulic radius, R, is chosen as the depth related parameter and K is the constant product of $1.49k_s i^{1/2}P$, then the exponent, n, will be equal to 5/3.

With a set of discharge measurements and their accompanying cross-sectional parameters, an evaluation can be made to determine if the calculated K is really constant (Table 2.1).

Table 2.1. Discharge measurements at the head of Hakra Branch Canal offtaking from the tail of Eastern Sadiqia Canal.

Date:		10/21/96	10/22/96	12/29/96
Q:	cusecs	2296	1957	.1065
A:	$feet^2$	950.5	849.5	658.0
P:	feet	151.2	149.2	145.0
R = A / P:	feet	6.29	5.69	4.54
K:	$feet^{(4/3)}/s$	107.3	107.8	85.6
K / P:	$feet^{(1/3)}/s$	0.709	0.723	0.590

Although the discharges vary, the value of K for the first two discharge measurements is reasonably constant. The K for the third discharge measurement varies significantly from the other two values. If K is divided by the wetted perimeter, P, the same trend is maintained. The reason for the discrepancy may be explained by the different dates of the discharge measurements. The first two dates are in October of 1996, while the last measurement was taken in December 1996. The bed level and / or the roughness coefficient of the canal might have changed with time and season. A likewise observation can be made when analyzing discharge measurements taken at the head of Malik Branch Canal (Table 2.2).

Table 2.2. Discharge measurements at the head of Malik Branch Canal offtaking from the tail of Eastern Sadiqia Canal.

Date:		10/10/96	12/26/96	12/24/96
Q:	cusecs	1979	1096	767
A:	$feet^2$	790.9	634.2	505.5
P:	feet	111.1	106.7	104.0
R = A / P:	feet	7.12	5.94	4.86
K:	$feet^{(4/3)}/s$	75.08	56.21	54.96
K / P:	$feet^{(1/3)}/s$	0.676	0.527	0.529

For the last two discharge measurement taken during December, the values for K are similar. However, the first discharge measurement made in October has a different value for K. Dividing K by the wetted perimeter, P, does not change this situation (Table 2.2).

Even within a small period of time between the discharge measurements, the K values may vary. This is especially true for the canals with lower design discharges, such as Table 2.3, where the columns are arranged in descending order from the highest discharge rate to the lowest.

Table 2.3. Discharge measurements at head of Hakra 6-R Distributary offtaking from Hakra Branch Canal.

Date:		10/13/96	10/28/96	10/24/96	10/21/96
Q:	cusecs	742	676	437	106
A:	$feet^2$	206.7	205.3	156.3	77.6
P:	feet	49.0	48.0	44.2	38.1
R = A / P:	feet	4.22	4.27	3.53	2.04
K:	$feet^{(4/3)}/s$	67.456	60.035	53.315	32.427
K / P:	$feet^{(1/3)}/s$	1.376	1.250	1.206	0.851

Although the discharge measurements are all taken in October of 1996, the K values are decreasing with the amount of discharge. As this section of the Hakra 6-R Distributary is lined, the bed level gradient and the roughness coefficient can hardly change. This implies that the surface level gradient is changing with regard to the bed level gradient. Actually, this would mean that, in essence, that the Manning-Strickler equation is no longer applicable, as strictly speaking, there is no longer uniform flow in the canal. The discharges used in this analysis are a little different from the discharges used for this structure in the rest of the report. The reason is that the discharges used here are not the same as the discharges at the site of the gauge. There are two outlets located between the measurement site and the site of the gauge. In the rest of this report, for the actual downstream gauge rating, these outlets have been taken into consideration

A very good way to illustrate the above observations is the use of double-log graphs. As previously shown, the KD-formula, in its most general form, is:

$$Q = KD^n \tag{1.2}$$

This relation becomes linear (a straight line) when the logarithm is taken from both sides of the equation:

$$\log Q = \log K + n \log D \tag{2.8}$$

If this is plotted on double-log paper with the depth, D, on the abscissa and the discharge, Q, on the ordinate, the resulting straight line will have a slope n and will intercept the ordinate at the value of K when D=1.

In case of the hydraulic radius, R, and its exponent, n=5/3, this relation can be written as:

$$\log Q = \log K + \tfrac{5}{3} \log R \tag{2.9}$$

In Figures 2.1-2.3, straight lines have been plotted for the previously discussed measurement data for the example canal locations. If K is really constant for all data points per site, all points would be lying on a single straight line. As the actual values of K sometimes vary for the obtained data points, so the points where the straight lines cross the ordinate also vary. The result is a group of parallel lines with slope n=5/3. To avoid plotting lines for each data point individually, only the upper and lower boundary straight lines have been plotted.

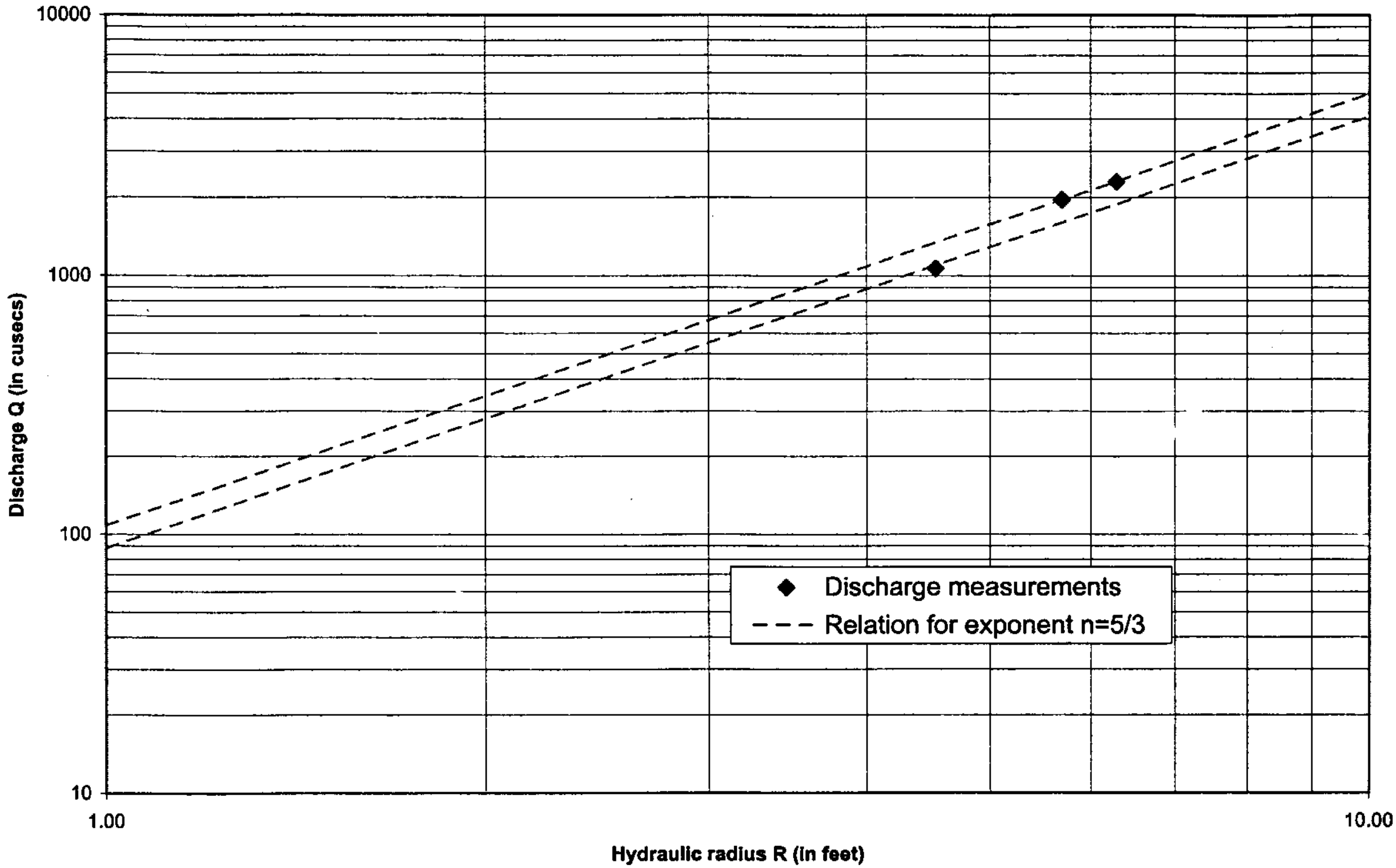


Figure 2.1. Relation between Discharge, Q, and Hydraulic Radius, R, at the head of Hakra Branch Canal.

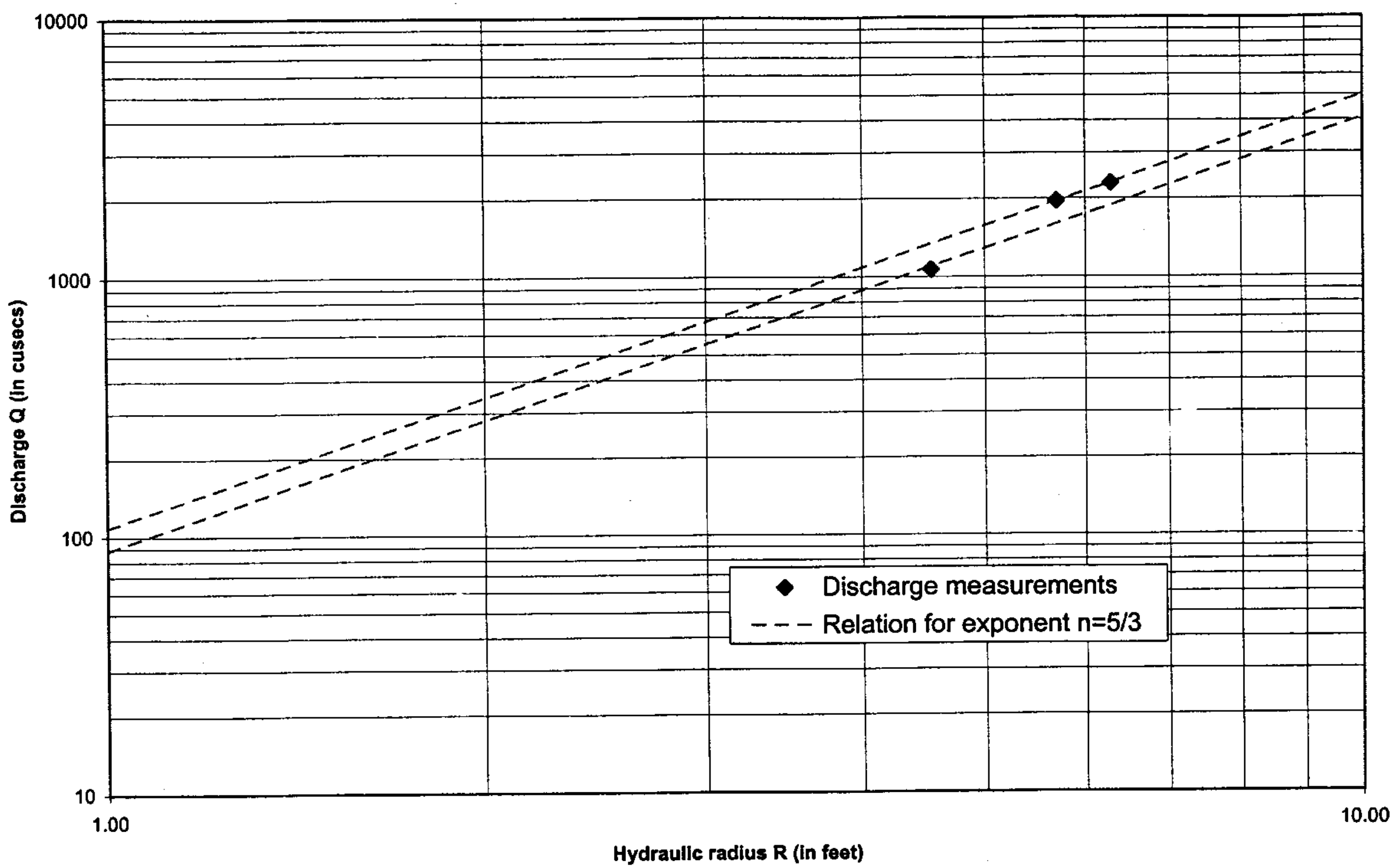


Figure 2.1. Relation between Discharge, Q, and Hydraulic Radius, R, at the head of Hakra Branch Canal.

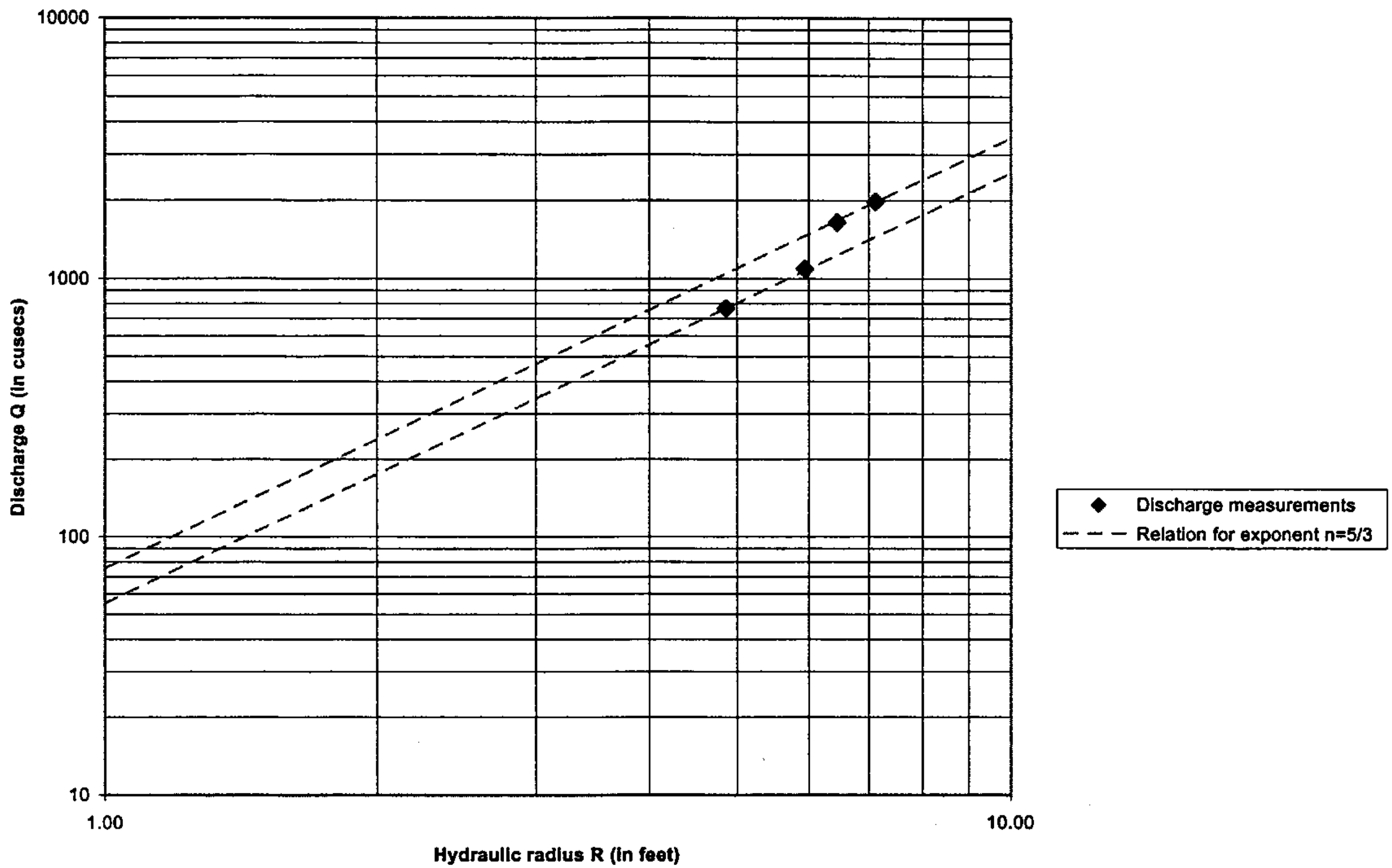


Figure 2.2. Relation between Discharge, Q, and Hydraulic Radius, R, at the head of Malik Branch Canal.

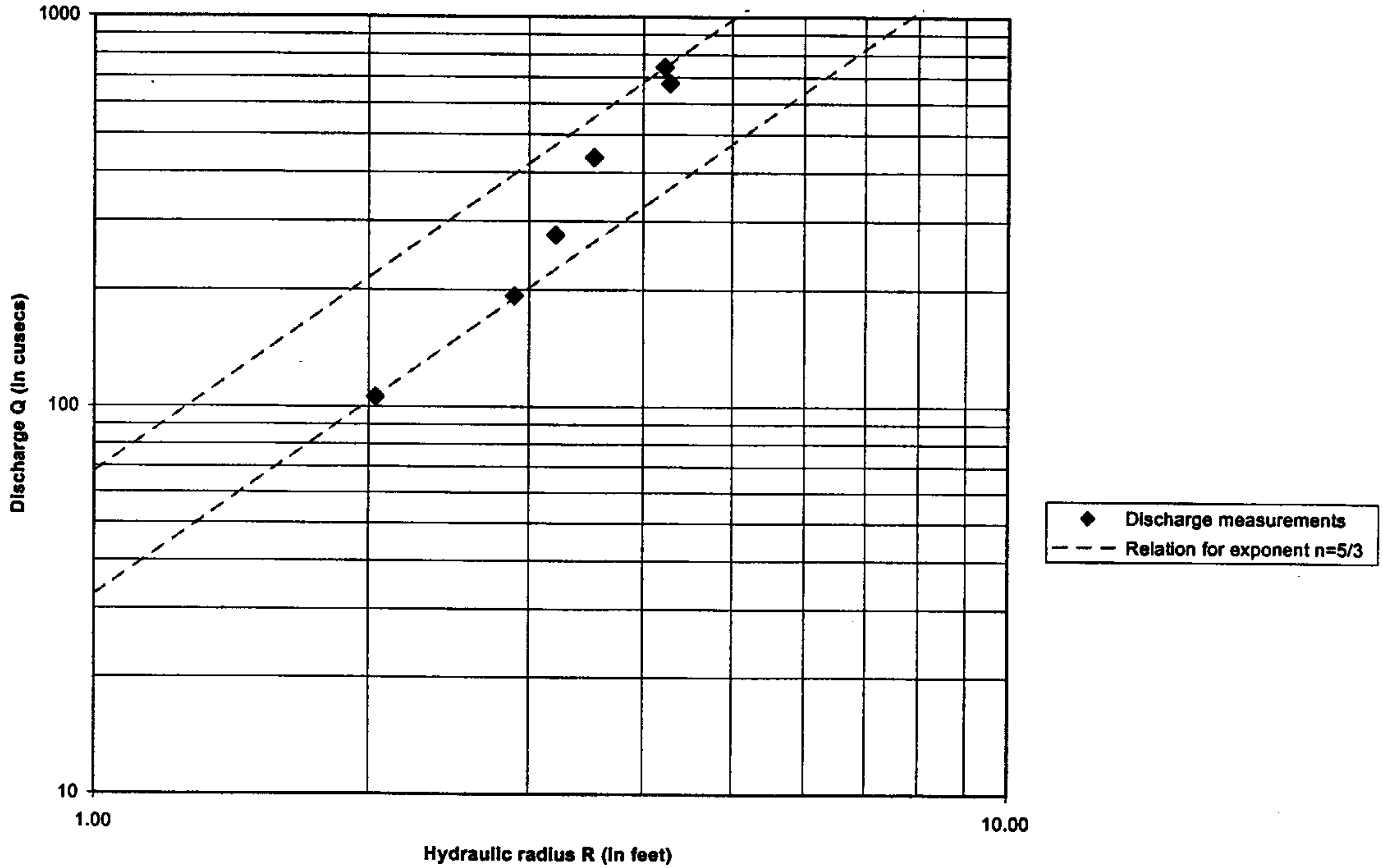


Figure 2.3. Relation between Discharge, Q, and Hydraulic Radius, R, at the head of Hakra 6 R Distributary.

The method of plotting measured data points on double-log paper is very good for evaluating any discrepancies or trends in the collected data. If data points are going to be used in developing a KD-relation, it is very helpful if they have been evaluated on the basis of the KD-frormula's origin in the equation of Manning-Strickler. Together with specific information on the channel and site, where the data was collected, and the different measurement dates, this leads to a choice as to what data to use in the preparation of a downstream gauge rating.

2.2. USE OF DIFFERENT PARAMETERS FOR DEPTH D

2.2.1. Use of Hydraulic Depth

Sometimes, for reasons of more easily available data, there is a preference to change from the use of the hydraulic radius, R, to the use of the hydraulic depth, D_{hy}. The hydraulic depth, D_{hy}, is derived by dividing the area, A, with the top surface width of the channel, W_T, instead of by the wetted perimeter, P, which is used in the case of the hydraulic radius, R.

$$D_{hy} = A / W_T \tag{2.10}$$

where

D_{hy}	=	Hydraulic depth (ft); and
W_T	=	Top surface width of the channel (ft).

As the wetted perimeter, P, is always larger than the top surface width, W_T, the hydraulic radius, R, will always be smaller than the hydraulic depth, D_{hy}.

From the Manning-Strickler formula, a relation can be obtained between the discharge, Q, and the hydraulic depth, D_{hy}:

$$Q = 1.49 k_s i^{1/2} W_T D_{hy}{}^{n} \tag{2.11}$$

The exponent, n, in this relation is no longer constant, nor equal to 5/3. Instead, it is dependent on the relation between the hydraulic depth, D_{hy}, and the hydraulic radius, R, or the relation between the top surface width, W_T, and the wetted perimeter, P. The exponent, n, can be written as follows:

$$n = 5/3 + \frac{2/3 \log\left(\frac{W_T}{P}\right)}{\log D_{hy}} \qquad \text{for } D_{hy} \neq 1 \tag{2.12}$$

For all distributaries and canals, values for D_{hy} can be expected to be larger than 1. In this case, n will be smaller than 5/3. Although n varies for different channels between 1.60 and 1.66, the values are nearly constant for the different discharges (and hydraulic depths) at a specific cross-section of a channel (Table 2.4).

Table 2.4. Values of the exponent, n, for discharge measurements at the head of Hakra 6-R Distributary using the hydraulic depth, D_{hy}.

Date:		10/13/96	10/28/96	10/24/96	10/21/96
Q:	cusecs	742	676	437	106
P:	feet	49.04	48.03	44.22	38.12
W:	feet	45.5	44.5	41.5	36.5
D_{hy}:	feet	4.54	4.61	3.77	2.13
K:	$feet^{(3-n)}/s$	62.59	55.62	50.03	31.05
n:		1.634	1.633	1.635	1.628

In Table 2.4, K is almost similar to the K used in the analysis with the hydraulic radius, R, except for the fact that the wetted perimeter, P, has been replaced with the top surface width, W_T. As was the case with the same example, while relating it to the hydraulic radius, R, the K value at this site is not constant for the different discharge measurements.

$$K = 1.49 k_s i^{1/2} W_T \qquad (2.13)$$

$$Q = K D_{hy}{}^{n} \qquad (2.14)$$

Table 2.4 shows that the value of n has a variation of less than 0.5% for the different discharge measurements. Although the value of n should be calculated for the different discharge measurements separately, which are used in the downstream gauge rating, the exponent, n, will mostly be constant. This is illustrated by Figure 2.4, where the value of the exponent, n, has been plotted for different hydraulic depths, D_{hy}. In this case, the value of the relation between top surface width, W_T, and wetted perimeter, P, has been put at a constant of 0.9 for all cross-sections / discharges. Clearly, the relation between D_{hy} and n is dominated by the influence of two crossing infinites; one at a value of $D_{hy}=1$ and the other at a value of n=5/3 (or 1.67). At $D_{hy}=1$, it does not make any difference what the value of n is in the KD-formula, the result of $(1)^n$ will always be equal to 1. For $D_{hy}>1$, the relation quickly moves towards the value of 5/3 (1.67), but it does not reach this value exactly and always remains a little less.

The assumption made in Figure 2.4 is that the value of the relation between top surface width, W_T, and wetted perimeter, P, for different hydraulic depths is constant, which is not entirely correct. In fact, this value decreases for larger hydraulic depths, when the top surface width, W_T, remains nearly the same, but the wetted perimeter, P, increases. In other words, there is a direct influence of the hydraulic depth D_{hy} on the wetted perimeter, which changes the behaviour of the exponent, n.

The influence of the wetted perimeter, P, is such that the exponent, n, will move slowly to n=5/3 as the hydraulic depth approaches infinity, but lies below n=5/3. This depends on the kind of relation between the wetted perimeter and the hydraulic depth. Practically, it means that for most channels and its different discharges that n is nearly constant at a value between n=1.60 and n=1.67.

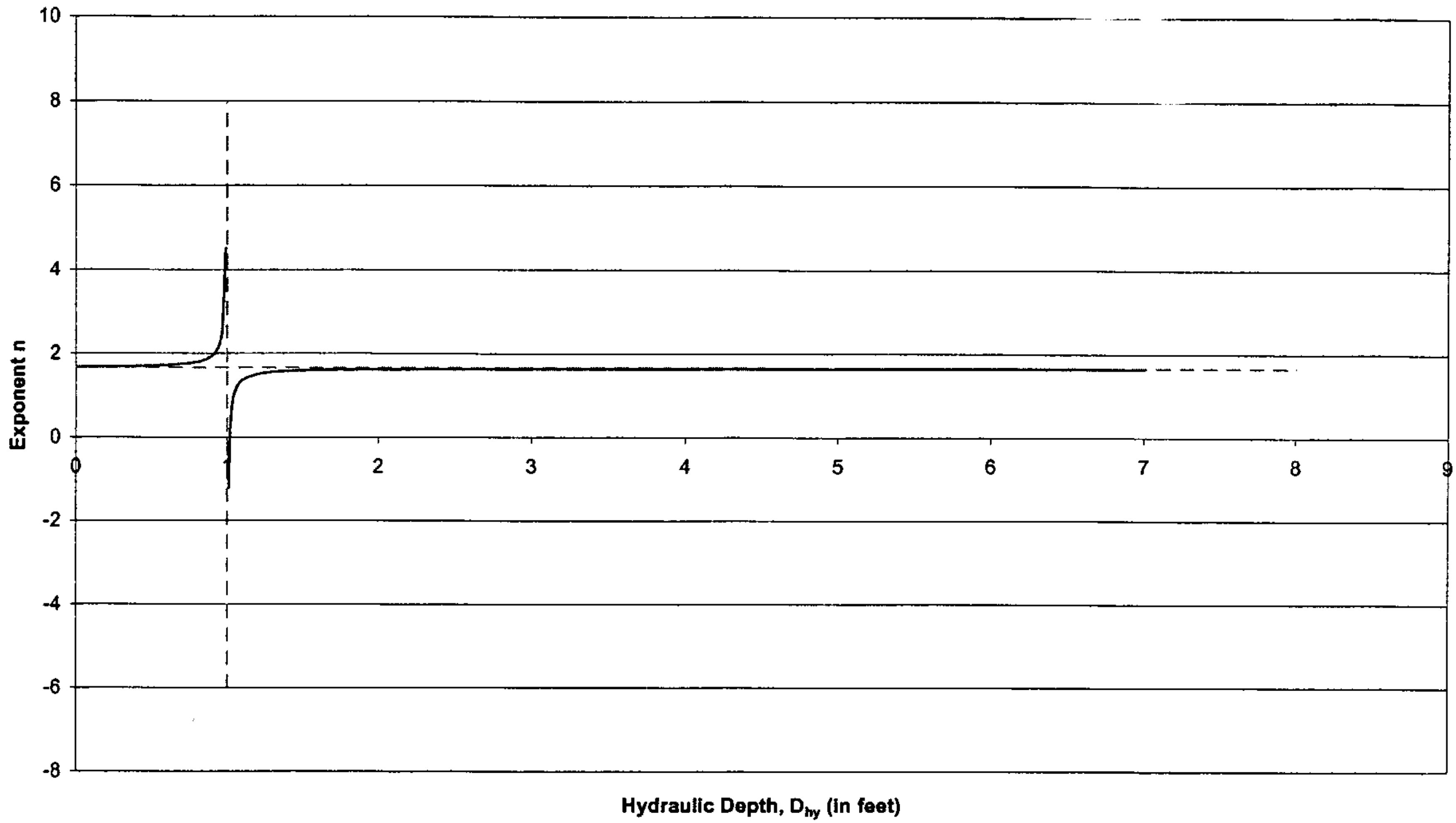


Figure 2.4. Relation between Hydraulic Depth, D_{hy}, and Exponent, n, for constant W_T/P.

2.2.2. Use of Other Depth Parameters

In the KD-formula, the depth parameter to be used for the depth, D, is not specified, which can be any chosen depth parameter. Mostly, the person making the downstream gauge rating will choose a depth parameter, which relates to the gauge. In a later section, this will be extensively discussed. However, it is important to know that any depth, D, can be related to the original equation of Manning-Strickler.

In this section, a general relation between the exponent, n, and the depth parameter, D, is given, similar to the one derived for the hydraulic depth, D_{hy}. For any depth parameter, D, the Manning-Strickler equation can be written in a similar way as already done with the hydraulic radius, R, and the hydraulic depth, D_{hy}.

$$Q = 1.49 k_s i^{1/2} \frac{A}{D} D^n \qquad (2.15)$$

In this it is assumed that the constant part of the equation is given by:

$$K = 1.49 k_s i^{1/2} \frac{A}{D} \qquad (2.16)$$

After relating this to the original equation of Manning-Strickler, the result for exponent, n, becomes:

$$n = 5/3 + \frac{2/3 \log\left(\frac{A}{PD}\right)}{\log D} \qquad (2.17)$$

This result is obviously similar to the one derived for the hydraulic depth, D_{hy}.

If the constant K given for this general derivation is not constant, but for instance the quotient A/D is still dependent on D, the relation for exponent, n, should be written differently, such as:

$$Q = 1.49 k_s i^{1/2} D^n \qquad (2.18)$$

where

$$K = 1.49 k_s i^{1/2} \qquad (2.19)$$

K is again assumed constant. Then, the result for the exponent, n, will be:

$$n = 5/3 + \frac{2/3 \log\left(\frac{A}{PD}\right)}{\log D} + \frac{\log\left(\frac{A}{D}\right)}{\log D} \qquad (2.20)$$

If this last result is compared with the previous result for the exponent, n, it can be concluded that n has been increased as a result of the last term.

3. ONE VARIABLE APPROACH TAKING K AS VARIABLE

3.1. INTRODUCTION

The KD formula is the most widely used formula for discharge observations in the subcontinent. One of the reasons for its adoptation on such a large scale is the versatility and the simplicity for using the formula and developing the rating. One of the important points that needs to be considered while using the formula is the updating of the rating from time-to-time. The Irrigation Managers, while revising the rating table, have an attitude to revise the rating table based on a single discharge observation by fixing the value of n=5/3 and calculating the value of K, then applying the same K value for the whole discharge range. This section has been dedicated to learning and explaining the effect of this practice regarding accuracy and variation of the value of K, along with the representativeness of the calculated discharges.

3.2. CALCULATION STEPS

For developing the rating table, the data for different sizes of canals located in different areas were used. The discharges were observed over a range from low flow to full supply level and the corresponding gauge readings were recorded . Later, the hydraulic depth of flow was calculated for each discharge observation and the gauge readings were transformed into the depth of flow by using the hydraulic depth. The value of K was calculated for each discharge observation by fixing the value of the exponent at 5/3. The average value of K was calculated and used for the calculation of the discharges and developing the rating table.

The above procedure can be explained graphically in that it was assumed that the slope of the Log D-Log Q curve is always 5/3, and represents all of the discharges for the corresponding depths of flow. Also, the hydraulic depth of flow was considered equal to the average depth of flow. In one way, this method is the simplest form for using the KD formula as the calculations involved are very simple and without involvement of any graphical or iterative method.

3.3. VARIATION OF K

The approach of using one discharge observation for developing the rating table was intentionally not adopted in the detailed analysis, so that the reader can get a better picture of the K variation over a range of the discharges from low to high flows.

Table 3.1 shows the variation of calculated K values for canals from Punjab, NWFP and Sindh provinces. From Table 3.1, one thing is clear, that the value of K is not constant over a range of discharges. Either it increases or decreases with the discharge. This table reveals that the variation of the K value is quite significant. Like in the case of Hakra 6-R Distributary, the K value varies from 31.06 to 49.55, which results in an

Table 3.1. Variation of K with changing discharge and discharge rating accuracy.

One Variable Approach										
Hakra 6-R Distributary, Hakra Branch Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
2.7	108	2.16	0.54	2.11	1.67	31.06	43.17	150	-42	-39.0
4.54	440	4.01	0.53	3.95	1.67	44.62	43.17	426	14	3.2
5.46	664	4.86	0.6	4.87	1.67	47.46	43.17	604	60	9.0
5.67	744	4.98	0.69	5.08	1.67	49.55	43.17	648	96	12.9
Head Hakra Branch Canal, Eastern Sadiqia Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
5.91	1065	4.7	1.21	4.43	1.67	89.26	93.48	1115	-50	-4.7
7.62	1957	5.94	1.68	6.14	1.67	95.13	93.48	1922	34	1.7
8.2	2296	6.64	1.56	6.72	1.67	96.04	93.48	2235	61	2.7
Head Malik Branch Canal, Eastern Sadiqia Canal										
G	Q	Hyd Depth	Delta G	D	N	K	K avg	Q cal	Q-Q cal	% Diff
4.85	767	5.16	-0.31	5.26	1.67	48.24	55.69	886	-119	-15.5
5.67	1096	6.25	-0.58	6.08	1.67	54.16	55.69	1127	-31	-2.8
7.38	1979	7.72	-0.34	7.79	1.67	64.69	55.69	1704	275	13.9
Geodi Minor, Lower Swat Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
0.52	2	0.45	0.07	0.43	1.67	9.24	12.36	3	-1	-33.7
0.52	3	0.43	0.09	0.43	1.67	10.44	12.36	3	0	-18.4
0.86	8	0.76	0.1	0.77	1.67	12.06	12.36	8	0	-2.5
0.86	8	0.76	0.1	0.77	1.67	11.95	12.36	8	0	-3.4
1.08	12	0.99	0.09	0.99	1.67	12.66	12.36	12	0	2.4
1.08	12	1.11	-0.03	0.99	1.67	12.44	12.36	12	0	0.7
1.25	17	1.15	0.1	1.16	1.67	13.1	12.36	16	1	5.7
1.28	17	1.15	0.13	1.19	1.67	12.99	12.36	16	1	4.9
1.62	27	1.46	0.16	1.53	1.67	13.53	12.36	25	2	8.6
1.62	27	1.46	0.16	1.53	1.67	13.27	12.36	25	2	6.8
2.1	43	2	0.1	2.01	1.67	13.34	12.36	40	3	7.4
2.1	42	2.07	0.03	2.01	1.67	13.28	12.36	40	3	7.0
Khan Mahi Branch, Lower Swat Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
1.78	6	0.87	0.91	0.61	1.67	13.59	15.73	7	-1	-15.7
1.78	6	0.9	0.88	0.61	1.67	14.14	15.73	7	-1	-11.2
2.38	21	1.38	1	1.21	1.67	15.44	15.73	22	0	-1.9
2.38	22	1.4	0.98	1.21	1.67	15.74	15.73	22	0	0.1
2.7	33	1.6	1.1	1.53	1.67	16.35	15.73	32	1	3.8
2.72	34	1.62	1.1	1.55	1.67	16.58	15.73	33	2	5.1
3.05	43	1.86	1.19	1.88	1.67	15.16	15.73	45	-2	-3.8
3.05	45	1.9	1.15	1.88	1.67	15.64	15.73	45	0	-0.6
3.73	81	2.36	1.37	2.56	1.67	16.9	15.73	75	6	7.0
3.73	81	2.4	1.33	2.56	1.67	16.9	15.73	75	6	7.0
4.44	118	2.92	1.52	3.27	1.67	16.34	15.73	113	4	3.7
4.44	115	2.92	1.52	3.27	1.67	15.96	15.73	113	2	1.4
Lower Swat Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
3.01	302	2.12	0.89	2.04	1.67	92.52	88.77	290	12	4.1
3.01	302	2.13	0.88	2.04	1.67	92.46	88.77	290	12	4.0
3.54	443	2.6	0.94	2.57	1.67	92.11	88.77	427	16	3.6
4.49	738	3.55	0.94	3.52	1.67	90.76	88.77	721	16	2.2
4.96	856	3.95	1.01	3.99	1.67	85.49	88.77	889	-33	-3.8
5.46	1073	4.48	0.98	4.49	1.67	87.98	88.77	1083	-10	-0.9
5.49	1083	4.5	0.99	4.52	1.67	87.81	88.77	1095	-12	-1.1
6.16	1259	4.99	1.17	5.19	1.67	81.04	88.77	1379	-120	-9.5
Mirpur Distributary, Jamrao Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
1.938	64	1.58	0.36	1.95	1.67	21.18	19.87	60	4	6.2
2.113	71	2.39	-0.28	2.12	1.67	20.33	19.87	70	2	2.3
2.383	84	2.34	0.04	2.39	1.67	19.61	19.87	85	-1	-1.3
2.633	93	2.8	-0.17	2.64	1.67	18.36	19.87	100	-8	-8.2
Daulatpur Minor, Jamrao Canal										
G	Q	Hyd Depth	Delta G	D	n	K	K avg	Q cal	Q-Q cal	% Diff
2.08	33	1.58	0.36	1.69	1.67	13.68	13.18	31	1	3.7
2.09	32	2.39	-0.28	1.7	1.67	13.43	13.18	32	1	1.9
2.35	40	2.34	0.04	1.96	1.67	13.03	13.18	40	0	-1.1
2.73	52	2.8	-0.17	2.34	1.67	12.56	13.18	54	-3	-4.9

inaccuracy of 39% in the low flow range and 13% in the high flow range. The same figures for the Khan Mahi Branch Canal is 15.7% and 1.4%, while for Daulatpur Minor it is 4% and 3%, respectively. As the method involves first calculating the K values and then averaging them, this results in the rating table being more accurate in the middle range of flows and more inaccurate for the extremes (i.e. full and low supply ranges).

