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EXAMINATION OF FLOW MEASURING ALTERNATIVES FOR MILK RIVER AT EASTERN CROSSING

by

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SUMMARY

There is a need to explore alternatives to present methods of obtaining discharge data at the hydrometric site on the Milk River at Eastern Crossing, Montana. The alternate methods of a) ultrasonic, b) electromagnetic and c) artificial controls have been examined. Five different designs of artificial controls have been selected for testing in a physical scale model. The design of the scale model is presented.

SOMMAIRE

Il semble nécessaire d'étudier d'autres méthodes que celles qui sont employées actuellement pour obtenir des données sur le débit au site de relevés hydrométriques de la rivière Milk à Eastern Crossing, Montana. Les méthodes suivantes ont été examinées : a) contrôle par ultrasons, b) analyses électromagnétiques et c) contrôles artificiels. Cinq différents contrôles artificiels ont été choisis et soumis à des essais dans un modèle à l'échelle. Le plan de ce modèle est présenté.

MANAGEMENT PERSPECTIVE

Accurate discharge data are required for gauge sites such as the one on the Milk River at Eastern crossing so that the apportionment of water between Canada and the United States as dictated by the Boundary Waters Treaty can be satisfied. However, the streamflow records at this site are inaccurate because of unstable stage-discharge relationships caused by shifting channel bed and migrating sand waves.

This report examines several alternate methods of obtaining discharge data and outlines the model tests which are required. The solution for the Milk River will be applicable to other similar gauge sites.

A/Chief
Hydraulics Division

PERSPECTIVE-GESTION

Il est important d'obtenir des données exactes sur le débit à des sites de jaugeage comme celui de la rivière Milk à Eastern Crossing de façon à être en mesure de respecter les stipulations du Traité des eaux limitrophes concernant la répartition des eaux entre le Canada et les États-Unis. Cependant, à ce site de jaugeage, les enregistrements des débits du cours d'eau sont imprécis à cause des relations niveau-débit instables causées par la migration du lit du canal et celle des dunes hydrauliques.

Dans ce rapport, nous examinons plusieurs autres méthodes permettant d'obtenir des données sur le débit et expliquons à quels essais sur le modèle il faut procéder. La méthode qui permettra de résoudre le problème de la rivière Milk sera appliquée à d'autres sites de jaugeage semblables.

Le chef intérimaire
Division de l'hydraulique

1.0 INTRODUCTION

The Milk River and its main tributary, the North Milk River, originate in the Rocky Mountains foothills of Montana, flow north-easterly into Alberta and join near the town of Milk River, (Figure 1). From this confluence, the Milk River flows south-easterly about 125 km before re-entering the state of Montana at Eastern Crossing and eventually joining the Missouri River.

During the irrigation season (April 1 to October 31) water is normally diverted from Lake Sherbourne, a storage reservoir in the adjoining St. Mary's River basin in Montana, via the St. Mary's Canal to the North Milk River. This diversion commenced on July 3, 1917 and has since diverted an annual flow of 178 x $10^6 m^3$/year, mostly during the irrigation season (Bradley, Smith, 1984). The Milk River is then used as a conveyance channel to provide water for irrigation in the lower portion of the Milk River in Montana after passing through Canada (Grove, 1985). The apportionment of the water between Canada and the United States is dictated by the Boundary Waters Treaty of 1909 and a 1921 order of the International Joint Commission. They provide that during the irrigation season Canada is entitled to three fourths of the natural flow of the St. Mary's River and the United States is entitled to three fourths of the natural flow of the Milk River for flows up to 19 m^3/s. Excess flows and flows during the non-irrigation season are divided equally (Bradley, Smith, 1984).

At the present time the waters between the United States and Canada are apportioned informally. The reason for this is that natural flow calculations are known to be crude, partly because of methods used, but largely because of inaccuracies in streamflow records for the Milk River at Eastern Crossing (Davis, 1984). Cooperative studies between the United States Geological Survey (USGS) and the Water Resources Branch (WRB) over the past three years are leading to an improved procedure for computing natural flows which requires that all input data are sufficiently accurate. All necessary data are considered to be satisfactory except the streamflow records for the Milk River at Eastern Crossing. Due to frequently shifting channel bed and rapidly migrating sandwaves, it has not been possible to obtain a stable stage-discharge relationship. As a result, the streamflow records can be considered to be only "fair". Clearly, a method must be found to improve the accuracy of these records.

WRB has requested the Hydraulics Division (HD) of the National Water Research Institute (NWRI) to develop a method for obtaining a stable and sufficiently sensitive stage-discharge relationship. The work is being done in collaboration with the Water Survey of Canada (WSC) Calgary, Alberta.

2.0 PRELIMINARY CONSIDERATIONS

2.1 Physical Setting

Throughout most of the Milk River basin which has a total area of 6800 km^2 at Eastern Crossing, the topography consists of gently rolling prairie. Rainfall is generally very low, averaging about 300-400 mm per year with about two thirds of this occurring between April and August. As a result, the countryside consists predominantly of grassland and is utilized mainly for ranching (MacLean, Beckstead, 1981).

Downstream of the town of Milk River, several wide coulees intersect the Milk River valley. These features represent former glacial melt-water channels which flowed at the end of the last continental glaciation. For most of its lower course, the Milk River flows in one of these melt-water channels which has the appearance of a steep-walled box canyon. Throughout part of this lower reach the combination of low rainfall, lack of vegetation, and presence of erodible valley deposits has produced extensive areas of badlands which contribute large quantities of silt and sand-sized sediment to the Milk River (MacLean and Beckstead, 1981).

At the Eastern Crossing Hydrometric site the river channel is straight for 350 m upstream from the present gauge, where the river bends sharply to the south, and straight for 800 m downstream from the gauge. The river reach and existing hydrometric facilities are shown in Figure 2. The bed of the stream is composed of sand and silt underlain by sandstone and coal. Both banks are clean except for weeds and very high stages. The right bank (looking downstream) is not subject to overflow, whereas the left bank may overflow during very high floods or as a result of ice jams.

2.2 Changes in Cross-Sectional Shape

The channel bed goes through very dramatic changes with changes in discharge as shown for very high flows and two flows, somewhat above and below the average during the irrigation season (Figure 3). The plots in Figure 3 show that at high flows of 2950 ft^3/s (83 m^3/s) the river scours its bed predominantly in the first 80 ft (24 m) from the right bank to a depth of about 7 ft (2.1 m). When the discharge is about 701 ft^3/s (20 m^3/s) the scour along the right bank side is less with a tendency of increasing the depth toward the left bank. At a discharge of 511 ft^3/s (14.5 m^3/s) the river bed cross-section has become almost uniform. The shape of the cross-section is in part related to the magnitude of the flow. At the highest flows the predominant scour towards the right bank reflects the effect of the sharp bed in the channel about 300 m upstream. The influence of the bend becomes progressively less as the flow decreases.

The change in the cross-section shape requires the movement of large quantities of sediment as can be seen from the superimposed profiles in Figure 3. This variability in the cross-section shape and changes in bed elevations are the reason why it has not been possible to establish a stable stage-discharge relationship at the Eastern Crossing Hydrometric station.

Although the shape of the cross-section is shown to be related to the magnitude of the flow and the upstream bend in the channel, other factors contribute. The most significant of these are the rate of change of flow (shape of the hydrograph) and the sediment introduced into the flow as a result of bank slumping and other sediment sources. Therefore, the river bed cross-section shape will be quite random with the macro features, indicated by the profiles in Figure 3 providing an overall tendency which are largely flow dependent.

2.3 River Flow Variability

Flow measurement methods must take into account the flow variability as well as the flow range in order to achieve the greatest possible accuracy. In addition, because cross-sectional geometry of the river is dependent largely on the discharge, then flow variability becomes important also in selecting a measuring method which is compatible with the expected behaviour of the river bed in the measuring reach.

To determine the flow variability, the 75 years of discharge records for the Milk River at Eastern Crossing were examined. Monthly mean flows were plotted in Figure 4 for the month of March through to the end of October. This time period was selected because the record was complete during this time. The plot shows how the discharge is distributed over the seasons, indicating that the highest flows can be expected during the month of June. An indication of the daily variability can be obtained from Figure 5 which shows the hydrographs of daily mean flows for April, June and October in 1985. The hydrographs show that the change in flow from day to day can be quite substantial as shown by the sharp peak for the month of June and the rapid rise in the flow during the month of April. Such rapid changes in the flow will result in accompanying rapid changes in the cross-sectional shape of the river cross-section.

Along with flow variability, the selection of a method of flow measurement will also depend on the percent of time that a particular discharge is equalled or exceeded. For this purpose a flow duration curve for daily mean flows, based on 75 years of records at Eastern Crossing is given in Figure 6. The flat slope of the curve in the region of the medium flow reveals the effect of the flow regulation as a result of the diversion of flows from the St. Mary's River. The curve in Figure 6 also shows that the median flow is about 12 m^3/s (424 ft^3/s) whereas 10% of the time the flows are greater than 23 m^3/s (812 ft^3/s) and smaller than 1.3 m^3/s (46 ft^3/s). Considering that flows during the irrigation season are on average of the order of 20 m^3/s (710 ft^3/s), it is obvious that a method which measures discharges accurately enough for computation of natural flows will account for 90% of the total flow.

2.4 Stage-Discharge Relationships

In an alluvial channel both the water surface and the flow boundary are free because their profiles are not fixed or known apriori. This is what makes alluvial channel flows so much more complicated than flows with rigid boundaries. Rigid channels have their geometry fixed by man or nature and flow depths can be predicted with accuracy adequate for most practical requirements. Stage discharge relationships in these cases are usually sufficiently stable except at high flows when the control may be drowned out or changed by shifting of the bed material. For alluvial streams, the problem is more complicated. The reason for this is that the geometric characteristics of the channel depend on the depth, velocity and sediment transport rate of the flow. These flow properties are in turn strongly dependent on the channel configuration and its hydraulic roughness. As a result of such behaviour it has been difficult or impossible to develop consistent stage-discharge relationships for such streams. A case in point is shown in Figure 7, which shows gauge height plotted versus discharge for the Huerfano River in Colorado (Dawdy 1961). It is quite clear from this plot that there is no definable stage discharge relationship.

In the case of the Milk River at Eastern Crossing, the situation is not quite so severe. Nevertheless, the USGS and the WSC have not been able to establish a definitive stage-discharge curve at this location. To demonstrate this, data for different time periods were plotted as gauge height versus discharge in Figure 8. The plot shows that there is a clearly discernible trend, implying much greater stability in the control than that exhibited in Figure 7. An average curve was fitted through the data, and in terms of the accuracy usually obtainable in sediment transport measurements, one might be tempted to say that the fit is quite good. However, as a means for determining reliable discharge records having an accuracy of 5% or better, the curve in Figure 8 leaves much to be desired. Most of the flows during the irrigation season, will be between 6 m^3/s (200 ft^3/s) and 28 m^3/s (1000 ft^3/s). Examination of Figure 8 shows that the curve is quite flat as can be expected for a natural control in a wide shallow cross-section. As a result, small errors in gauge height will result in large errors in discharge. This situation becomes progressively worse towards the lower flows as the slope of the curve becomes ever flatter. Indeed, measurements below 1.4 m^3/s (50 ft^3/s) are very scarce and no curve is defined in this range.

It is clear from Figure 8 that the variances in the available stage-discharge curve are larger than desirable for the preparation of acceptable discharge records. A more sensitive method must be found to obtain accurate discharge records over the full range of flows.

2.5 Sediment Loading

The large variability in the river cross-section as indicated in Figure 3, results in large sediment loads carried by the flow. The concentration of suspended sediment is given as a function of discharge in Figure 9. The data were obtained between 1974 and 1982 (WRB, 1986). Although there is considerable scatter, an average curve gives a good indication of the sediment loading for different discharges. The maximum flow for which precise discharge records are required is about 30 m^3/s. However, the river passes sediment for flows much higher than this and any method adopted to measure the flow must not be affected by the sediment transport at any flow rate. At 30 m^3/s, the suspended sediment transported per day is about 8000 tonnes. At 100 m^3/s the sediment load would be about 86,000 tonnes per day. The suspended sediment particle sizes vary over a wide range as shown in Figure 10. It is quite clear from these plots that the sediments are very fine ranging over three orders of magnitude, with the D_{50} sizes being always less than 0.2 mm. In contrast to the potentially large suspended sediment load, the rate of bed load transport is much less. Records are virtually nonexistent, but estimates based on three measurements made in 1978 (Spitzer, 1986) indicate that the transport rate is of the order of 10 tonnes/day when the discharge is 30 m^3/s. The medium particle size of the bed load is indicated from bed material samples that are also of the order of less than 0.2 mm, which is similar to that of the suspended sediment. The grain size distribution of the bed material is shown in Figure 11.