3.4. EVALUATION OF A DISCHARGE RATING BASED ON SINGLE DISCHARGE OBSERVATION

The same phenomenon can be explained by considering the example of Hakra 6-R Distributary. If the discharge was observed only at a gauge reading of 5.67 feet, which was 743.9 cusecs, so that assuming n=1.67, the K value will be 49.55. Using these values, the calculated and observed discharges will be as shown in Table 3.2.

Table 3.2. Accuracy Variation in Case of Rating Table Based on One Discharge Observation at Head of Hakra 6-R Distributary.

Gauge	Depth of Flow	Observed Discharge	Calculated Discharge	Difference in Discharge	%age Error
Feet	Feet	Cusecs	Cusecs	Cusecs	
2.7	2.11	108	172	64	59.5
4.54	3.95	440	489	49	11.1
5.46	4.87	664	693	29	4.4
5.67	5.08	744	744	0	0.0

So, if the discharge table is developed based on one discharge observation (in this case, at the peak flow range), that rating will only be accurate for this discharge range and the rating for other flow ranges will likely be highly inaccurate. So, developing the discharge table by using only one discharge observation could be highly inaccurate.

As shown in Figure 3.1, the rating for the head of Hakra 6-R Distributary is quite accurate for the peak flows and the gap between the two curves is increasing with a decrease in gauge reading or discharge. Likewise, the response of the rating developed by using a discharge observation in the low flow range will result in a difference of discharges, or gap, between the two curves, which will increase with the increasing gauge or discharge value.

Table 3.1 shows the variation of K values and the accuracy variation over a range of discharges for the selected canals. The overall results have been summarized in Table 3.3.

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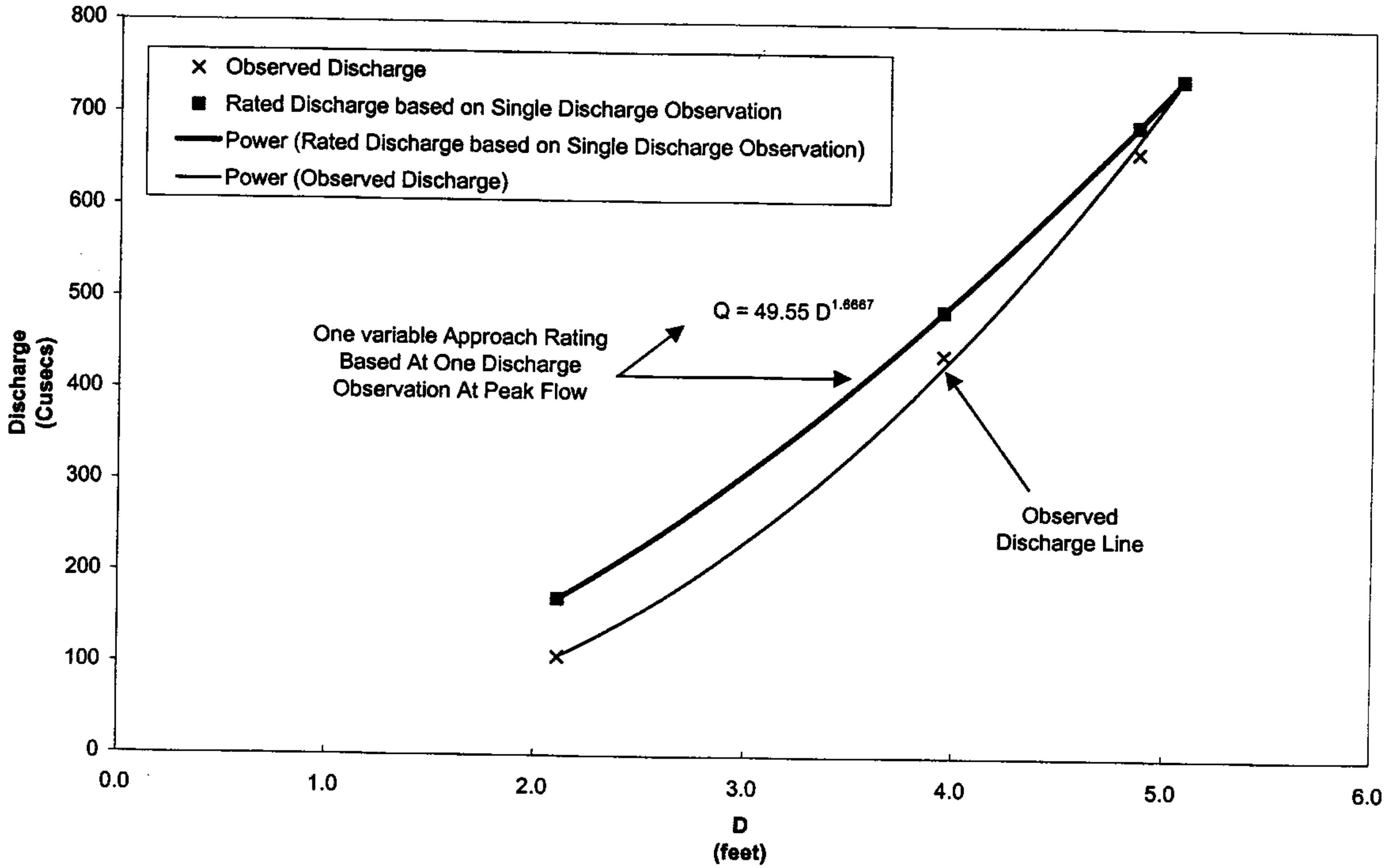


Figure 3.1. Graphical Presentation of Rating Curve and Measured Discharge Trend Line.

Table 3.3. Summarized Results of One Variable Approach.

Canal Name	Authorized Discharge (Cusecs)	One variable Approach				
		K variable, n=5/3 & ΔG from Hyd; Depth				
		K	ΔG (Ft)	Average Difference (Cusecs)	% Error [(Qm-Qcal)/Qm]*100	
					Maximum	Minimum
Geodi Minor	35	12.36	0.09	1	33.7	0.7
Daulatpur Minor	49	13.18	0.4	1	4.9	1.1
Mirpur Distributary	64	18.36	-0.01	4	8.2	1.3
Khan Mahi Branch	103	15.73	1.17	2	15.7	0.1
6-R Hakra Disty	459	43.17	0.59	53	39.0	3.2
Malik Branch	1538	48.24	-0.41	141	15.5	2.8
Lower Swat Canal	1940	88.77	0.98	29	9.5	0.9
Hakra Branch	2785	93.48	1.48	49	4.7	1.7

The above results reveal that the value of K increases with the discharge. This statement is true for all of the canals except Khan Mahi, which deviates a little bit from the statement. Looking at the average difference, it can also be perceived from the above table that the rating produced by this technique is more accurate for the smaller channels. The reason could be that, in the case of smaller channels, the variation of bed level in a section is less, so the representativeness of K increases, while for larger canals with more bed variation, the formula is more inaccurate.

3.5. DEVELOPMENT OF K - G RELATIONSHIP

As seen in Table 3.1, the use of an average value for K affects the accuracy in the extreme flow ranges (i.e. in the high and low flow ranges). This makes the total rating inaccurate. So it is better to seek a relationship between the K and G parameters and use the K values corresponding to the G values from that relationship. This can be seen as accepting the fact that the value of K varies with the discharge. This approach will improve the accuracy over a discharge range.

For obtaining the values of instantaneous K, it was assumed that K is a function of G^x which mathematically can be written as

$$K = f(G)^x$$

The easiest way of obtaining the K value corresponding to G is from a graph between G and K . So, a graph between K and G was drawn and the values of K for the corresponding G were read from the graph. This can be very easily done on the computer by drawing a power function trend line through the points and determining the relationship between G and K . By using that relationship, the values of K corresponding to G can be easily calculated.

To illustrate the procedure, the data for Hakra 6-R Distributary is used. The average depth was taken as the depth parameter. The gauge correction was found and the gauge was corrected to obtain the value of D. Then, the value of K was calculated for the individual discharge observations by assuming that Q=f (D)$^{(5/3)}$. The graph between K and G was drawn as shown in Figure 3.2.

Then the values of K were read against the downstream (D/S) gauge reading and used for the calculation of the discharge. The results of the above exercise are presented below in Table 3.4.

Table 3.4. Comparison of varying K to average K approach for three sites.

One Variable Approach									
Hakra 6-R Head, Hakra Branch Canal									
Using K From D~K Curve								**Using Average K**	
G	**Qm**	**D**	**K**	**n**	**Qcal**	**Qm-Qc**	**%Diff**	**Q cal**	**% Diff**
Ft	**Cusecs**	**Ft**			**Cusecs**	**Cusecs**		**Cusecs**	
2.7	108	2.16	31.63	1.67	114	-6	-5.9	150	-39.0
4.54	440	4.01	43.74	1.67	444	-4	-0.9	426	3.2
5.46	664	4.86	48.41	1.67	679	-15	-2.3	604	9.0
5.67	744	4.98	49.04	1.67	717	27	3.6	648	12.9
Mirpur Distributary, Jamrao Canal									
Using K From D~K Curve								**Using Average K**	
G	**Qm**	**D**	**K**	**n**	**Qcal**	**Qm-Qc**	**%Diff**	**Q cal**	**% Diff**
Ft	**Cusecs**	**Ft**			**Cusecs**	**Cusecs**		**Cusecs**	
1.94	64	1.95	21.22	1.67	65	0	-0.4	60	6.2
2.11	71	2.12	20.42	1.67	72	0	-0.7	70	2.3
2.38	84	2.39	19.36	1.67	83	1	1.0	85	-1.3
2.63	93	2.64	18.52	1.67	94	-1	-1.2	100	-8.2
Geodi Minor, Lower Swat Canal									
Using K From D~K Curve								**Using Average K**	
G	**Qm**	**D**	**K**	**n**	**Qcal**	**Qm-Qc**	**%Diff**	**Q cal**	**% Diff**
Ft	**Cusecs**	**Ft**			**Cusecs**	**Cusecs**		**Cusecs**	
0.52	2	0.43	10.3	1.67	3	0	-11.1	3	-33.7
0.52	3	0.43	10.3	1.67	3	0	1.6	3	-18.4
0.86	8	0.77	11.59	1.67	7	0	4.0	8	-2.5
0.86	8	0.77	11.59	1.67	7	0	3.1	8	-3.4
1.08	12	0.99	12.19	1.67	12	0	3.7	12	2.4
1.08	12	0.99	12.19	1.67	12	0	2.0	12	0.7
1.25	17	1.16	12.59	1.67	16	1	3.9	16	5.7
1.28	17	1.19	12.66	1.67	17	0	2.5	16	4.9
1.62	27	1.53	13.32	1.67	27	0	1.4	25	8.6
1.62	27	1.53	13.32	1.67	27	0	-0.5	25	6.8
2.1	43	2.01	14.07	1.67	45	-2	-5.7	40	7.4
2.1	42	2.01	14.07	1.67	45	-3	-6.2	40	7.0

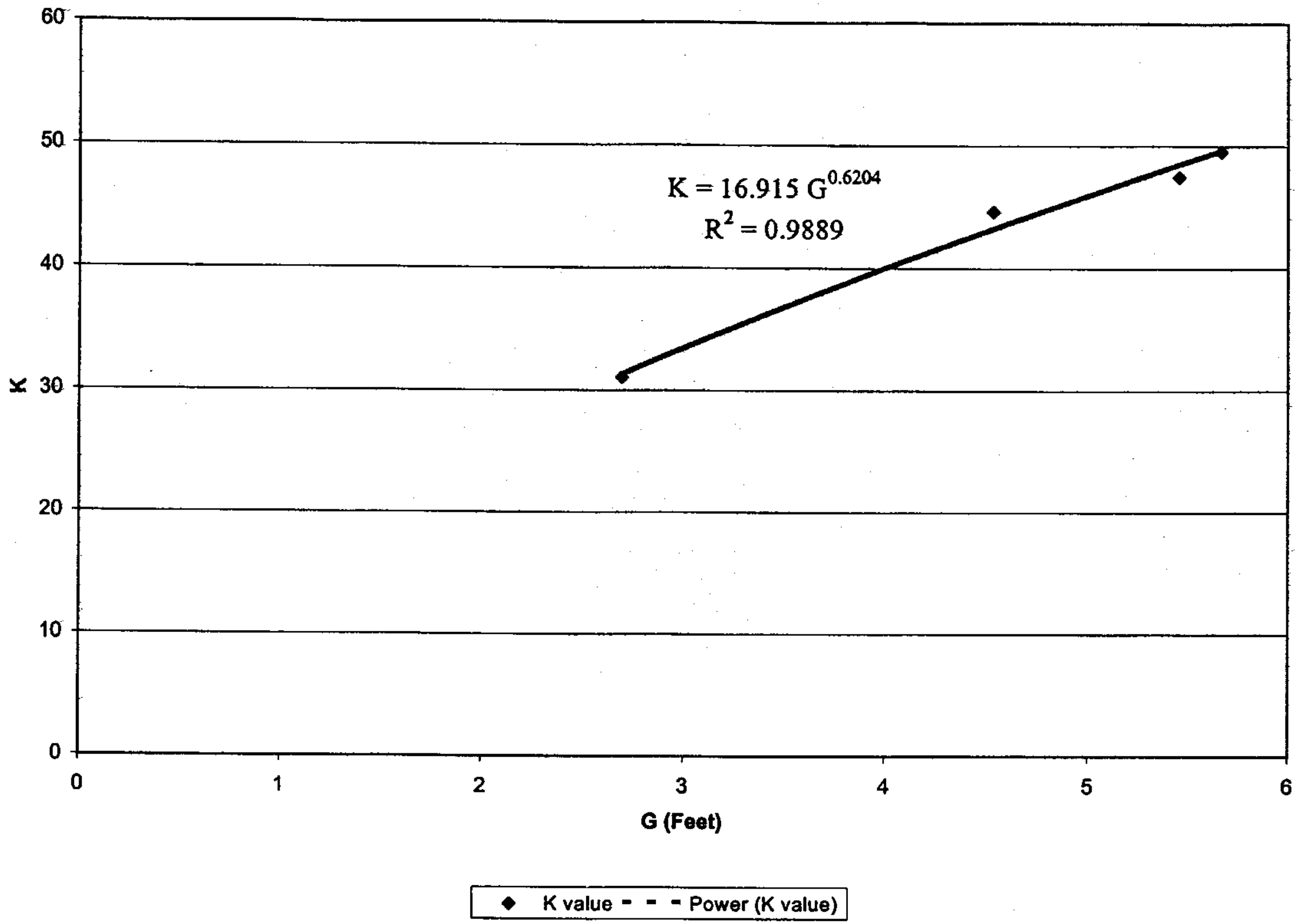


Figure 3.2. Relation between coefficient, K, and downstream gauge, G, for Hakra 6-R Distributary.

Table 3.4 shows significant improvement in the accuracy. In case of the Hakra 6-R Distributary, the results are on the average more than four times as accurate. In the cases of Mirpur Distributary and Geodi Minor, the accuracy improves on average by more than six and more than three times, respectively.

So, this section can be concluded with the comment that by assuming the value of n equal to 5/3, it is still possible to get the required accuracy by following the procedure described under Section 3.5. The rating based on a single discharge observation or the use of single (average) K value leads to inaccuracies beyond a tolerable limit.

The value of K either increases or decreases with an increase or decrease in the gauge reading, or the depth of flow, as shown earlier in this report. Also, the averaging of the K value makes the rating inaccurate in the extreme flow ranges. One solution to the problem, as discussed before, is by establishing a relationship between the G and K values and by extending the curve. Finding the value of K corresponding to a particular value of G becomes a singular unique value of K. Then, this unique value of K can be used for calculating the discharge. As shown in Table 3.4, the results of this approach, using unique values of K, are quite encouraging.

The next thought which may come to mind is why the value of K varies with the discharge? And is there any method for arriving at one single value of K or a set of K values with the varying depth having a minimum standard deviation? The following section will deal with a methodology leading to an almost constant value of K for the discharge range.

3.6. K ~ G RELATIONSHIPS

The curve relating K and G always shows a trend. The values of K either increase or decrease with an increase or decrease in the values of the gauge reading. As mentioned before, this relationship will be the main issue in developing an approach for arriving at a constant value of K that will provide a good discharge rating. For developing this approach, two canals -- Hakra 6-R and Mirpur distributaries – have been selected. The Hakra 6-R Distributary has a positive relationship between K and G, whereas the same relationship for Mirpur Distributary is negative as shown in Figures 3.3 and 3.4. The curves are comparable with the curves relating the coefficient of discharge (C_d) and gate opening (G_o). The introduction of a correction in the reference elevation for a zero gate opening results in a constant value of C_d over the flow range and takes care of the errors in establishing the reference level or leakage through the gates. So, on the same pattern, a correction factor was introduced for the gauge reading to make it representative of the depth, instead of using the hydraulic depth, or any other factor. The procedure adopted was to vary the gauge correction (ΔG) values and the one yielding a nearly constant value of K was used for calculating the discharges. For developing a graphical solution, the above statement can be restated that the ΔG value yielding the best straight vertical line in the $(G-\Delta G)$~K relationship becomes both the targeted gauge correction value and K value as schematically shown in Figure 3.5.

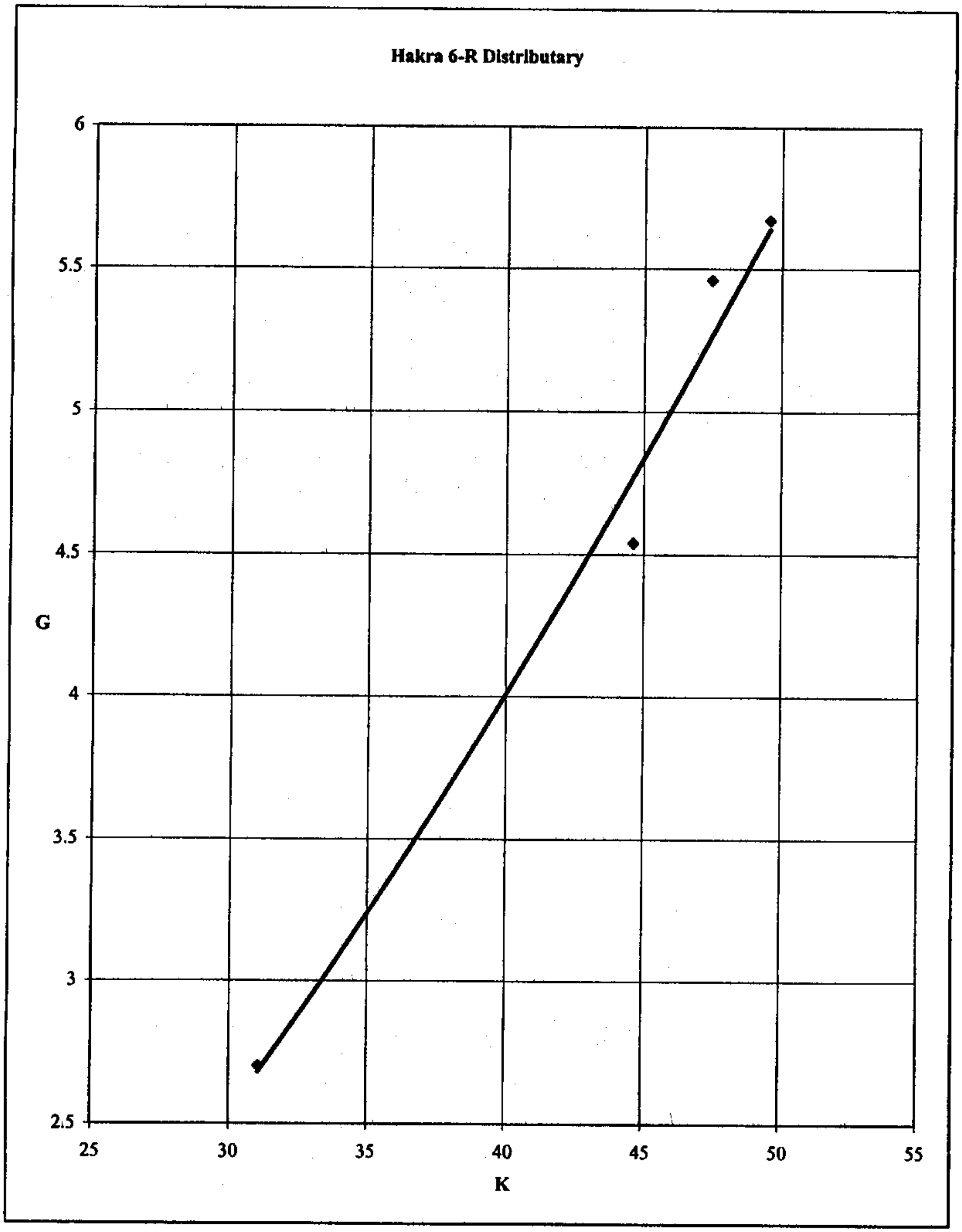


Figure 3.3. Variation of the K coefficient with the gauge reading for Hakra 6-R Distributary.

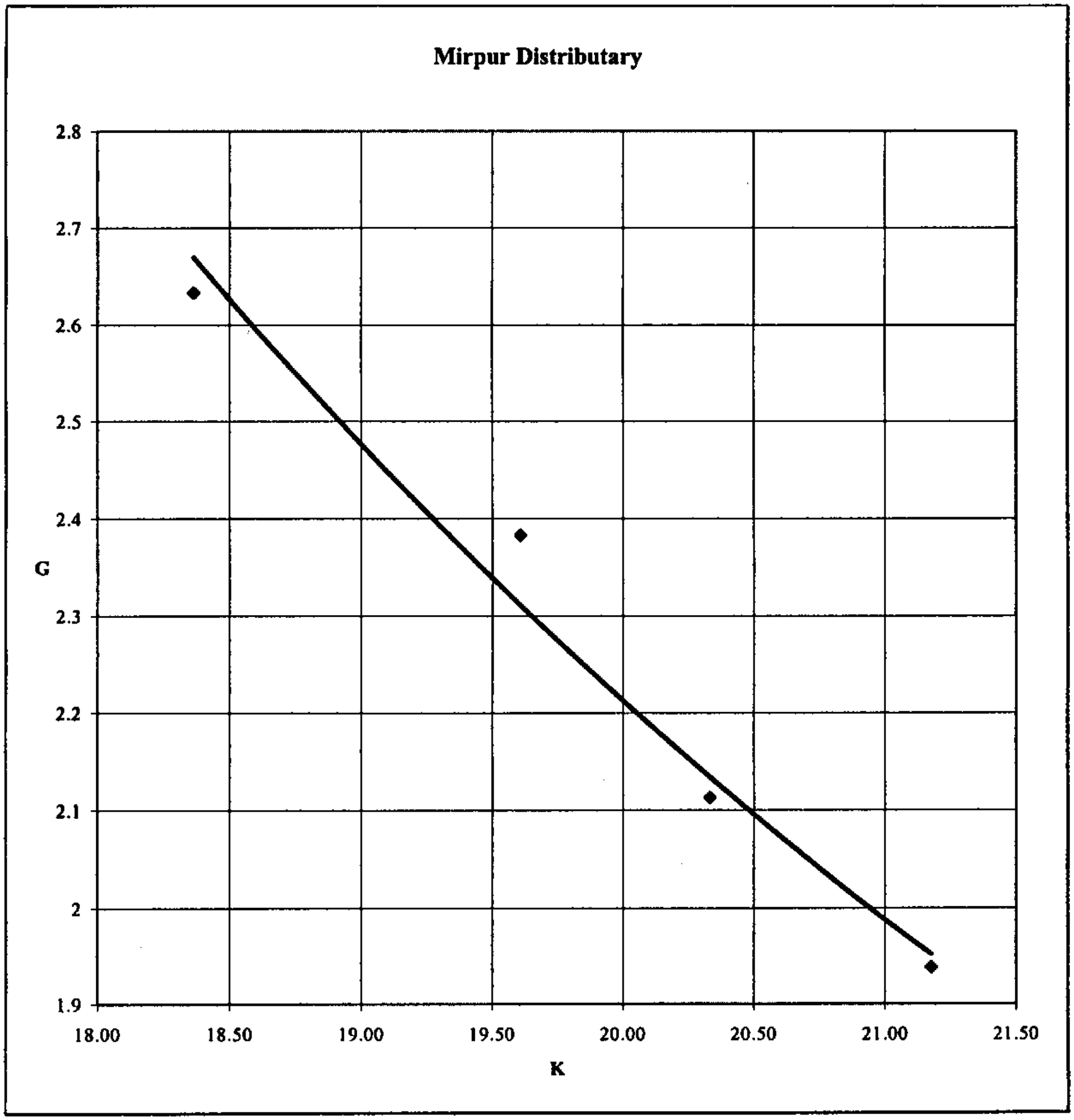


Figure 3.4. Variation of the K coefficient with the gauge reading for Mirpur Distributary.

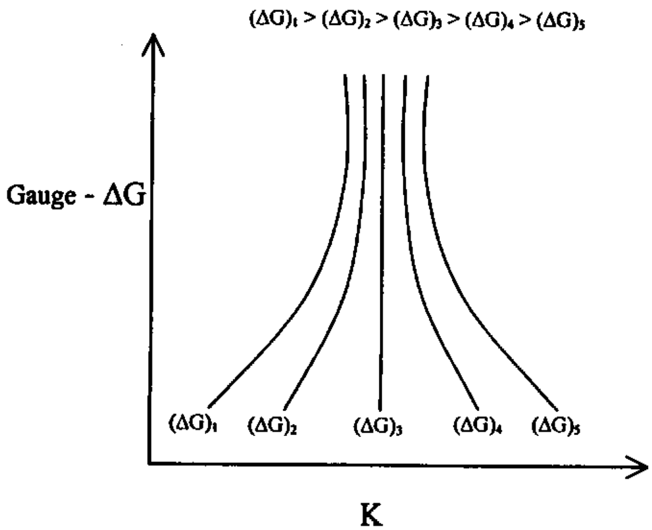


Figure 3.5. Schematic of variation in relationships between K and G – ΔG by introducing different gauge corrections.

Figures 3.3 and 3.4 show the graphical relationships between K and G without introducing any gauge correction factor. The trend in variation of each curve can be easily seen from these graphs; one important point worth mentioning here is that the curve can be either negatively or positively sloped. As the Hakra 6-R and Mirpur distributaries are inversely sloped, so these two canals were selected for developing a general technique. Different gauge corrections were employed for both of the canals and for each ΔG value, a G-ΔG and K curve was drawn. The gauge correction (ΔG), yielding the best straight vertical line was selected for identifying the values of both ΔG and K for use in the KD formula (Equation 1.5). The results of the exercise are presented in Table 3.5 and the variation of the curves by the introduction of different gauge corrections are shown as Figures 3.6 and 3.7.

The following conclusions can be drawn by looking at the results:

- The results of the approach using unique values of K from the G~K curve are comparable with the ΔG and K variable approach; and
- The negative or positive slope of a G~K curve indicates the sign of the gauge correction (i.e. for a negatively sloped G~K curve, the gauge correction is negative thus resulting in the D value being higher than value of G, and vice versa for a positively sloped G~K curve).

If all of the three approaches are compared using $n=5/3$, the following statements can be made:

- ➢ that K varies with the variation of the discharge and the hydraulic depth doesn't give a good estimate of ΔG for use with $n=5/3$.
- ➢ As the relationship of G with the depth is still unknown, so the idea of finding ΔG by trial and error is good and yields an accurate discharge rating.
- ➢ The only drawback in the technique is that it involves a lot of calculations and graphs.

Table 3.5. Comparison of approaches using n=5/3 for selected channels.

Comparison of approaches for which n=5/3 is used																	
Basic Data			**K Variable Approach**					**Using K obtained from G~K Curve**					**Delta G & K variable Approach**				
Hakra 6-R Disty			**K avg=43.17**					**$K=16.915G^{0.6204}$**					**$\Delta G=$**	1.324		**K Avg=**	63.1971
G	**Q**	**n**	**K**	**$(G-\Delta G_{hyd})$**	**Q cal**	**Q-Q cal**	**% Diff**	**K**	**$(G-\Delta G_{hyd})$**	**Q cal**	**Q-Q cal**	**% Diff**	**K**	**D**	**Qcal**	**Q-Qcal**	**%Error**
2.7	108	1.67	31.06	2.11	150	-42	-39.0	31.32	2.11	109	-1	-0.8	63.35	1.38	108	0	0.2
4.54	440	1.67	44.62	3.95	426	14	3.2	43.24	3.95	427	14	3.1	62.85	3.22	443	-2	-0.6
5.46	664	1.67	47.46	4.87	604	60	9.0	48.49	4.87	678	-14	-2.2	62.31	4.14	674	-9	-1.4
5.67	744	1.67	49.55	5.08	649	96	12.9	49.64	5.08	745	-1	-0.2	64.27	4.35	731	12	1.7
Mirpur Distributary			**K avg=19.87**					**$K=28.46G^{-0.4444}$**					**$\Delta G=$**	-0.841		**K Avg=**	11.7593
G	**Q**	**n**	**K**	**$(G-\Delta G_{hyd})$**	**Q cal**	**Q-Q cal**	**% Diff**	**K**	**$(G-\Delta G_{hyd})$**	**Q cal**	**Q-Q cal**	**% Diff**	**K**	**D**	**Qcal**	**Q-Qcal**	**%Error**
1.94	64	1.67	21.18	1.95	60	4	6.2	21.21	1.95	64	-0	-0.2	11.72	3	65	-0	-0.3
2.11	71	1.67	20.33	2.12	70	2	2.3	20.41	2.12	72	-0	-0.4	11.73	3	72	-0	-0.2
2.38	84	1.67	19.61	2.39	85	-1	-1.3	19.35	2.39	83	1	1.3	11.94	3	83	1	1.5
2.63	93	1.67	18.36	2.64	100	-8	-8.2	18.51	2.64	94	-1	-0.8	11.65	3	94	-0	-1.0

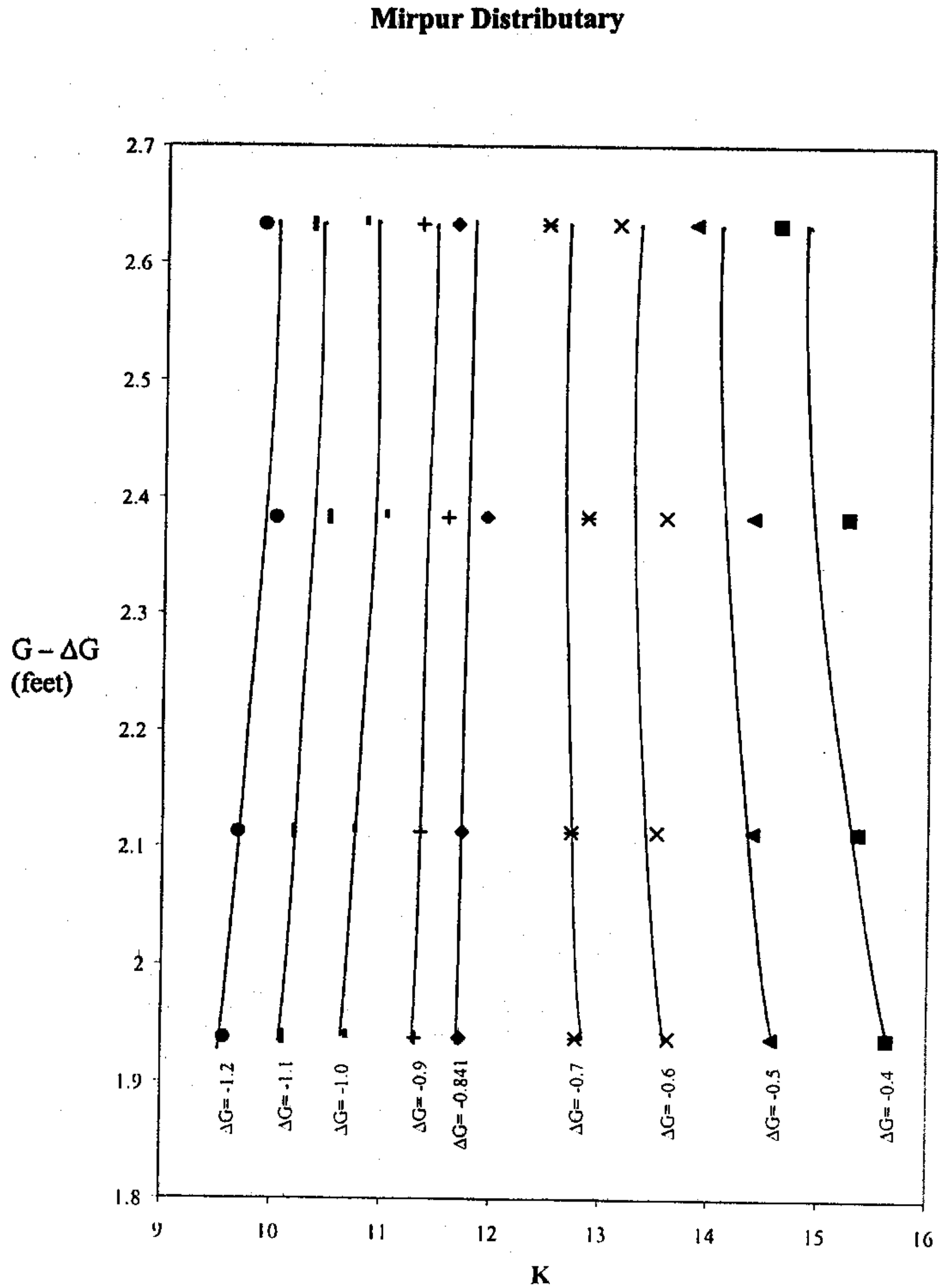


Figure 3.6. Curve variations by introducing different gauge corrections for Mirpur Distributary.

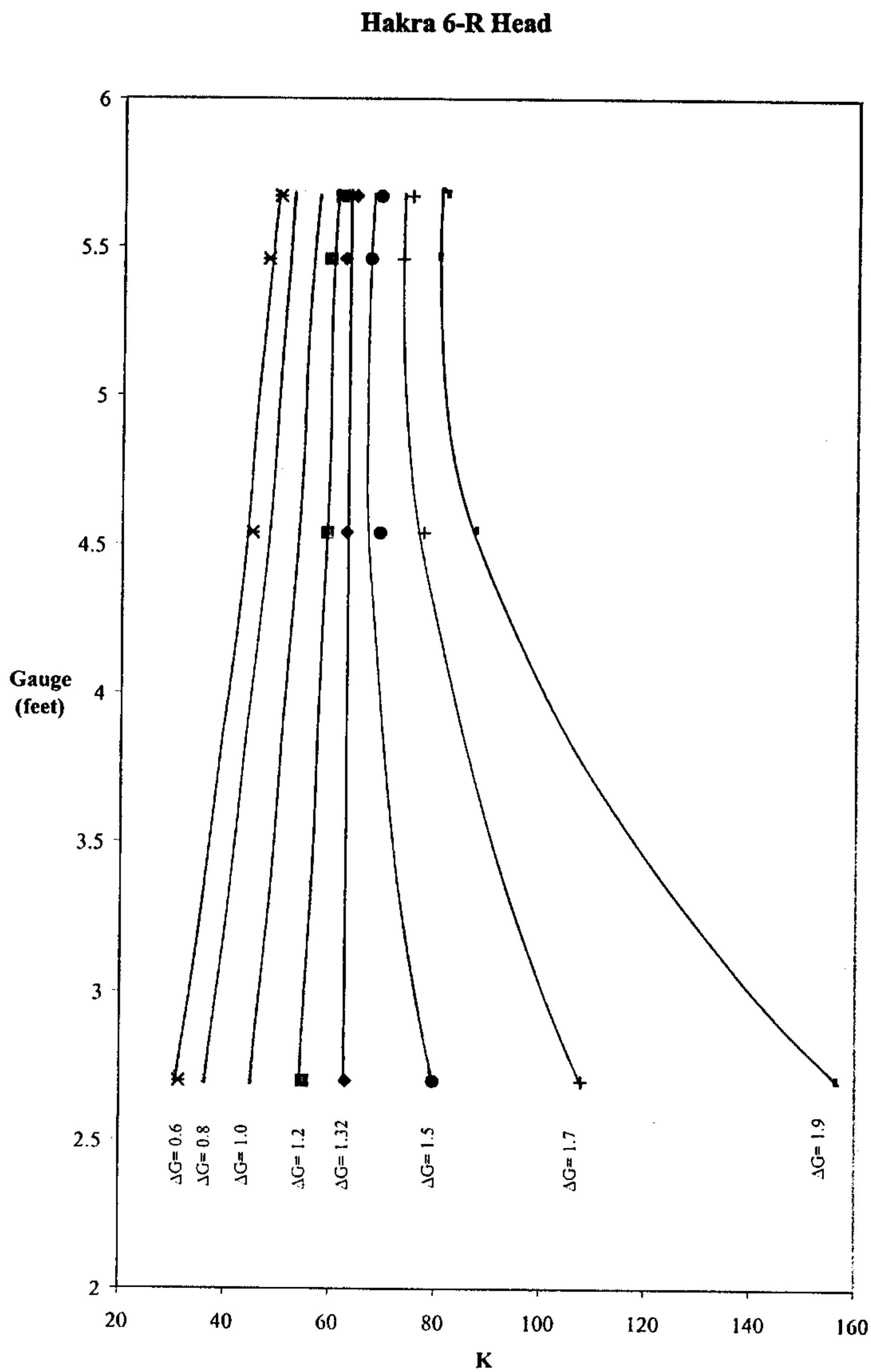


Figure 3.7. Curve variations by introducing different gauage corrections for Hakra 6-R Distributary.

4. DOWNSTREAM GAUGE RATING USING TWO VARIABLES APPROACH

In this section, the two variables approach will be evaluated. The approach will be used in two different ways by making two different choices for the set of two variables in the formula $Q=K(G+\Delta G)^n$. In one set, the variables K and n will be used, while ΔG is established by using the hydraulic depth. In the second case, the two variables will be K and ΔG, with the value of n being fixed as 5/3. The results will then be compared for accuracy and for any trend, or possible correlation, between the different variables. The two approaches can be explained by the table below.