3.0 EXAMINATION OF AVAILABLE METHODS

Examination of the stage-discharge relationship has clearly shown that present methods of obtaining daily mean discharge records are not satisfactory. An alternative method must be found which provides greater accuracy and continuity while at the same time is not affected by the great variability in channel cross-section and sediment load. The characteristics of the Milk River at Eastern Crossing which must be taken into account are given as:

a) range of flow to be measured within 5% accuracy is 0.05 m^3/s to 30 m^3/s;
b) average depth at 30 m^3/s is 0.70 m;
c) width to depth ratio > 50;
d) flat gradient;
e) heavy sediment load.

A review of available methods which would provide a continuous record or incremented average flow over short time intervals (i.e., of the order of 15 minutes) indicates the following for consideration:

a) Ultrasonic Method
b) Electromagnetic Method
c) Flow Measuring Structures.

These methods are examined briefly in the following sections to see if any of them should be considered for the Milk River.

3.1 Ultrasonic Method

3.1.1 Principle of operation

The ultrasonic method, sometimes known as the acoustic method, of stream gauging is now well established and to date there are many such stations in operation in a number of countries (Hershey 1978). The use of this method of measuring discharge dates back to the mid 1950's (Schwengel and Hess, 1955, Schwengel et al., 1955), development has proceeded and complete systems are now commercially available (Newman, 1982).

Multiple pairs of sonar transducers are mounted on opposite banks of the river at a number of defined depths with each transducer acting alternately as a transmitter and receiver of pulses of ultra sound, (Figure 12a). Pulses of sound are transmitted from an opposite pair of transducers which form a path which is angled to the direction of flow. The pulses are received by the alternate transducers after a period of time depending on the path length, the velocity of sound along the path and the resolved component of the velocity of the water. Because of the flow of the water, one pulse will arrive earlier than the other, and the time difference, say, Δt is used to determine the velocity of the flow, V. The velocity thus obtained is the average across the section at a particular elevation, say, y (i.e., position of the transducer pair relative to a datum). The average discharge at a given elevation is obtained from a relationship such as

$$q_n = \frac{1}{2} C^2 \Delta t_n \Delta y \, B_n \tan\theta \qquad (1)$$

from which the total discharge through the entire cross-section can then be computed by simply summing up the incremental discharges q_n as

$$Q = \sum_{n=1}^{m} q_n \qquad (2)$$

where n signifies the transducer elevation, m = the number of measurements made (i.e., the number of different elevations), Q = the total discharge, c = speed of sound, Δt_n = difference in transit times of sound signal, Δy = increment in elevation, θ = angle between sound path and direction of flow vector and B_n = width of nth element.

It is quite evident from Equation (1) that the accuracy of determining the discharge is primarily dependent on the ability to measure Δt_n and θ. The high complexity of making the required measurements and the resultant computation of flows in the cross-section has been greatly reduced in recent years by the use of micro processors.

3.1.2 Advantages of the ultrasonic system

The advantages of the ultrasonic system are that:

a) no stage-discharge relationship required;
b) does not obstruct flow or create significant backwater
c) high accuracy with automatic continuous measurement of flow
d) cost of equipment is virtually independent of the size of the river
e) suitable for rivers over 200 m wide
f) continuous or intermittent transmission of data.

3.1.3 Requirements for ultrasonic gauge site

In order to have satisfactory operation of these ultrasonic gauges, Newmann (1982) and Hershey (1978) suggest that

a) the channel must be straight for at least ten times the river width;
b) the channel cross-section should be stable, symmetrical and level;
c) the channel should be free of weed growth which can disperse the sound beam.
d) entrained air caused by rapids, waterfalls, etc., must be avoided as this will attenuate the signal or disperse the beam;
e) upstream inflows to the river at different temperatures or laden with high salt content should be avoided as the beam can be refracted. Similarly, gauging should not be attempted just downstream from a confluence since uneven mixing can cause beam refraction;
f) wide, shallow rivers should be avoided, because the beam may reflect from the surface of the bed and thus cause gross inaccuracies in measurement of Δt_n.
g) rivers with low velocities should be avoided where possible since temperature stratification can cause refraction of the ultrasonic beam;

h) rivers with high sediment concentrations should be avoided because suspended solids distort the pulse signal. In one particular case sediment load in excess of 2000 mg/ℓ in a river 64 m wide with ultrasonic equipment operating at 50 Hz, resulted in complete loss of signal (Newman, 1982). Such a situation is typical for the Milk River at Eastern Crossing.

Although the attributes of the ultrasonic gauging system are indeed quite tempting, the basic characteristics of the river reach at Eastern Crossing do not permit the use of this method. This is primarily because of conditions a, b, f and h.

3.2 Electromagnetic method

3.2.1 Principle of operation

The electromagnetic method has been developed primarily in the United Kingdom (Hershey, 1978; Newmann, 1982) and continental Europe (Grils, 1970; Rolff and Starks, 1973). The development of this method was primarily motivated in the U.K. by the profuse growth of weeds at gauging stations which resulted in unstable stage-discharge relationships because these varied with conditions of weed growth. The principle of this method may be described as follows. If an electrical conductor moves through a magnetic field so that the magnetic force lines are cut, an induction voltage is generated in the electrical conductor proportional to the latter's velocity of motion.

In applying this principle to the measurement of stream flow, the water is taken to be the moving conductor having a width equal to that of the flow. The potential is measured by placing an electrode in each stream bank so that the plane through the electrode is perpendicular to the direction of flow. A schematic illustration of the electromagnetic method is shown in Figure 13. In practice the induced voltage is reduced because of signal attenuation. In a natural river section, the electric conductivity of the water and the bed will allow electric current to escape through the bed. This can be represented by an attenuation factor which may depend on flow depth say $\delta(y)$ where $\delta(y) < 1$ (Hershey 1978). In addition, electric current will tend to flow outside the artificially produced magnetic field and this reduces the signal voltage by a factor of say, β, known as the end shortening effect where $\beta < 1$ (Hershey 1978). However, for a given coil configuration β may be taken as being constant. In a rectangular cross-section the discharge can be determined from the relationship

$$Q = f\left[\frac{hV}{\delta(y)K}\right] \tag{3}$$

in which Q = discharge, f denotes a function, h = depth of the flow, V = voltage measured at the electrodes, $\delta(y)$ = attenuation factor due to current loss through the river bed, K = a systems constant. A particular gauging site must be calibrated in order to obtain a reliable relation of Q vs $hV/\delta(y)K$.

Values of $\delta(y)$ can be obtained from additional measurements of water resistance and bed resistance (Hershey, 1978). The effect of the conductivity attenuation can be eliminated (i.e., $\delta(y)$ becomes 1), by placing an insulating membrane in the river bed at the elevation of the base of the electrodes and covering an area, slightly larger than that of the magnetic coil as shown in Figure 13. However, this solution may only be practical in small streams having widths less than 10 metres. For larger streams, values of $\delta(y)$ must be determined and because $\delta(y) < 1$, this together with the effect of β ($\beta<1$) further reduces an already small signal which must be separated from larger unwanted noise. Nevertheless, Hershey (1978) claims success with this method, without the membrane, for streams up to 25 m in width.

The electro-magnetic gauge is unaffected by changes in bed elevations as a result of sediment deposition or scour as long as scour does not extend below the base of the electrodes. This was confirmed experimentally in a laboratory model by Engel and Roy (1986).

3.2.2 Advantages of electromagnetic system

The advantages of the electromagnetic system are that:

a) no stage-discharge relationship is required;
b) does not obstruct flow;
c) not affected by suspended sediment;
d) sediment deposits do not affect the flow measurement.
e) zero flow, negative flow and positive flow are automatically integrated;
f) changes in cross-section shape do not affect the flow measurement as long as such changes do not extend below the elevation of the base of the electrodes.

3.2.3 Requirements for an electromagnetic gauge site

The following criteria should be met in order to have a satisfactory installation (Hershey, 1978; Newman, 1982).

a) the channel should be straight for at least three times the river width;
b) the section of the channel should be symmetrical about the river centre line.
c) the banks should preferably be near vertical or have a one in two slope, otherwise field or scale model calibration will be necessary
d) there should be no major source of electrical interference near the site;

e) heavily reinforced concrete or steel piling can distort the magnetic field and should preferably be avoided, otherwise on-site calibration will be necessary;

f) the channel should be lined with an insulating membrane in order to obtain the best results. A degraded version of the electromagnetic gauge may be possible with no membrane.

The requirements for an electromagnetic gauge are not as restrictive as those for the ultrasonic gauge. Perhaps the greatest problem is that there has been no experience with rivers having the width of the Milk River at Eastern Crossing. Rather, electromagnetic gauges have been limited to streams with widths less than 25 m. On such small streams accuracies of ±10% at velocites as low as 0.03 m/s and ±5% at higher flows have been observed (Rowse, 1985). One difficulty will be the placement of an insulating membrane in the river bed covering the area of the magnetic coil over the full width of the river. Alternatively, there is the uncertainty regarding the achievable accuracy when no insulating membrane is used over a width of 50 m required at Eastern Crossing. Experience on small streams has shown that the uninsulated river bed reduces the voltage signal by a factor of about 5 to 10, the amount depending on bed and river water conductivity. However, on rivers in areas of low electrical noise, concrete or gabions may be found to stabilize the signal attenuation and permit acceptable measurement accuracies to be obtained from a gauge without a plastic membrane (Rowse, 1985), but this needs to be confirmed by further studies.

Another important consideration is the maintenance of cross-section geometry between the electrodes at the banks, in accordance with points b and c in this section. Traditional methods such as sheet piling would significantly distort the magnetic field as pointed out in e. Gabions made of plastic mesh may be a useful alternative together with "rip-rap" on the channel bed. This would be particularly useful at Eastern Crossing, because of the availability of local materials to fill the gabion and provide the "rip-rap". There are other, engineering considerations such as providing adequate power supply and the design of the magnetic coils and their placement.

3.2.4 Present State of Development

About 40 electromagnetic flow gauges have been installed in England with the majority by the Severn Trent Authority. They have installed 17 such gauges. All instrumentation was designed or adapted for this specific application and the specifications were developed by close cooperation between a specialist R&D unit in the Severn Trent Authority and a contractor. As well as measuring flow, at all sites water levels are monitored by a gas bubble gauge. Commonly an OTT gauge of the beam type is used to measure the gas pressure and these gauges have been fitted with shaft encoders to obtain electronic readout of the water level. Data are transmitted to a central location by telephone line automatically at intervals or by direct interrogation. All electronics are industrial quality and designed for easy checking and maintenance.

Some of these gauges were inspected during a visit to the Severn Trent Authority in England in April 1986. The dominant aspect of all the installations is that these gauges are no longer experimental but completely and reliably operational.

The oldest installation was placed at a site originally intended to measure flow by means of a flume. The site, on the River Tame is shown in Fig. 14 with a view of the instrumentation in Fig. 15. Coils to induce a magnetic flux are always placed under the river. At this site the insulating membrane is bitumen and paper which is no longer recommended and is to be replaced.

Another typical installation was observed at the River Soar. Figure 16 shows the electrode in a perforated pipe but exposed outside the bank of the river. This bank is stabilized with plastic gabions. No steel wire can be used as it causes spurious signals. Metallic rubbish such as bed springs in the measuring section can also cause trouble. The insulating membrane is a 4 mm high density polyethylene under the rip-rap in gabions. This gauge uses a mercury manometer to provide electronic water level data. Fig. 17 shows the stainless steel end of the electrode and Fig. 18 shows the wires making up the coil. Each cable contains 11 conductors which are connected in the gauge house to provide in all 110 turns in the coil. The current is 5 amperes, which is reversed every second. River discharge values are logged at the end of 15 minutes and represent a rolling mean. The telemetry unit can hold data for up to 15 days.

Another installation in the River Sence is shown in Figure 19. The river bank is stabilized by a stone revetment with the electrode incorporated into the revetment. Fig. 20 shows the electrode installed within the bank rip-rap.