Table 4.1. Two approaches for evaluating two variables for downstream gauge ratings.

	n	K	ΔG
K & n Variable	Variable	Variable	Constant=Average ΔG, from Discharge Observation
K & ΔG Variable	Constant=5/3	Variable	Variable

4.1. APPROACH WITH K & n AS VARIABLES

4.1.1. Introduction

The two variables approach in this form is most popular and is taken as a standard procedure for developing the discharge tables in the Irrigation and Drainage Authorities of Pakistan. The procedure as listed in the Manual of Irrigation Practice is:

> The General method employed is that a series of discharges are to be observed at round about steady full, three quarters, half and one quarter full supply. For each series of observations, one mean value of "Q", the gauge reading "G" and the area of waterway is to be observed . Any obvious erroneous observation should be rejected. "D" the mean water depth is then worked out by dividing the area by mean width, and from the four values of "Q" and "D" thus arrived at, the values of K and n in the equation $Q=KD^n$ is determined by the method of least squares.

4.1.2. Calculation Steps

The data required for the two variables (K & n) approach is discharge, gauge readings, area of waterway and top surface width for each discharge measurement. The calculation procedure is:

a) By dividing the area of waterway by the top surface width, the Hydraulic Mean Depth can be calculated, which corresponds with the average depth of flow;
b) The Hydraulic Mean Depth was subtracted from the Gauge Reading to determine the gauge correction, ΔG, which is illustrated in Figure 4.1;

c) ΔG was calculated for all of the discharge measurements and then averaged to arrive at a single value of gauge correction; and

d) By subtracting the average ΔG from each gauge reading, G, the depth of flow "D" can be calculated for each discharge measurement.

As the equation format is $Q=KD^n$, the unknowns "n" & "K" can be calculated by regression or determined using a graphical method.

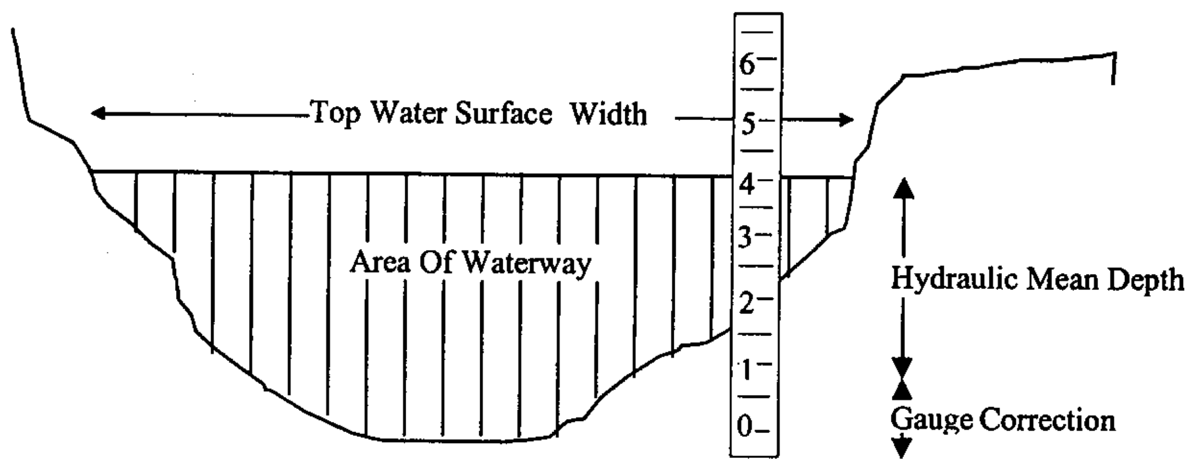


Figure 4.1. Illustration of using the hydraulic mean depth to derive the gauge correction.

4.1.2.1. Regression Analysis (Ordinary Least Squares)

Once the value of D is calculated , as explained above, the values of the variables n & K can be found by the method of least squares. For this, some mathematics are applied for reaching a solution.

The equation is

$$Q = KD^n \tag{1.2}$$

The equation can be rewritten after applying logarithms as

$$\text{Log } Q = \text{Log } K + n \text{ Log } D \qquad (2.8)$$

which is a straight line equation, with a slope n and the Y intercept at log K. For the solution of these types of equations, a data set of two or more observations is required. A larger number of observations will more likely result in a more accurate solution.

Let number of observations = T

The solution for n is

$$n = \frac{Sxy}{Sxx} \qquad (4.1)$$

where

$$Sxy = \left[\sum (LogD \times LogQ) - \frac{(\sum LogD)(\sum LogQ)}{T} \right] \qquad (4.2)$$

and

$$Sxx = \left[\sum (LogD)^2 - \frac{(\sum LogD)^2}{T} \right] \qquad (4.3)$$

Once the value of "n" is calculated, the value of "K " can be found by placing the value of n in Equation 4.4.

$$\text{Log } K = \text{Average Log } Q - n \times \text{Average Log } D \qquad (4.4)$$

4.1.2.2. Graphical Solution

The solution for the stage discharge relationship can also be determined through graphical means. Equation 2.8, Log Q = Log K + n Log D, is a general form of a straight line equation with the slope "n" and the Y intercept on the ordinate corresponding with log D = 0 (D = 1) is "Log K" as shown in Figure 4.2.

So, by plotting, Log Q on the Y axis and log D on the X axis, the values of n and K can be calculated from the graph as n is the slope of the line and K is the anti-log of the Y Intercept ($10^{(\text{Y Intercept})}$) at D = 1 (log D = 0).

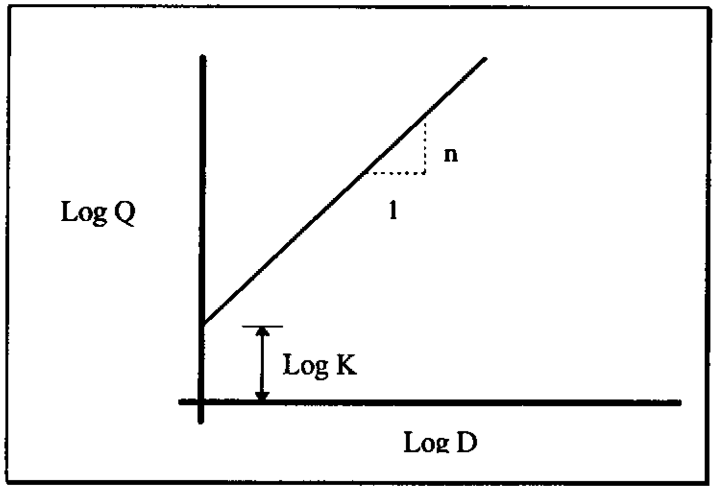


Figure 4.2. Illustration of graphical solution for the two variables K & n.

4.2. APPROACH WITH K & ΔG AS VARIABLES

In the second of the two variable approaches, the idea is to keep the n- slope of the Log D and Log Q line equal to 5/3 and vary ΔG so that the K values are determined with a minimum standard deviation, resulting in the rating curve which will best fit the measured discharge values. Although, for this approach, ΔG can be solved by taking any of two discharge observations as there are two unknowns, so only two equations are required. But it is not necessary that the considered points will lie on the mean curve line; so, for obtaining a more accurate solution, all of the discharge observations (excluding the obviously wrong discharge observations) were considered and ΔG was calculated by using an iterative least squares method.

4.3. COMPARISON OF APPROACHES

In the second approach used for arriving at a stage-discharge relationship, in which only K and ΔG are taken as the variables, the value of n is fixed at a value of 5/3. The idea behind the approach is, on the one hand, to standardize the values of the exponent and, on the other hand, to maintain the accuracy within reasonable limits. The approach is then compared with the standard procedures used in the Irrigation Departments (i.e. the first approach in which a series of the discharge measurements were used to arrive at the values of n & K).

As shown in Figure 4.3, the first approach (K & ΔG variable), fixes the slope of the line to be "n", but in actuality, i, ii & iii are the points representing the actual observations in the field .The first approach line will necessarily be at a slope n, whereas the average line, achieved by the second two variables approach, could be at any other slope n. The controlling factors in opting for one of the approaches would be the accuracy and simplicity in the procedures.

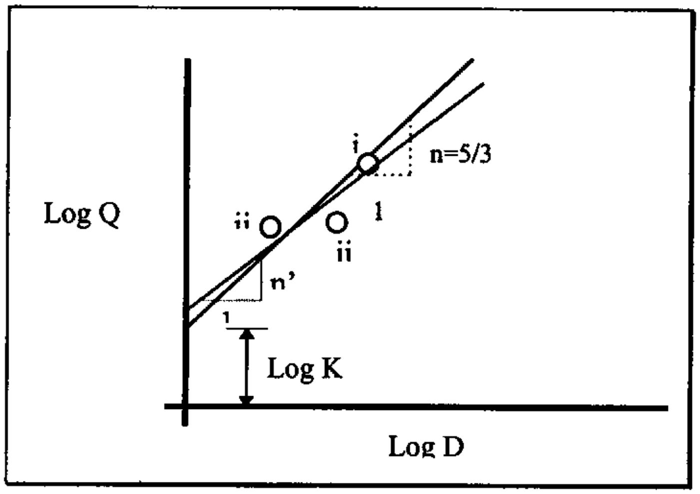


Figure 4.3. Graphical illustration of K & ΔG as variables with n = 5/3.

In the K & n as variable approach, and the K & ΔG as variable approach, the resulting ΔG values differ. The value in the first case is calculated by using the mean hydraulic depth, which represents the actual physical situation (more or less) in the field, whereas in the other case, ΔG is considered as a variable for achieving a best fit line. So, the value of the gauge correction sometimes deviates significantly between the two approaches, which needs to be investigated.

Table 4.2. Comparison of gauge corrections from the two variables approaches using some example irrigation channels.

Canal Name	Authorized Discharge (Cusecs)	Two Variables Approaches	
		n & K variable and ΔG from hyd depth	K & ΔG variable and n=5/3
		ΔG (Ft)	ΔG (Ft)
Geodi Minor	35	0.09	0.18
Daulatpur Minor	49	1.17	0.07
Mirpur Distributary	64	0.59	-0.84
Khan Mahi Branch	103	-0.41	1.24
6-R Hakra Disty	459	0.98	1.32
Malik Branch	1538	1.48	1.70
Lower Sawat Canal	1940	0.40	0.77
Hakra Branch	2785	-0.01	1.69

The values of the gauge correction resulting from considering ΔG as variable are lower as compared to the values resulting from considering the hydraulic depth for calculating the gauge correction for the same canal.

Looking at Table 4.2, the values of gauge correction are quite in concurrence with each other except in the case of Daulatpur Minor, where the difference in gauge correction values is quite significant as compared to the size of canal. Although the difference in gauge correction is even more for the case of Hakra Branch Canal, but from a physical acceptability point of view, this difference is not much for a 150 feet wide canal.

The variation of n values is also very interesting. A comparison of the KD formula and discharge formula through triangular and rectangular notches, with the assumption that the channel shape varies from triangular to rectangular, then the value of n should vary between 3/2 to 5/2 depending on the shape of the cross section at the site. In one of the two variables approaches, the value of n was fixed at 5/3 (i.e. 1.67), which is most of the time considered as best suiting the canal shapes in regime. In the other case, the value of n was a variable to best describe the flow in the canal at different gauge readings. The results show that the average value of n for the considered canals is 1.79, while two values of 1.43 for Daulatpur Minor and 1.22 for the Mirpur Disty are below 1.5. But, it is very difficult to establish any relationship between the value of n and the size or shape of the channel, which strengthens the idea that the gauge correction factor has influence over the value of n. The results are shown in the table below (Table 4.3).

Table 4.3. Comparison of exponents from the two variables approaches using some example irrigation channels.

Canal Name	Authorized Discharge (Cusecs)	Two Variables Approaches	
		n & K variable and ΔG from hyd Depth	K & ΔG variable and n=5/3
		n	n
Geodi Minor	35	1.87	1.67
Daulatpur Minor	49	1.43	1.67
Mirpur Distributary	64	1.22	1.67
Khan Mahi Branch	103	1.76	1.67
6-R Hakra Disty	459	2.19	1.67
Malik Branch	1538	2.41	1.67
Lower Swat Canal	1940	1.56	1.67
Hakra Branch	2785	1.85	1.67

As referred in the above discussion , the factors important for developing a good canal rating using the KD formula are that the formula should be serviceable (i.e. the gauge correction and n values should be in concurrence with the physical situation of the canal); secondly, the rating should be accurate enough; and thirdly, the method for developing the rating should not be complex.

Table 4.4. Comparison of discharge rating accuracy from the two variables approaches using some example irrigation channels.

Canal Name	Discharge (Cusecs)	Two Variables Approaches					
		n & K variable and ΔG from hyd Depth			K & ΔG variable and n=5/3		
		Average Difference (Cusecs)	% Error [(Qcal-Qm)/Qm]*100		Average Difference (Cusecs)	% Error [(Qcal-Qm)/Qm]*100	
			Maximum	Minimum		Maximum	Minimum
Geodi Minor	35	0.66	11.41	0.37	0.29	7.43	0.05
Daulatpur Minor	49	0.2	0.94	0.18	0.16	0.9	0
Mirpur Distributary	64	0.56	1.32	0.16	0.63	1.49	0.24
Khan Mahi Branch	103	1.83	5.72	0.06	1.48	6.13	0.91
6-R Hakra Disty	459	6.68	2.68	0.04	6.13	1.6	0.19
Malik Branch	1538	3.81	0.49	0.18	54.04	6.44	0.24
Lower Swat Canal	1940	18.91	4.49	0.99	20.36	0.05	0.01
Hakra Branch	2785	5.09	0.38	0.08	56.29	3.54	1.76

The above table (Table 4.4.) reveals the results from the second approach, that is, considering K and n as variables, which are more accurate as compared with the approach considering K & ΔG as variables. This difference in accuracy is quite significant in the case of Malik and Hakra head regulators, where the average difference in actual and calculated discharges is even more than 10 times the difference in the case of using K and n as variables, but for the rest of the canals, the accuracy is not much different.

As much as the simplicity of the method is considered, the K and n variable approach can be adopted by involving the regression formulae, whereas the K & ΔG variable approach is iterative so that more work is involved with the calculations.

5. DOWNSTREAM GAUGE RATING USING THREE VARIABLES APPROACH

5.1. INTRODUCTION

All of the previous approaches were more or less based on hydraulic principals for justification – one variable was linked with Maning's equation, which can be reduced to the form $Q=KD^n$, as explained in section two of this report; likewise, one of the two variables approaches was also linked to the same principle and the second was with the assumption that the hydraulic depth is equal to the average depth of flow. But in the three variables approach, the hydraulic principles have been set aside and regression means (curve fitting) have been used to arrive at an accurate rating curve. All of the three factors (i.e. K, D and n) have been assumed as varying; by trying different values of ΔG, the values of n and K have been determined for an accurate rating.

5.2. CALCULATION STEPS

For arriving at the best ratings, the method of least squares have been adopted. The value of Gauge Correction (ΔG) was varied and values of n and K were calculated by regression for each value of ΔG. The discharge was calculated for each value of ΔG or D and the difference of calculated and measured discharges was calculated and squared. The values of ΔG, K and n giving the least value of $(Q_{measued}-Q_{calculated})^2$ were used as discharge rating parameters. The same variation of $(Q_{measued}-Q_{calculated})^2$ can be explained from Figure 5.1, which shows that with an increase in ΔG, the value of $(Q_{measued}-Q_{calculated})^2$ decreases and after reaching a minimum value, $(Q_{measued}-Q_{calculated})^2$ starts increasing with an increase in the value of ΔG. The lowest point on the curve shows that the values of the three variables are in the best combination for fitting the mean observed discharge line on the rating curve.

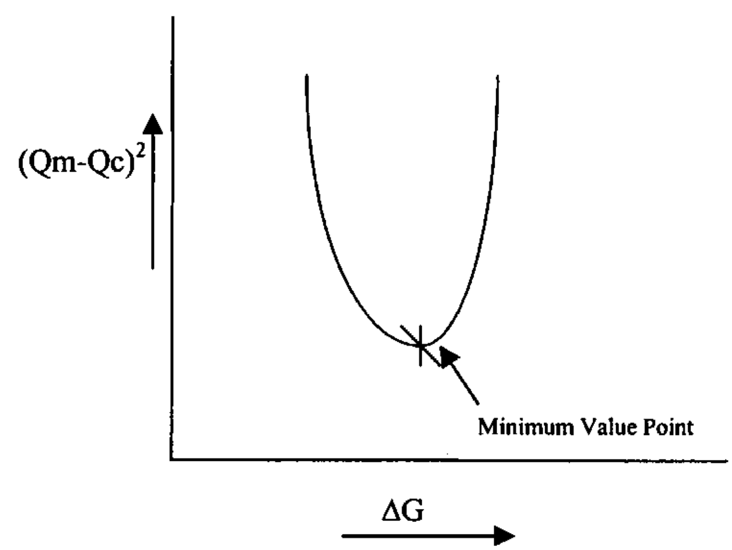


Figure 5.1. Graphical illustration of determining the gauge correction that will minimize the inaccuracy of the discharge rating.

5.3. DOWNSTREAM GAUGE RATING PARAMETERS

In order to reach a sound conclusion, field discharge observations have covered a wide range of measurements depending upon the size of the channel i.e. from less than 3 cusecs (Geodi Minor) to 2296 cusecs (Hakra Branch Canal).

Table 5.1. Variation of Downstream Gauge Rating Parameters.

Canal Name	Authorized Discharge (Cusecs)	Three Variables Approach			Two Variables Approach	
		K, n & ΔG Variables Approach			From Hydraulic Depth	
		n	K	ΔG (Ft)	ΔG (Ft)	n
Geodi Minor	35	1.57	15.79	0.22	0.09	1.87
Daulatpur Minor	49	2.08	4.52	-0.5	0.4	1.43
Mirpur Distributary	64	0.63	70.13	1.07	-0.01	1.22
Khan Mahi Branch	103	1.59	18.82	1.29	1.17	1.76
Hakra 6-R Disty	459	1.83	45.7	1.1	0.59	2.19
Malik Branch	1538	2.07	36.67	0.5	-0.41	2.41
Lower Swat Canal	1940	1.16	229.79	1.75	0.98	1.56
Hakra Branch	2785	1.47	180.65	2.57	1.48	1.85

By looking through the final values of n, K & ΔG in the order of increasing discharge, it is very difficult to conclude anything. The other point worth mentioning is that the value of n (i.e. the exponent in the KD formula) is usually assumed to range from 1.5 to 2.5 is also not very valid in this case. The variation of the n value in this case is from 0.63 to 2.08. The same way of looking at values of K resulting from the adoption of the approach gives a weak increasing trend of K values with the increasing discharge. These unusual values can easily be associated with the fact that this approach is clearly looking for a precise or accurate solution without considering any physical requirements from the field or hydraulics. The errors or inaccuracies of the discharge observations are incorporated in the approach at the cost of the norms for the values of rating parameters.

Looking at the gauge correction (ΔG) values for the three variables approach (which as mentioned before, is part of the best combination of the rating parameters), the values calculated for the same canal by using the hydraulic depth are different. The minimum difference between corresponding ΔG values is 0.12 in the case of Khan Mahi Branch Canal and the maximum value for this difference is 1.09, which is quite large and may make the method less trustworthy.

5.4. DISCHARGE ACCURACY VARIATION OF THREE VARIABLES APROACH

The strongest point for the three variables approach is that its objective is to derive the most accurate rating, which can be well seen in Tables 5.2 and 5.3. The maximum average difference is 14.25 cusecs in the case of Lower Swat Canal, which is less than 1% of the authorized discharge for this canal. Looking at the % error in calculated and measure discharges, although 6.61 looks quite high, but actually this difference is in the low flow range which can be seen from Table 5.2. Apparently, there is no trend or correlation between the discharge or size of canal and the accuracy. And even for the same canal, the same is the trend.

Table 5.2. Discharge accuracy variation for three variables approach over a range of discharges for the example irrigation channels.

Three Variables Approach							
Hakra 6-R Distributary, Hakra Branch Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
2.70	107.84	1.60	0.20	2.03	108.01	0.03	-0.16
4.54	440.41	3.44	0.54	2.64	438.35	4.26	0.47
5.46	664.04	4.36	0.64	2.82	676.36	151.73	-1.85
5.67	743.91	4.57	0.66	2.87	737.16	45.54	0.91
Head Hakra Branch Canal, Eastern Sadiqia Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
8.20	2296.06	5.63	0.75	3.36	2295.98	0.01	0.00
7.62	1956.52	5.05	0.70	3.29	1956.61	0.01	0.00
5.91	1065.04	3.34	0.52	3.03	1065.03	0.00	0.00
Head Malik Branch Canal, Eastern Sadiqia Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
4.85	766.95	4.35	0.64	2.88	767.02	0.00	-0.01
5.67	1096.42	5.17	0.71	3.04	1096.27	0.02	0.01
7.38	1979.44	6.88	0.84	3.30	1979.54	0.01	-0.01
Geodi Minor Canal, Lower Swat Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
0.52	2.25	0.30	-0.52	0.35	2.37	0.01	-5.42
0.52	2.54	0.30	-0.52	0.40	2.37	0.03	6.61
0.86	7.77	0.64	-0.19	0.89	7.82	0.00	-0.67
0.86	7.70	0.64	-0.19	0.89	7.82	0.01	-1.58
1.08	12.41	0.86	-0.07	1.09	12.46	0.00	-0.37
1.08	12.20	0.86	-0.07	1.09	12.46	0.07	-2.10
1.25	16.74	1.03	0.01	1.22	16.55	0.04	1.15
1.28	17.32	1.06	0.03	1.24	17.31	0.00	0.04
1.62	27.43	1.40	0.15	1.44	26.83	0.36	2.18
1.62	26.90	1.40	0.15	1.43	26.83	0.00	0.26
2.10	42.66	1.88	0.27	1.63	42.68	0.00	-0.05
2.10	42.47	1.88	0.27	1.63	42.68	0.05	-0.50
Khan Mahi Branch Canal, Lower Swat Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
1.78	5.95	0.49	-0.31	0.77	6.04	0.01	-1.49
1.78	6.19	0.49	-0.31	0.79	6.04	0.02	2.44
2.38	21.19	1.09	0.04	1.33	21.60	0.16	-1.91
2.38	21.60	1.09	0.04	1.33	21.60	0.00	0.02
2.70	33.19	1.41	0.15	1.52	32.55	0.41	1.93
2.72	34.39	1.43	0.16	1.54	33.29	1.22	3.21
3.05	43.37	1.76	0.25	1.64	46.34	8.85	-6.86
3.05	44.76	1.76	0.25	1.65	46.34	2.51	-3.54
3.73	80.94	2.44	0.39	1.91	78.00	8.62	3.63
3.73	80.94	2.44	0.39	1.91	78.00	8.62	3.63
4.44	117.63	3.15	0.50	2.07	117.19	0.19	0.37
4.44	114.90	3.15	0.50	2.06	117.19	5.26	-2.00
Lower Swat Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
3.01	302.34	1.26	0.10	2.48	300.18	4.67	0.72
3.01	302.15	1.26	0.10	2.48	300.18	3.89	0.65
3.54	442.72	1.79	0.25	2.65	450.47	60.14	-1.75
4.49	737.53	2.74	0.44	2.87	736.95	0.34	0.08
4.96	856.28	3.21	0.51	2.93	884.97	823.13	-3.35
5.46	1073.12	3.71	0.57	3.03	1046.20	724.77	2.51
5.49	1083.03	3.74	0.57	3.03	1055.99	731.42	2.50
6.16	1258.74	4.41	0.64	3.10	1277.61	356.21	-1.50
Mirpur Distributary, Jamrao Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
1.94	64.38	0.87	-0.06	1.81	64.11	0.07	0.42
2.11	71.34	1.04	0.02	1.85	72.02	0.46	-0.95
2.38	83.99	1.31	0.12	1.92	83.33	0.43	0.78
2.63	92.82	1.56	0.19	1.97	93.06	0.06	-0.26
Daulatpur Minor, Jamrao Canal							
G	Q	D	Log D	Log Qm	Pred Q	(Qm-Qp)2	% Diff
2.08	32.64	2.58	0.41	1.51	32.38	0.07	0.81
2.09	32.36	2.59	0.41	1.51	32.64	0.08	-0.86
2.35	39.83	2.85	0.45	1.60	39.81	0.00	0.05
2.73	51.62	3.23	0.51	1.71	51.62	0.00	-0.01

Table 5.3. Discharge accuracy variation of the Three Variables Approach.

Canal Name	Authorised Discharge (Cusecs)	Three Variables Approach					
		n, K & ΔG Variables					
		n	K	ΔG (Ft)	Average Difference (Cusecs)	% Error [(Qcal-Qm)/Qm]*100	
						Maximum	Minimum
Geodi Minor	35	1.57	15.79	0.22	0.16	6.61	0.04
Daulatpur Minor	49	2.08	4.52	-0.5	0.14	0.86	0.01
Mirpur Distributary	64	0.63	70.13	1.07	0.46	0.95	0.26
Khan Mahi Branch	103	1.59	18.82	1.29	1.3	6.86	0.02
Hakra 6-R Disty	459	1.83	45.7	1.1	5.32	1.85	0.16
Malik Branch	1538	2.07	36.67	0.5	0.11	0.01	0.01
Lower Swat Canal	1940	1.16	229.79	1.75	14.25	3.35	0.08
Hakra Branch	2785	1.47	180.65	2.57	0.06	0.00	0.00

6. REVIEW OF GAUGE CORRECTIONS AND THEIR RELATION WITH CHANNEL CROSS-SECTION DATA

6.1. INTRODUCTION

Various methodologies have been presented to develop an appropriate KD-relationship for a downstream gauge rating. These methodologies have all been developed to derive an accurate rating, which falls close to the actual measured discharges.

In this section, the relationship is analyzed between, on the one side, the gauge readings and the different depth parameters used for developing a rating and, on the other side, the values of the coefficient K and the exponent n in the derived KD-relationship.

An important aspect to consider in the preparation of a downstream gauge rating is how the gauge reading is corrected to obtain a depth parameter which reflects the physical characteristics of the channel cross-section. It would be very convenient if gauge corrections used for adjusting gauge readings to provide depth, D are directly related to the channel cross-section. In case the bed level would change, this gauge correction would then be adjusted to the new difference between the bed and the gauge.

6.2. GAUGE READING AND DEPTH PARAMETER

In a downstream gauge rating, a relationship is developed between the gauge reading, which reflects the water level, and the discharge running through the channel cross-section. This relationship needs to be a unique one. Only then can it be described in the following form:

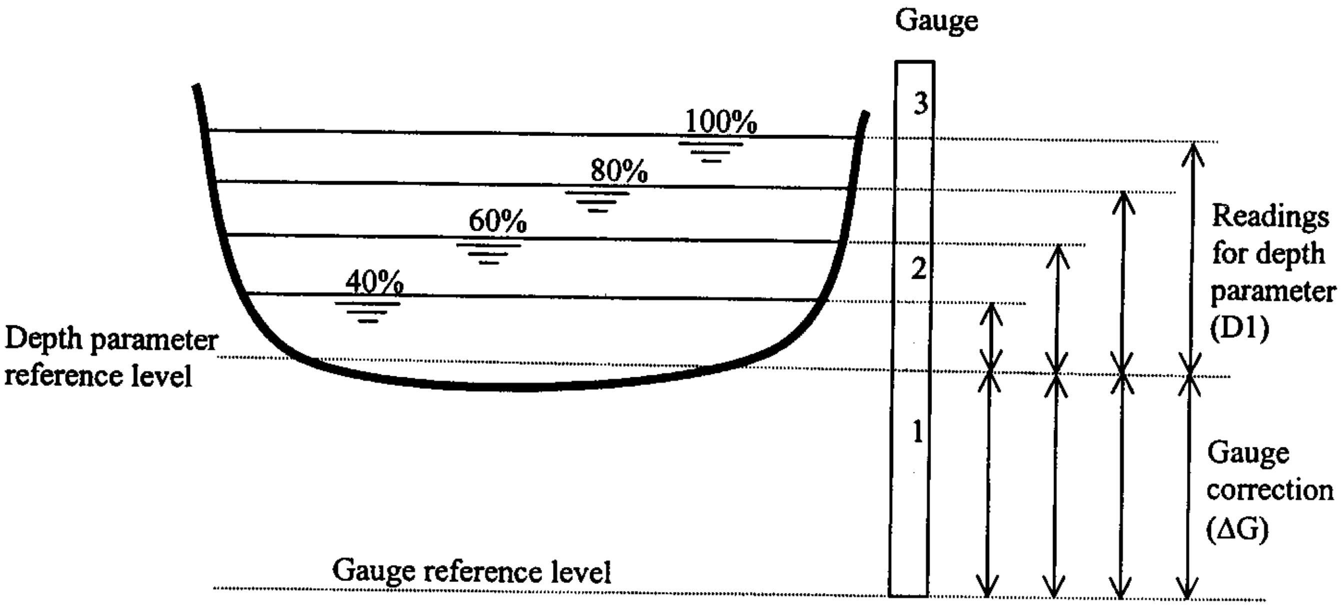
$$Q = K \ (G - \Delta G)^{n} \qquad (1.5)$$

For a gauge reading, the reference level is the gauge zero point. The gauge zero point should reflect the channel bed level at the time of design of the channel. However, sediment deposition and scouring often change the bed level over time.

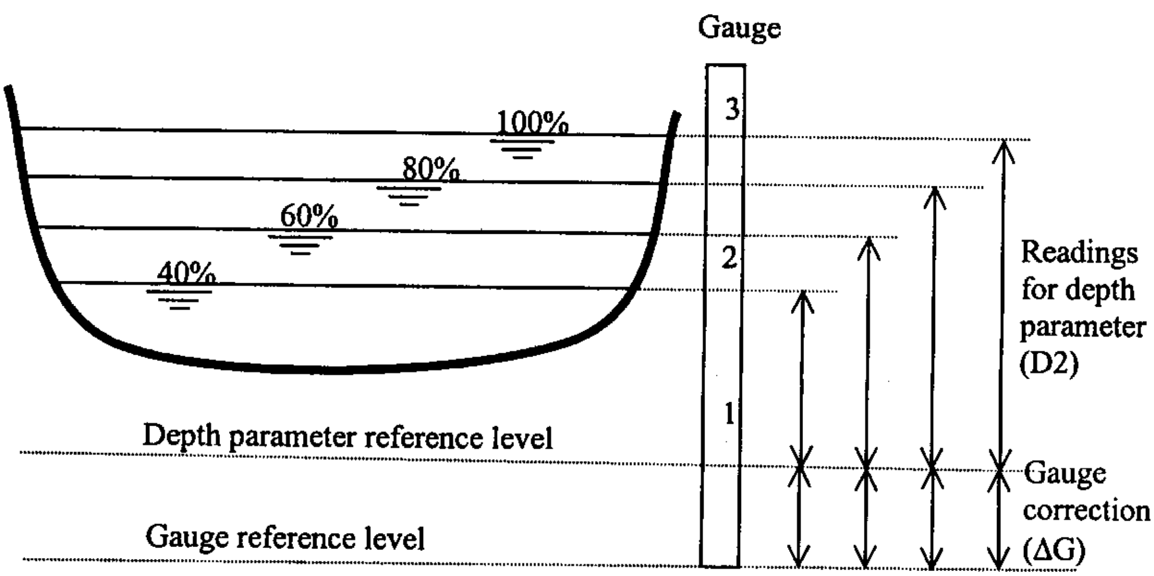
The gauge correction is nothing more than a way to change the reference level for a gauge reading from the gauge zero point to another elevation. The result of using a new reference level for the gauge reading is a change of the depth parameter, D, in which D is:

$$D = G - \Delta G \qquad (1.3)$$

Figures 6.1a and b reflect the use of two different depth parameters (D1 and D2) in the preparation of a downstream gauge discharge relation. In both cases, the four discharge stages and their respective gauge readings are the same. What has changed is the reference level for the depth parameter, which basically means a change in the gauge correction, ΔG. The result is a difference in the values of D1 and D2 for each of the discharge stages.



(a) Use of gauge correction for depth parameter, D1, lying above the channel bed level.



(b) Use of gauge correction for depth parameter, D2, lying below the channel bed level.

Figure 6.1. Gauge corrections above and below the channel bed level .

Figure 6.2 shows the implications of the different values for D1 and D2 on the downstream gauge rating. The double-log plot of the depth parameter value against discharge gives two different ratings with differing slopes and Y-axis intercepts. Both ratings seem to fit the measured discharge points quite well. As shown in a previous section of this report, the slope of the rating line depicts the value of the exponent, n, in the KD-formula, while the Y-axis intercept gives the value of the coefficient K. Thus, a change in gauge correction changes the depth parameter, which then results in a change of exponent, n, and the coefficient, K.

Later, in this section, it will be shown that a rating can be developed for any reference level or gauge correction. With a certain gauge correction, there will always be a coefficient, K, and exponent, n, to fit the data. Only, the question arises which combination of gauge correction, ΔG, exponent, n, and coefficient, K, gives a combination of the best and easiest obtainable rating.

6.3. GAUGE CORRECTIONS THAT REFLECT CHANNEL CROSS-SECTION DATA

The idea behind the use of a gauge correction is to apply a depth parameter that has a physical meaning with regard to the channel cross-section. Two approaches for achieving this will be evaluated. One is used to obtain a downstream gauge rating which comes close to the Manning-Strickler formula. The other approach tries to obtain a depth parameter which comes close to the actual average depth of the channel and thus uses a gauge correction, which represents the channel bed level.

6.3.1. Gauge Correction From Hydraulic Mean Depth

In the second section of this report, the background of the KD formula was explained in relation to the equation of Manning-Strickler. Originally, Manning-Strickler uses the hydraulic radius as a depth parameter and, in that case, the exponent n in the KD formula should have the value 5/3. Assuming that the equation of Manning-Strickler best describes the flow through a channel section, it is preferable that the parameter values in the KD formula are related to those in the Manning-Strickler equation. Of course, there are some practical restrictions in choosing the parameter values in the KD formula. One restriction is that the parameter values should be easily derived from measurement data. The other restriction is that, unlike the Manning-Strickler formula, the KD formula needs to relate to a fixed gauge.

With the first restriction in mind, the hydraulic radius is replaced by the hydraulic mean depth. A series of discharge measurements consist of about steady full supply (100%), as well as 80%, 60% and 40% of full supply. For each series of observations, the discharge, Q, downstream gauge, G, and area, A, of the waterway are calculated after rejecting any obviously erroneous observations. Where hydraulic radius requires the wetted perimeter and the total area of the channel cross-section, the hydraulic (mean) depth is calculated with the use of the area of the channel cross-section, A, and the top surface width, W_T. The hydraulic mean depth, denoted by D_{hy}, is expressed as:

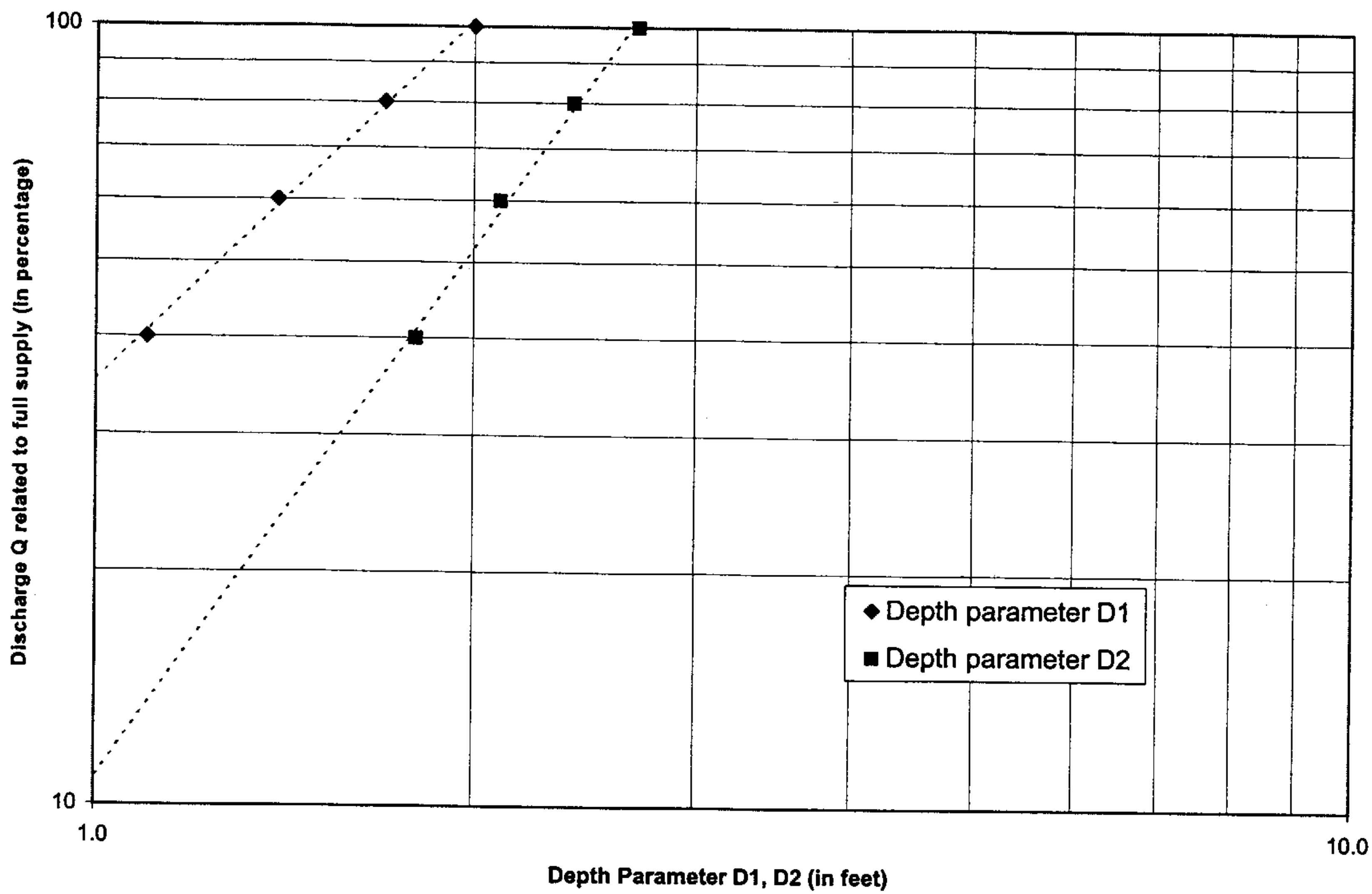


Figure 6.2. Relation between Discharge Q and Depth Parameter D1, D2.

$$D_{hy} = \frac{A}{(W_T)} \tag{2.10}$$

where

D_{hy} = hydraulic mean depth;
A = cross-sectional area of flow; and
W_T = top width of the water surface.