3.2.5 Costs and other factors

No electromagnetic gauge has yet been installed in a river as wide or as remote as the Milk River. The Milk River has a very active bed load which the gauge should handle providing the membrane and electrodes remain in place. Gauges in England have been operating successfully and reliably for over five years. Similar gauges are used for the direct measurement of combined sewer flows.

The systems work well for the Severn Trent Authority because they have a high technology development unit which is responsible for the operational introduction of new ideas. Also, the prime contractor, Sarasota Inc., works closely with them. Sarasota sees no technical reason why an electromagnetic meter should not work in the Milk River. The power estimates are about 1,000 to 1,500 watts for the coil with cost estimates for the electronics, etc., of about $100,000. Gauge instrumentation is sensitive to fluorescent lights which should not be used. The entire data collection system can be automated to give both water levels and flow measurements.

3.3 Discharge Measurement Structures

The control of flow in an open channel is defined in various ways. As used here the term means the establishment of a definitive flow condition in the channel or, more specifically, a definitive relationship between the stage and the discharge of the flow (Chow, 1959). The section of the channel in which such a control is achieved is then called a control section.

The relationship between stage and discharge is used for the operation of gauging stations in that gauge height (stage) is recorded and the discharge is determined from it. This is only possible as long as the river bed retains its form along the entire stretch which has an influence on the stage-discharge relationship. The cross-section of the river in the control reach has to be stable. Raising or lowering of the bed at this section will change this relationship (Gonzalez et al. 1969; Walser, 1970). Further, the slope of the river bed in the control reach must be stable, because the average velocity of approach depends on it. The conditions of a stable river bed is not fulfilled in sand bed streams such as the Milk River. Even if the control section is stabilized with a weir, larger or smaller sediment deposits upstream of it may significantly change the velocity of approach. An example of this is given by Walser (1970) and is shown in Figure 21. An existing weir was adapted to serve as a gauging station. Although the weir crest was always free of sediment, the relation between discharge and gauge height changed by as much as 10% as shown by the curves in Figure 21. It is therefore very important to choose a structure to serve as stable control which is not affected by sediment movement and deposition or which will keep the gauging reach free of deposition or scour.

There is a wide range of flow measuring structures available most of which have been developed for particular applications. A summary of these is presented by Bos (1976). After consideration of the characteristics of the Milk River at Eastern Crossing and the requirements for accurate measurement of discharge, three generic types of structures were selected for further consideration, namely:

1. Broad Crested Weirs
2. Short Crested Weirs
3. Long Throated Critical Depth Flume.

3.3.1 Broad crested weirs

In its simplest form a broad crested weir is an overflow structure with a horizontal crest above which the deviation from hydrostatic pressure distribution may be neglected. In other words, the stream lines are practically straight and parallel. To ensure that this condition is achieved, the length of the weir crest, say L, in the direction of flow (Figure 22) should be such that $0.08 \leq H_1/L \leq 0.5$ (H_1 = total energy head over the weir) (Bos, 1976). It is necessary to have $H_1/L \geq 0.08$ in order to be able to neglect the energy losses over the crest and $H_1/L \leq 0.50$ so that only slight curvature of stream lines occurs above the crest and hydrostatic pressure may be assumed.

3.3.1.1 Triangular broad crested weir

On a natural stream where it is necessary to measure a wide range of discharges, a triangular control has several advantages over the more simple rectangular cross-section. A triangular control provides a large breadth at high flows thus minimizing back water effects while providing greater sensitivity at low flows. A broad crested weir with triangular cross-section is shown schematically in Figure 23.

The limitations and pertinent operational characteristics of the triangular broad crested weir are given in Table 1. Hydraulic Model Studies must be conducted to determine the performance of this structure in the Milk River at Eastern Crossing.

3.3.2 Short crested weir

For a short crested weir the length of the crest is such that $H_1/L > 0.5$ and for such cases the curvature of the streamlines above the crest cannot be neglected. Indeed, the same measuring structure can act as a broad crested weir for low heads ($H_1/L < 0.5$, while with an increase of head ($H_1/L > 0.5$), the influence of the stream line curvature becomes significant and the structure acts as a short crested weir. However, such dual performance is not desirable.

The head-discharge relationships of short crested weirs of given cross-sectional shape are structurally similar to those corresponding to similar shapes of broad crested weirs.

However, owing to the non hydrostatic pressure distribution above the crest, the discharge coefficient C_d of a short crested weir is higher than that of a broad crested weir, depending on the degree of curvature in the stream lines.

3.3.2.1 Triangular profile flat V weir

Because of the advantages of a triangular control section, the Hydraulic Research Station at Wallingford (White, 1966) investigated the characteristics of a triangular profile, flat V weir with cross slopes of 1:10 and 1:20, and in the direction of flow, an upstream slope of 1:2 and a downstream slope of either 1:2 or 1:5, as shown in Figure 24.

The 1:2/1:2 weir is much more sensitive to bed level elevation of the tail water channel relative to crest level than the 1:2/1:5 weir. Tests have also shown that the modular flow limit (i.e., beginning of submergence of control) for the 1:2/1:5 weir does not depend on as many variables as the 1:2/1:2 shape. Finally, tests have also shown that the modular flow limit is independent of the cross slope of the weir (i.e., angle or V notch). These are good reasons to select the 1:2/1:5 weir for the conditions to be encountered on the Milk River at Eastern Crossing.

The important operational characteristics of the flat V weir with 1:2/1:5 longitudinal slope are given in Table 1. Hydraulic model studies must be conducted to assess its performance under conditions encountered in sand bed rivers.

3.3.2.2 USGS Design No. 1

A variation of the flat V weir proposed by White (1966) was developed by the USGS on the basis of model studies. A design shown in (Figure 25) was recommended by Harris and Richardson, (1964) for installation in the Rio Grande conveyance channel at Bernardo, New Mexico. This design will be designated as USGS Design No. 1.

The downstream apron was designed to include a sampling sill from which total sediment load samples could be obtained. The cross slope of the weir was designed to be 1:35, however, better sensitivity can be obtained by increasing this to 1:20, as used by White (1966). Tests have shown that backwater has no effect on the stage discharge relationship for values of submergence less than 90%, which is a useful feature for the Milk River. Discharge measurements at this structure were made to establish a stage-discharge curve, which is given in Figure (26). A good fit to the data can be obtained for all flows except those greater than 600 ft/s ($17 m^3/s$) during the period from April 29, 1965 to October 20, 1965. It is not clear why there is such a difference for these data, but it is possible that this may be due to sediment deposition upstream near the control structure. This points out the need for very careful studies of the behaviour of sediment near any control structure considered for the Milk River. The scatter about the curve for the remaining data is primarily due to error of making the discharge measurements with current meters.

The important operational characteristics of the USGS No. 1 design are given in Table 1. Careful tests will be conducted to determine or confirm the characteristics of this structure.

3.3.2.3 USGS Design No. 2

A simple design, designated here as USGS Design No. 2 has been proposed by Kilpatrick (1985). This design was developed for the Eastern Crossing gauging site, and is shown schematically on Figure (27). The designers believe that this configuration will provide satisfactory sensitivity at low to medium flows and will prevent deposition of sediment. It is expected that modular flows will be maintained for flows up to 28 m^3/s (1000 ft^3/s). A weir of this design was recently installed on the Souris River near Westhope in North Dakota but information of its performance was not available at the writing of this report. Careful testing must be conducted to evaluate its performance relative to the other structures being considered.

3.3.3 Long throated critical depth flume

The discharge in an open channel may be measured by means of a flume, consisting essentially of contractions in the sides and/or bottom of the channel, thus forming a throat. When the reduction in the cross sectional area exceeds a certain value, critical depth occurs in the constriction and the stage discharge relationship becomes independent of conditions in the channel downstream of the structure. The device is then said to be free flowing and constitutes a critical depth measuring flume (Hershey, 1978).

Long throated flumes are those structures which have a throat section in which the stream lines run parallel to each other over a sufficiently long distance so that hydrostatic pressure distribution can be assumed at the control section.

3.3.3.1 Long throated flume with triangular throat

The hydraulic behaviour of a critical depth flume is essentially the same as that of a broad-crested weir. Consequently, the stage discharge equations for critical depth flumes are derived in exactly the same way as those for broad-crested weirs of the same shape. A flume with triangular throat section is shown in Figure 28.

Long throated flumes are usually equipped with gradual expansions downstream of their control sections in order to obtain a gradual conversion of kinetic energy into potential energy and thus obtain a high modular limit. There is no data available which relates the submergence ratio H_2/H_1 at the modular limit to the angle of divergence of the flume expansion. The smaller the angle of divergence, the more gradual is the flume expansion. Obviously, a more sudden expansion is less costly to build. However, the effect of this on the performance of the structure must be determined from tests.

The important operational characteristics of the long throated flume with triangular throat are given in Table 1. Hydraulic model tests must be conducted to determine the performance of this type of flume for a large cross-section such as that of the Milk River at Eastern Crossing.

4.0 HYDRAULIC MODEL TESTING

The large sediment loads of the Milk River at Eastern Crossing will make it necessary to conduct some careful laboratory tests to assess the most probable behaviour of different control structures under prototype conditions. It is particularly important to avoid the accumulation of sediment between the head measuring section and the control section. Tests will be conducted on the five structures presented in the previous section, namely a triangular broad crested weir, flat V weir, USGS Design No. 1, USGS Design No. 2, and the triangular long throated flume.

4.1 Modelling Considerations

4.1.1 Considerations for the control structure

The primary concern regarding a control structure at Eastern Crossing is the deposition of sediment near the structure and the effect that this will have on the stage-discharge relationship. The deposition of sediment will affect the upstream height P_1 of the control structure and this in turn will affect the ratio h_1/P_1 (h_1 and P_1 are shown in Figure 29). For a given head h_1 there is a limiting value of P_1 below which the discharge coefficient of the structure is affected. Therefore, it is important to determine the value of P_1 for a given magnitude of flow and sediment load. The average value of P_1 with reference to Figure 29 may be expressed in general terms as

$$P_1 = f[h_1, H_b, D_{50}, \rho, \mu, \gamma_s, n, g, u, c_s] \tag{44}$$

where P_1, h_1, H_b, D_{50} and n are shown in Figure 29, ρ = density of fluid, μ = viscosity of the fluid, γ_s = submerged unit weight of sediment, g = acceleration due to gravity, u = the velocity of approach and c_s = sediment concentrations of the incoming flow.

Using dimensional analyses, one obtains

$$\frac{P_1}{H_b} = f_1 \left(\frac{h_1}{H_b}, \frac{D_{50}}{H_b}, \frac{g^{1/2} H_b^{3/2} \rho}{\mu}, \frac{\gamma_s}{\rho g}, n, \frac{u}{g^{1/2} H_b^{1/2}}, C_s\right) \tag{45}$$

In the prototype the flow in the river reach at Eastern Crossing is in the upper smooth turbulent flow regime. However, if a structure is placed across the stream, a coarse rip-rap approach apron will be required to stablize the cross-section. In the presence of the increased roughness, the flow regime will most certainly change to one of greater turbulence. As a result, one might consider that viscous effects are much reduced. Furthermore, as one is in the near vicinity of the structure itself, the flow changes from gradually varying to rapidly varying and this too will result in lesser viscous effects. As a result, viscous effects near the structure may be considered to be of secondary importance, and $g^{1/2}H_b^{3/2}\rho/\mu$ may be eliminated from further consideration. Equation (45) can thus be reduced to

$$\frac{P_1}{H_b} = f_2\left[\frac{h_1}{H_b}, \frac{D_{50}}{H_b}, \frac{\gamma_s}{\rho g}, n, \frac{u}{g^{1/2}H_b^{1/2}}, C_s\right] \quad (46)$$

In order to have dynamic similarity between the model and the prototype one must have

$$\lambda\pi_1=1,\ \lambda\pi_2=1,\ \lambda\pi_3=1,\ \lambda\pi_4=1,\ \lambda\pi_5=1,\ \lambda\pi_6=1,\ \lambda\pi_7=1 \quad (47)$$

in which

$$\pi_1 = \frac{P_1}{H_b},\quad \pi_2 = \frac{h_1}{H_b},\quad \pi_3 = \frac{D_{50}}{H_b},\quad \pi_4 = \frac{\gamma_s}{\rho g},\quad \pi_5 = n,$$

$$\pi_6 = \frac{u}{g^{1/2}H_b^{1/2}},\quad \pi_7 = C_s$$

and λ denotes a scale ratio.