The top width of the water surface, W_T, is easily obtained from a tag line which locates the edge of the water surface at each bank, where the difference between the two tag line measurements is W_T. The cross-sectional area of flow, A, which is also required for the discharge measurement itself is obtained by summing the section area between each set of two verticals across the full width of the cross-section. In contrast, the wetted perimeter required for the calculation of the hydraulic radius is more difficult to assess. It needs to be specially measured or specially calculated.

For each current meter measurement, the hydraulic mean depth, D_{hy}, is calculated by using Equation 6.1. This calculated value of D_{hy} can be subtracted from the observed gauge reading during the current meter measurement to obtain the gauge correction, ΔG, for that particular discharge measurement. This can be accomplished by rearranging Equation 1.3,

$$\Delta G = G - D_{hy} \tag{6.1}$$

where D_{hy} is substituted as an appropriate measure of D, so that

$$\Delta G = G - D \tag{1.3}$$

When developing a downstream gauge rating, each current meter measurement will most likely provide a slightly different value of the estimated gauge correction. The variation of ΔG for some selected flow control structures is listed in Table 6.1.

Although not reflected in the observed Malik Branch Canal discharge measurements and the second discharge measurement in Hakra Branch Canal, the general trend of the gauge corrections for the hydraulic mean depth is that they decrease with decreasing discharge.

The KD formula does not allow for a gauge correction changing with different discharges. For this reason, an average gauge correction is calculated from the individual gauge corrections. This average gauge correction is illustrated in Figure 6.3. The average gauge correction is used to create the reference level for the depth parameter actually used in the downstream gauge rating. Although the values for the depth parameter will be near to the hydraulic mean depth values of the observed discharges, there will be a difference.

Table 6.1. Variation in estimated gauge correction using hydraulic mean depth for selected head regulators under the Eastern Sadiqia Canal.

Discharge (cusecs)	Gauge ,G (feet)	Area (sq. feet)	W_T (feet)	D_{hy} (feet)	ΔG (feet)
Malik Branch Canal					
1979	7.38	790.9	105.0	7.53	-0.15
1096	5.67	634.2	102.0	6.22	-0.55
767	4.85	505.5	101.0	5.00	-0.15
Hakra Branch Canal					
2296	8.20	950.5	147.0	6.47	1.73
1957	7.62	849.5	145.5	5.84	1.78
1065	5.91	658.0	143.0	4.60	1.31
Hakra 6-R Distributary					
744	5.67	206.7	45.5	4.54	1.13
664	5.46	205.3	44.5	4.61	0.85
440	4.54	156.3	41.5	3.77	0.77
108	2.70	77.6	36.5	2.13	0.57

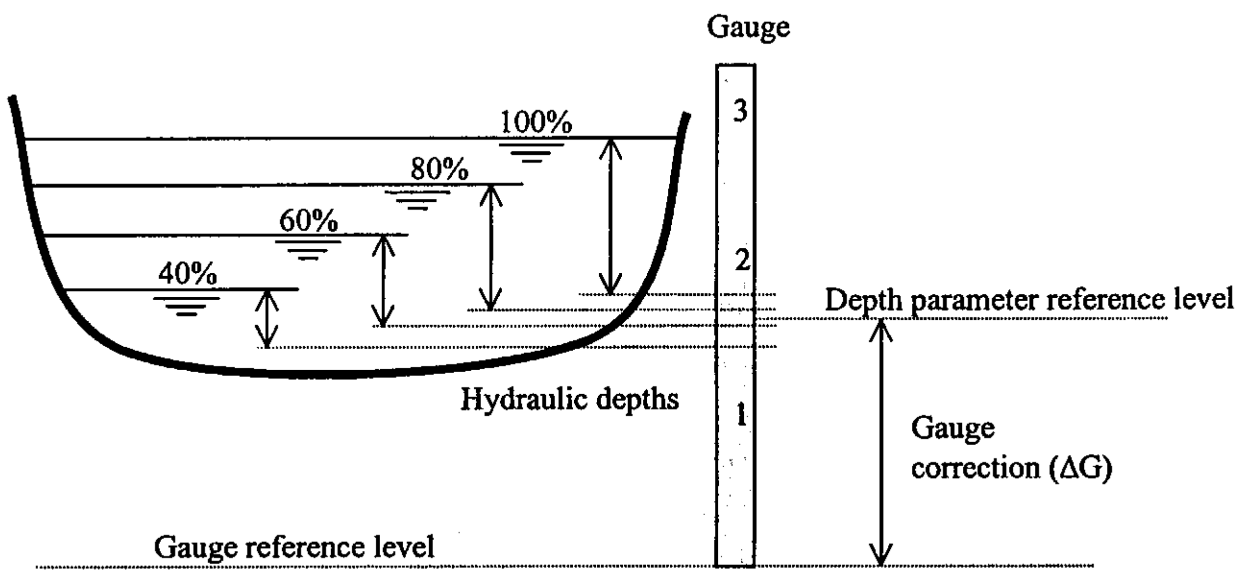


Figure 6.3. Averaged Gauge Correction with new Depth Parameter Reference Level for Mean Hydraulic Depth.

Both described restrictions, the use of hydraulic mean depth instead of hydraulic radius and the averaging of the individual gauge corrections, will change the downstream gauge rating. Thus, it can be expected that the exponent n will no longer be equal to the value of 5/3 used in the Manning-Strickler equation.

6.3.2. Gauge Corrections from Channel Bed Level

The irrigation systems in the Indian Subcontinent mostly convey higher sediment loads during the kharif season (April to October) as compared with the rabi season (October to April). Based on field measurements of channel cross-sections, it is quite evident that due to sediment deposition and changes between the design and operating parameters, either the channel's bed may rise due to sediment deposition or the bed may scour over a relatively small amount of time.

Another way to use channel cross section data in establishing a gauge correction is to have the gauge correction reflect the channel bed level. The advantage of this would be that after some time it can be easily evaluated whether the channel bed has changed. If the difference between the gauge zero point and the channel bed level is no longer equal to the gauge correction, either the gauge correction has to be adjusted or the whole downstream gauge rating would have to be revised.

Two methods were explored for obtaining the channel bed level from the cross-section data. One uses a certain number of verticals in the deepest part of the channel cross-section, the mid-cross-section mean depth, while the other tries to take the mean depth of the whole middle flatter part of the cross-section, the cross-sectional mean depth.

The mid-cross-section mean depth will be denoted by D_{mx}, where the subscript m refers to mid, while subscript x refers to cross-section, so that the subscript mx represents mid-cross-section. Establishing a value for D_{mx} needs to be tested. For this purpose, current meter measurements for three selected discharge regulating structures located in the Eastern Sadiqia Canal command area will be used. A comparison will be made using: (1) the single vertical having the greatest flow depth, $(D_{mx})_1$; (2) the average flow depth for four adjacent (consecutive) verticals having the greatest flow depths, $(D_{mx})_4$; (3) the average flow depth for eight adjacent verticals having the greatest flow depth, $(D_{mx})_8$; and (4) the flow depth for twelve adjacent verticals representing the maximum value of D_{mx} for any combination of twelve adjacent verticals in the cross-section, $(D_{mx})_{12}$. For any of these simple techniques, the mid-cross-section mean depth is an average calculated from Equation 6.2.

$$D_{mx} = \frac{\Sigma(\text{depth of observed verticals})}{\text{number of observations}} \qquad (6.2)$$

where d is the flow depth at a vertical used while making a current meter measurement.

Using the three selected head regulators under the Eastern Sadiqia Canal, tne four techniques for calculating the mid-cross-section mean depth are listed in Table 6.2 for each current meter measurement.

Table 6.2. Comparison of four techniques for calculating the mid-cross-section mean depth.

Discharge (cusecs)	Downstream Gauge (feet)	Mid-cross-section Mean Depth (all in feet)			
		$(D_{mx})_1$	$(D_{mx})_4$	$(D_{mx})_8$	$(D_{mx})_{12}$
Malik Branch Canal					
1979	7.38	8.60	8.40	8.35	8.28
1096	5.67	8.30	7.88	7.44	7.18
767	4.85	6.10	5.93	5.91	5.82
Hakra Branch Canal					
2296	8.20	8.20	8.13	8.03	7.85
1957	7.62	7.50	7.38	7.28	7.08
1065	5.91	5.90	5.75	5.55	5.30
Hakra 6-R Distributary					
744	5.67	5.50	5.50	5.46	5.44
664	5.46	5.50	5.50	5.50	5.50
440	4.54	4.50	4.45	4.43	4.42
108	2.70	2.50	2.50	2.49	2.46

From the analysis of information listed in Table 6.2, it is quite clear that, in the case of Malik Branch Canal, the value of $(D_{mx})_1 > (D_{mx})_4$ and $(D_{mx})_4 > (D_{mx})_8$ and $(D_{mx})_8 > (D_{mx})_{12}$. The gauge correction difference decreases as more verticals are taken to calculate the mean depth. The trend in Hakra Branch Canal is similar to the trend in Malik Branch Canal. However, in Hakra 6-R Distributary, there is a variation in trend; possible measurement errors could be the cause of this variation (as it is a lined distributary).

Keeping in mind that the gauge zeropoint should reflect the design bed level, it is quite obvious from Table 6.2 that none of the zero gauge readings really coincide with the actual bed level. For the Head Regulator of Malik Branch Canal at the tail of Eastern Sadiqia Canal, it seems that the channel bed has eroded and the depth of water is more than what is indicated by the downstream gauge reading. On the other hand, in the case of

Hakra Branch Canal, as well as Hakra 6-R Distributary, the gauge correction values reflect that the zero reading for the downstream gauge is below the channel bed and the actual depth of water is less than what is indicated by the downstream gauge reading.

For calculating the cross-sectional mean depth, D_m, all verticals were selected from the flat-portioned part of the channel cross-section (see Table 6.3.) :

$$D_m = \frac{\sum(\text{depth of verticals})}{\text{number of verticals}} \qquad (6.3)$$

Table 6.3. The verticals used in the middle portion of canal section to calculate the cross-sectional mean depth for selected gauge readings.

S. No.	Name of Channel	Total verticals	Verticals used in mean depth	Mean Depth	D/S gauge
1.	Malik Branch Canal	24	16	8.19	7.38
2.	Hakra Branch Canal	30	24	8.24	9.28
3.	Hakra 6R Disty	20	15	5.5	5.76*

***This is a lined channel where the downstream gauge was not installed, but instead an auxiliary gauge has been developed with help of a pointed white mark (WM) used for elevation.**

The above calculations of the channel bed level do not give one clear bed level, so that the chosen bed level becomes quite arbitrary. This means that any future changes of the channel bed level might be difficult to assess.

6.4. INFLUENCE OF THE GAUGE CORRECTION

This section has the purpose to review the influence of the gauge correction on the downstream gauge rating. In the first step, the different gauge corrections obtained with cross-sectional analysis and with the different downstream gauge rating methods have been put together in Table 6.4 to provide insights. Where the gauge corrections are different for different discharge measurements, an average gauge correction has been calculated. The columns with the observed discharge and the gauge reading are followed the gauge corrections for four mid-cross-section mean depths, the mean hydraulic depth, and the hydraulic radius. The last three columns are for gauge corrections calculated in different downstream gauge rating methods. The first of these is the one variable approach, which is corrected with a changed gauge correction; the second, the two variable approach with K and ΔG as variables; while the third is the three variable approach. The one variable approach and the two variable approach with K and n as variables both use the average for the mean hydraulic depth as the gauge correction.

Table 6.4. Overview of gauge corrections (or delta gauge) obtained with different cross-sectional analysis and downstream gauge rating methods.

Discharge (cusecs)	Downstream Gauge (feet)	Delta Gauge with: $(D_{mx})1$ (feet)	$(D_{mx})4$ (feet)	$(D_{mx})8$ (feet)	$(D_{mx})12$ (feet)	D_{hy} (feet)	R (feet)	$1V+\Delta G$ (feet)	2V (feet)	3V (feet)
Malik Branch Canal										
1979	7.38	-1.22	-1.02	-0.97	-0.90	-0.15	0.26	1.56	1.70	0.50
1096	5.67	-2.63	-2.21	-1.77	-1.51	-0.58	-0.27	1.56	1.70	0.50
767	4.85	-1.25	-1.08	-1.06	-0.97	-0.15	-0.01	1.56	1.70	0.50
Mean Delta Gauge:		-1.70	-1.44	-1.27	-1.13	-0.29	-0.01	1.56	1.70	0.50
Hakra Branch Canal										
2296	8.20	0.00	0.07	0.17	0.35	1.70	1.91	2.00	1.69	2.57
1957	7.62	0.12	0.24	0.34	0.54	1.78	1.93	2.00	1.69	2.57
1063	5.91	0.01	0.16	0.36	0.61	1.31	1.37	2.00	1.69	2.57
Mean Delta Gauge:		0.04	0.16	0.29	0.50	1.60	1.74	2.00	1.69	2.57
Hakra 6-R Distributary										
744	5.67	0.17	0.17	0.21	0.23	1.13	1.45	1.32	1.32	1.10
664	5.46	-0.04	-0.04	-0.04	-0.04	0.81	1.19	1.32	1.32	1.10
440	4.54	0.04	0.09	0.11	0.12	0.77	1.01	1.32	1.32	1.10
108	2.70	0.20	0.20	0.21	0.24	0.57	0.66	1.32	1.32	1.10
Mean Delta Gauge:		0.09	0.11	0.12	0.14	0.82	1.08	1.32	1.32	1.10

Table 6.4. shows interesting results. The mean hydraulic depth and the hydraulic radius give gauge corrections that are clearly higher than the channel bed level. In turn, the hydraulic radius is located higher than the mean hydraulic depth. The gauge corrections derived from the downstream gauge rating methods are generally the highest of all.

To be able to analyze the results a little further, the depth readings obtained from the difference between gauge reading and the gauge correction were plotted in a double-log graph. Beforehand, individual gauge corrections have been averaged for each of the depth parameters related to cross-sectional data. Figure 6.4 presents the results for the head of Hakra Branch Canal . The outcome is even more interesting. There seem to be ratings possible by drawing straight lines through the data points having a similar gauge correction. The slope of these lines are decreasing with increasing gauge correction, while the K value, which comes from the Y-Axis intercept, is increasing with increasing gauge correction. In Figure 6.4, only the rating lines have been drawn for the datasets with the highest and lowest gauge correction.

To understand this concept, rating results of the example structure sites were plotted with the exponent, n, against the gauge corrections. As already seen in Table 6.4, different calculation methods for obtaining ratings yielded different gauge corrections. The result of the relation between the exponent, n, and the gauge correction, ΔG, is seen in Figure 6.5. The outcome could hardly be more clear. There is a linear relationship between the gauge correction and exponent. With increasing gauge correction, the exponent, n, decreases.

Figure 6.6. explores the relationship between gauge corrections and the coefficient, K, in the KD formula. In this case, the relationship is exponential. With an increasing gauge correction, the coefficient, K, increases as well, but in a more rapid pace.

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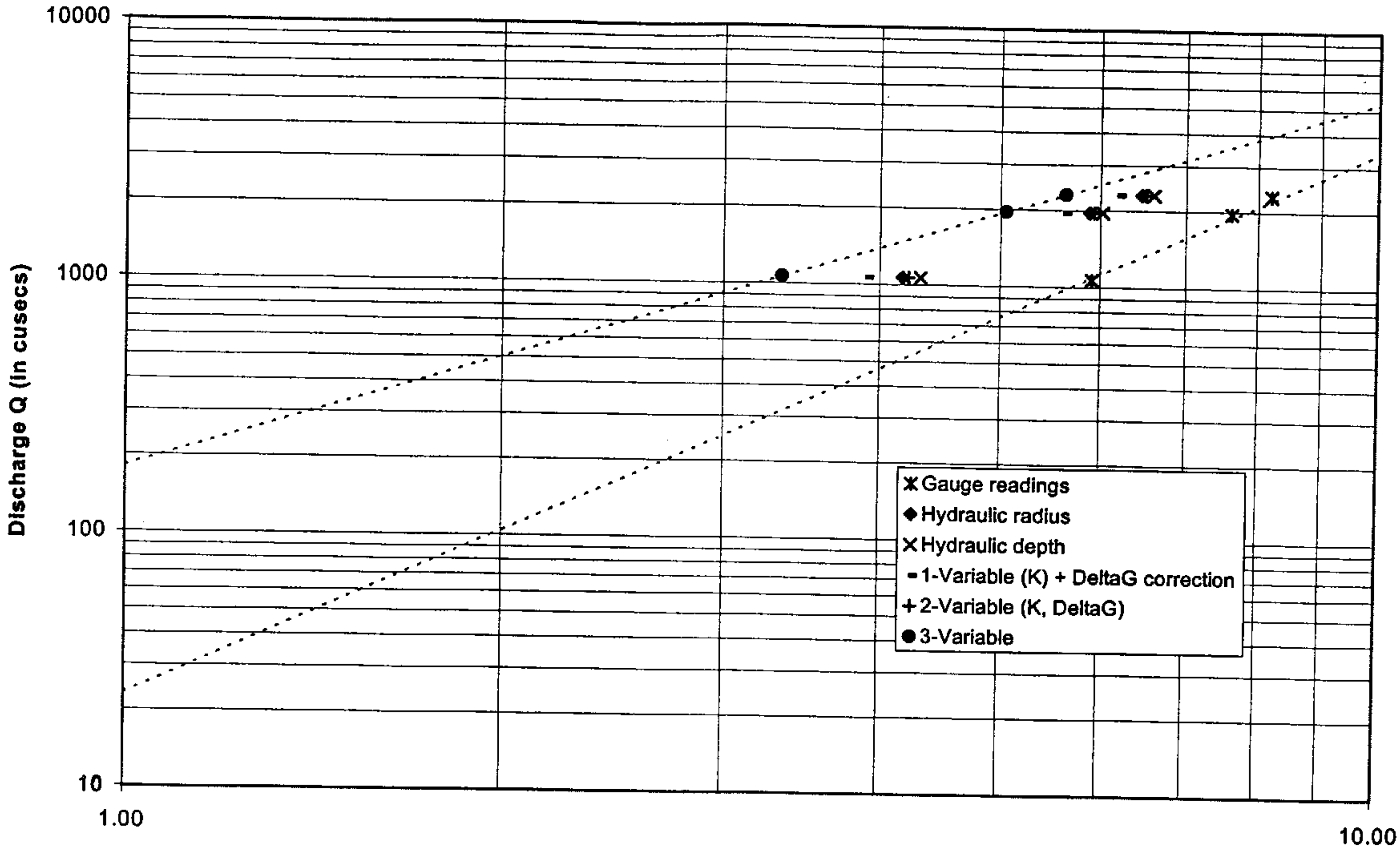


Figure 6.4. Relation between Discharge and Depth Parameters for Head of Hakra Branch Canal.

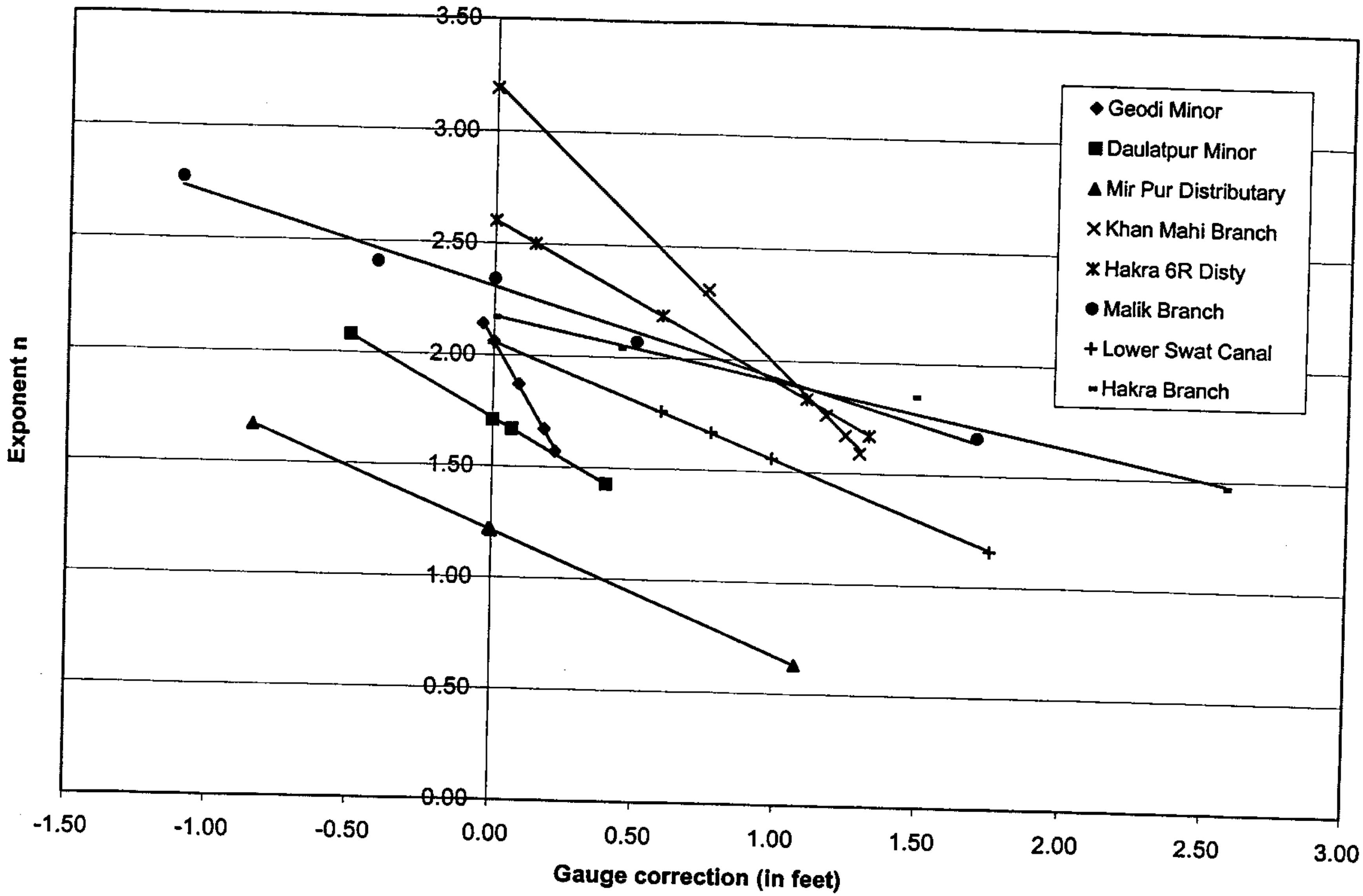


Figure 6.5. Relation between gauge corrections and rating exponents for different channels.

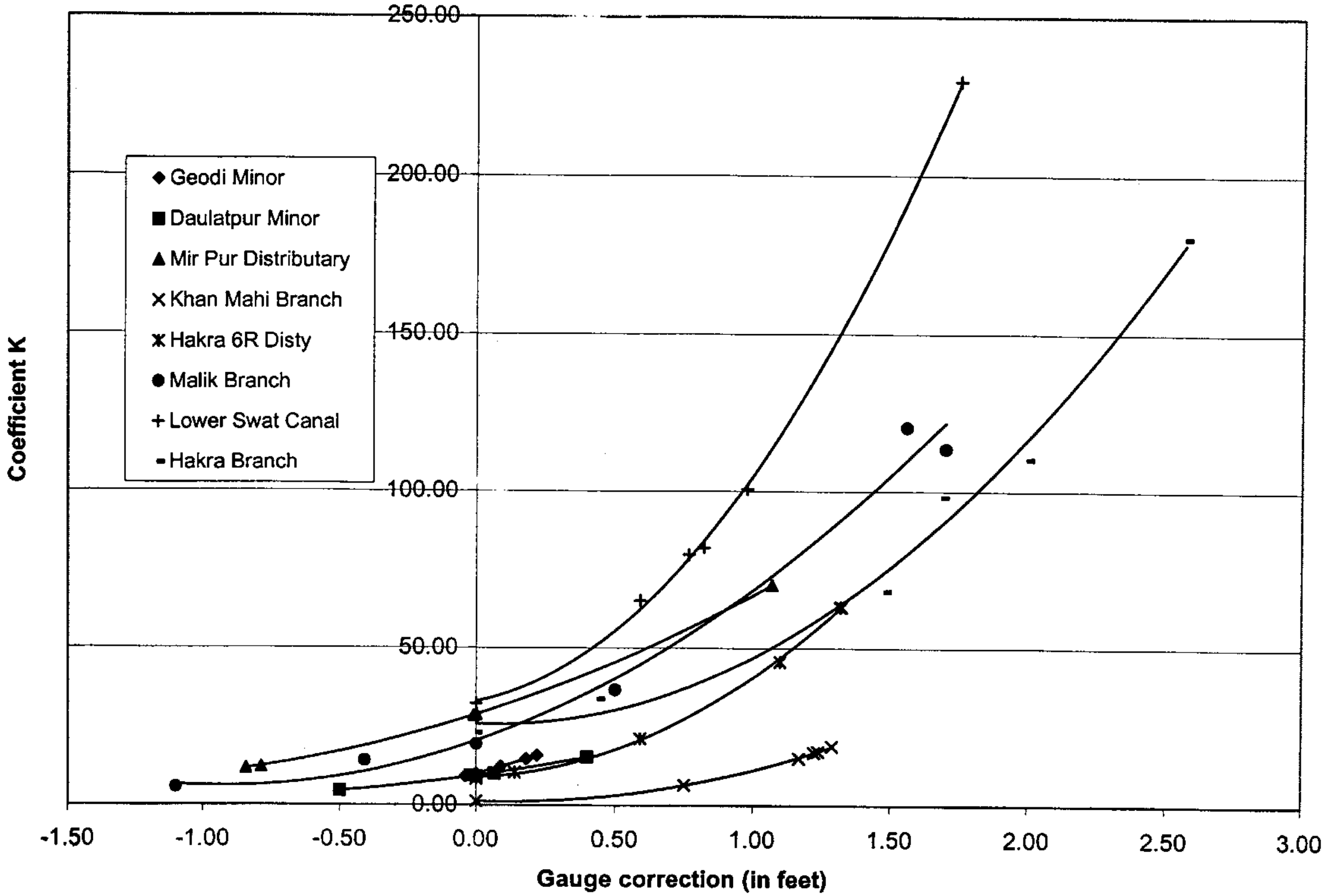


Figure 6.6. Relation between gauge corrections and the coefficient, K, for different channels.

These clear relationships are explained by seeing the gauge correction as a means to change the reference level for the water levels in the channel from the gauge zero point to another reference level. With the changed reference level, the value of the depth parameter changes. At each of the water levels, the discharge in the channel remains the same. This means that no matter what reference level is chosen, an increase in the channel water level will always mean the same increase in discharge. With a large depth parameter, or low lying reference level, an increase in the channel water level will have a relatively smaller effect on the KD term as compared with a small depth parameter or high lying reference level. To compensate for this effect, the power function in the KD term depicted by n will decrease for smaller depth parameters, which comes with an increased gauge correction and higher-lying reference level. Of course, no matter what reference level is used, the discharges calculated with the KD formula need to remain the same as the measured discharges.

Furthermore, the results seem to suggest that it does not matter which gauge correction is used, rating can always be obtained a with the appropriate exponent, n, and coefficient, K. However, this conclusion is too simple. A rating may be found for any gauge correction, but it remains to be seen whether that rating will be accurate enough. The three variable approach gives (per definition) the most accurate rating to fit the measured discharges. After that comes the two variable approaches. The resulting ratings of these methods have quite high gauge corrections. In general, these gauge corrections are a lot higher than the gauge corrections depicting the channel bed level and even higher than the hydraulic radius.

7. PERIODIC ADJUSTMENTS TO THE DOWNSTREAM GAUGE RATINGS

Once a KD-relation between the downstream gauge and the discharge has been established, it would be nice to be able to continue using the relationship for a number of years. Unfortunately, any change in the downstream cross-section has consequences for the KD-relation. Within a monsoon period, channel bed levels can change considerably, causing errors in recently developed gauge-discharge relationships. In this section, two methods will be given to tune the KD-relation to limited bed level changes. By using these methods, the durability of the developed KD-relation can be extended with minor efforts.

The primary objective of both methods is to be able to check the developed KD-relation with a new observation of the discharge. The downstream gauge reading taken during the discharge observation should lie on the curve, which gives the relationship between the gauge and the discharge. This curve can be drawn on graph paper by taking a number of values for the downstream gauge and plotting them against the discharge found by filling in the developed KD-formula. In Figure 7.1, the gauge-discharge curve for Head Malik Branch has been plotted according to the KD-formula derived by using the double-variable method. In this case, the downstream gauge has been plotted on the y-axis (ordinate) to show its vertical orientation.

After the monsoon period had ended in the Punjab, field observations in the Malik Branch command area give reasons to doubt whether the discharge given by the downstream gauge rating at the Head of Malik Branch was still accurate. To end the doubts, the Irrigation Department decided to conduct a new discharge observation with use of current metering. They found the discharge to be 1447 cusecs at a downstream gauge reading of 6.59 feet. The point has been depicted in Figure 7.1 by a cross. According to the developed and plotted gauge-discharge curve, the discharge at a gauge of 6.59 feet should have been about 1525 cusecs, a reduction in discharge of little more than 5 %.

The size of the reduction may give reasons to doubt the current metering, but the Irrigation Department made sure to conduct the current metering by following accurately the taught procedures with a well calibrated current meter. There is only one explanation possible. The bed level in the downstream section of the Head of Malik Branch structure has risen, reducing the flow area and mean depth at a downstream gauge of 6.59 feet. A discharge of 1447 cusecs would in the original gauge-discharge curve occur at a gauge of 6.44 feet. This means that the flow area and mean depth at the gauge of 6.44 feet before the monsoon period are the same as the flow area and mean depth at a gauge of 6.59 feet after the monsoon period. The bed level has apparently risen by 6.59–6.44 = 0.15 feet. Through accounting for this bed level change by adjusting the Delta gauge for an additional +0.15 foot, an accurate KD-relation can be regained.

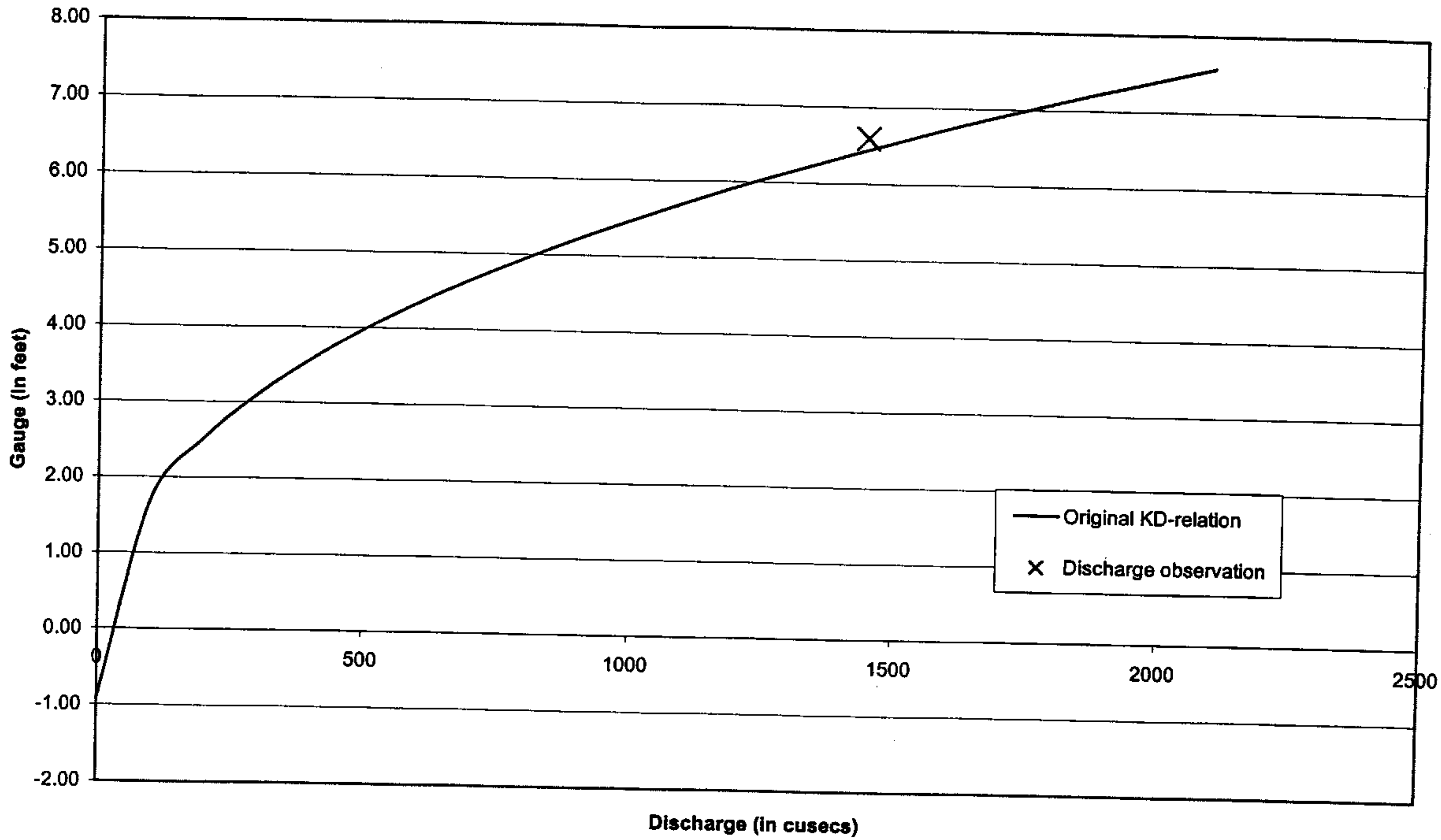


Figure 7.1. Discharge-gauge relation for Head of Malik Branch Canal.

In Figure 7.2, the new KD-relation with the adjusted Delta gauge has been plotted. Note that the curve has been shifted upward along the vertical axis with the mentioned +0.15 feet. The new curve goes nicely through the point depicting the discharge observed with the current metering.

Of course, the same kind of adjustment can be done for a lowered bed level. At RD 0+130 Khan Mahi Branch, Lower Swat Canal a current metering is conducted by the NWFP Irrigation Department to check their earlier developed KD-relation. The current metering results in a discharge of 98 cusecs at a downstream gauge of 3.98 feet. This implies that the bed level has lowered as the original gauge discharge curve for the double-variable method links a downstream gauge of 4.08 feet to a discharge of 98 cusecs. The bed level has scoured by 3.98-4.08=-0.10 feet. Again, this bed level change can be adjusted by changing the Delta gauge with an additional –0.10 feet. The result is seen in Figure 7.3. The original gauge discharge curve has been shifted downwards by 0.10 feet thereby fitting it with the new discharge observation.

By reviewing the above-described tuning method, it becomes quite obvious that an accurate current metering is essential. If the discharge is off the mark, the whole new gauge rating will be off the mark as well. Furthermore, the method is fit for small changes in bed level. If the shape of the channel changes with larger bed level fluctuations and moving banks, the values for K and n will be affected as well. In that case, it is time to take a full series of current meterings and develop a whole new KD-relation.

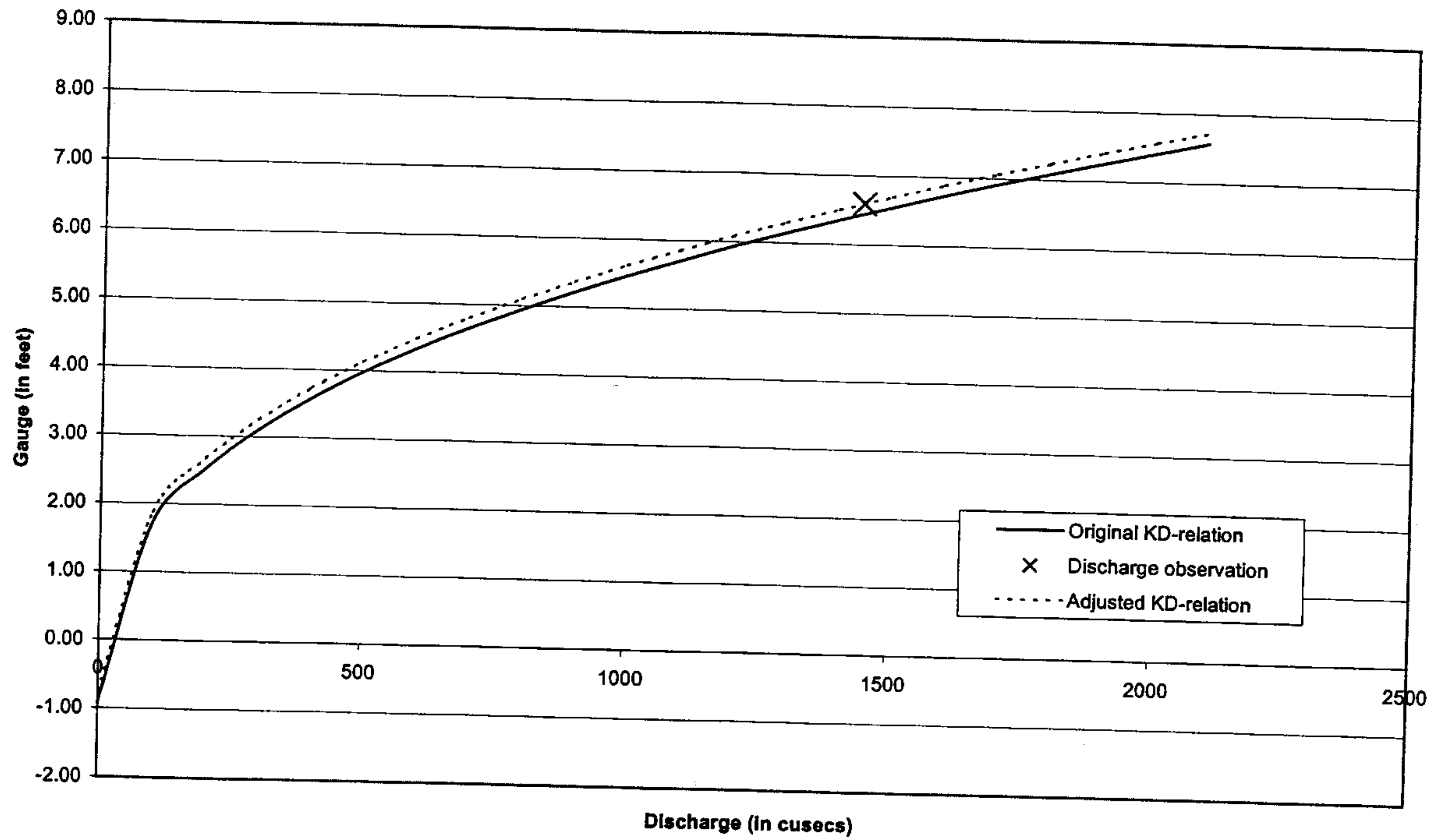


Figure 7.2. Discharge-gauge relation for Head Malik.

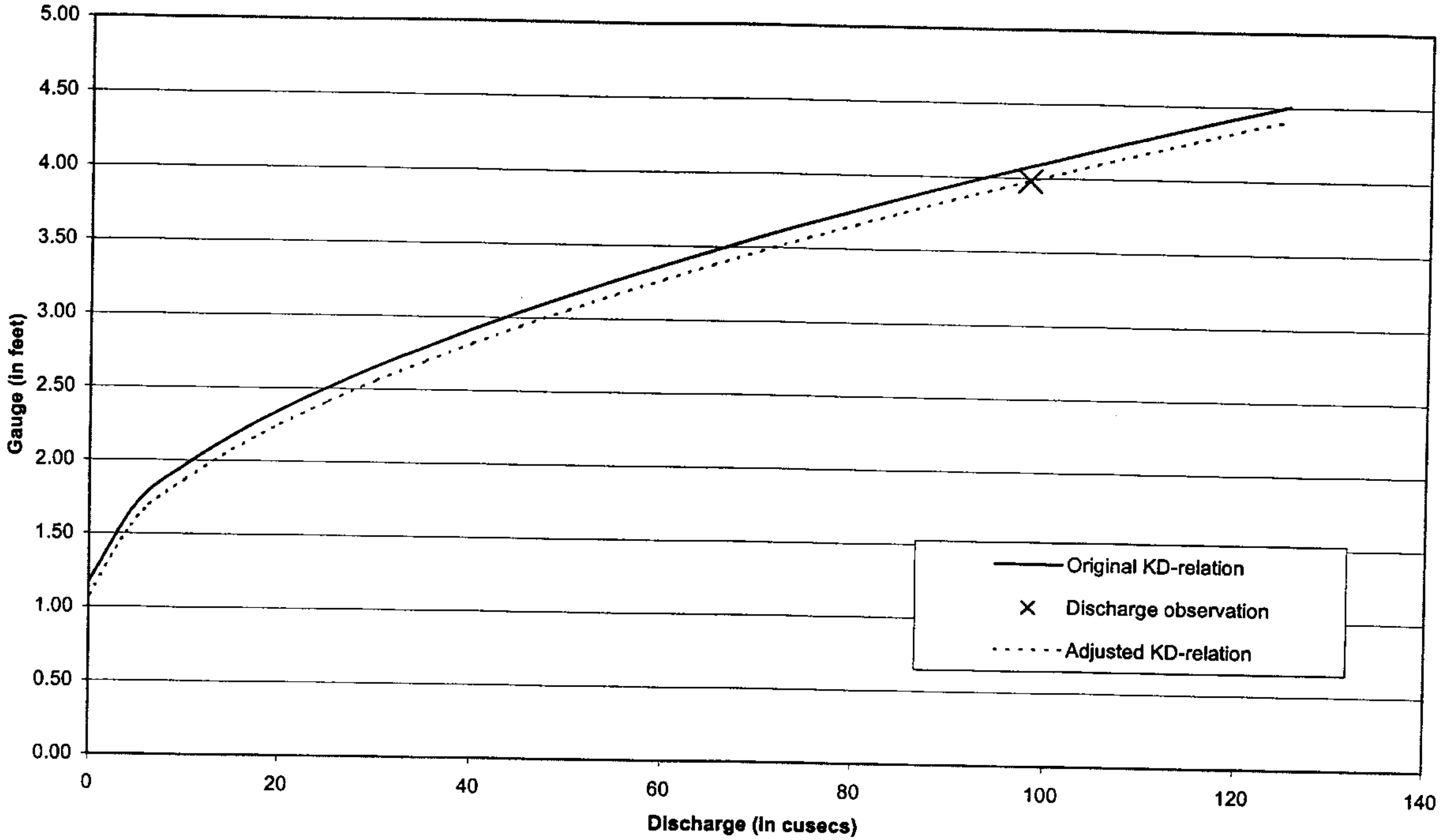


Figure 7.3. Discharge-gauge relation for RD 0+130 of Khan Mahi Branch.

8. USING FLOW CONTROL STRUCTURE RATING

At the time that current meter measurements are taken to develop the downstream gauge rating, these same measurements can be used to calibrate the flow control structure. For a gate structure, this would require that the gate opening could be measured, along with a flow depth upstream from the structure if free orifice flow occurs, while submerged orifice flow requires the measurement of both an upstream and downstream depth. These measurements have to be referenced with the same elevation, which is commonly the gate sill or gate seat.

The advantage in developing a discharge calibration for the structure is that this rating should remain stable for a number of years. Thus, it can be used to periodically adjust the zero level for the downstream gauge.

8.1 RATING ORIFICES

Any type of opening in which the upstream water level is higher then the top of the opening is referred to as an orifice. In this case, if the jet of water emanating from the orifice discharges freely into the air or downstream channel without a backwater or tailwater effect, then the orifice is operating under free-flow conditions. If the upstream water level is below the top of the opening, then the opening is hydraulically performing as a weir structure. For free-flow conditions through an orifice, the discharge equation is:

$$Q_f = C_d C_v A \sqrt{2 g h_u} \qquad (8.1)$$

where C_d is the dimensionless discharge coefficient, C_v is the dimensionless velocity head coefficient, A is the cross- sectional area of the orifice opening, g is the acceleration due to gravity, and h_u is measured from the centroid of the orifice to the upstream water level as shown in Figure 8.1 (a).

The upstream depth, h_u, can also be measured from the bottom of the orifice opening if the downstream depth is taken to be about 0.611 times the vertical orifice opening, which takes into account the theoretical flow contraction just downstream of the orifice. Otherwise, the inferred assumption is that the downstream depth is equal to one-half the opening, and h_u is effectively measured from the area centroid of the opening. Either of these two assumptions may be adequate in rating the orifice for free-flow conditions, and in defining the governing equation, but it should be noted that the choice will affect the value of the discharge coefficient.

If the downstream water level is also above the top of the orifice as illustrated in Figure 8.1 (b), then submerged conditions exist and the discharge equation becomes:

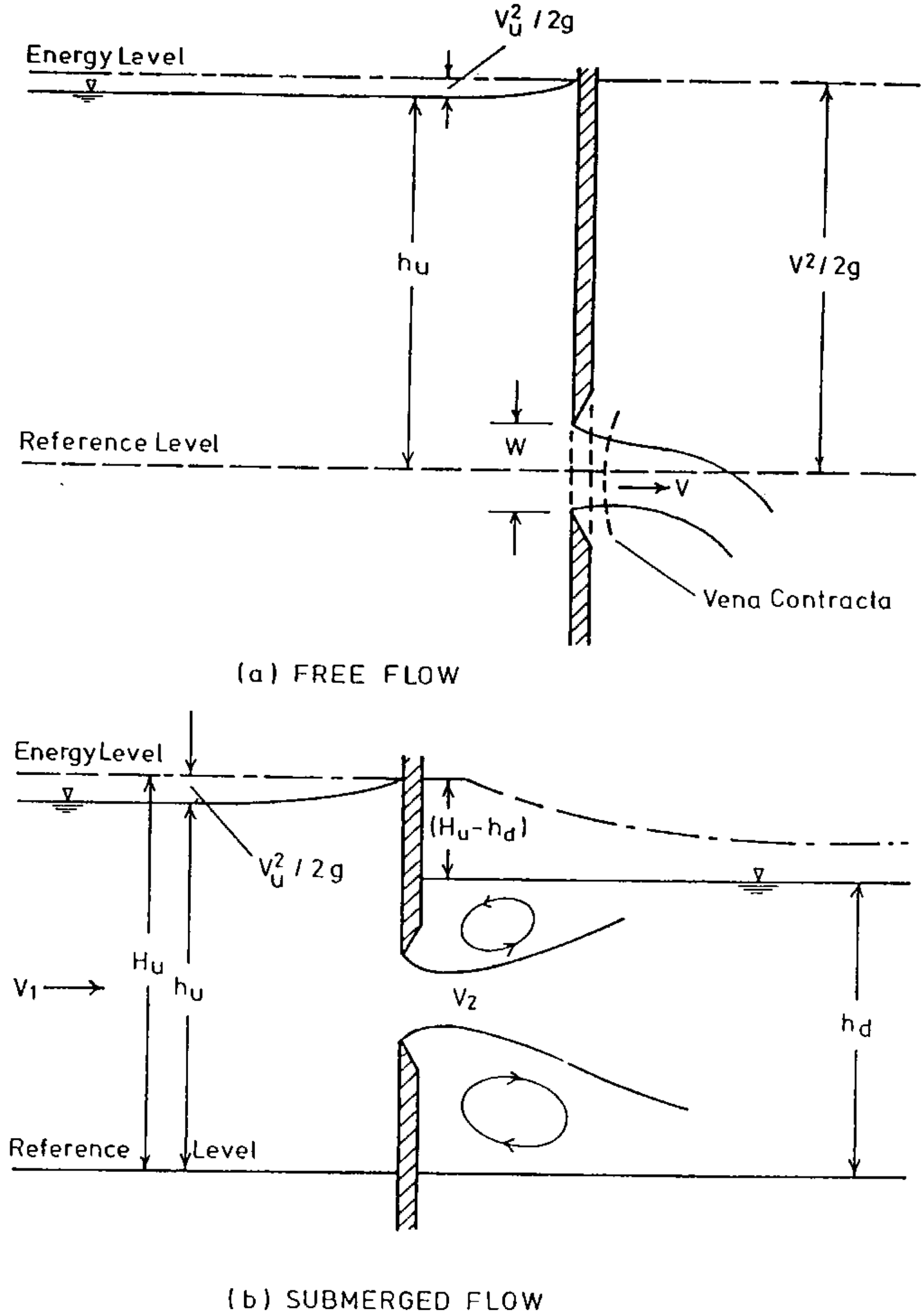


Figure 8.1. Definition sketch of orifice flow.

$$Q_s = C_d\, C_v\, A\sqrt{2g(h_u - h_d)} \tag{8.2}$$

Where h_u-h_d is the difference in water surface elevations upstream and downstream of the submerged orifice.

The velocity head coefficient, C_v, approaches unity as the approach velocity to the orifice decreases to zero. In an irrigation system, C_v can usually be assumed to be unity since most irrigation channels have flat gradients and the flow velocities are low (usually less than 1 m/s).

An orifice can be used as a highly accurate flow measuring device in an irrigation system. If the orifice structure has not been previously rated in the laboratory, then it can easily be rated in the field. The hydraulic head term, h_u, or (h_u-h_d), can be relied upon to have the exponent 1/2, which means that a single field rating measurement, if accurately made, will provide an accurate determination of the coefficient of discharge, C_d. However, the use of a single rating measurement implies the assumption of a constant C_d value, which is the case for an orifice having fixed dimensions (e.g., a culvert or an outlet structure), but this is not the case for an orifice having adjustable dimensions.

8.2 STRUCTURES WITH ADJUSTABLE GATES

Adjustments to the basic orifice equations for free- and submerged-flow are often made to more accurately represent the structure rating as a function of flow depths and gate openings. The following sections present some alternative equation forms for taking into account the variability in the discharge coefficient under different operating conditions. Orifices usually have C_d values of about 0.60 to 0.80, depending on the geometry and installation of the structure, but values ranging from about 0.3 to 0.9 have been measured in the field.

8.2.1 Free-Flow Rectangular Gate Structures

A definition sketch for a rectangular gate structure having free orifice flow is shown in Figure 8.2. For a rectangular gate having a gate opening, G_o, and a gate width, W, the free-flow discharge equation can be obtained from Equation 8.1, assuming that the dimensionless velocity head coefficient is unity.

$$Q_f = C_d\, G_o\, W\sqrt{2g(h_u - G_o/2)} \tag{8.3}$$

where G_o is the vertical gate opening, W is the gate width, and G_ow W is the area, A, of the orifice opening.

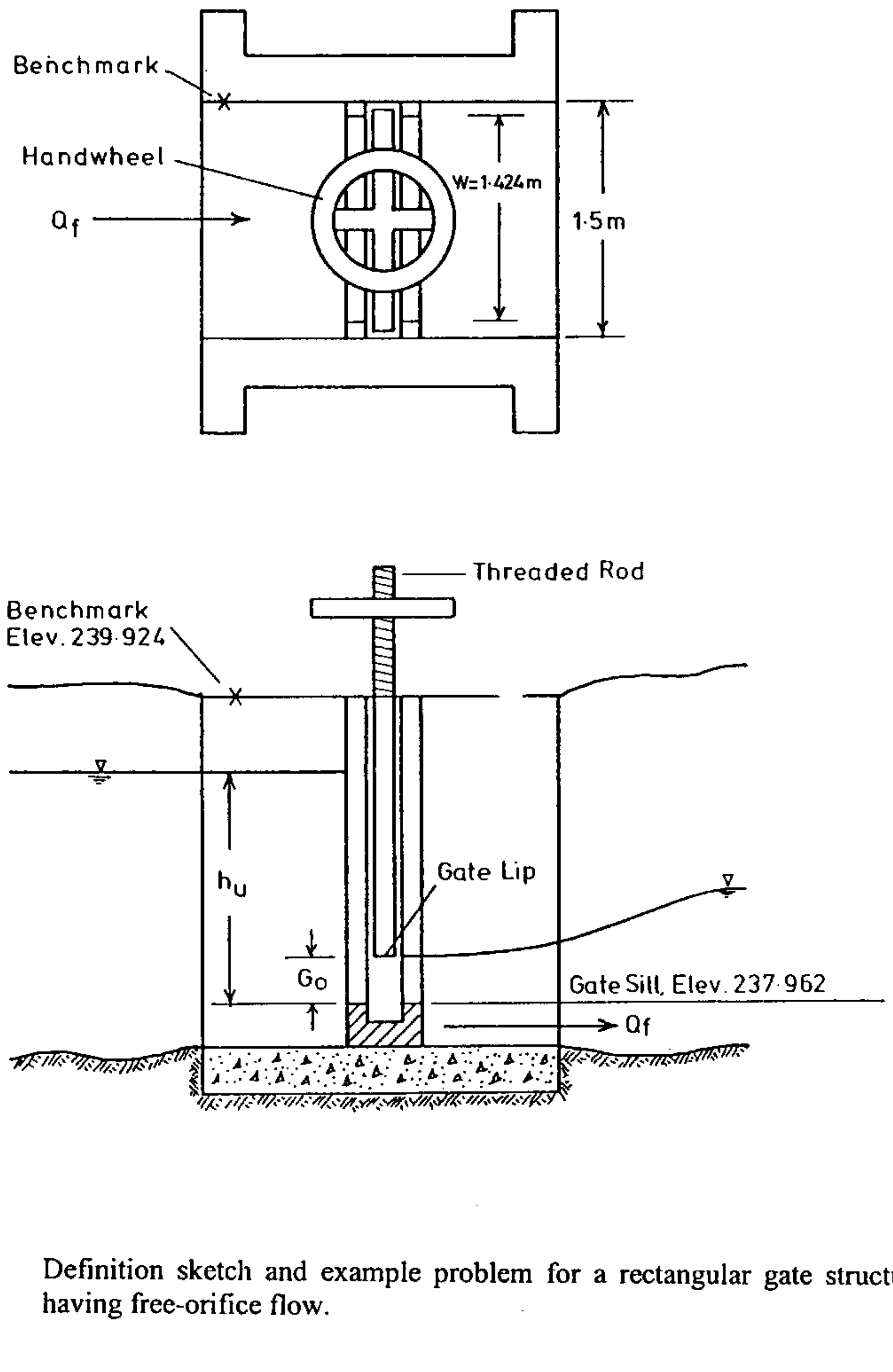


Figure 8.2. Definition sketch and example problem for a rectangular gate structure having free-orifice flow.

The upstream flow depth, h_u, can be measured anywhere upstream of the gate, including the upstream face of the gate. The value of h_u will vary a small amount depending on the location chosen for measuring h_u. Consequently, the value of the coefficient of discharge, C_d, will also vary according to the location selected for measuring h_u.

One of the most difficult tasks in calibrating a gate structure is obtaining a highly accurate measurement of the gate opening, G_o. For gates having a threaded rod that rises as the gate opening is increased, the gate opening is read from the top of the handwheel to the top of the rod with the gate closed, and then set at some opening, G_o. This very likely represents a measurement of gate opening from where the gate is totally seated, rather than a measurement from the gate sill; therefore, the measured value of G_o from the threadrod will usually be greater than the true gate opening, unless special precautions (described below) are taken to calibrate the threadrod.

Likewise, when the gate lip is set at the same elevation as the gate sill, there will undoubtedly be some flow or leakage through the gate. This implies that the datum for measuring the gate opening is below the gate sill. In fact, there is often leakage from a gate even when it is totally seated (closed) because of inadequate maintenance. An example problem will be used to illustrate the procedure for determining an appropriate zero datum for the gate opening.

For the rectangular gate structure shown in Figure 8.2, the calibration data listed in Table 8.1 was collected. The data reduction is listed in Table 8.2 where the coefficient of discharge, C_d, was calculated from Equation 8.3.

A rectangular coordinate plot of C_d versus the gate opening, G_o, listed in Table 8.2 is shown graphically in Figure 8.3. The value of C_d continues to decrease with larger gate openings. To determine if a constant value of C_d can be derived, Equation 8.3 can be rewritten in the following format:

$$Q_f = C_d (G_o + \Delta G_o) W \sqrt{2g\left[(h_u)_{\Delta G_o} - \frac{G_o + \Delta G_o}{2}\right]} \qquad (8.4)$$

Where ΔG_o is a measure of the zero datum level from the gate sill, where

$$(h_u)\, \Delta_{Go} = h_u + \Delta G \qquad (8.5)$$

is shown in Figure 8.4. An appropriate value of ΔG_o will be determined by trial-and-error for the example problem. Assuming values of ΔG_o equal to 1mm, 2mm, 3mm,etc., the computations for determining C_d can be made using Equation 8.4. The results for ΔG_o equal to 1mm, 2mm, 3mm, 4mm, 5mm, 6mm, 7mm, 8mm, and 12mm(gate seated) are listed in Table 8.3. The best results are obtained from ΔG_o of 3mm; this result is plotted in Figure 8.5, which shows that C_d varies from 0.582 to 0.593 with the average value of

Table 8.1. Example of field calibration data for a rectangular gate structure having free orifice flow.

Discharge, Q_f m^3/s	Gate Opening, G_o m	Upstream Benchmark Tape Measurement m
0.0646	0.010	0.124
0.0708	0.020	1.264
0.0742	0.030	1.587
0.0755	0.040	1.720
0.0763	0.050	1.787
0.0767	0.060	1.825

Table 8.2. Data reduction for example rectangular gate structure having free orifice flow.

Q_f m^3/s	G_o m	h_u m	C_d (see note below)
0.0646	0.010	1.838	0.756
0.0708	0.020	0.698	0.677
0.0742	0.030	0.375	0.654
0.0755	0.040	0.242	0.635
0.0763	0.050	0.175	0.625
0.0767	0.060	0.137	0.620

Note: The discharge coefficient, C_d, was calculated using the following equation:

$$Q_f = C_d G_o W \sqrt{2g(h_u - G_o/2)}$$

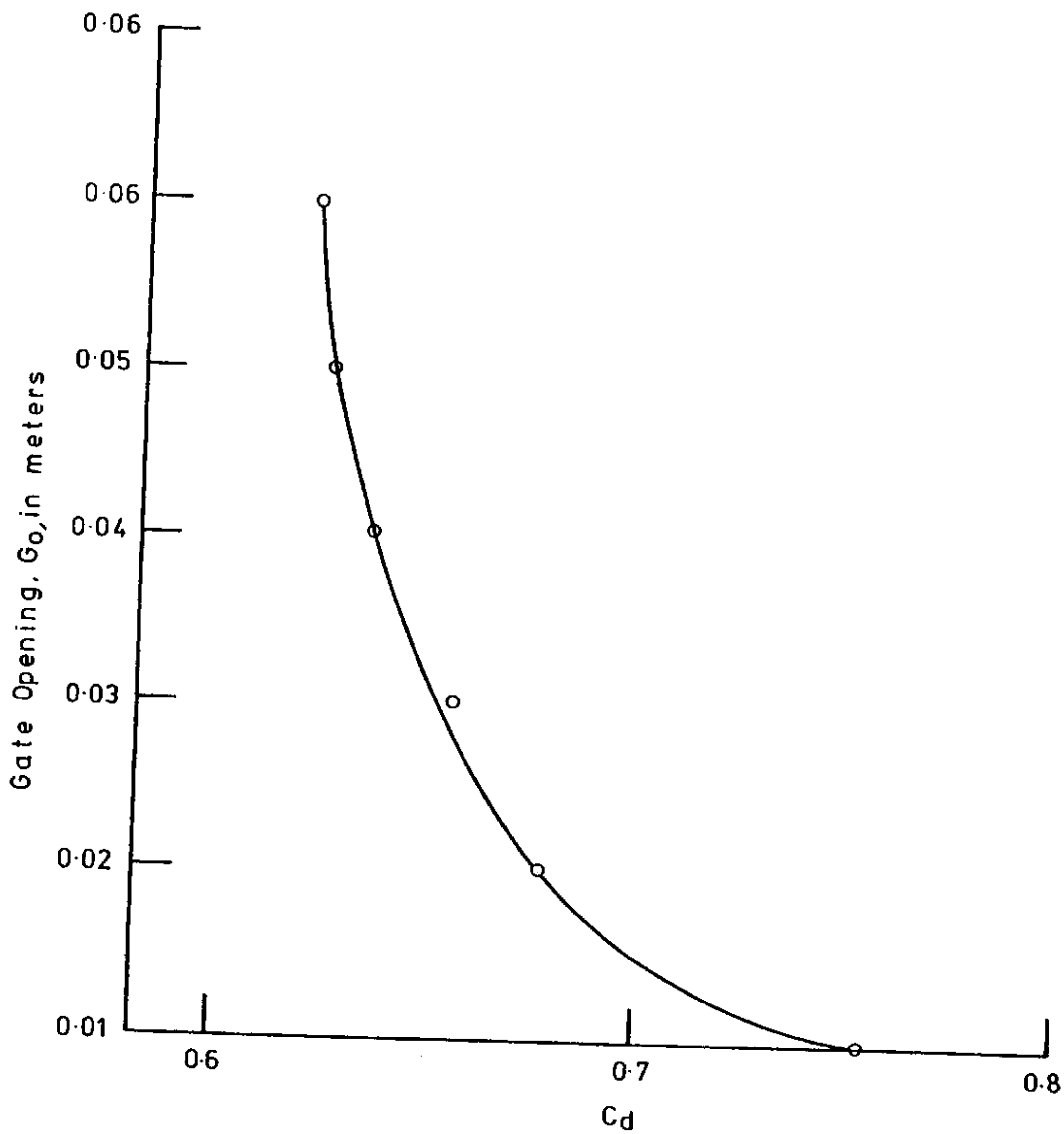


Figure 8.3. Variation in the discharge coefficient, C_d, with gate opening, G_o, for the example rectangular gate structure with free orifice flow.

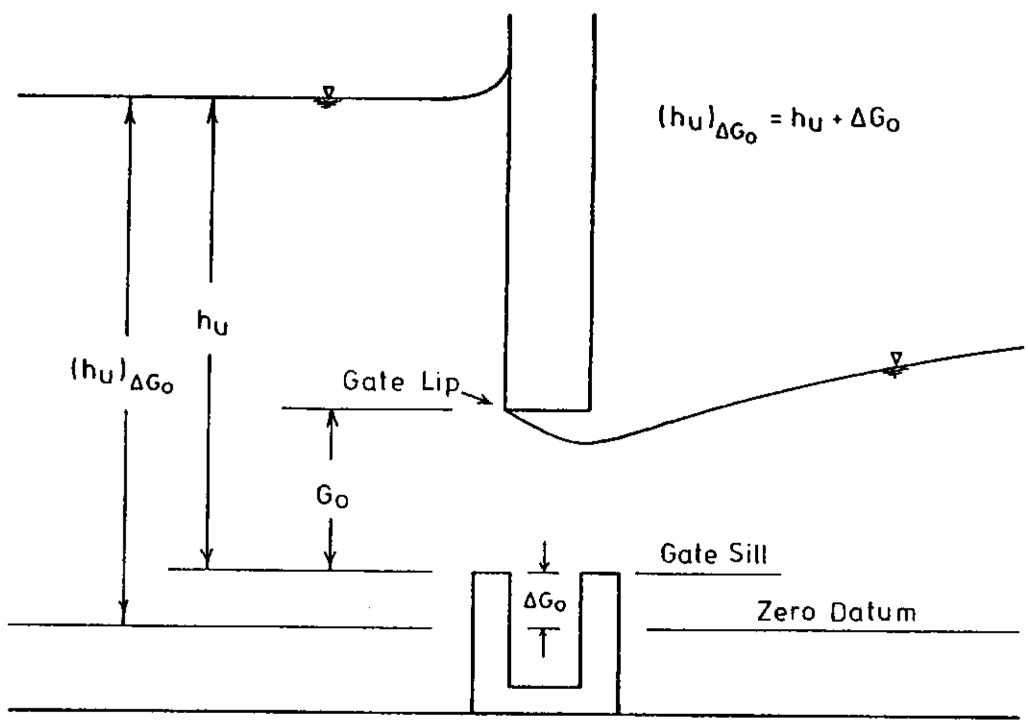


Figure 8.4. Definition sketch of the zero datum for gate opening and upstream flow depth for a rectangular gate structure.

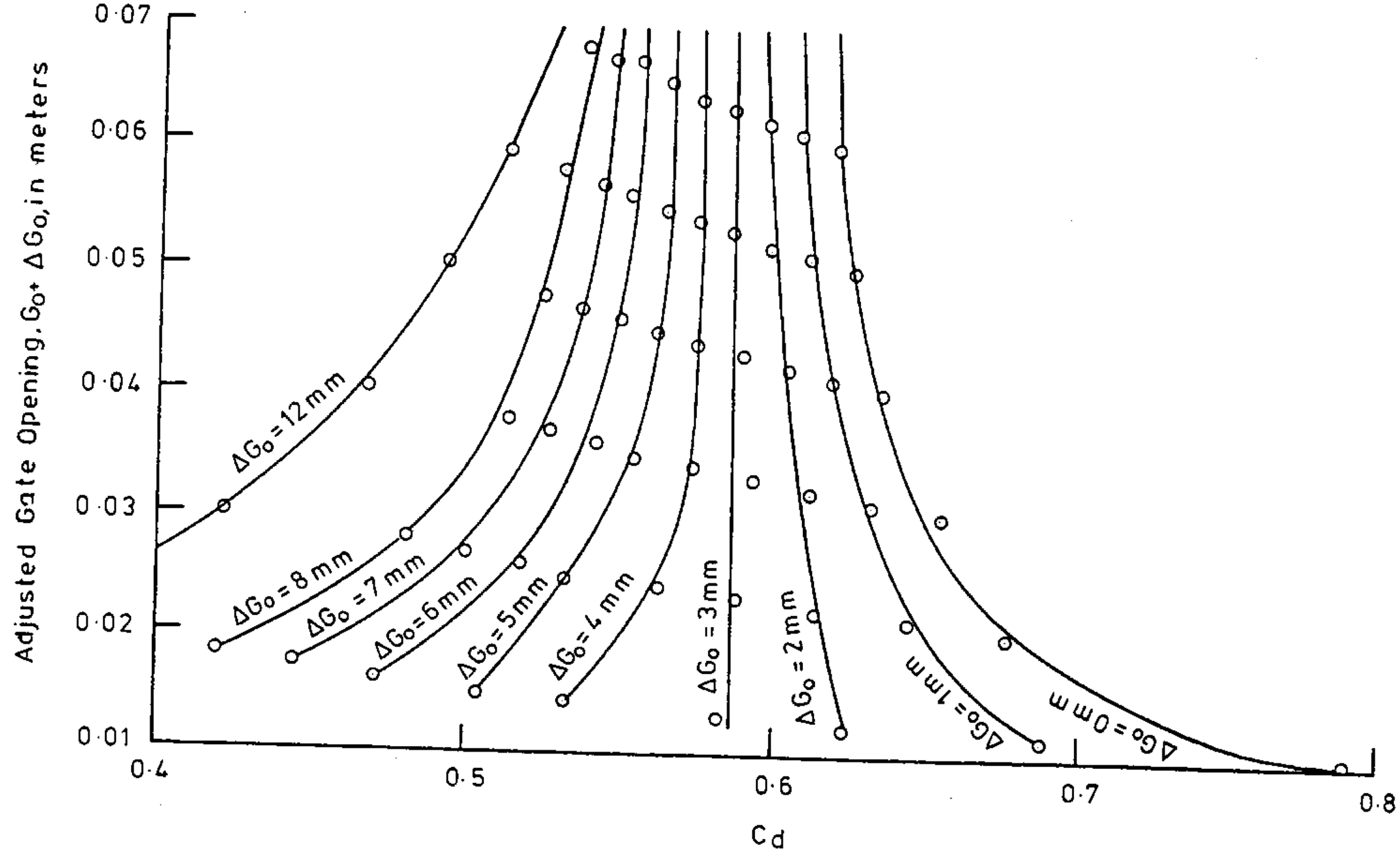


Figure 8.5. Variation in the discharge coefficient, C_d, with the adjusted gate opening, $G_o + \Delta G_o$, for the example rectangular gate structure with free orifice flow.

Table 8.3 Computation of the discharge coefficient, C_d, for adjusted values of gate opening and upstream flow depth for the example rectangular gate structure having free orifice flow.

Q_f m^3/s	G_o m	h_u m	Discharge Coefficient, C_d (see note below)									
			ΔG_o 0mm	ΔG_o 1mm	ΔG_o 2mm	ΔG_o 3mm	ΔG_o 4mm	ΔG_o 5mm	ΔG_o 6mm	ΔG_o 7mm	ΔG_o 8mm	ΔG_o 12mm
0.0646	0.010	1.838	0.756	0.688	0.630	0.582	0.540	0.504	0.472	0.445	0.420	0.344
0.0708	0.020	0.698	0.677	0.644	0.615	0.588	0.563	0.540	0.519	0.500	0.482	0.425
0.0742	0.030	0.375	0.654	0.632	0.612	0.593	0.575	558	0.542	0.527	0.513	0.471
0.0755	0.040	0.242	0.635	0.619	0.604	0.589	0.575	0.561	0.549	0.536	0.525	0.495
0.0763	0.050	0.175	0.625	0.611	0.599	0.586	0.575	0.563	0.552	0.542	0.531	0.514
0.0767	0.060	0.137	0.620	0.608	0.597	0.586	0.575	0.565	0.556	0.546	0.537	0.531

Note: The last column with ΔG_o = 12 mm is for the gate totally seated (closed). The discharge coefficient, C_d, was calculated from:

$$Q_f = C_d (G_o + \Delta G_o) W \sqrt{2g\left[(h_u)_{\Delta G_o} - \frac{G_o + \Delta G_o}{2}\right]}$$

C_d being 0.587. For this particular structure, the discharge normally varies between 200 and 300 lps, and the gate opening is normally operated between 40-60 mm, so that a constant value of C_d = 0.587 can be used when the zero datum for G_o and h_u is taken as 3 mm below the gate sill (another alternative would be to use a constant value of C_d = 0.575 for ΔG_o = 4mm rather than 3 mm).

8.2.2 Submerged-Flow Rectangular Gate Structures

Submerged-flow gate structures are the most common constrictions employed in irrigation networks. The gates are used to regulate the water levels upstream and the discharge downstream. For this reason, they are very important structures that need to be field calibrated. Fortunately, they are one of the easiest structures to field calibrate for discharge measurement.

A definition sketch for a rectangular gate structure having submerged orifice flow is shown in Figure 8.6. Assuming that the dimensionless velocity head coefficient in Equation 8.2 is unity, the submerged-flow discharge equation for a rectangular gate having an opening, G_o, and a width, W, becomes:

$$Q_s = C_d G_o W \sqrt{2g(h_u - h_d)} \tag{8.6}$$

where G_o W, is the area, A, of the orifice.

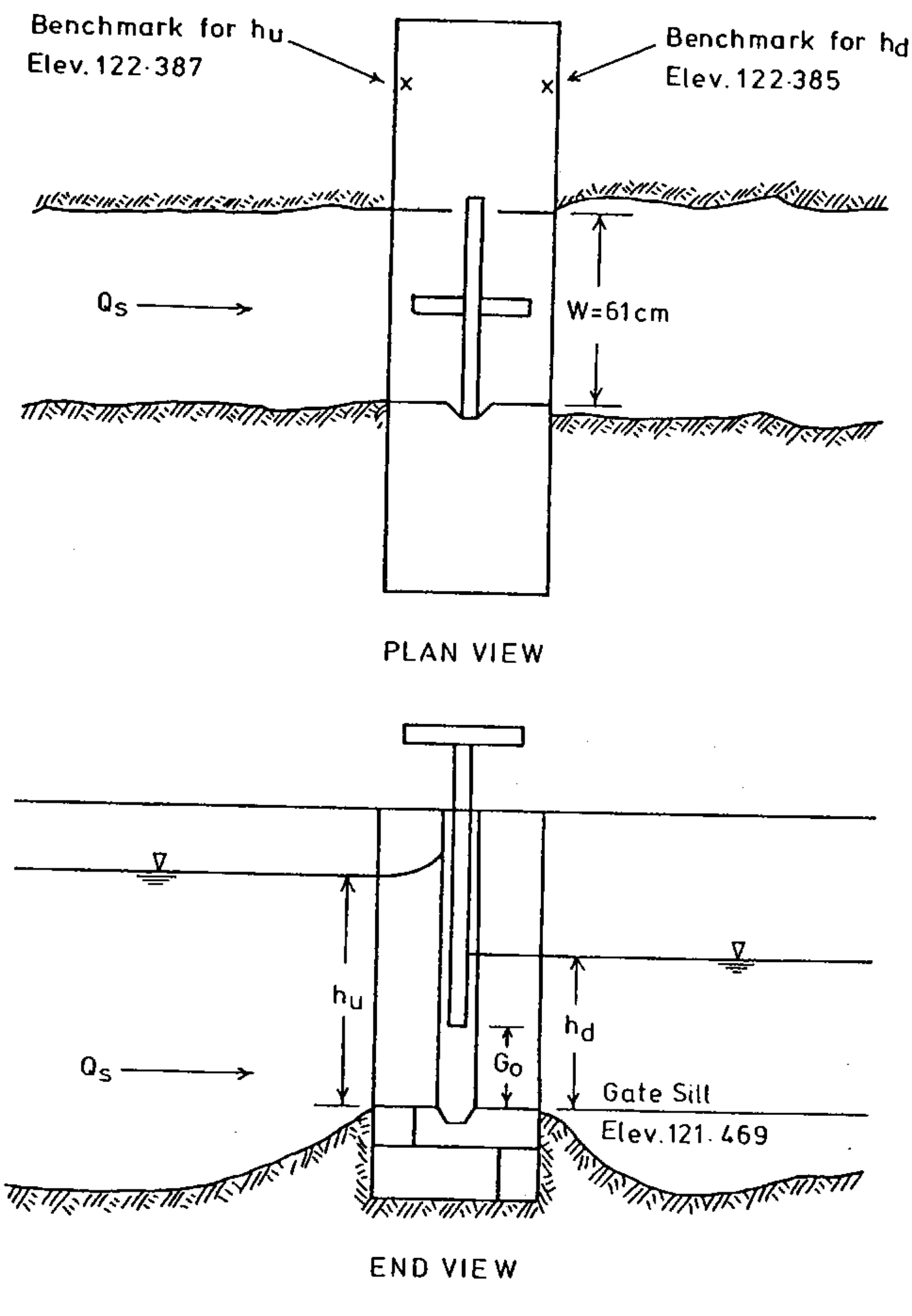


Figure 8.6. Definition sketch and example problem for a rectangular gate structure having submerged orifice flow.

The upstream flow depth, h_u, can be measured anywhere upstream of the gate, including the upstream face of the gate. Likewise, the downstream flow depth, h_d, can be measured anywhere downstream of the gate, including the downstream face of the gate. Many times, h_u and h_d will be measured at the gate because only one reference benchmark is needed on top of the gate structure in order to mark tape measurements down to the water surface. This is satisfactory if the water surfaces on the gate are smooth. If not, h_u and h_d should be measured at locations where the water surface is smooth, not turbulent and fluctuating.

All of the information in the previous section regarding the measurement of gate opening,G_o, applies equally well for submerged gates.