If one now adopts a scale ratio of say λ_y for all linear length dimensions, then one obtains in accordance with the conditions of Equation (47)

$$\lambda\pi_1 = 1: \quad \lambda_{P_1} = \lambda_{H_b} = \lambda_y \tag{48a}$$

$$\lambda\pi_2 = 1: \quad \lambda_{h_1} = \lambda_{H_b} = \lambda_y \tag{48b}$$

$$\lambda\pi_3 = 1: \quad \lambda_{D_{50}} = \lambda_{H_b} = \lambda_y \tag{48c}$$

$$\lambda\pi_4 = 1: \quad \lambda_{\gamma_s} = 1 \qquad (\lambda_\rho = \lambda_g = 1) \tag{48d}$$

$$\lambda\pi_5 = 1: \quad \lambda_n = 1 \tag{48e}$$

$$\lambda\pi_7 = 1: \quad \lambda_{c_s} = 1 \tag{48f}$$

Examination of Equations (48) shows that the conditions of Equation (48c) cannot be met. In order to satisfy $\lambda\pi_3 = 1$, it is necessary that the model grain size be smaller than that of the prototype by the factor λy. This is not possible because the prototype gain size is already very small (D_{50} = 0.15 m). As a result, assuming a realistic factor of, say, $\lambda y = 1/10$, the grain size of the model would have to be of the order of 0.015 mm which is too small for physical modelling. Unfortunately, one cannot simplify the problem by omitting π_3 because P_1/H_b in Equation (47) is quite sensitive to changes in D_{50}/H_b, particularly at higher flows as shown in Figure (38), (White, 1971). Therefore, a dynamically similar model is not possible for the processes near the structures. The best that can be done is to conduct tests of relative performance of the structures under consideration and select the best one. In order to conduct such tests it is necessary to model the sediment load which is brought to the control structure reach by the flow upstream.

4.1.2 Modelling of sediment transport

Given that it is desirable to maintain an undistorted model, then for the flow approaching the gauging reach one must have, according to Yalin (1971)

$$\lambda\pi_8 = 1, \quad \lambda\pi_9 = 1, \quad \lambda\pi_{10} = 1 \tag{49}$$

where

$$\pi_8 = \frac{u_* D_{50}}{\nu}, \quad \pi_9 = \frac{\rho u_*^2}{\gamma_s D_{50}}, \quad \pi_{10} = \frac{u}{g^{1/2} h^{1/2}}$$

in which u_* = shear velocity, ν = kinematic viscosity, h = mean depth of flow and all other terms have been previously defined. Given that $u_* = \sqrt{ghs}$ (s = water surface slope) and that $\lambda_h = \lambda_y$, then from conditions $\lambda\pi_8 = 1$ and $\lambda\pi_9 = 1$, one may write

$$\sqrt{\lambda_y \lambda_s}\, \lambda_D = 1 \tag{50}$$

and

$$\frac{\lambda_y \lambda_s}{\lambda_{\gamma_s} \lambda_D} = 1 \tag{51}$$

in which λ_s = slope ratio, λ_D = grain size ratio, and λ_{γ_s} = the submerged unit weight ratio. Combining Equations (50) and (51) one obtains

$$\lambda_D = \frac{1}{\sqrt{\lambda_y \lambda_s}} \tag{52}$$

and

$$\lambda_{\gamma_s} = (\lambda_y \lambda_s)^{3/2} \tag{53}$$

From the conditions embodied in Equations (50) and (51) it can also be shown that

$$\lambda_{q_s} = (\lambda_y \lambda_s)^{3/2} \tag{54}$$

where q_s = the sediment transport per unit width of flow. Finally, from the condition $\lambda_{\pi_{10}} = 1$ it can be shown that

$$\lambda_u = \sqrt{\lambda_y} \tag{55}$$

Equations (52), (53), (54), and (55) are used to determine the required parameters for the model approach channel to the structure and the channel immediately downstream. There are two possible approaches to utilizing the scaling relations, namely, a) model without channel slope distortion; and b) model with slope distortion.

A model without slope distortion requires that $\lambda_s = 1$. As a result, one then obtains from Equations (52) and (53) that $\lambda_D = 1/\sqrt{\lambda_y}$, $\lambda_{\gamma_s} = \lambda_y^{3/2}$ and $\lambda_{q_s} = \lambda_y^{3/2}$. Clearly, because $\lambda_y < 1$, it follows that $\lambda_D > 1$ and $\lambda_{\gamma_s} < 1$ which means that the grain size in the model will be larger than that of the prototype and its submerged weight will be less.

A model with slope distortion provides the option of using the prototype bed material. That is, one takes $\lambda_D = 1$ and $\lambda_{\gamma s} = 1$, which then yields $\lambda_s = 1/\lambda_y$, showing that the model slope will be greater than that of the prototype by a factor of $1/\lambda_y$.

From an economical point of view it is more desirable to use a material which is the same as that in the prototype. The alternative of taking $\lambda_D > 1$ and $\lambda_{\gamma s} < 1$ requires the acquisition of synthetic or other substances such as coal depending on the most suitable value of λ_y. The effect of λ_y on the value of γ_s is shown in Figure 31 which shows γ_s plotted as a function of $1/\lambda_y$. The curve clearly shows that as $1/\lambda_y \to 1.0$, γ_s approaches the prototype value of 2.65. The curve also shows that γ_s is very sensitive to changes in λ_y until $1/\lambda_y \approx 8$. For values of $1/\lambda_y > 8$ changes in γ_s are small as $1/\lambda_y$ increases. Clearly small values of $1/\lambda_y$ will result in large hydraulic models, with associated large costs.

A reasonable sized model for testing the control structures would be one for which $\lambda_y = 1/10$. Therefore, in order to have a model without slope distortion, a lightweight material with $\gamma_s/\gamma = 1.05$ must be used, having a diameter of about 0.5 mm. An available material having this specific gravity is polystyrene. Alternately, for a slope distorted model, the slope must be 10 times larger than that of the prototype. Values of the primary parameters for a model for which $\lambda_y = 1/10$ are shown for the case with no slope distortion in Table 2 and for the case of slope distortion in Table 3.

5.0 CONCLUSIONS

1. The Milk River carries a heavy sediment load, the magnitude of which is largely dependent on the water discharge. As a result there are large variations in cross-sectional shape of the river and changes in bed elevations, which has made it impossible to establish a natural stage-discharge relationship at the hydrometric site at Eastern Crossing. It is necessary to determine a method of obtaining reliable discharge data at this site. Three different methods have been considered:
    1. Ultrasonic Method
    2. Electromagnetic Method
    3. Artificial Controls.

2. The ultrasonic method is not suitable for the flow conditions at Eastern Crossing because the flow is too shallow relative to its width and the sediment concentration for most flows are too high.

3. Electromagnetic measurement of flow is entirely feasible and well past the experimental stage provided experienced contractors are involved. Success also depends on the professional and technical capabilities of the client.

   The electronic technical problems are solved but the civil engineering problem of installation and maintaining an insulating membrane are not. Civil engineering costs will far outweigh the electrical and electronic costs.

4. If a stable artificial control is to be used at Eastern Crossing it is important to choose a design which is not affected by sediment movement and deposition or which will keep the gauging reach free of deposition and scour. A structure having a flat "V" notch crest will increase the sensitivity at low flows.

5. Five artificial control designs have been selected from a large number of available alternatives. These designs are summarized in Table 3. The performance of these designs under sediment transport conditions will be tested in a physical model having a scale ratio 1:10.

6. If an artificial control meeting the necessary requirements can be found, then this design should be compared with the electromagnetic method by conducting a feasibility study.

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TABLE 1

Significant Operational Characteristics of Selected Control Structures

Structure	$\frac{Q_{max}}{Q_{min}}$	$\frac{H_2}{H_1}$	Sensitivity %/0.01 m at min. head	Accuracy E	Debris Passing Capacity	Sediment Passing Capacity	Remarks
Triangular Broadcrested Weir	830	0.80 to 0.95	42	$E \geq 10\%$ for low flows $E \geq 5\%$ for med. to high flows	good to fair	fair	$0.08 \leq \frac{H_1}{L} \leq 0.5$
Flat V Weir	≥ 17500	0.67	42	"	very good	good	low sensitivity to changes in bed elevation up and down stream
Triangular Long Throated Flume	≤ 315	0.70 to 0.90*	28 to 42	"	very good	very good	depends on down-stream transition
USGS Design #1	>1000	0.7 to 0.90	42	"	good	good	only one prototype with good success
USGS Design #2	>1000	-	-	-	-	-	a new design

TABLE 2

Parameters for Model With No Slope Distortion for $\lambda y = 1/10$

	Prototype	Model
Max. depth (cm)	183	18.3
Max. width (m)	48	4.8
Max. discharge m^3/s	85	0.269
Grain size D_{50} (mm)	0.15	0.47
Unit weight λ_D (kg/m^3)	2.65	1.05 (polystyrene)
Slope (bed)	0.000787	0.000787

TABLE 3

Parameters for Model with Slope Distortion for $\lambda y = 1/10$

	Prototype	Model
Max. depth (cm)	183	18.3
Max. width (m)	48	4.8
Max. discharge m^3/s	85	0.269
Grain size D_{50} (mm)	0.15	0.15
Unit weight λ_D (kg/m^3)	2.65	2.65 (sand)
Slope (bed)	0.000787	0.00787

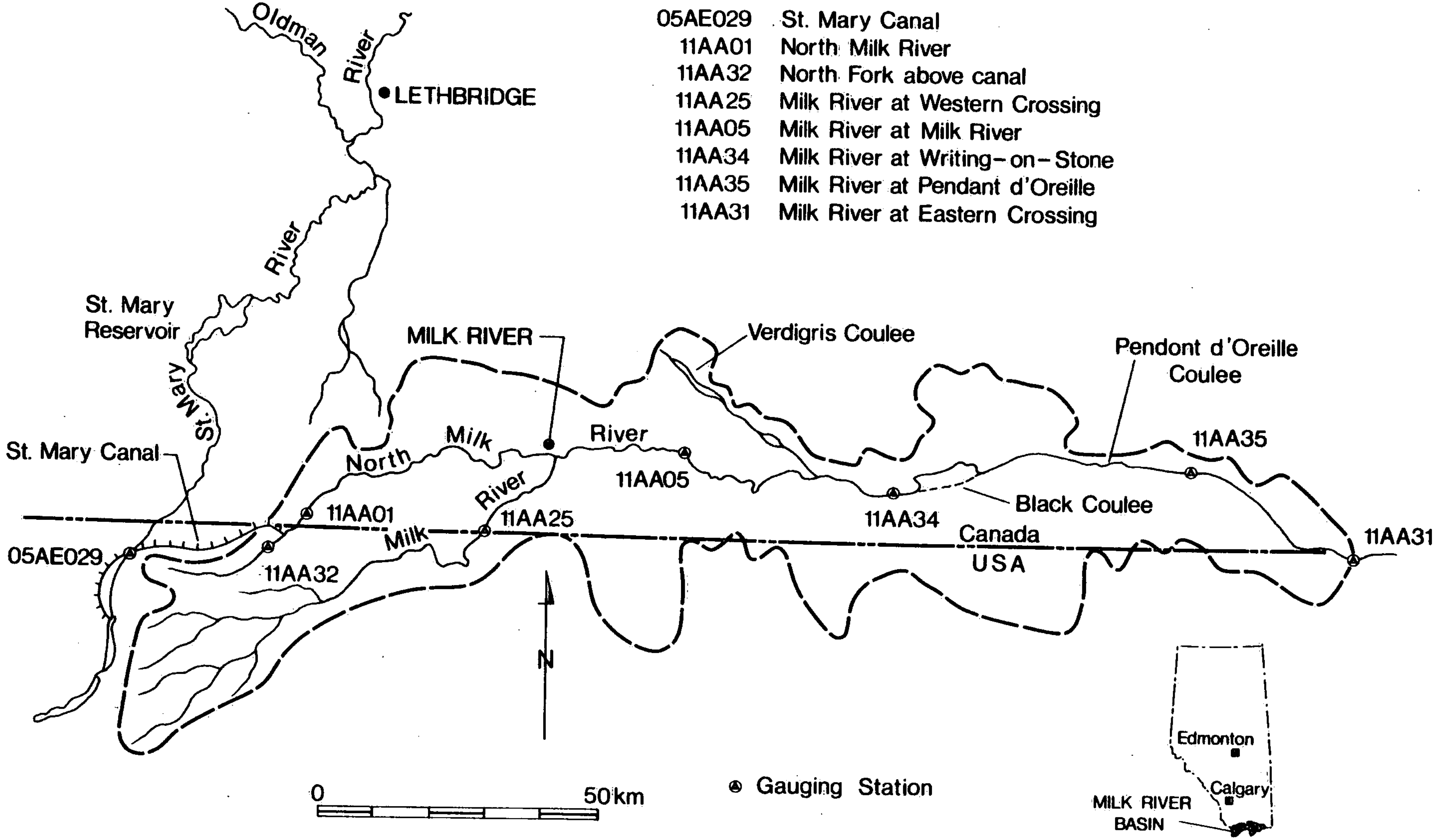

FIGURE 1. Milk River basin (from MacLean & Beckstead 1981)