For the rectangular gate structure shown in Figure 8.6, the field calibration data is listed in Table 8.4. Note that for this type of slide gate, the gate opening was measured both on the left side $(G_o)_L$, and the right side $(G_o)_R$, because the gate lip is not always horizontal. The data reduction is listed in Table 8.5 where the coefficient of discharge C_d, was calculated from Equation 8.6. The variation in C_d, with gate opening, G_o, is plotted in Figure 8.7

As in the case of the free-flow orifice calibration in the previous section, a trial-and-error approach can be used to determine a more precise zero datum for the gate opening (see Figure 8.6). In this case, Equation 8.6 would be rewritten as:

$$Q_s = C_d (G_o + \Delta G_o) W \sqrt{(2g(h_u - h_d)} \qquad (8.7)$$

where ΔG_o is the vertical distance from the gate sill downwords to the zero datum level. The criteria for determining ΔG_o is to obtain a nearly constant value of C_d. A value of ΔG_o must be assumed and then C_d calculated from Equation 8.7 for each data set. Then, another value of ΔG_o is assumed, and another, etc, These computations are shown in Table 8.6 (the calculations for ΔG_o = 2mm have been omitted). A value for ΔG_o of 6mm provides the best results, which are plotted in Figure 8.8, where an average value of C_d = 0.640 is appropriate.

In the previous free-flow orifice calibration, when adjusting the gate opening (G_o+ ΔG_o), the value of upstream flow depth, h_u,also had to be adjusted. For this submerged-flow calibration, the term h_u-h_d is used, so any reference level can be used provided it is the same for both h_u and h_d. Thus, even though the values of h_u-h_d could be adjusted by the value for ΔG_o, there would be no change in the value of h_u-h_d.

Table 8.4. Example field calibration data for a rectangular gate structure having submerged orifice flow.

Discharge, Q_s M^3/s	Gate Opening		Benchmark Tape Measurement	
	$(G_o)_L$ m	$(G_o)_R$ m	Upstream m	Downstream m
0.079	0.101	0.103	0.095	0.273
0.095	0.123	0.119	0.099	0.283
0.111	0.139	0.139	0.102	0.296
0.126	0.161	0.163	0.105	0.290
0.141	0.180	0.178	0.108	0.301
0.155	0.199	0.197	0.110	0.301

Table 8.5. Data reduction for example rectangular gate structure having submerged orifice flow.

Q_s m^3/S	G_o m	h_u m	h_d m	H_u-h_d m	C_d
0.079	0.102	0.823	0.643	0.180	0.676
0.095	0.121	0.819	0.633	0.187	0.674
0.111	0.139	0.816	0.620	0.196	0.668
0.126	0.162	0.813	0.626	0.187	0.666
0.141	0.179	0.810	0.615	0.195	0.660
0.155	0.198	0.808	0.615	0.193	0.659

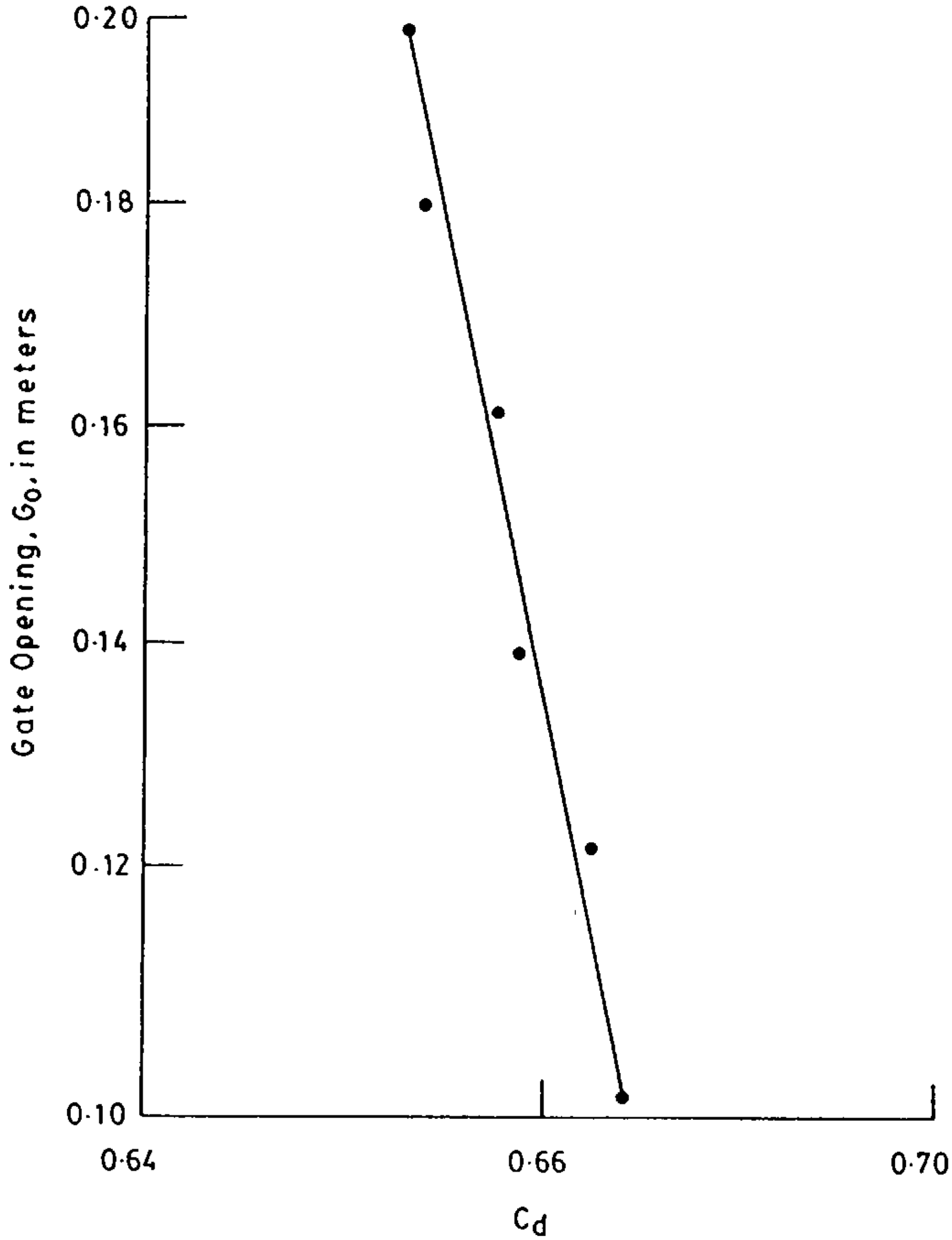


Figure 8.7. Variation in the discharge coefficient, C_d, with gate opening, G_o, for the example rectangular gate structure with submerged orifice flow.

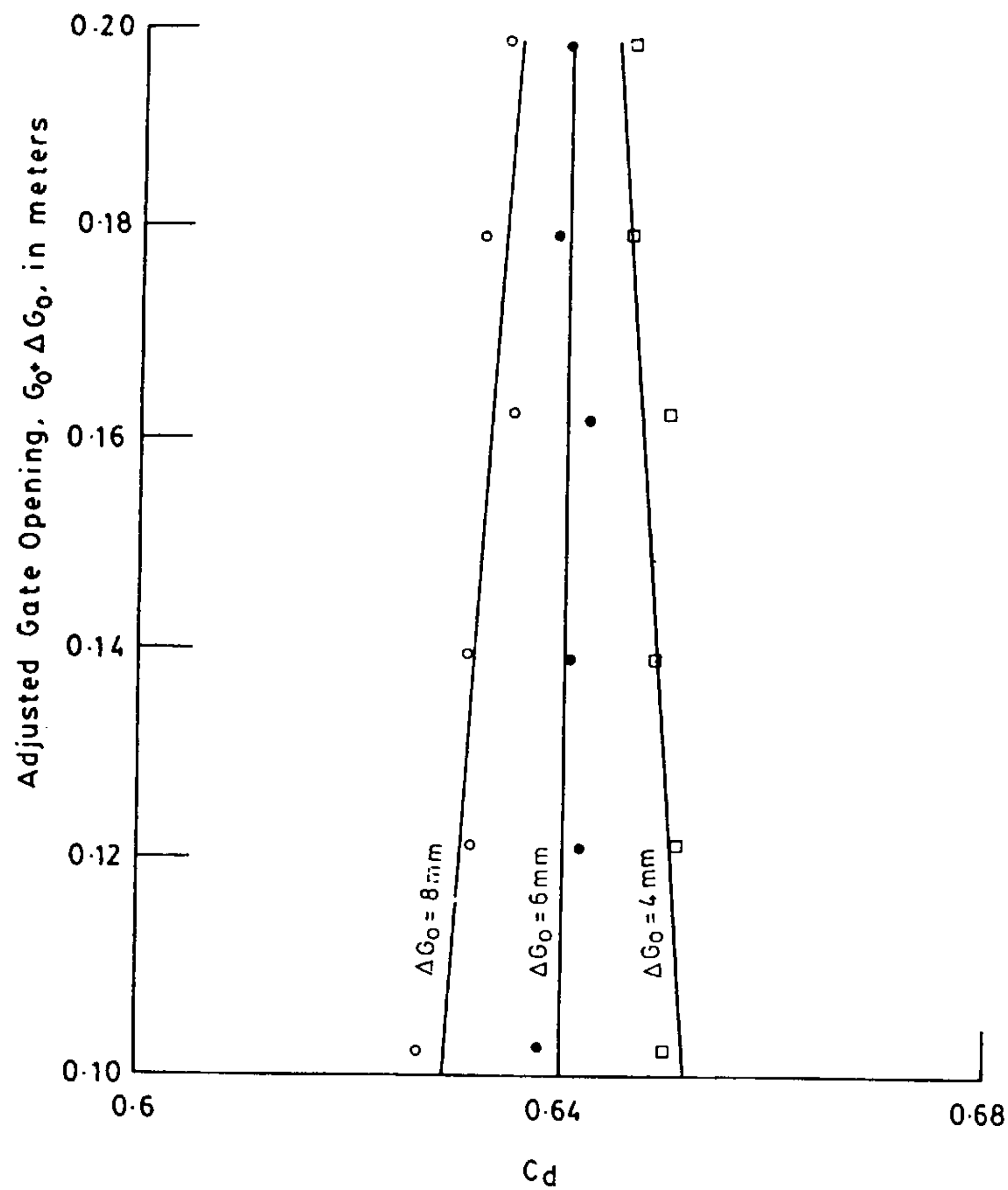


Figure 8.8. Variation in the discharge coefficient, C_d, with the adjusted gate opening, $G_o + \Delta G_o$, for the example rectangular gate structure with submerged orifice flow.

Table 8.6. Computation of the discharge coefficient, C_d, for adjusted values of gate opening, $G_o+\Delta G_o$, for the example rectangular gate structure having submerged orifice flow.

Q_s m^3/s	G_o m	h_u-h_d m	C_d (see note below)		
			$\Delta G_o = 4$ mm	$\Delta G_o = 6$ mm	$\Delta G_o = 8$ mm
0.079	0.102	0.1801	0.650	0.638	0.626
0.095	0.121	0.1865	0.651	0.641	0.631
0.111	0.139	0.1960	0.649	0.640	0.631
0.126	0162	0.1869	0.650	0.642	0.635
0.141	0.179	0.1949	0.646	0.639	0.632
0.155	0.198	0.1931	0.646	0.640	0.634

Note: That discharge coefficient, C_d, was calculated from:

$$Q_s = C_d(G_o + \Delta G_o)\, W \sqrt{2g(h_u - h_d)}$$

8.3 APPLICATION TO DOWNSTREAM GAUGE RATING

During the fall of 1996, IIMI and ISRIP conducted discharge measurements in the Fordwah Eastern Sadiqia area on request of the Irrigation Department. The discharge measurements were done at different stages of flow downstream of main cross-regulators in the canal system and were aimed at establishing new discharge tables. As the Irrigation Department is accustomed to the KD-relation, the main aim was to develop the KD-relation using the downstream gauge. Apart from data required for the KD-relation, including a downstream cross-section, IIMI also collected the necessary calibration data for deriving the structure calibration. This structure calibration provides an opportunity for periodically adjusting the developed KD-relation over time as a result of changes bed-level.

The head-regulator of Malik Branch Canal will be used as an example. In a previous section, the results of developing the KD-relation with the double-variable method was shown. The presented results were derived from the discharge measurements collected in the fall of 1996. Using the same discharge measurements, the head-regulator can be calibrated using the structure formula. As the Malik Branch Head Regulator is a rectangular-gate structure having free orifice flow, Equation 8.3 is used as the basis for developing the calibrated structure formula.

Using this formula, the value of C_d will most likely change with the gate opening, G_o, as illustrated in Figure 8.3. By using Equation 8.4, the reference level for both the upstream head and the gate opening can be evaluated by assigning assumed values to ΔG_o in order to derive one constant value for the coefficient of discharge, C_d, for all gate openings as illustrated in Figure 8.5.

During the Kharif 1997 season, the Malik Branch fell dry due to problems in one of the main link canals supplying water to the Fordwah Eastern Sadiqia area. This resulted in a chance to observe the bed level at the downstream side of the structure around the start of June 1997. That bed level had scoured considerably at an average of just less than half a foot for the cross-section. Obviously, an adjustment of the downstream KD-relation was necessary.

On 30 July 1997, measurements were taken of the upstream and downstream gauges and the gate openings. Using the previously derived average coefficient of discharge, C_d, the discharge through the head-regulator was found to be 1342 cusecs according to the structure formula. However, the KD-relation developed for the old bed-level gave a discharge of only 1100 cusecs. As the structure formula is not influenced by bed level changes, the discharge found by this method is the correct one under the prevailing circumstances.

Like in the situation of an extra current meter measurement, the newly found discharge is used to correct the KD-relation for the change in bed level. A discharge of 1342 cusecs in the old KD-relation represents a downstream gauge of 6.23 feet. On the 30th of July, the downstream gauge read 5.71 feet, implying that the bed level has drawn down by 6.23-5.71=0.52 feet. Considering a time difference of just less than two months between the two bed level measurements, this is an unusual circumstance. However, there is reason to adjust the Delta Gauge Correction by +0.52 feet. Figure 8.9 gives the KD-relation adjusted by using a discharge observation made with help of the previously calibrated structure formula.

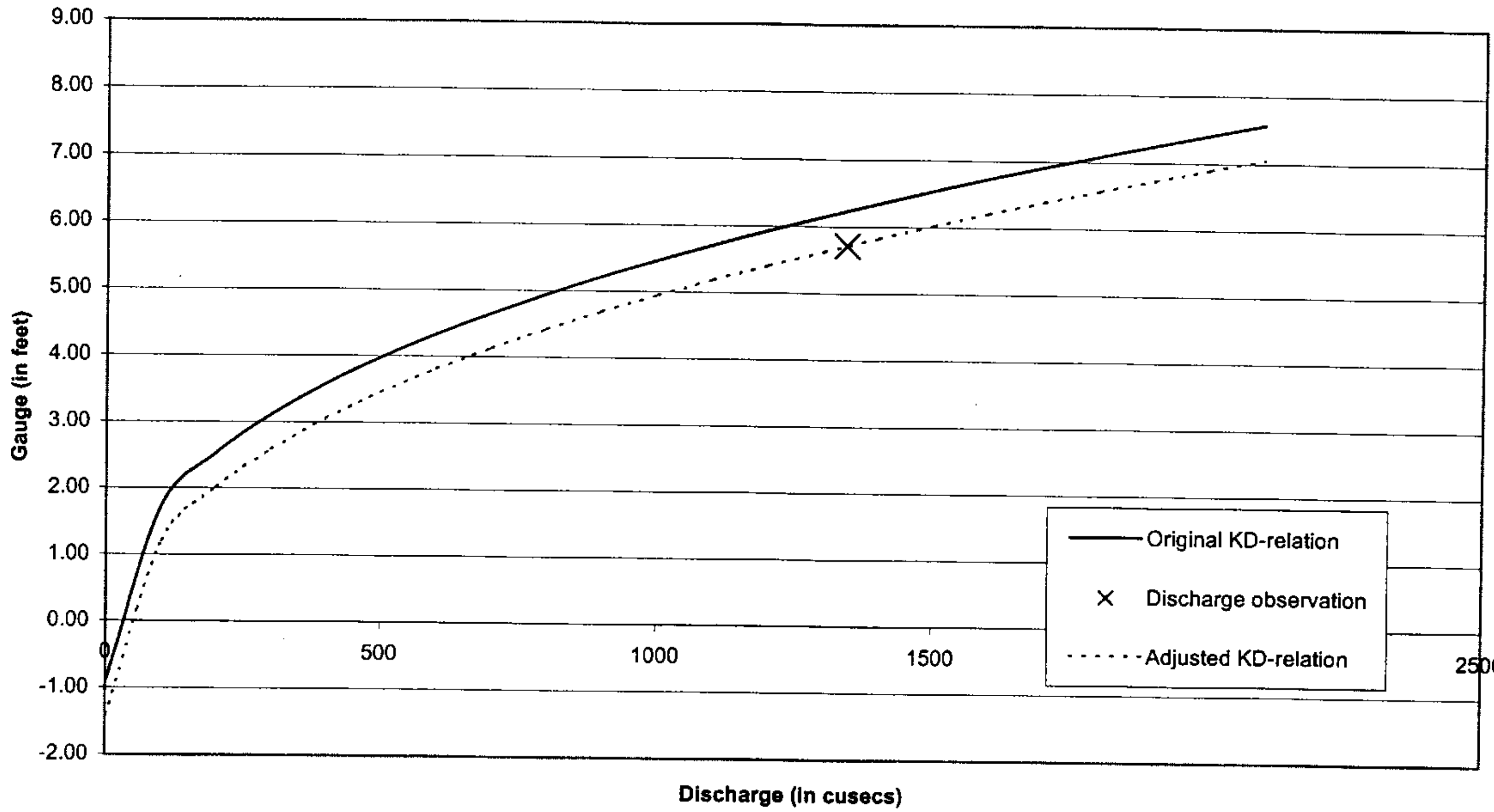


Figure 8.9. Discharge-gauge relation for head of Malik Branch adjusted on 30 July 1997 with structure formula.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1. CONCLUSIONS

A downstream gauge rating is a good method for determining the discharge running through a channel at places, where a structure calibration is difficult because of the way the structure is operated or because of the excess data required for a single reading.

In this report, different methodologies have been presented to obtain a downstream gauge rating from a series of measured discharges. These methodologies all have their origin in the three possible variables identified in the KD formula: the gauge correction, ΔG; the coefficient, K; and the exponent, n.

If none of the three variables are restricted, as seen in the Three Variables Approach, the result will always be a best fit of the measured discharges.

The basic relationships describing the flow in an open channel is the Manning-Strickler formula. This formula uses the hydraulic radius as the depth parameter and a value of 5/3 for the exponent, n. A useful method has been presented to check if the measured flows in the downstream channel section reflect the open channel flow according to Manning-Strickler. In many cases, the observed flows show a trend differing somewhat from the theory. This is caused by either large gaps in observation dates or non-uniformity of flow in the downstream channel section, or both.

Variables are restricted in the KD formula to obtain a result that approaches the Manning-Strickler formula. However, there are three difficulties in comparing the resulting ratings with the Manning-Strickler formula: (1) the need to relate the rating to a gauge reading; (2) the preference to use the mean hydraulic depth instead of hydraulic radius; and (3) the non-uniformity of channel flows.

If the gauge correction, ΔG, is restricted to reflect a mean gauge correction obtained from the mean hydraulic depth, while the exponent, n, is put at 5/3, there is only one variable left, the coefficient, K. This One Variable Approach is inaccurate. The most important reason is that the rating is forced to reflect the Manning-Strickler formula, which is not realistic keeping in mind the above stated disruptions. A method has been introduced to vary the gauge correction a little, thereby achieving an improved rating. This method achieves the desired results, but is rather cumbersome.

Restricting only one variable leaves enough space for obtaining an accurate rating. In this report, methods have been presented restricting either the gauge correction, ΔG, or the exponent, n. Both give quite accurate ratings, but practically the Two Variables Approach with the exponent, n, and coefficient, K, as variables and the gauge correction, ΔG, restricted, is more preferred. In this case, the rating can be made both graphically and mathematically in a single effort, while in the Two Variables Approach with exponent, n, restricted to 5/3, the rating has to be obtained by trial-and-error using an indicator for accuracy of the rating.

A downstream gauge rating is much less durable than a structure calibration. A change in the profile downstream of a structure affects the downstream gauge rating, but it does not disrupt the structure calibration. If the change in bed profile is limited to a small change in the bed level, the downstream gauge rating can be updated with a small likewise change in the gauge correction, ΔG. To check the downstream gauge rating, either an additional discharge measurement or a gauge reading using a structure calibration will be required.

To regularly monitor the downstream channel profile at a marked place with reference to an established benchmark is recommended. This will be an additional indicator if the downstream gauge rating needs to be revised.

9.2. RECOMMENDATIONS

Based upon the conclusions described above, the following recommendations can be stated:

1. By using the mean hydraulic depth to arrive at the gauge correction, ΔG, along with setting the exponent, n, as 5/3, then the coefficient, K, is the single variable that can be plotted against $G\text{-}G_0$ to arrive at a discharge rating, although the procedure is cumbersome.

2. The Two Variables Approach with the exponent, n, and coefficient, K, as variables, with the gauge correction, ΔG, being defined by the mean hydraulic depth, is preferred; in this case, the discharge rating can be established both graphically and mathematically in a single effort.

3. In the Three Variables Approach, the best fit to the discharge measurements will be obtained, but this is actually "curve fitting", which may not be founded on the physical hydraulic situation, particularly if one or two discharge measurements are in error by 5-10 percent.

4. Because the downstream gauge rating needs periodic adjustments, the most preferred approach, but also the most time consuming in the short-term, is to simultaneously calibrate the downstream gauge and the canal discharge regulating structure, which will save time in the long-term by facilitating frequent adjustments to the downstream gauge rating.

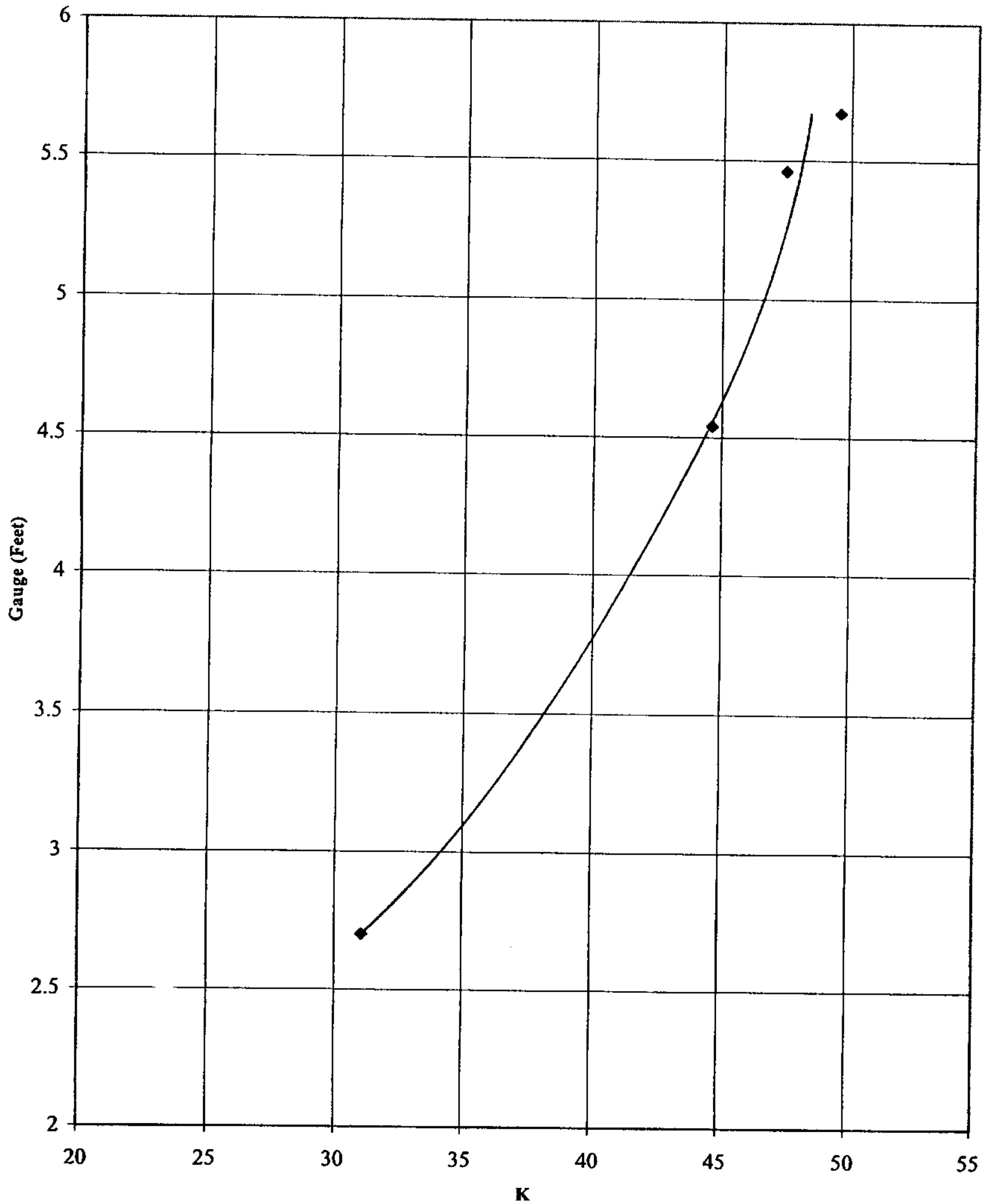
REFERENCES

Public Works Department (Irrigation Branch), 1961. A Manual of Irrigation Practice, Government Printing Press, Lahore.

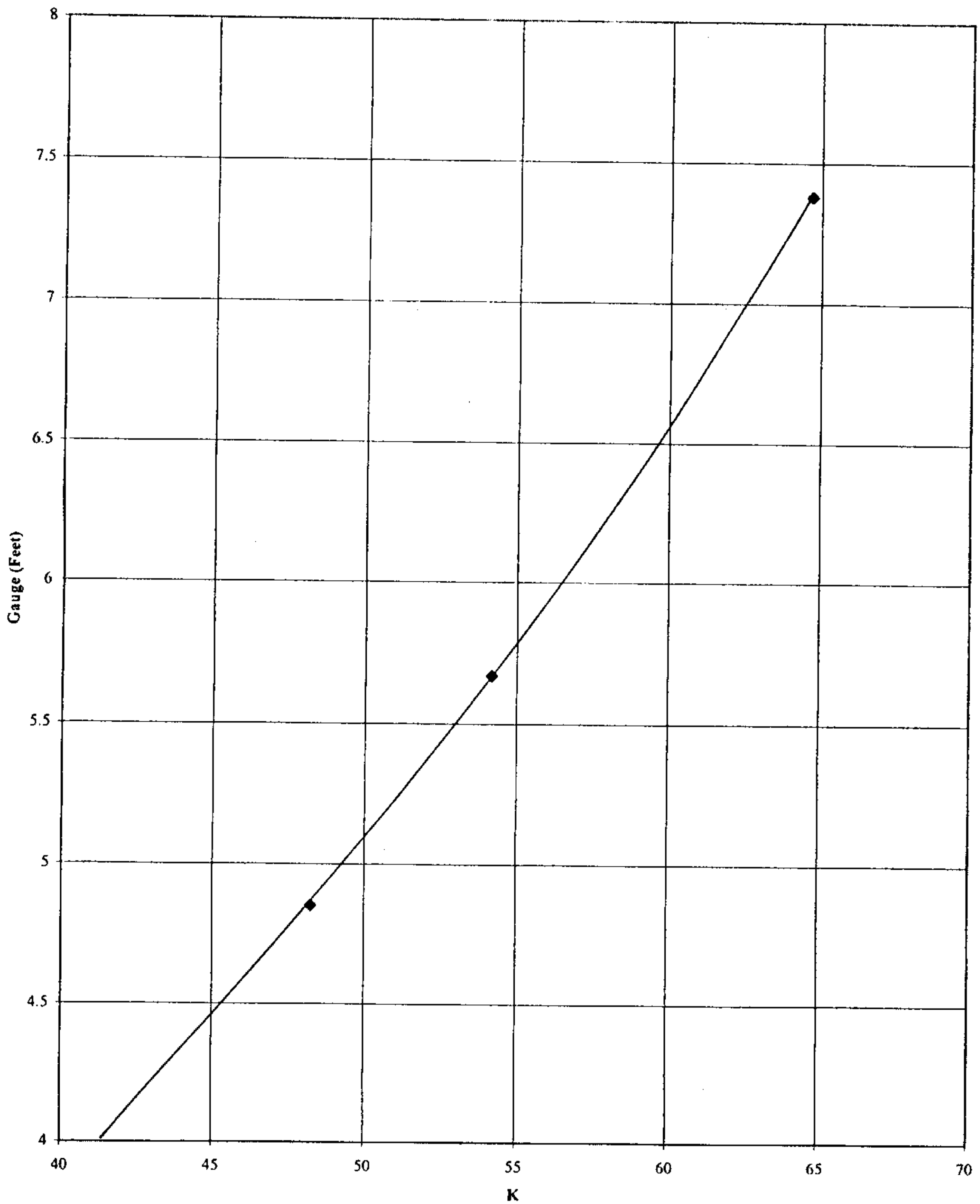
ANNEXURE A.

THE K~G RELATIONSHIPS FOR SELECTED IRRIGATION CHANNELS USING THE SINGLE VARIABLE APPROACH.