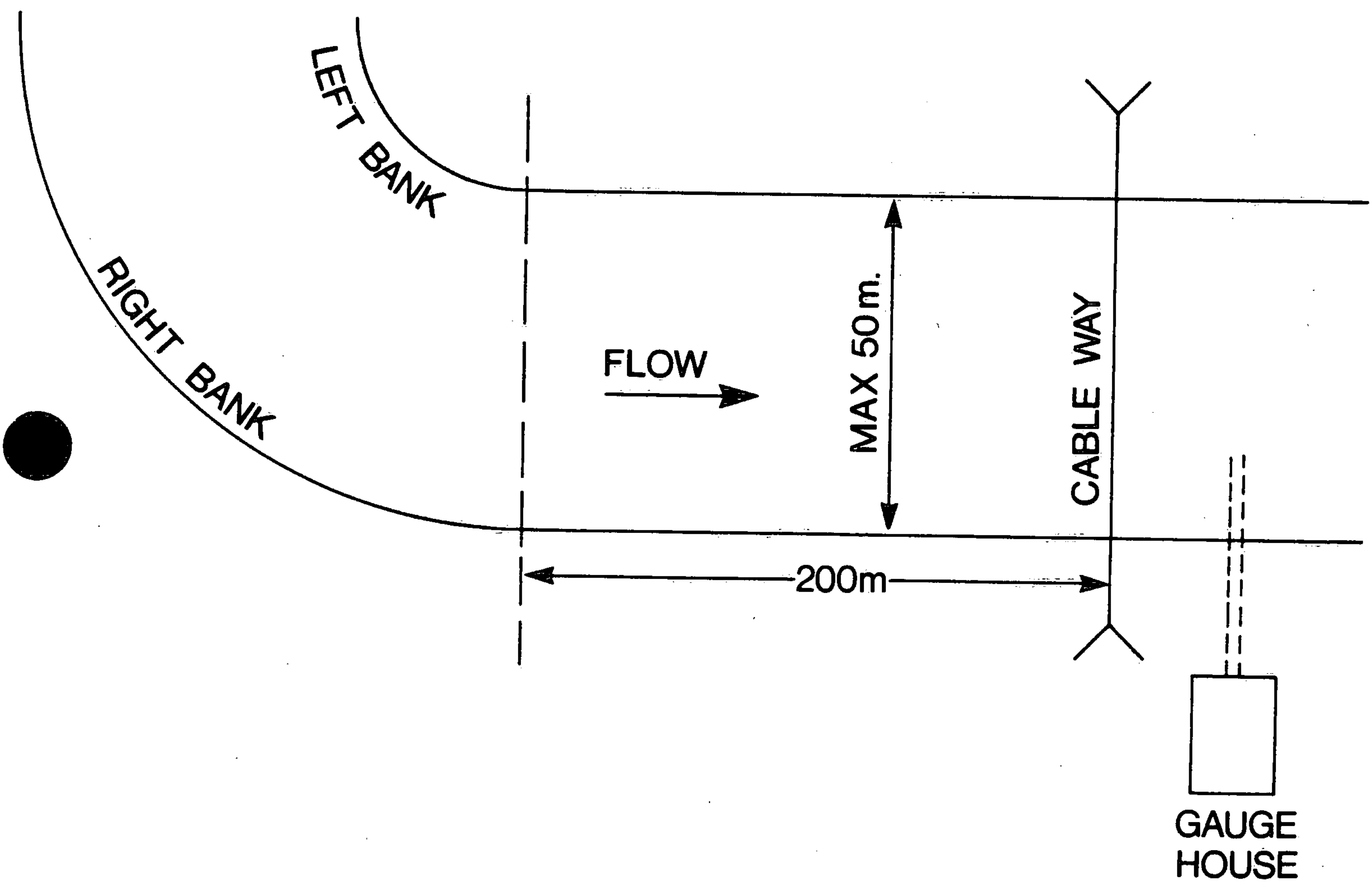

FIGURE 2. Approximate site plan at eastern crossing

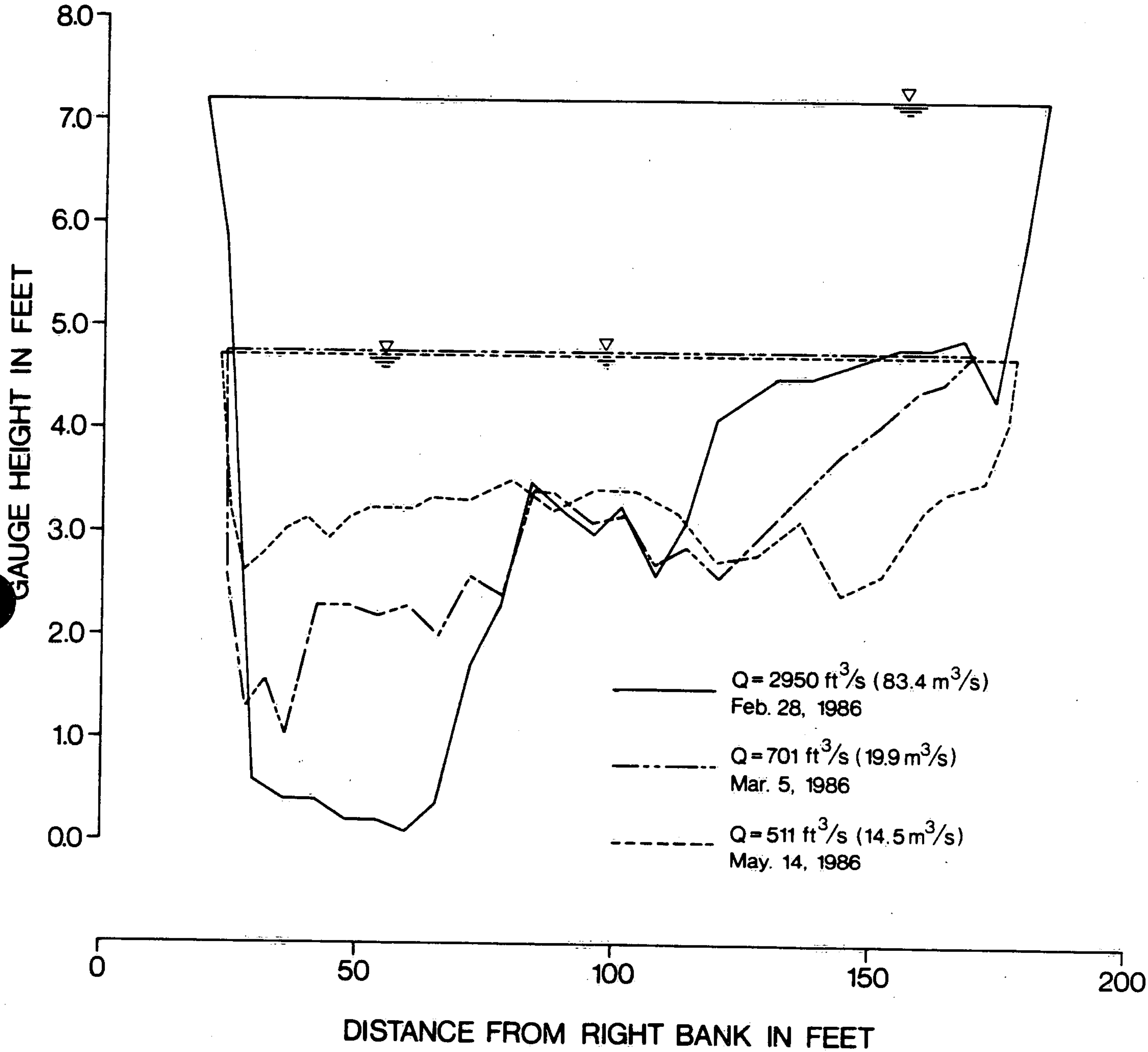

FIGURE 3. Variability of crossection

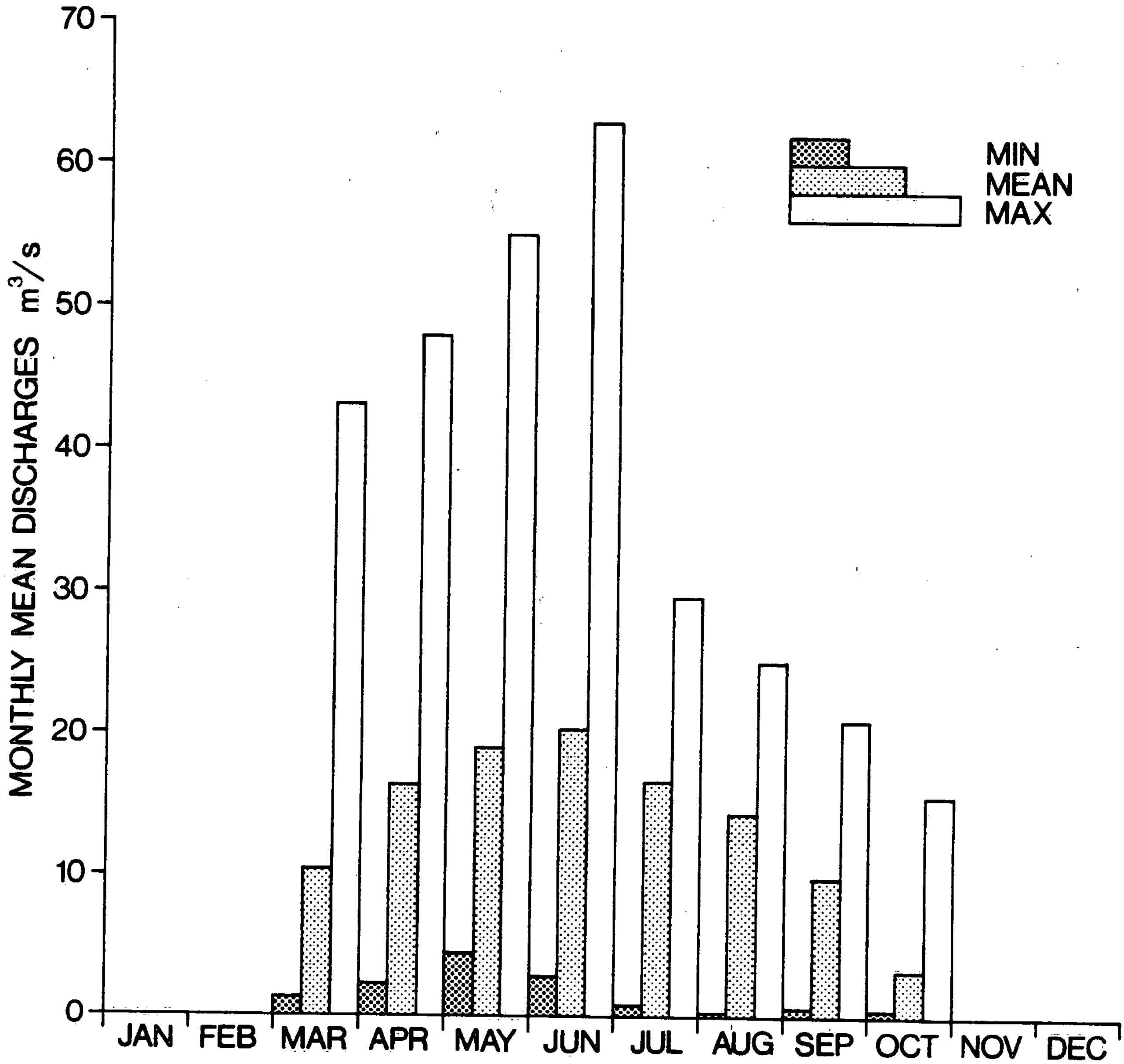

FIGURE 4. Distribution of monthly mean flows, for period of record.

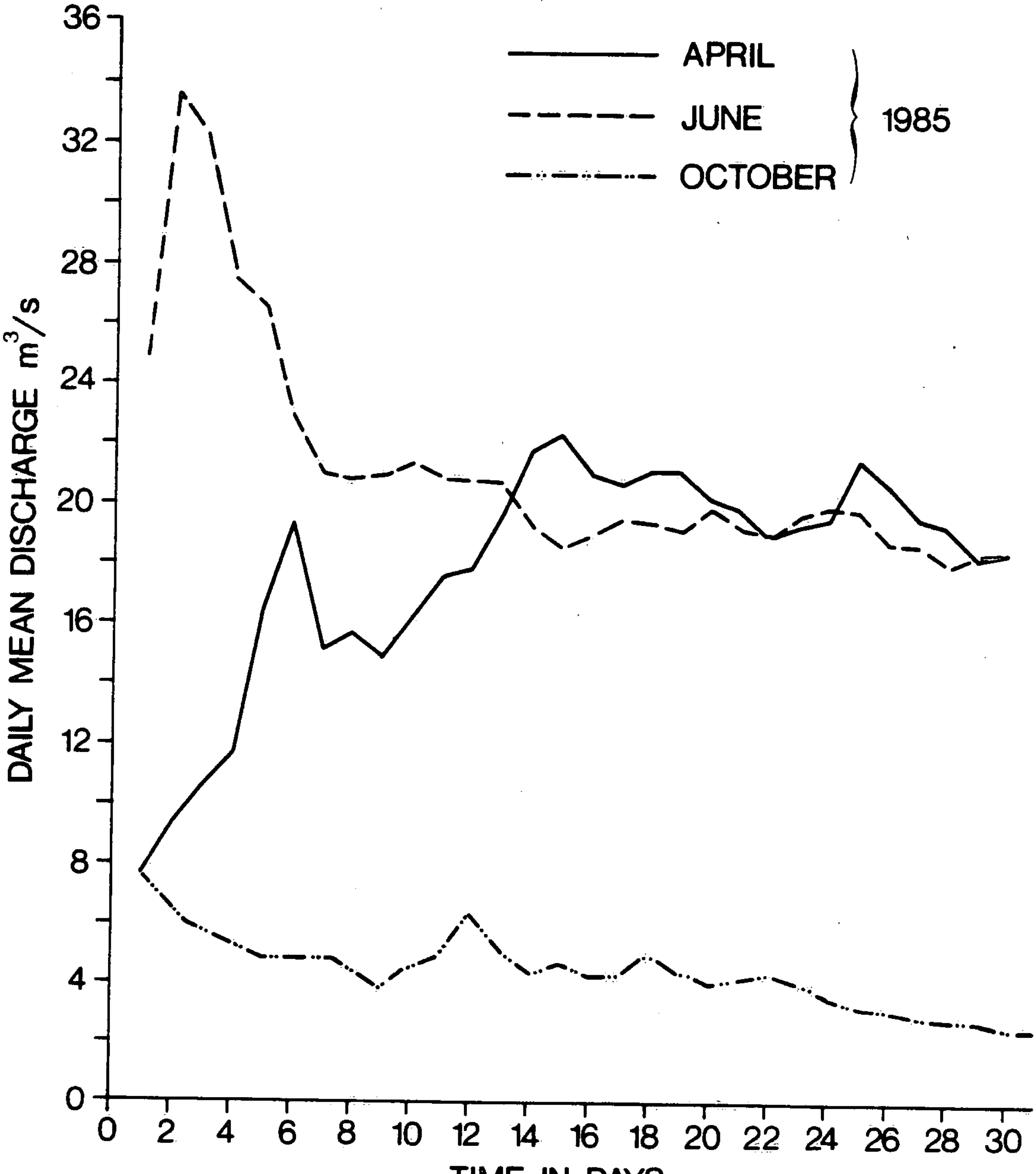

FIGURE 5. Hydrographs for daily mean flows.

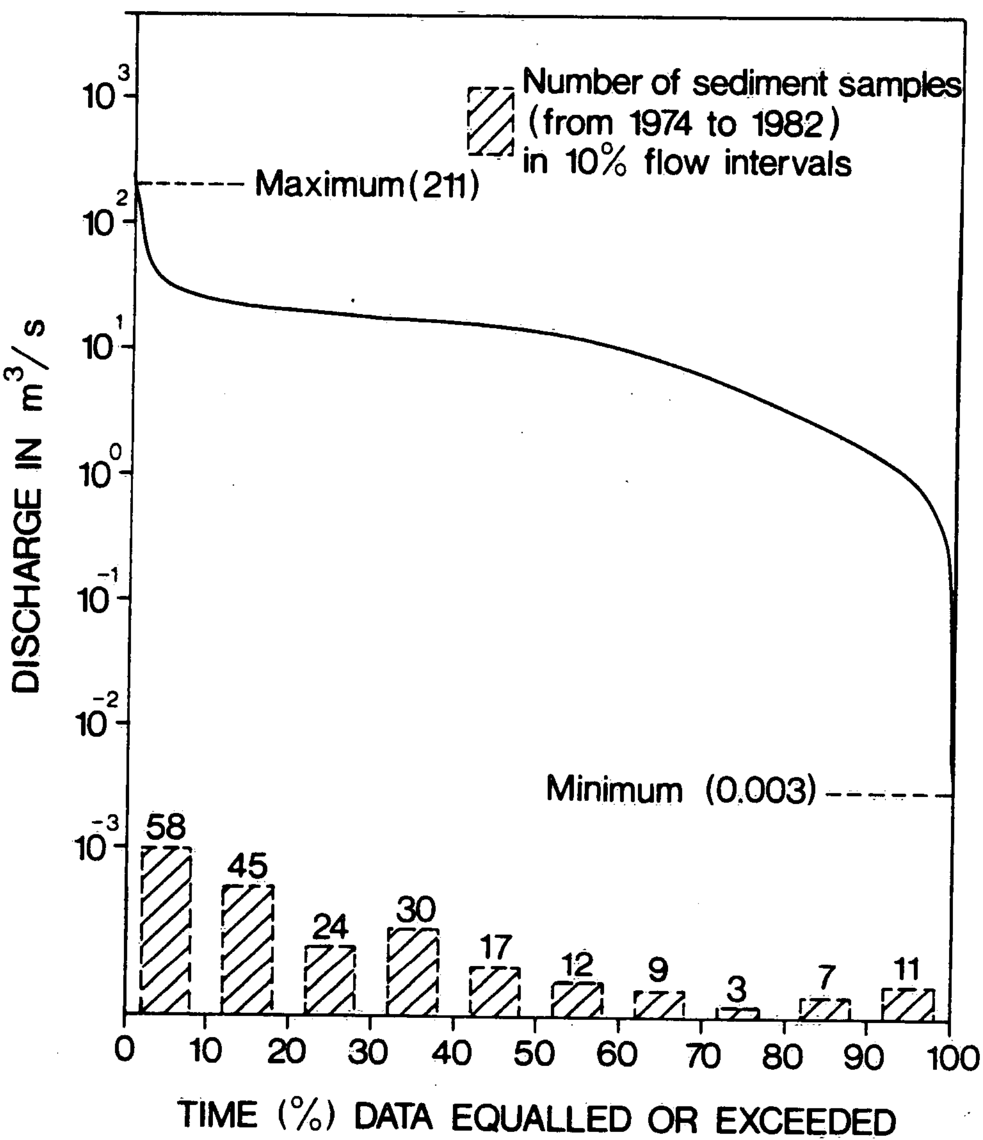

FIGURE 6 Flow duration at eastern crossing (W.R.B. 1986)

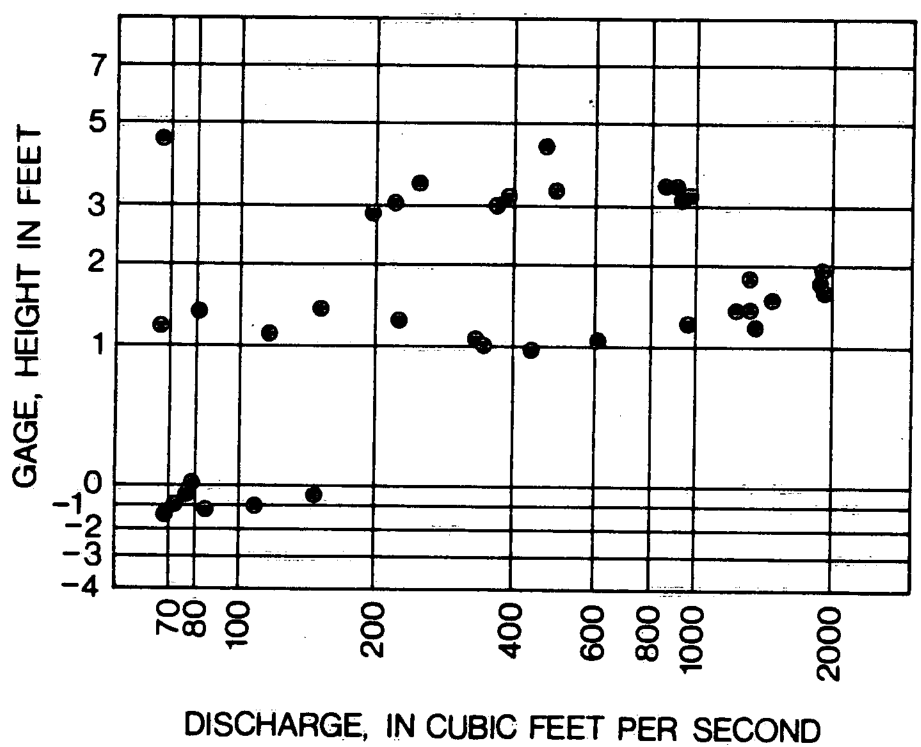

FIGURE 7. Stage – discharge relation for Huerfano River near Undercliffe, Colo. (DAWDY, 1961)

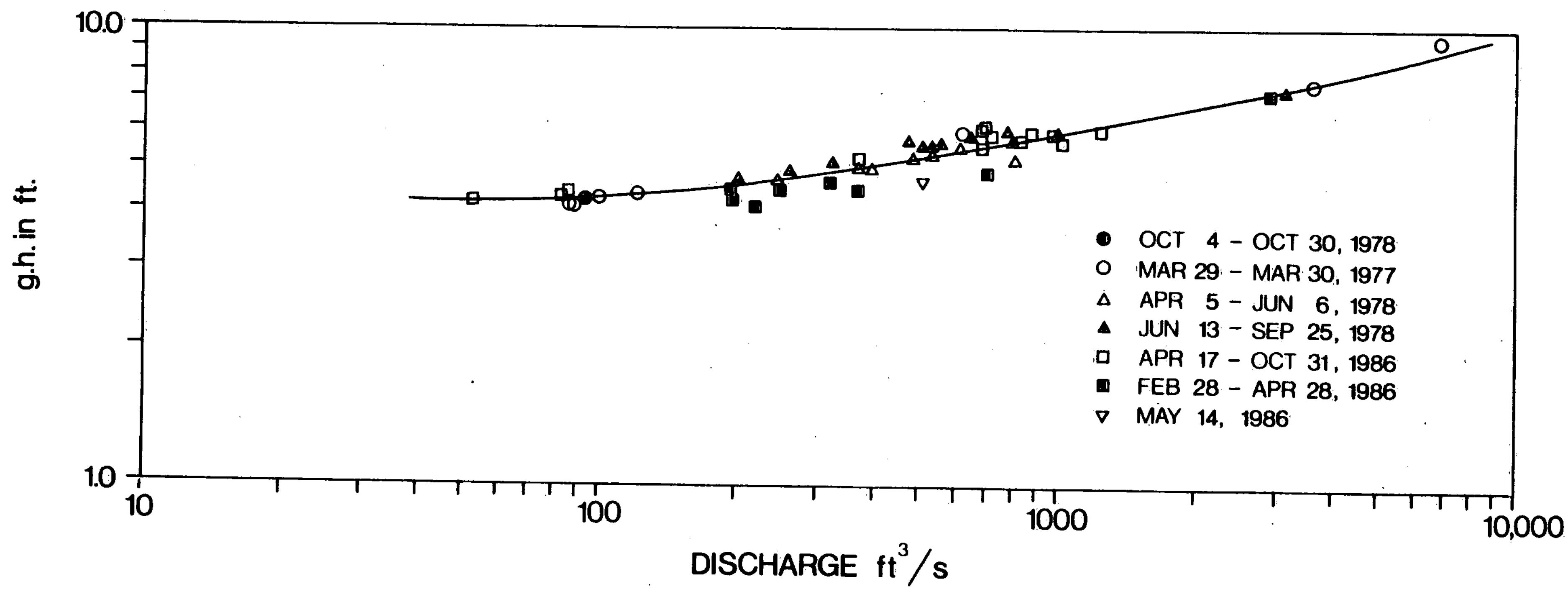

FIGURE 8. Typical stage – discharge relationship at eastern crossing.