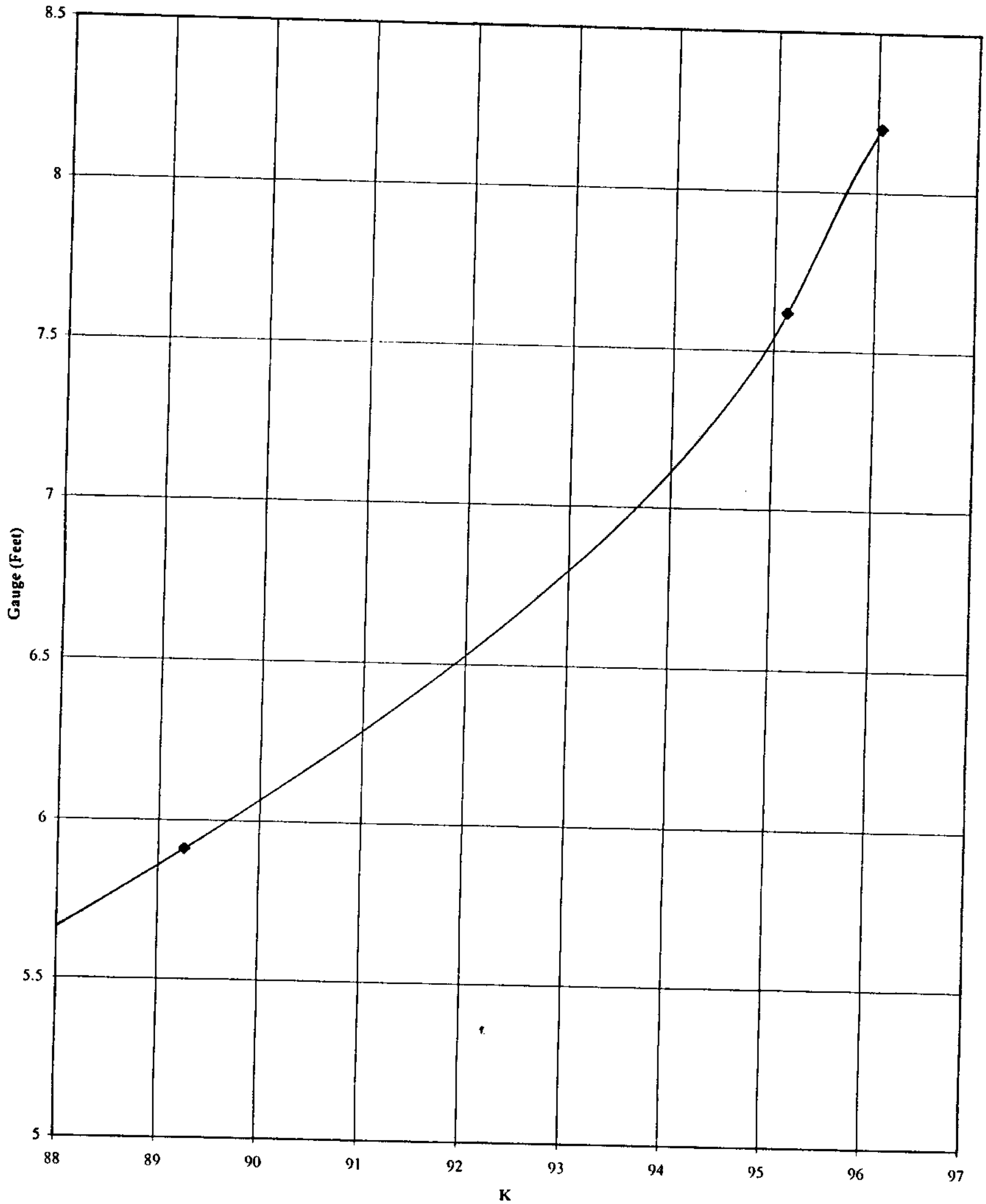
Head 6-R Distributary



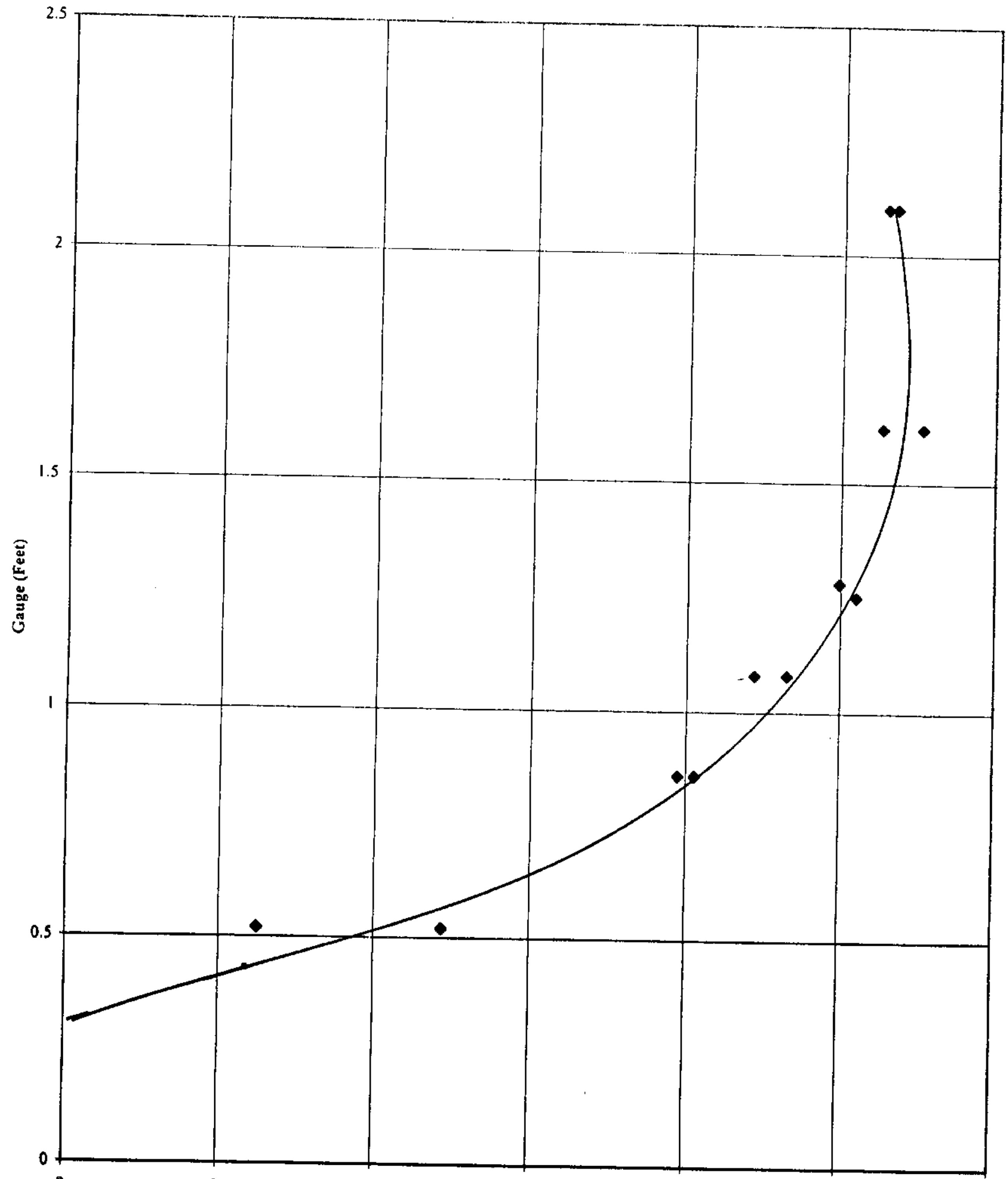
Head Malik Branch Canal



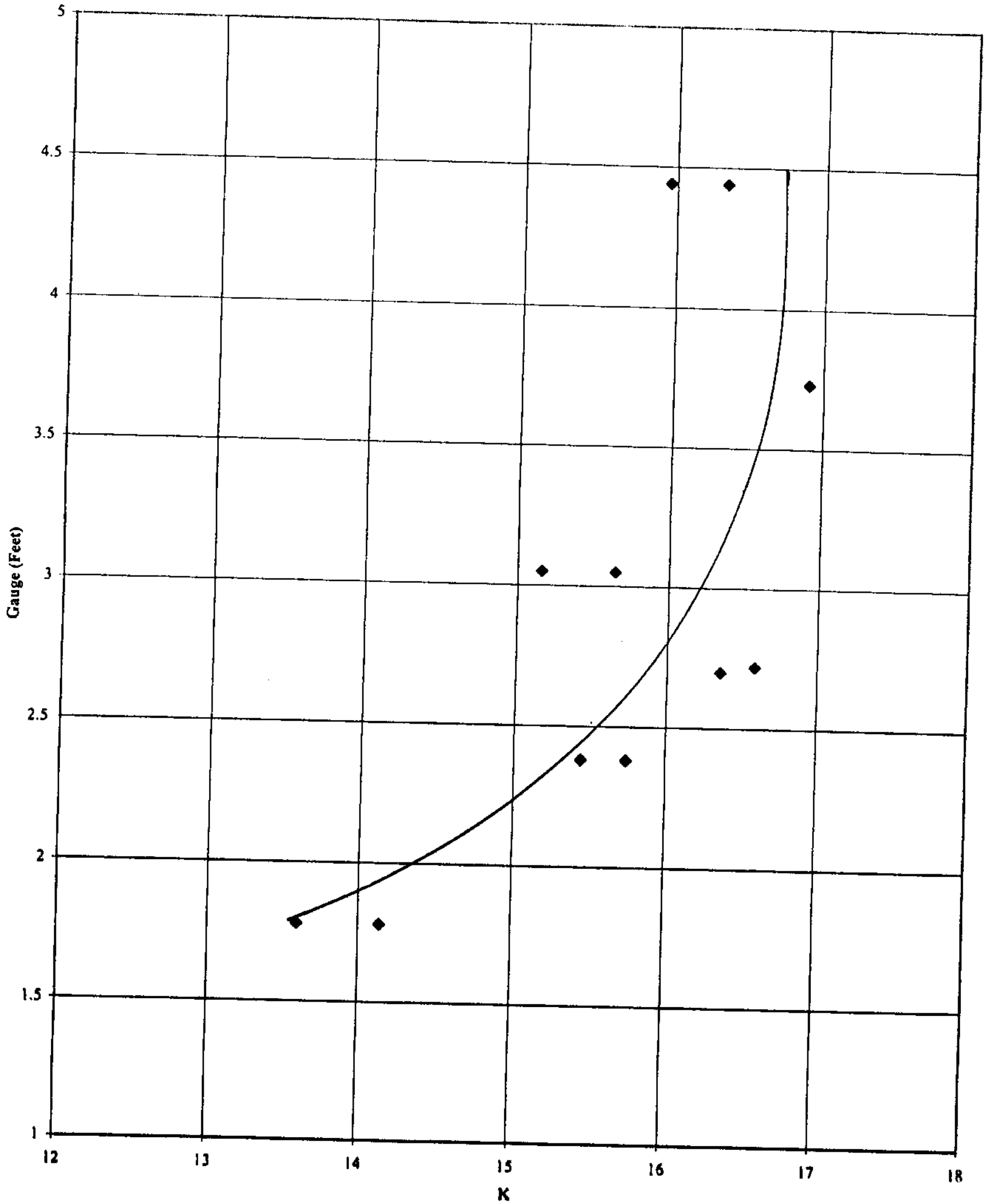
Head Hakra Branch Canal



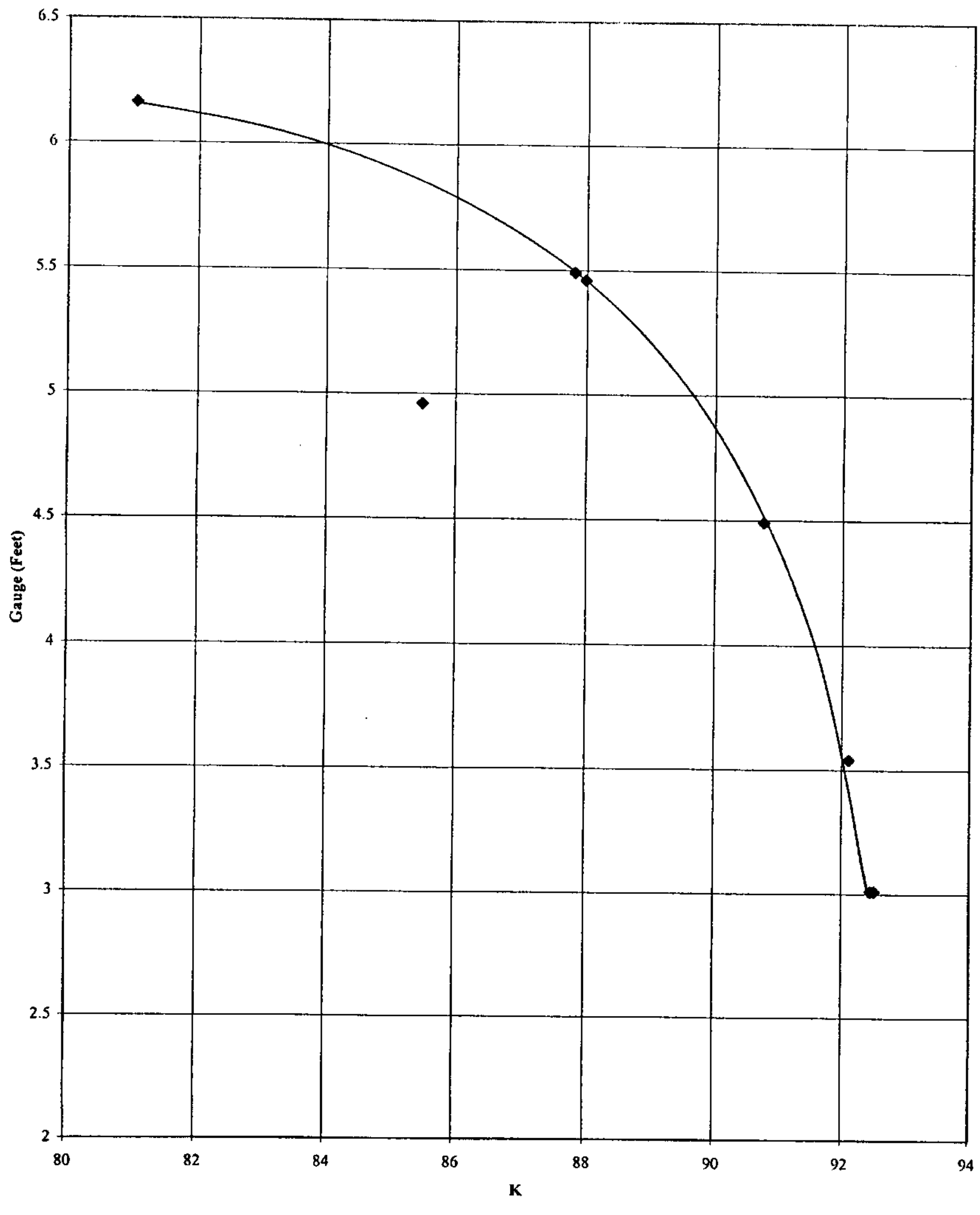
Geodi Minor Canal



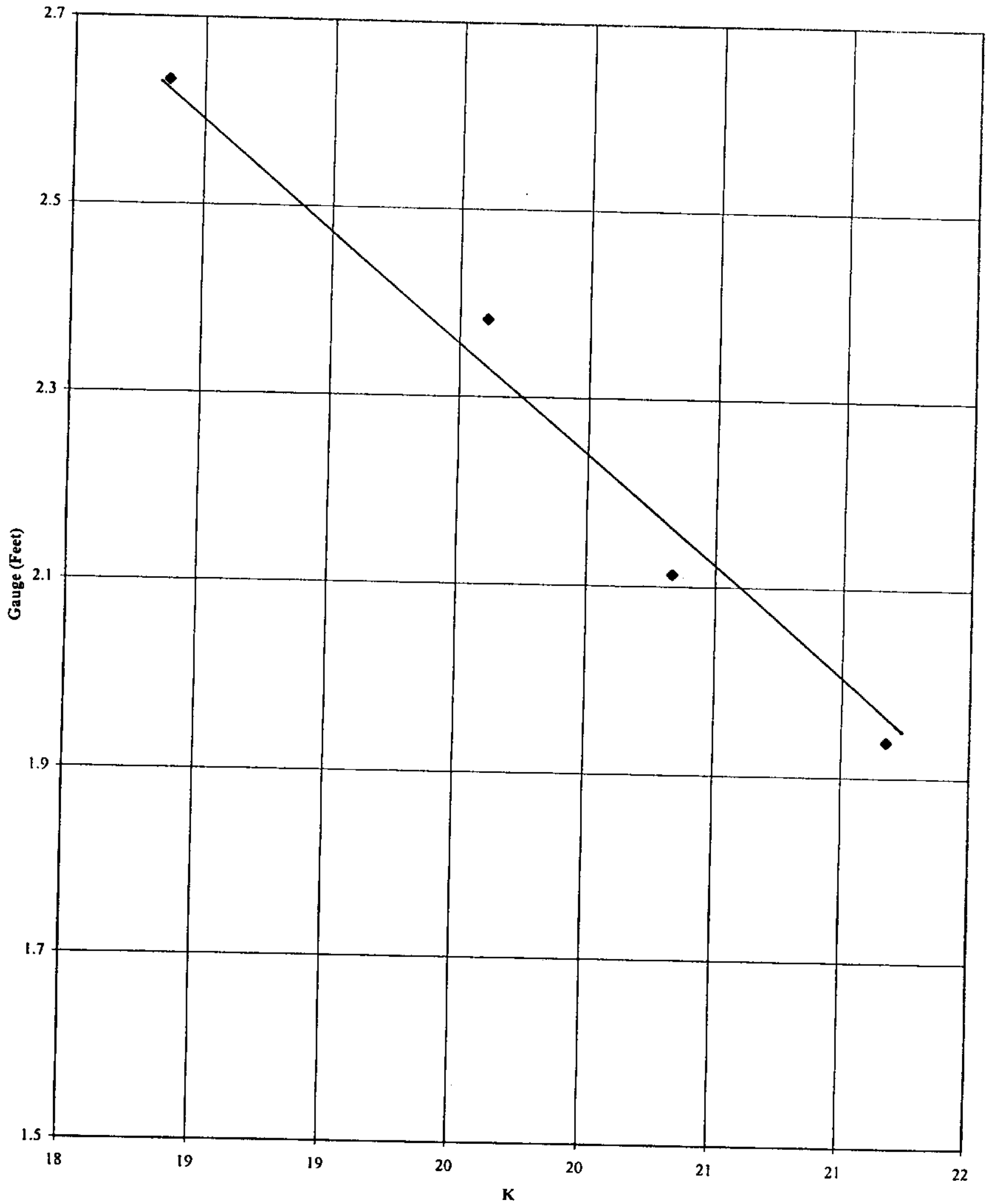
Khan Mahi Branch Canal



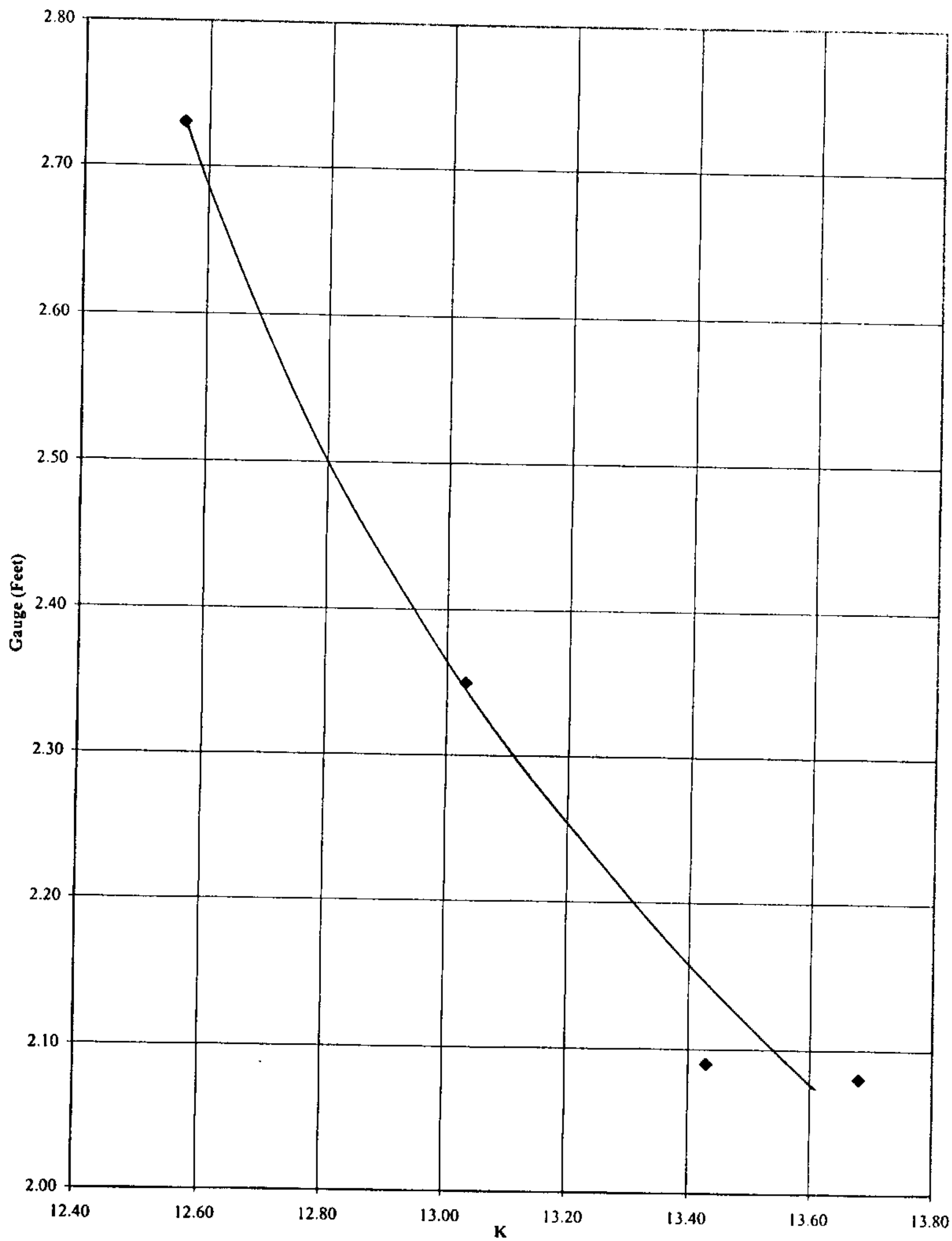
Lower Swat Canal



Mirpur Distributary

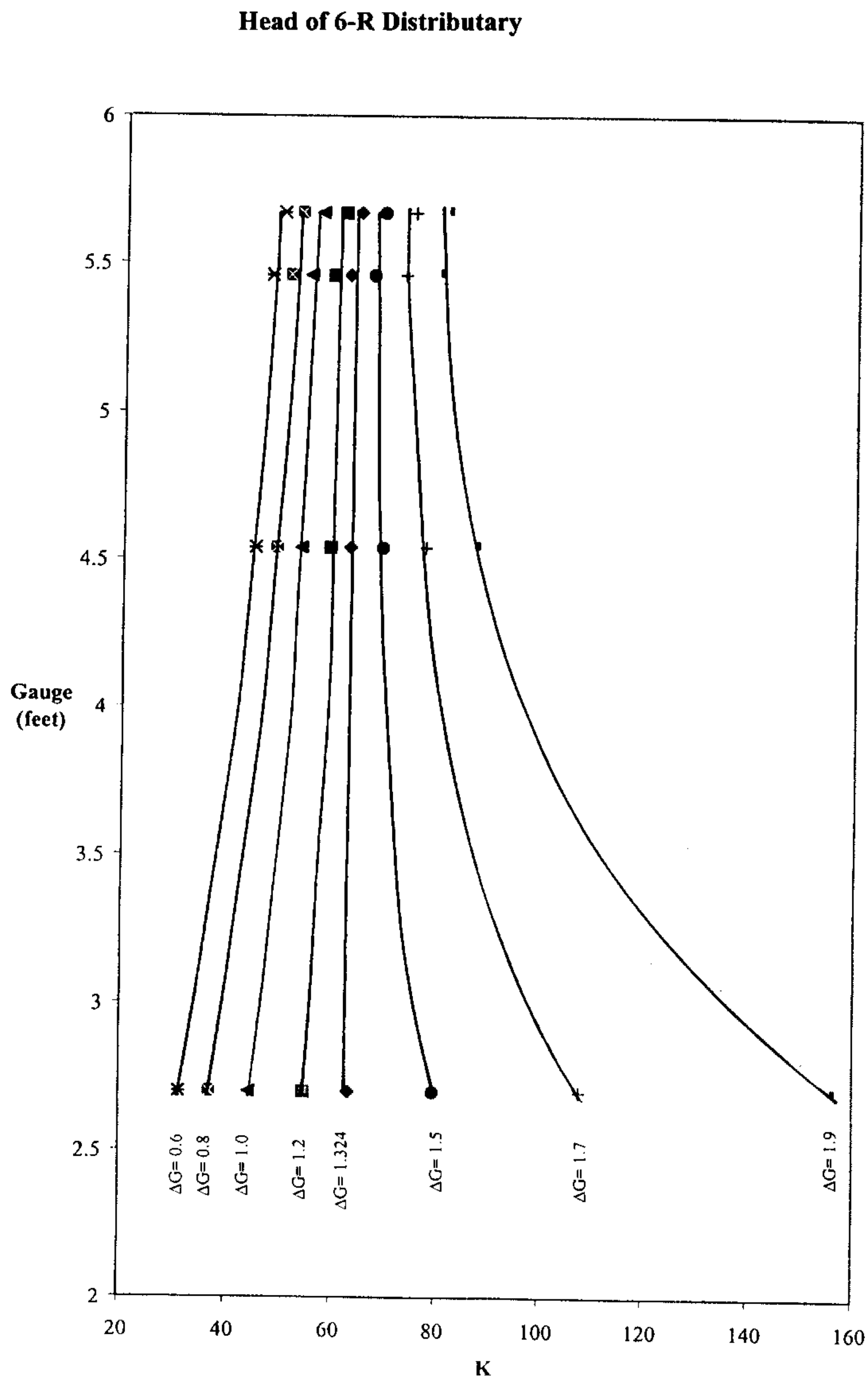


Daulatpur Minor Canal

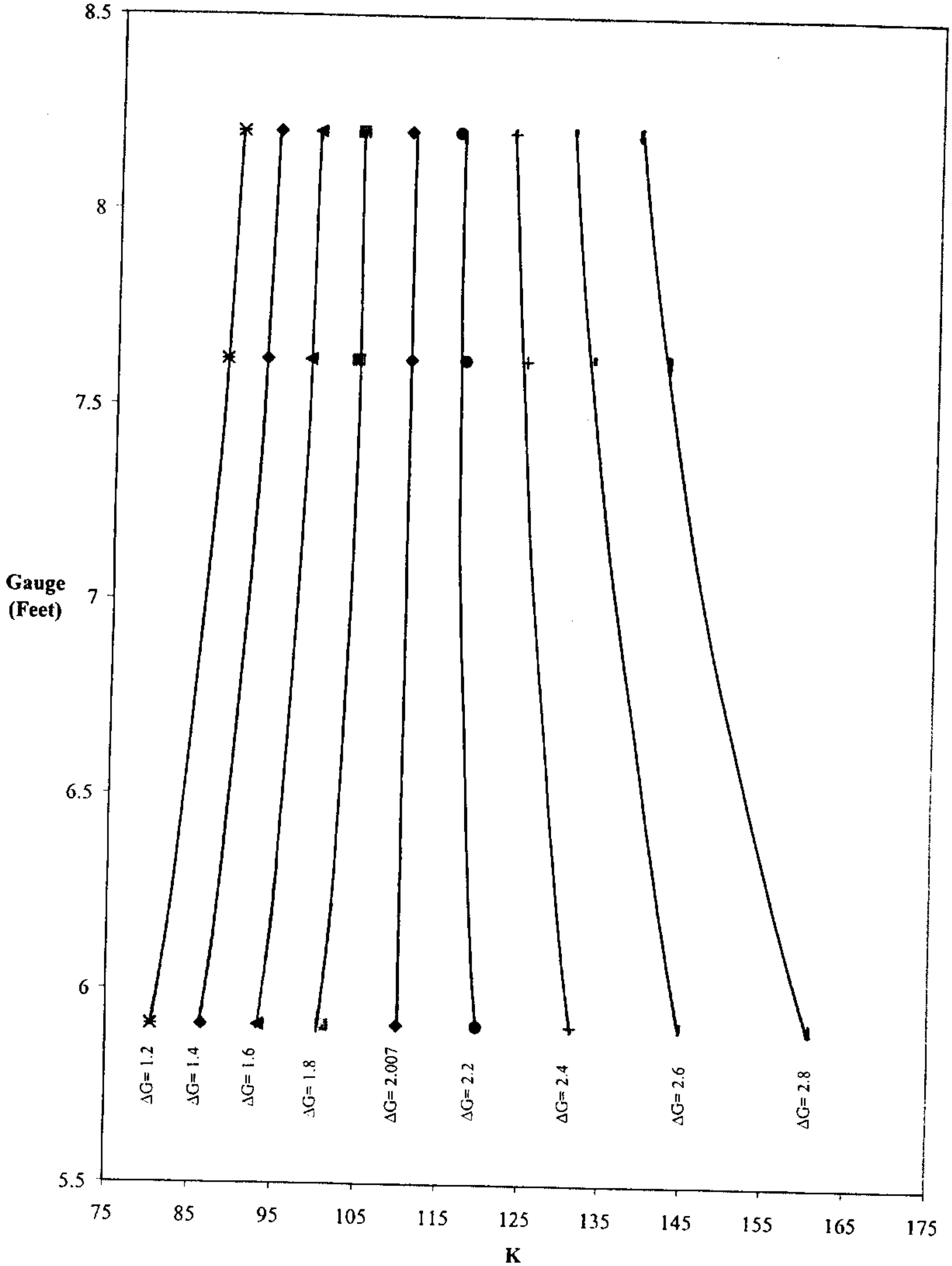


ANNEXURE B.

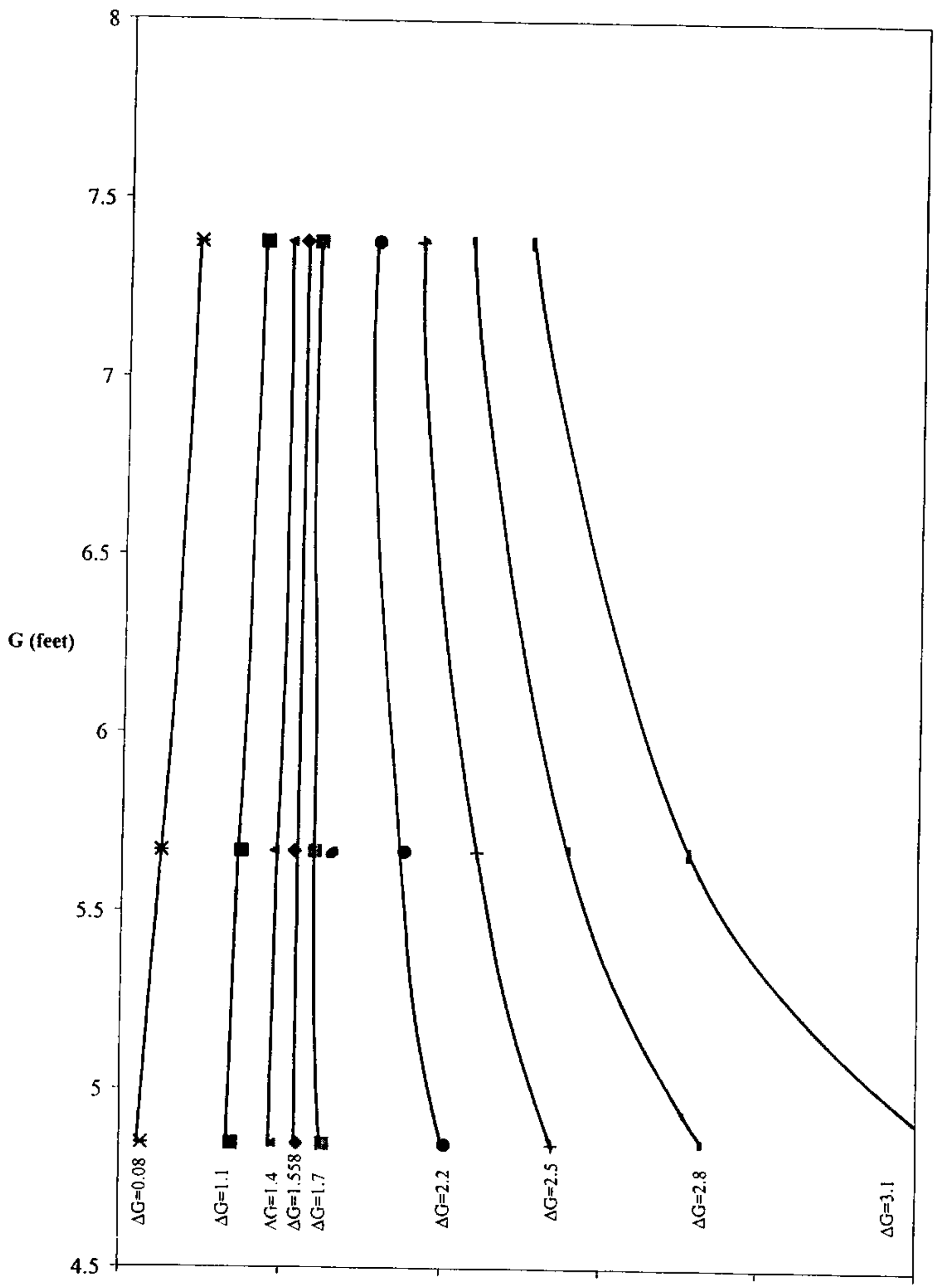
DERIVATION OF GAUGE CORRECTION YIELDING THE COEFFICIENT, K, AS A CONSTANT FOR SELECTED IRRIGATION CHANNELS USING THE SINGLE VARIABLE APPROACH.


Head of 6-R Distributary
Gauge
(feet)
6
5.5
5
4.5
4
3.5
3
2.5
2
20
40
60
80
100
120
140
160
K
ΔG= 0.6
ΔG= 0.8
ΔG= 1.0
ΔG= 1.2
ΔG= 1.324
ΔG= 1.5
ΔG= 1.7
ΔG= 1.9