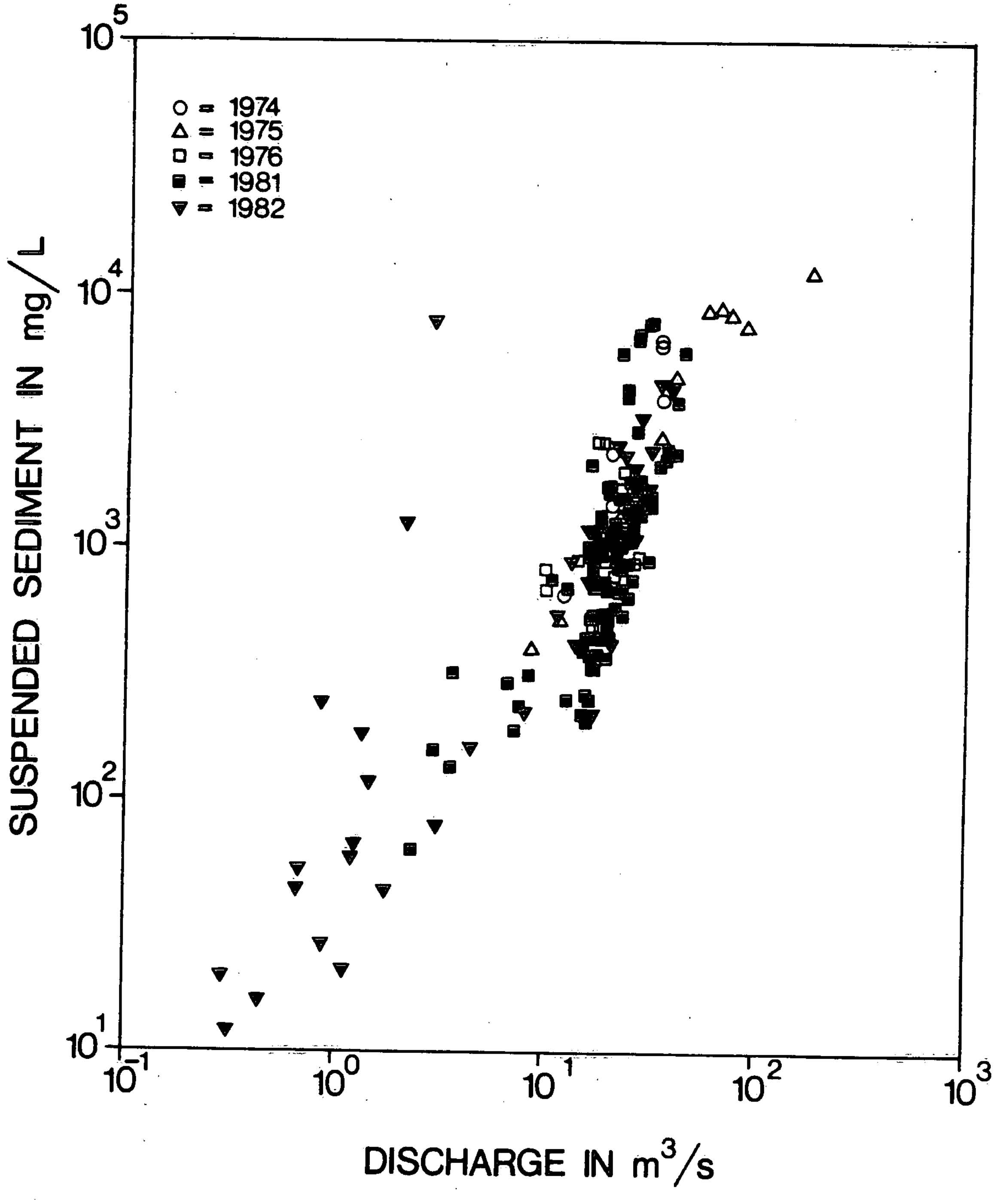

FIGURE 9. Suspended sediment concentrations (W.R.B. 1986)

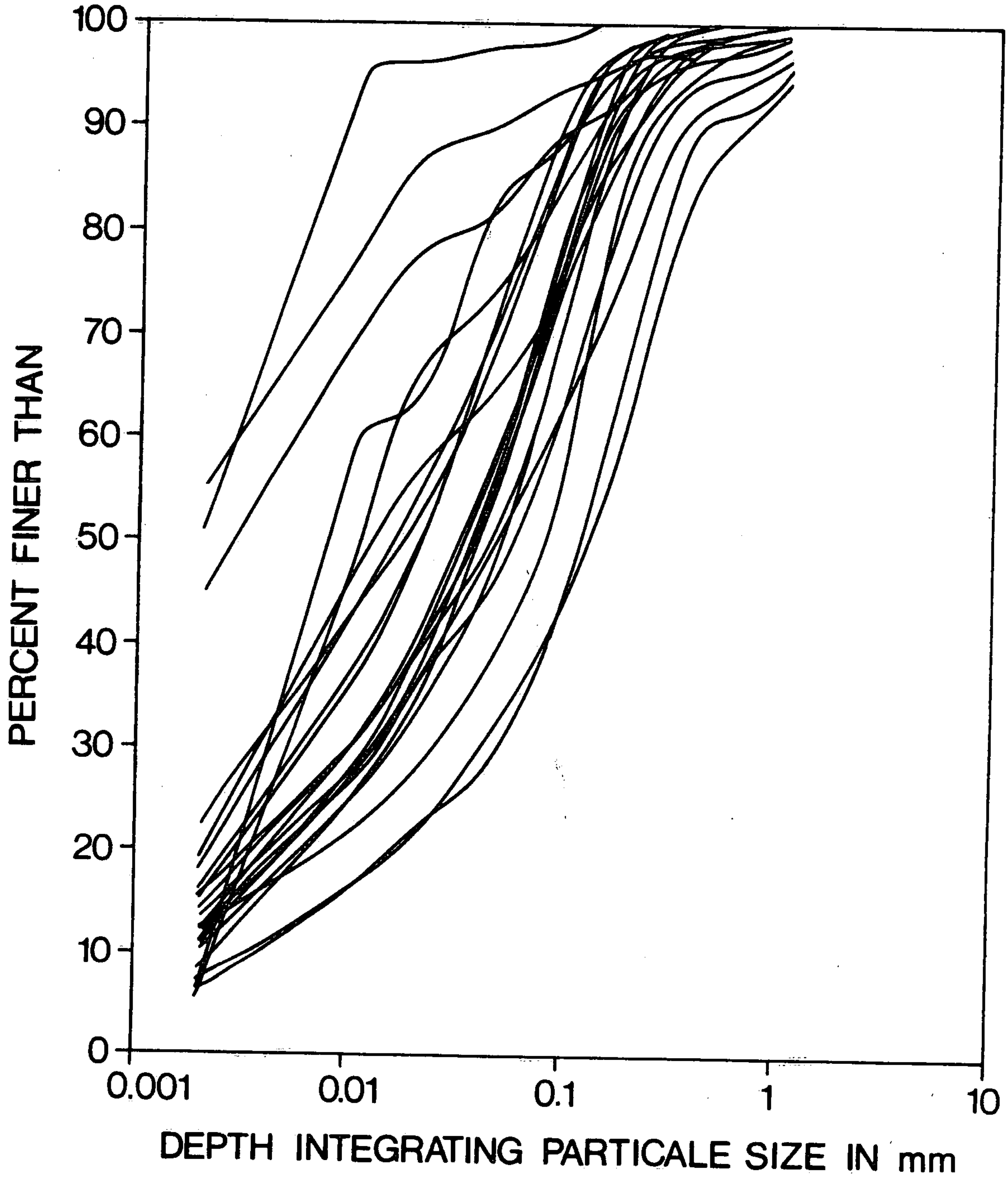

FIGURE 10. Grain size distribution of suspended sediment (W.R.B. 1982)

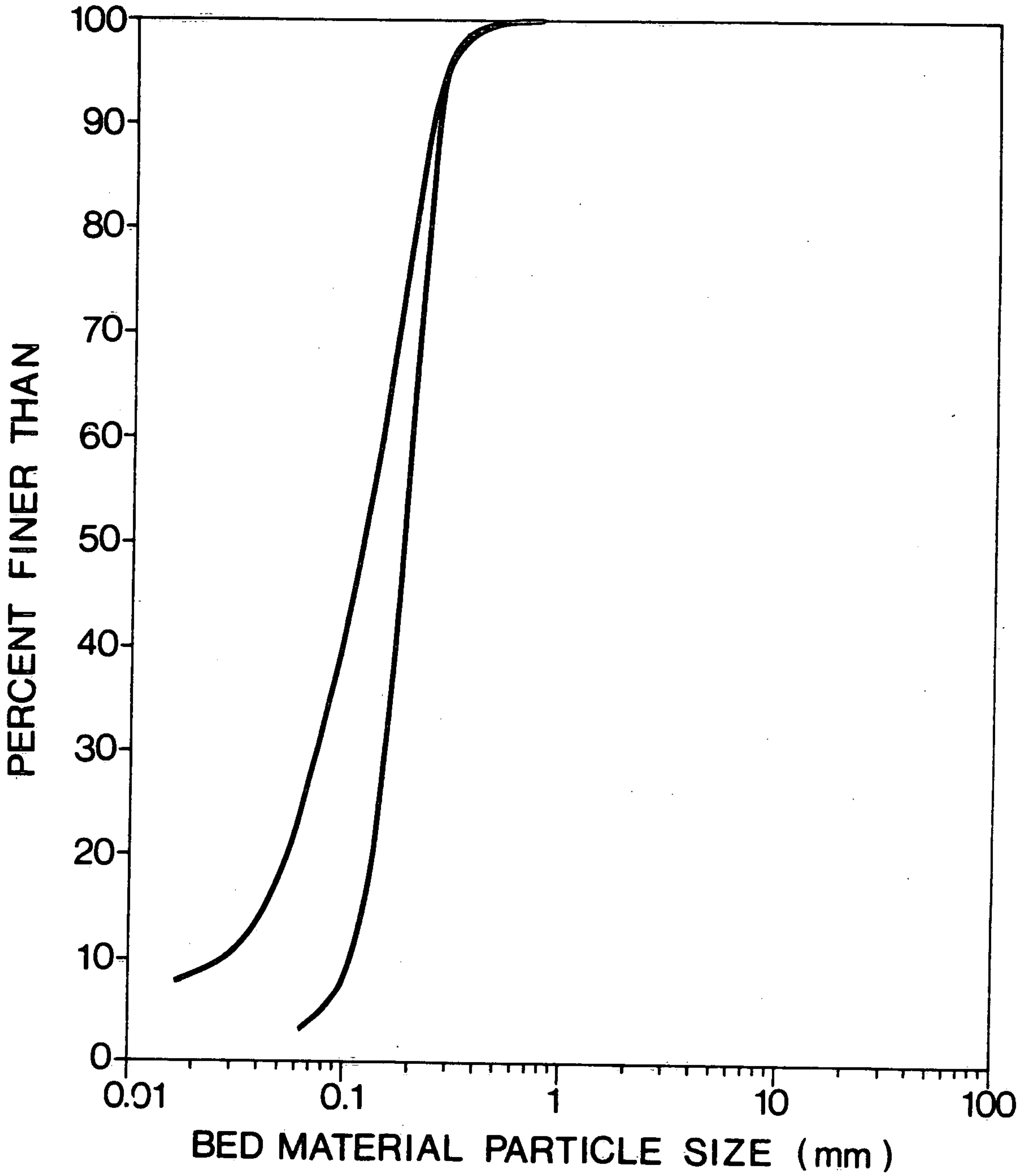

FIGURE 11. Grain size distribution for bed material (W.R.B. 1986)