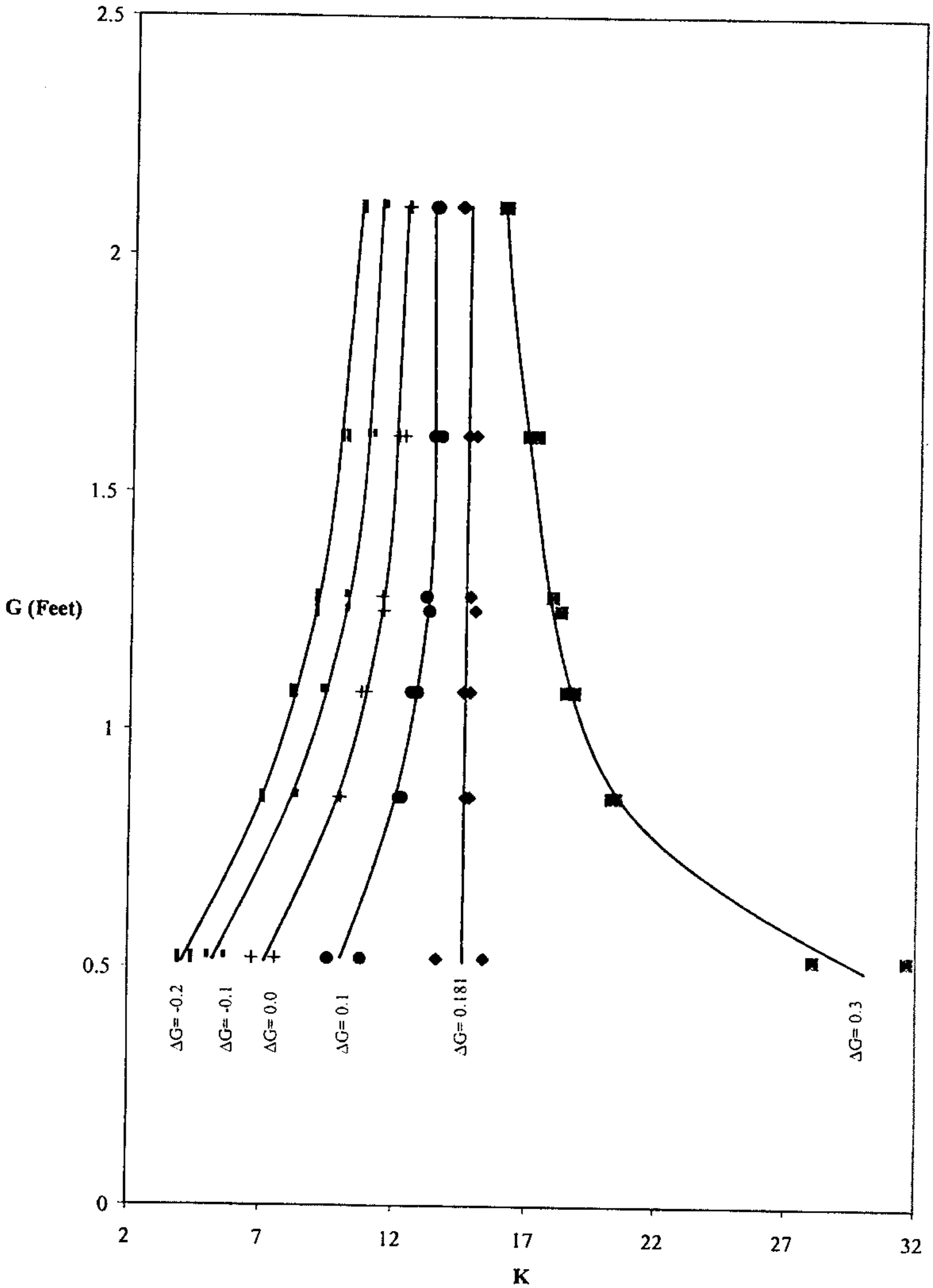
Head of Hakra Branch Canal



Head of Malik Branch Canal

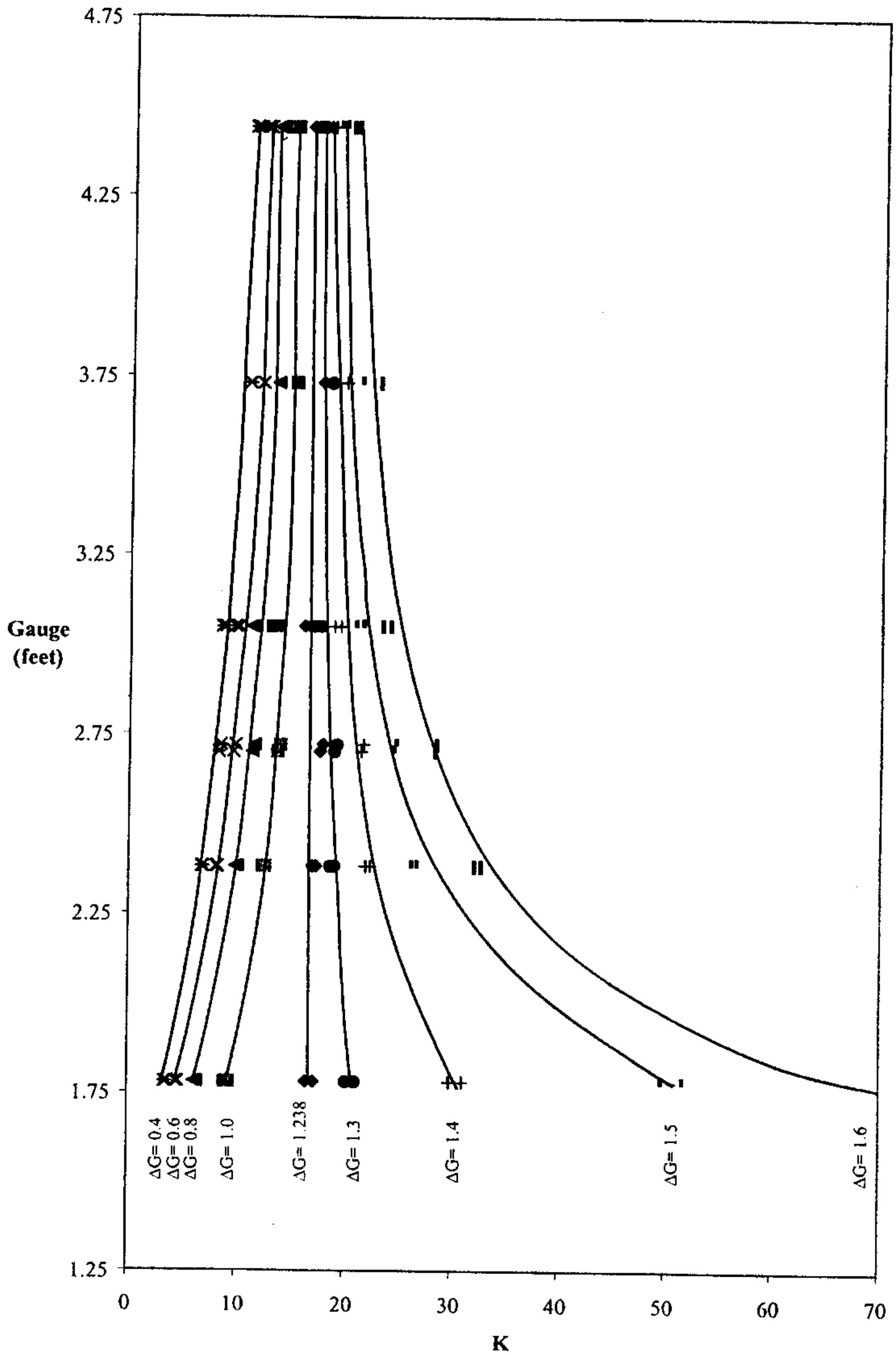


Geodi Minor

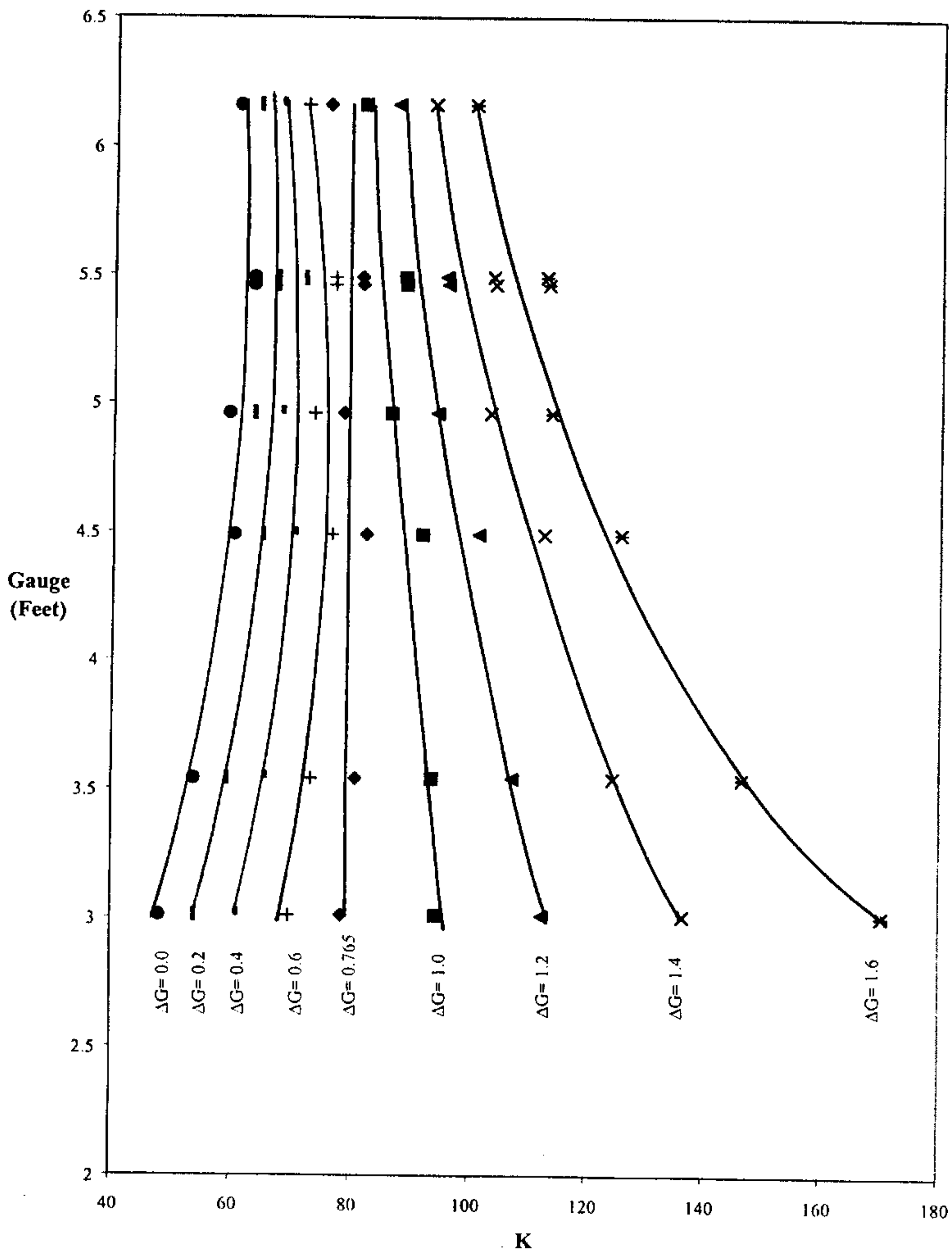


2.5
2
1.5
G (Feet)
1
0.5
0
2
7
12
17
22
27
32
K
ΔG= -0.2
ΔG= -0.1
ΔG= 0.0
ΔG= 0.1
ΔG= 0.181
ΔG= 0.3

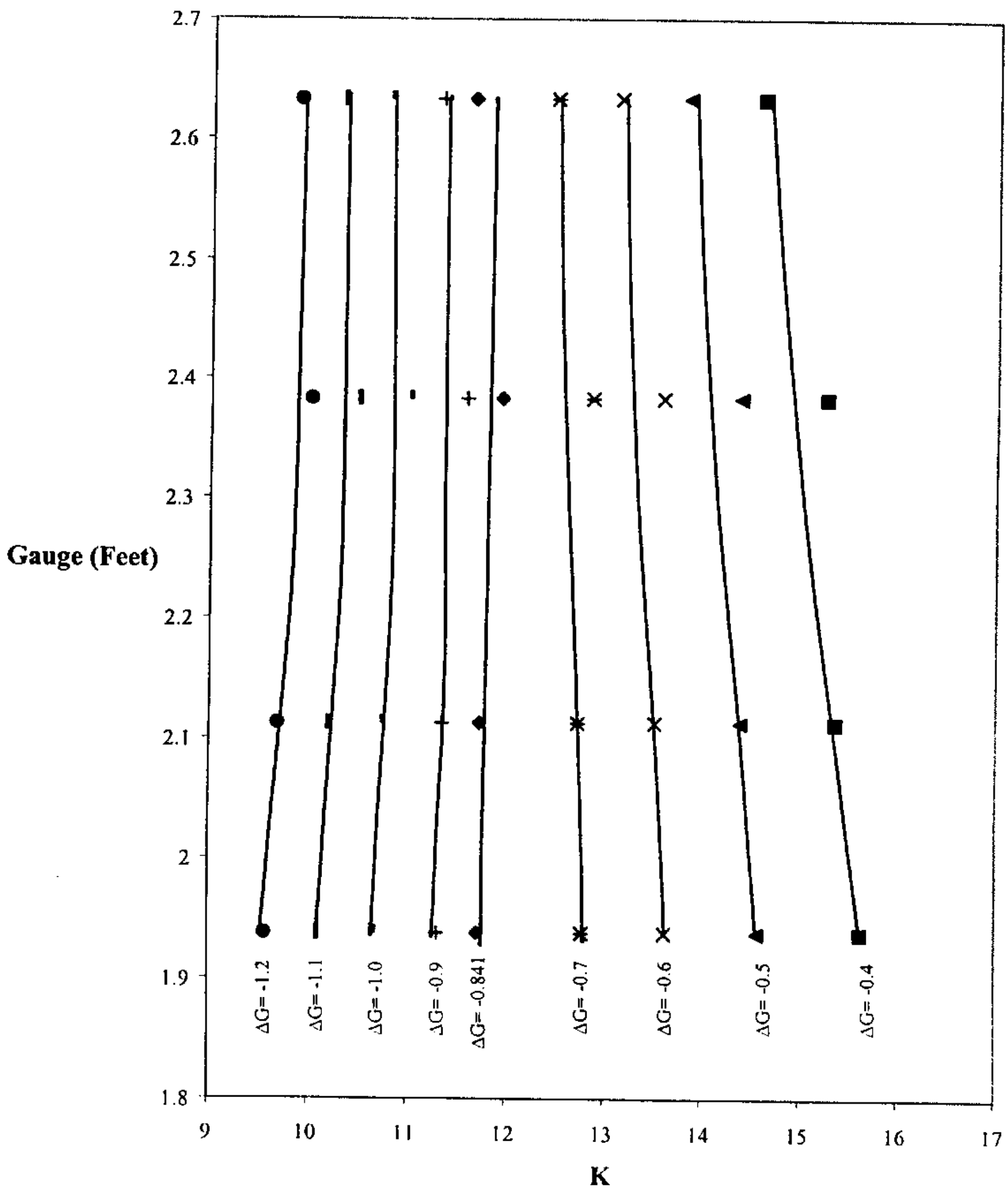
Khan Mahi Branch



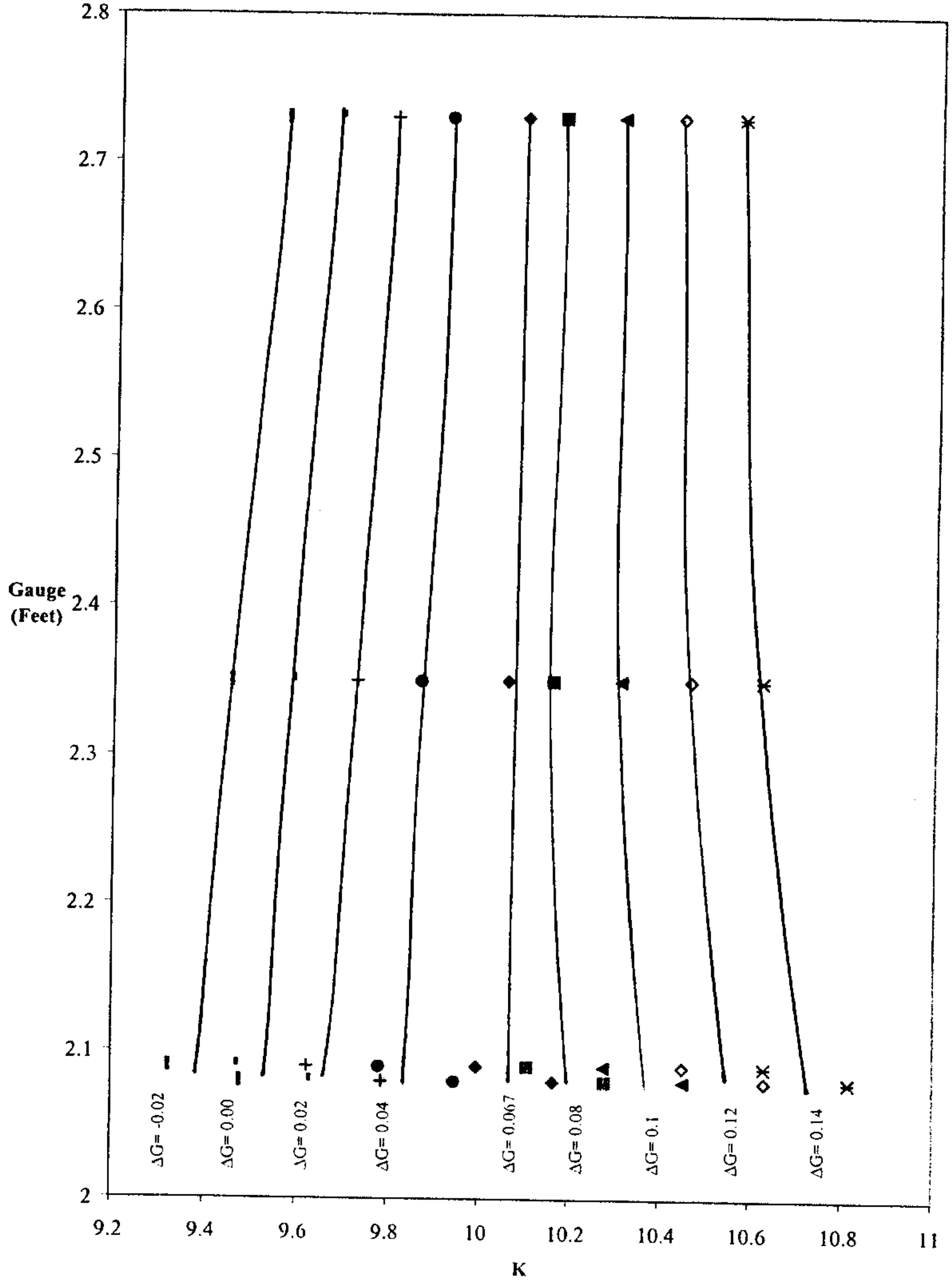
Lower Swat Canal



Mirpur Distributary



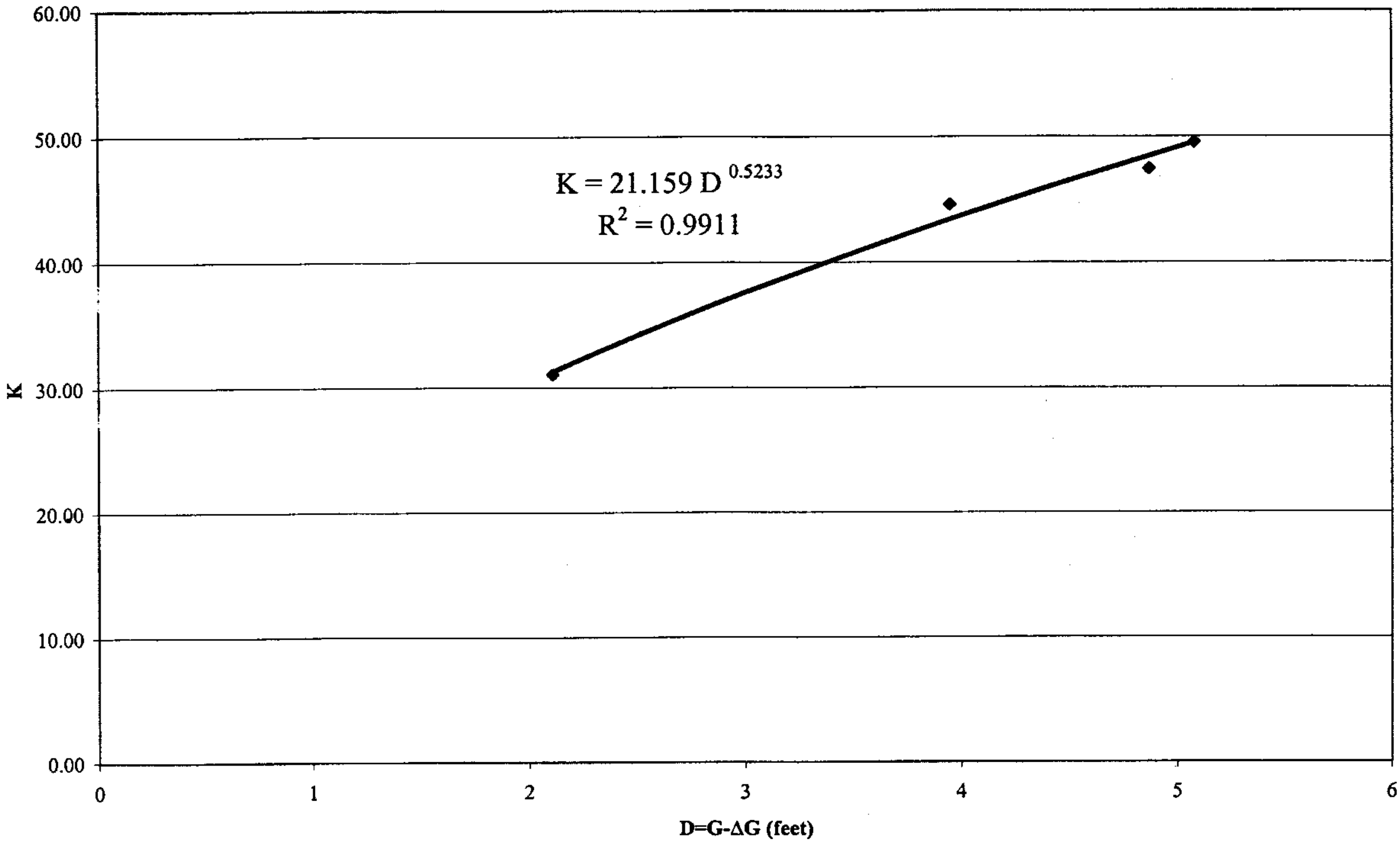
Daulatpur Minor

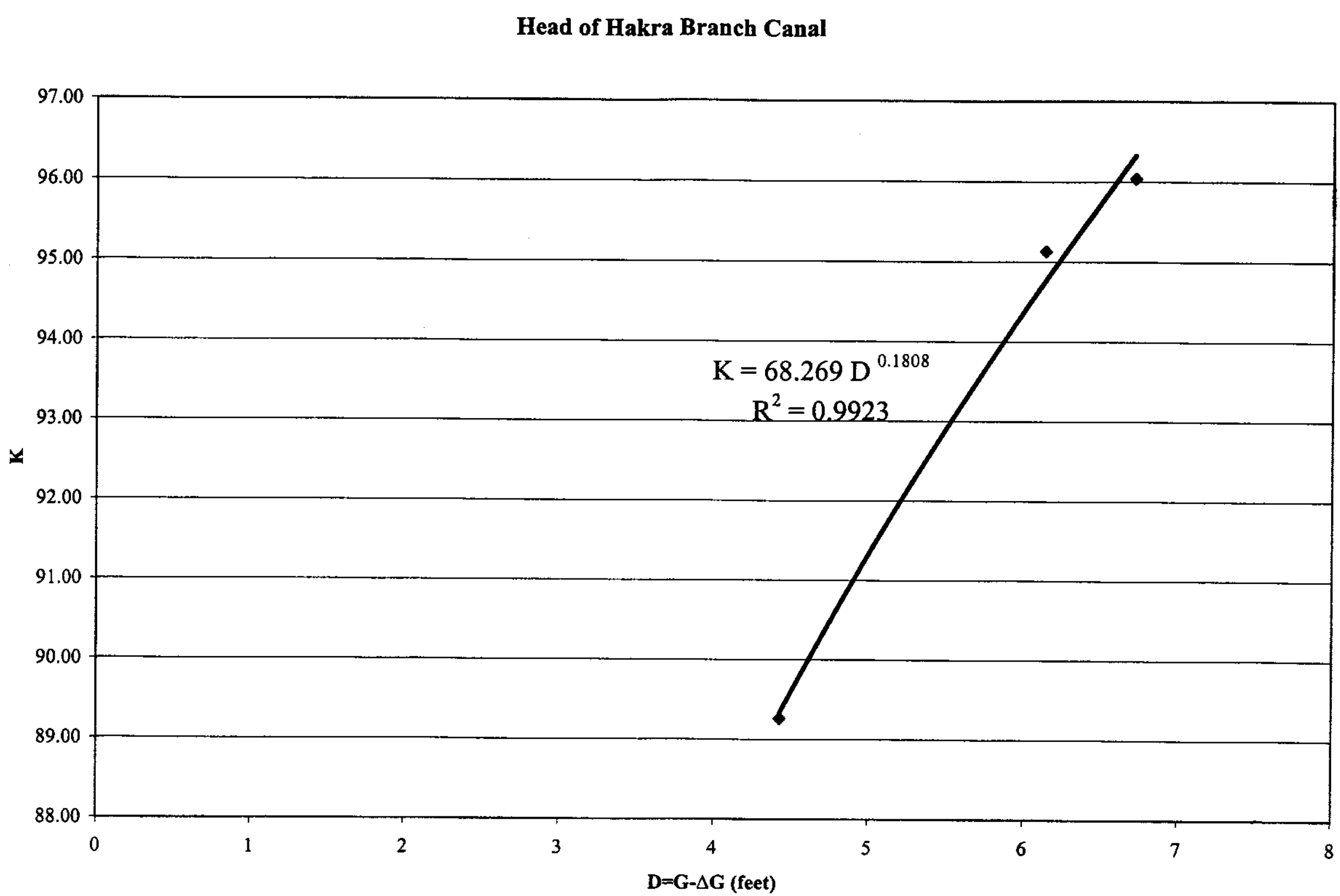


ANNEXURE C.

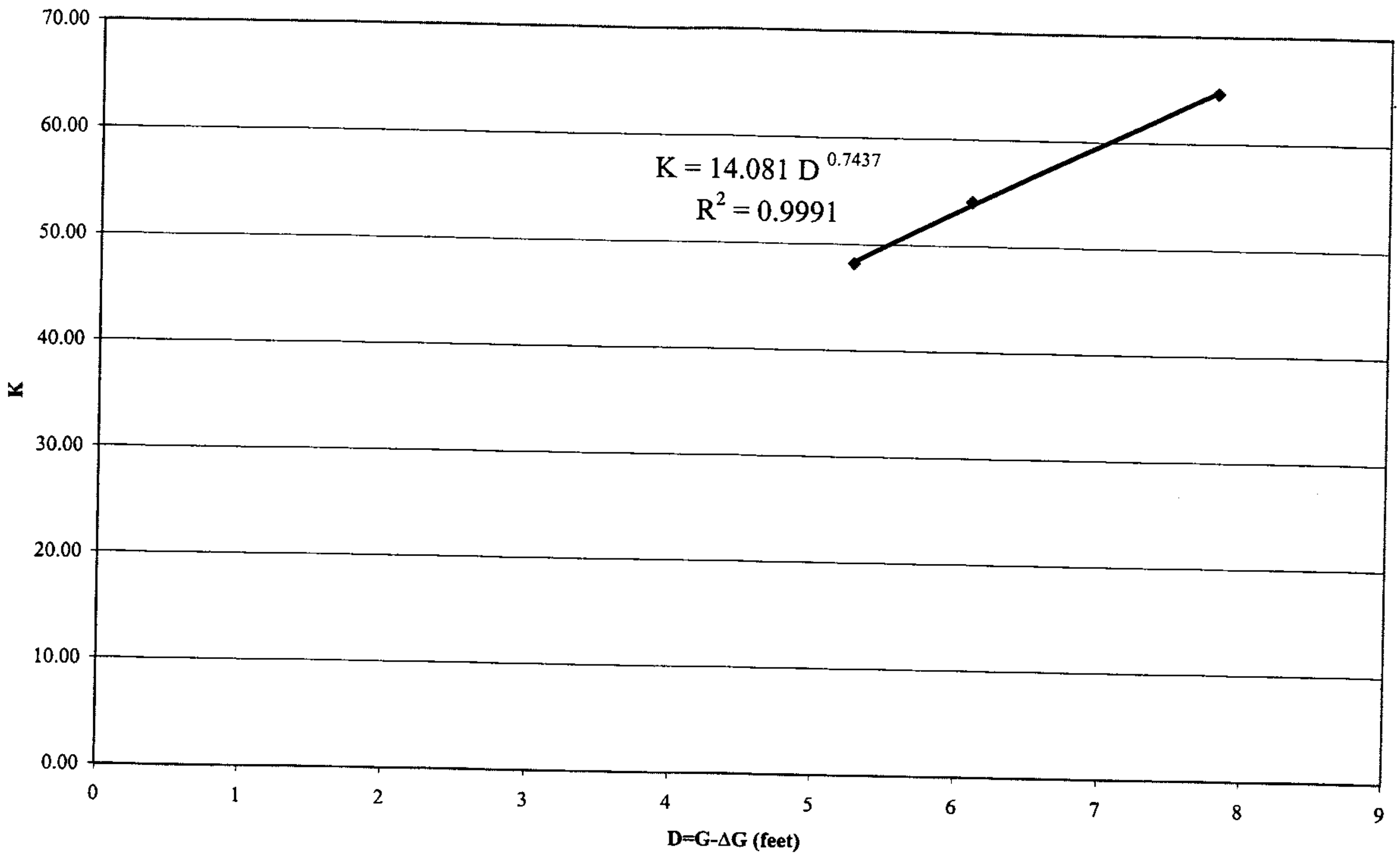
THE K-D RELATIONSHIPS FOR SELECTED IRRIGATION CHANNELS USING THE SINGLE VARIABLE APPROACH

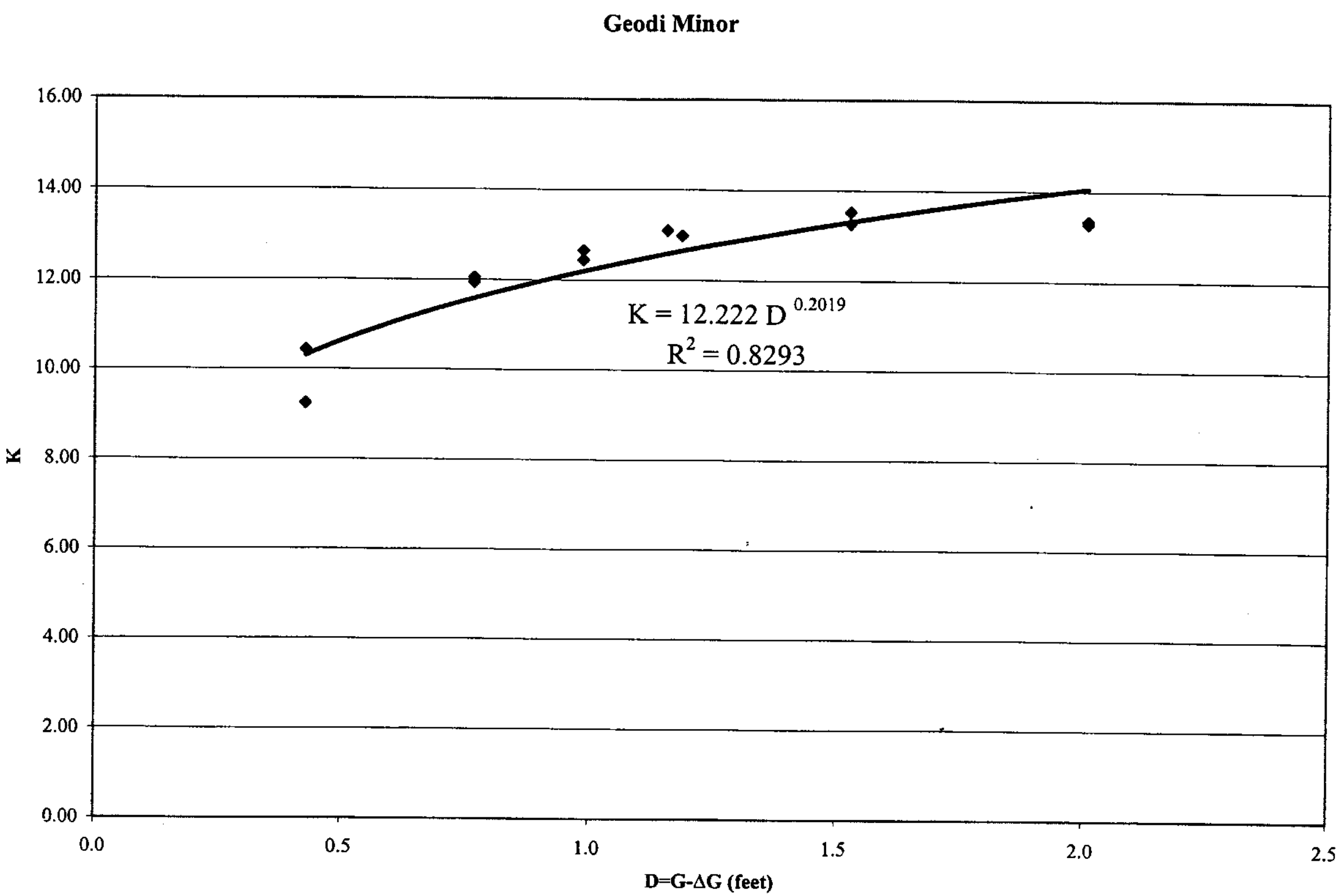
Head of Hakra 6-R Distributary




Head of Hakra Branch Canal
K = 68.269 D 0.1808
R2 = 0.9923
97.00
96.00
95.00
94.00
93.00
92.00
91.00
90.00
89.00
88.00
K
0
1
2
3
4
5
6
7
8
D=G-ΔG (feet)

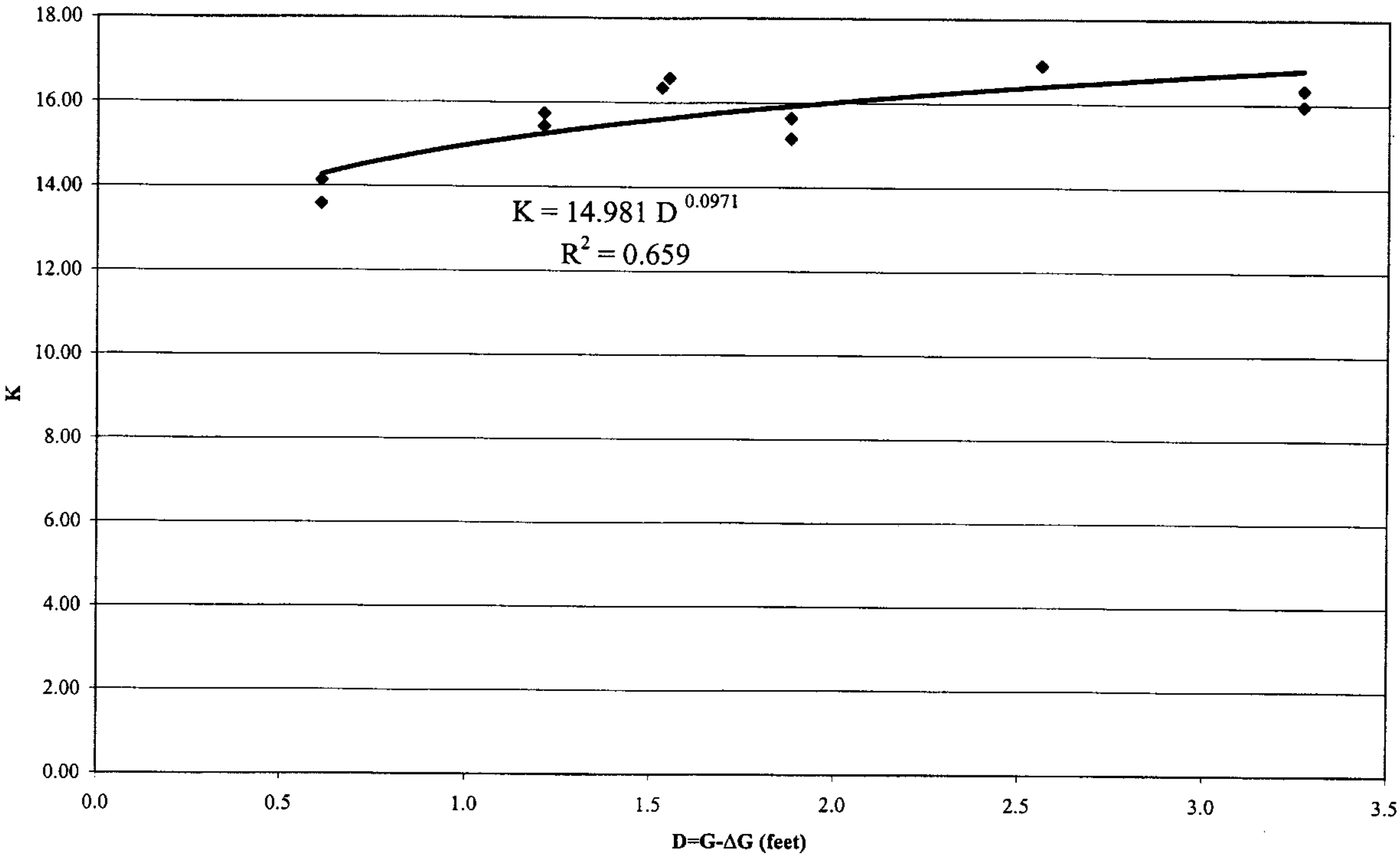
Head of Malik Branch Canal

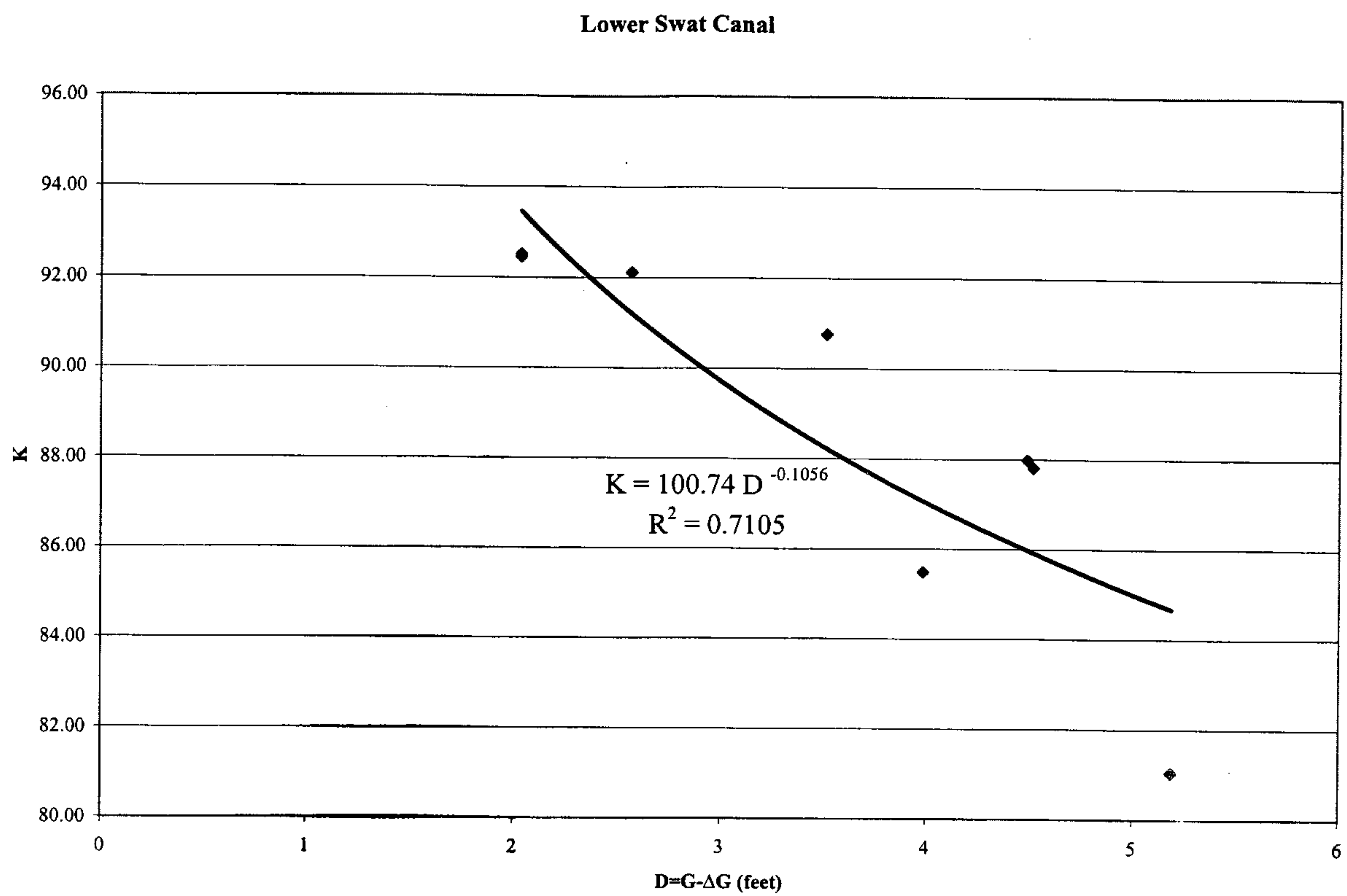


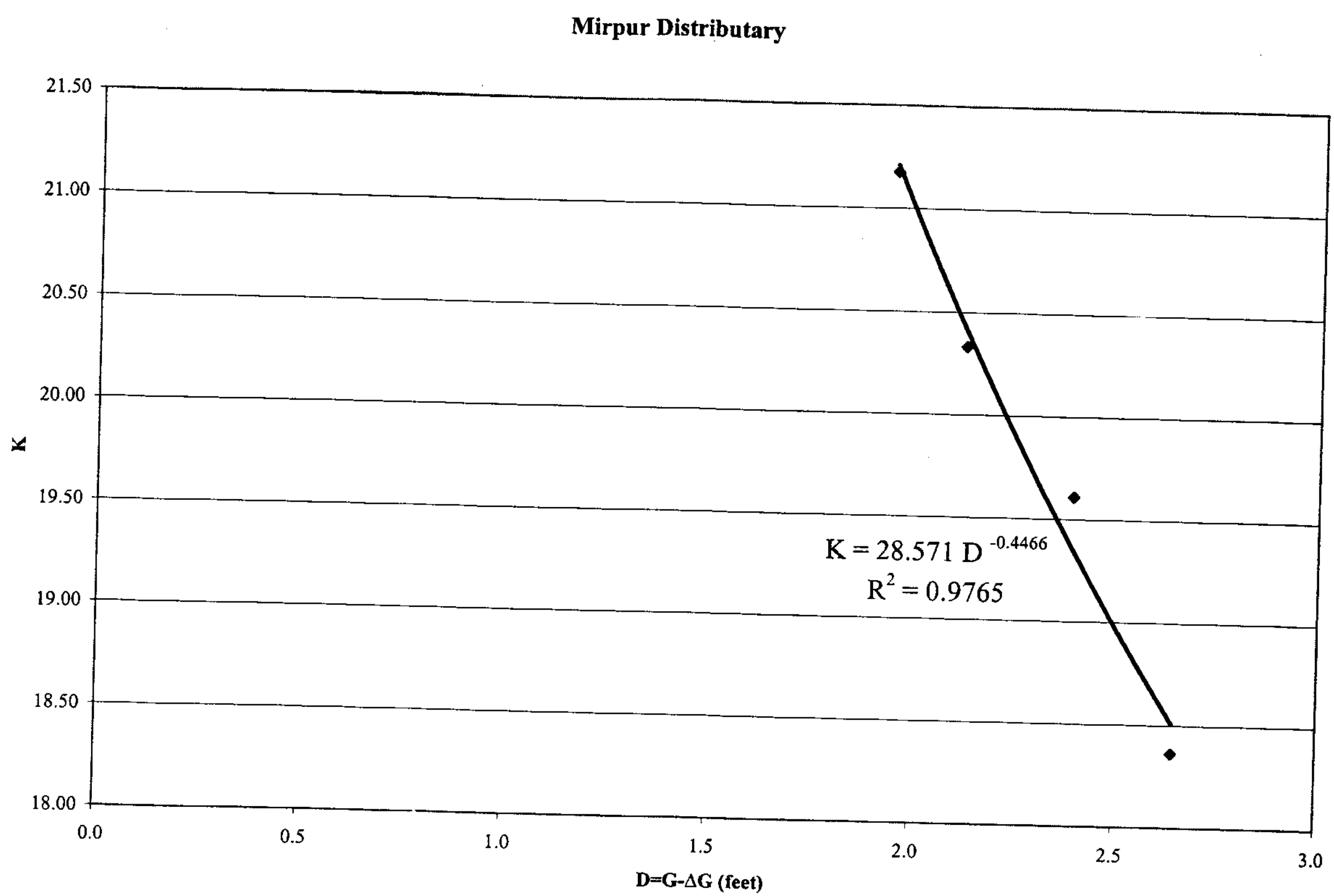


Geodi Minor
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
K
0.0
0.5
1.0
1.5
2.0
2.5
D=G-ΔG (feet)
K = 12.222 D 0.2019
R2 = 0.8293

Khan Mahi Branch Canal

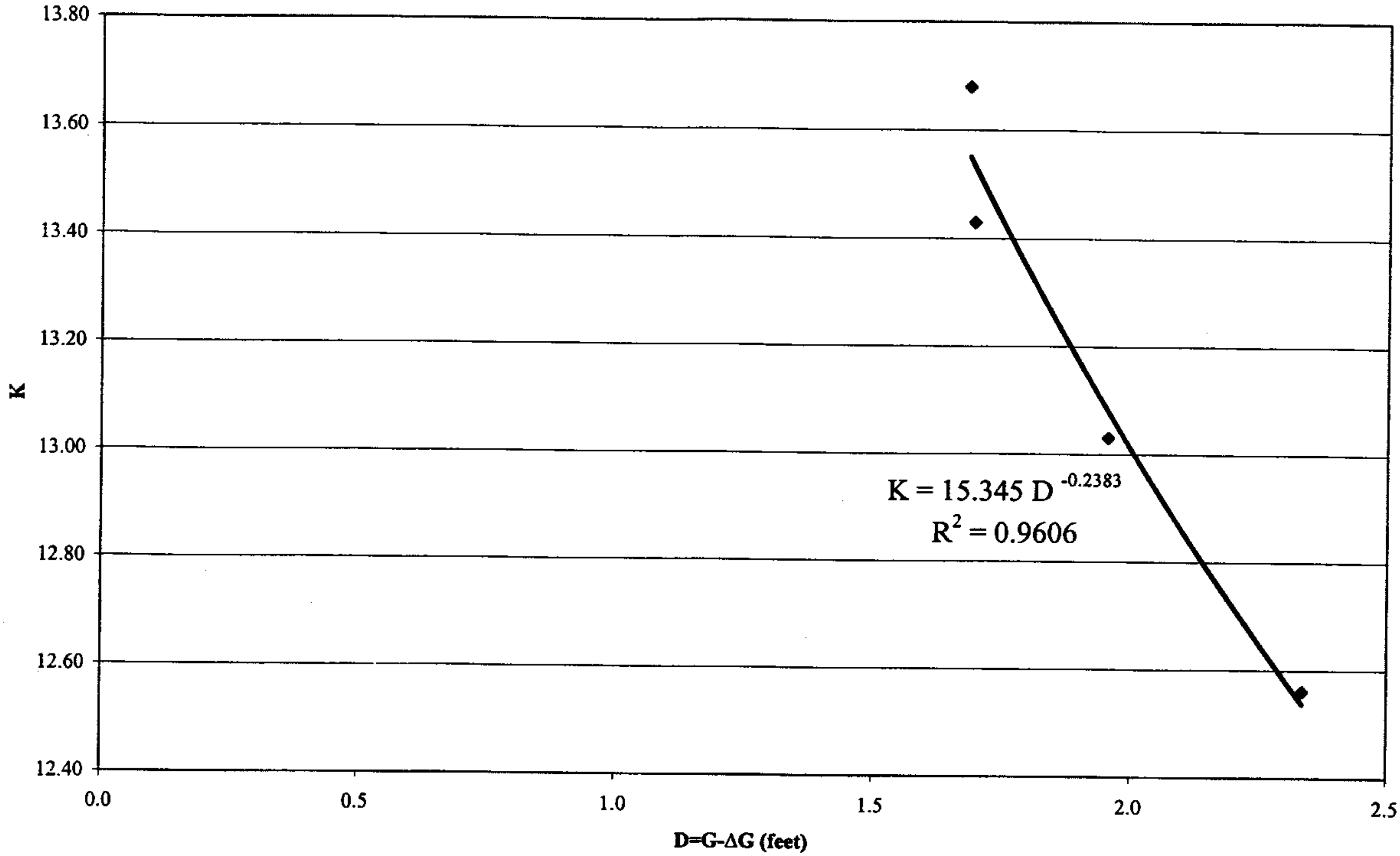



Lower Swat Canal
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
80.00
K
K = 100.74 D -0.1056
R2 = 0.7105
0
1
2
3
4
5
6
D=G-ΔG (feet)



Mirpur Distributary
K
D=G-ΔG (feet)
K = 28.571 D^-0.4466
R^2 = 0.9765
21.50
21.00
20.50
20.00
19.50
19.00
18.50
18.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0

Daulatpur Minor



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