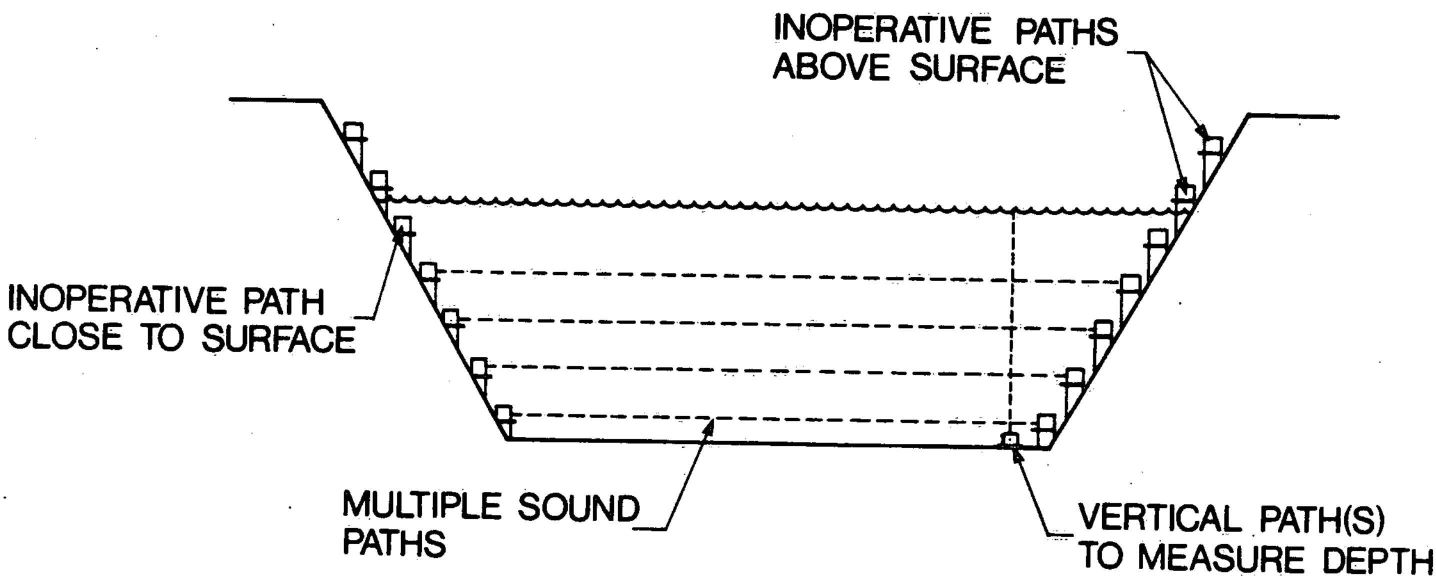

FIGURE 12a Schematic section of river showing multiple paths (Newman, 1982)

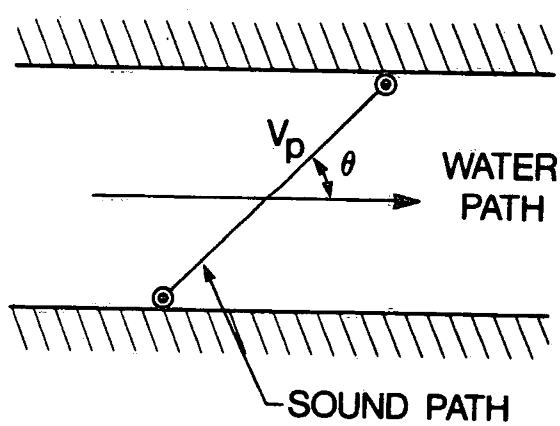

FIGURE 12b Schematic plan of river showing angled flight path. (Newman, 1982)

NOTE:
FIELD COIL CAN BE
PLACED ABOVE OR BELOW THE FLOW

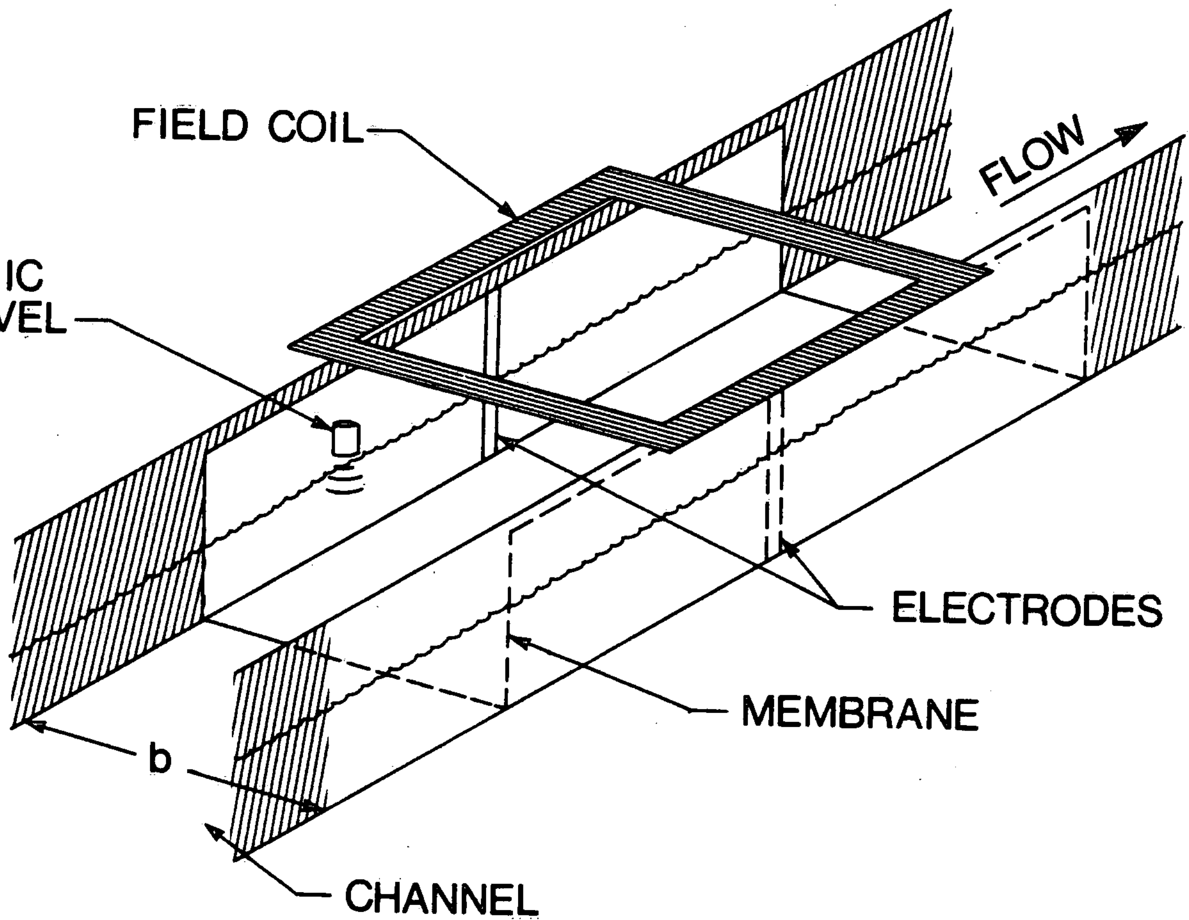

FIGURE 13. Schematic layout of open channel electromagnetic flow meter. (After Newman 1982)

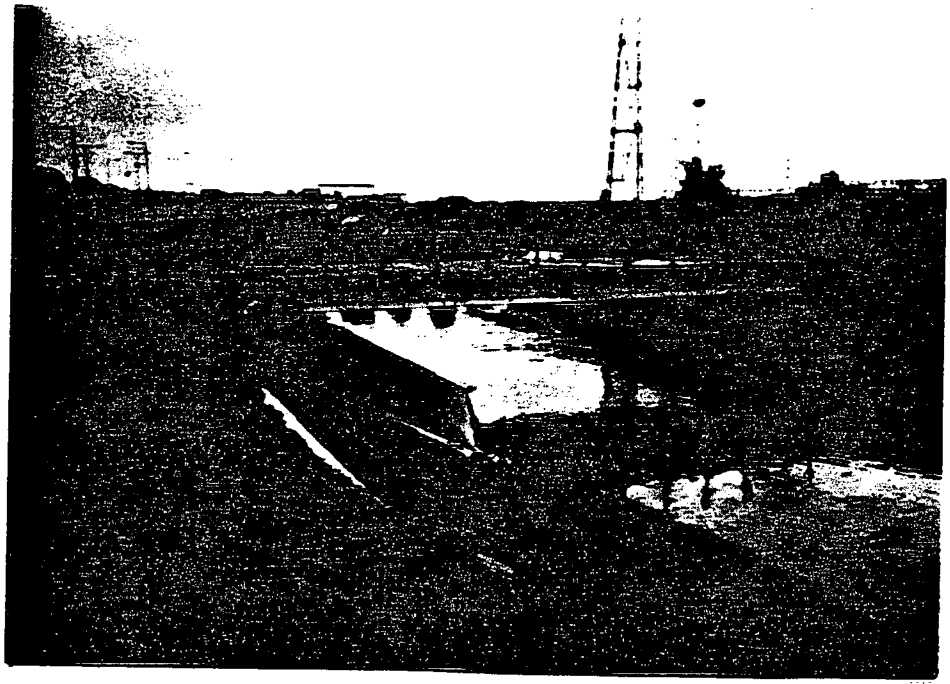

Fig. 14 Gauging station on the River Tame near Walsall. Flume gauge converted to electromagnetic measurement. Bank full flow is 59 m^3/s.

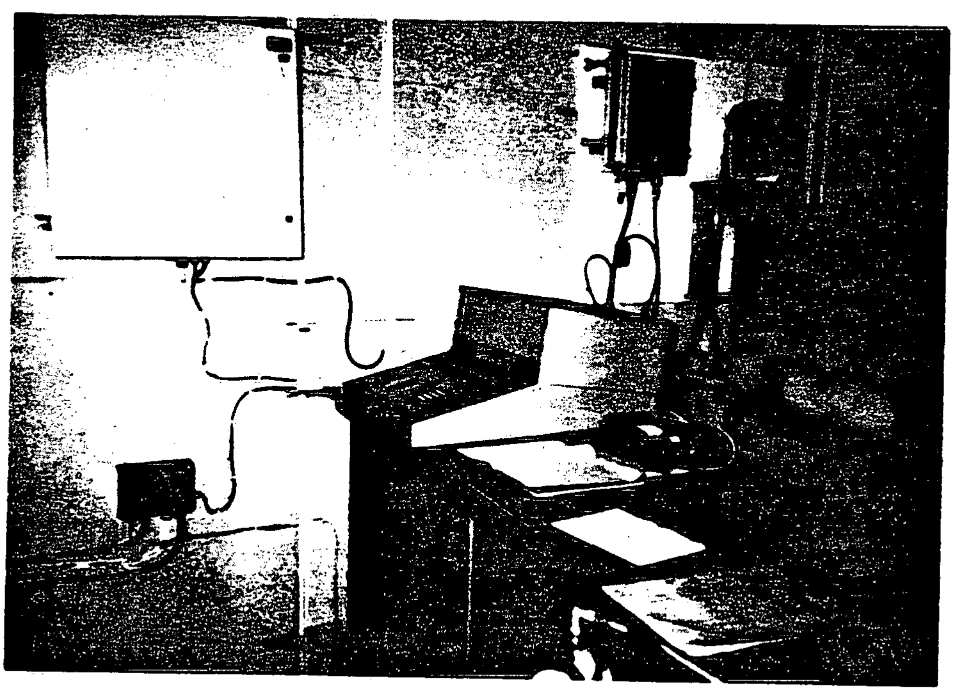

Fig. 15 River Tame gauge house showing control for electromagnetic measurement. Data transferred by telephone line to central office.

Fig. 16 River Soar Gauge at Little Thorpe Narborough. River bank is stabilized with plastic gabions. Photograph shows electrode tube. This gauge has been in place six years.

Fig. 17 River Soar Gauge. Manhole showing the end of the stainless steel electrode.

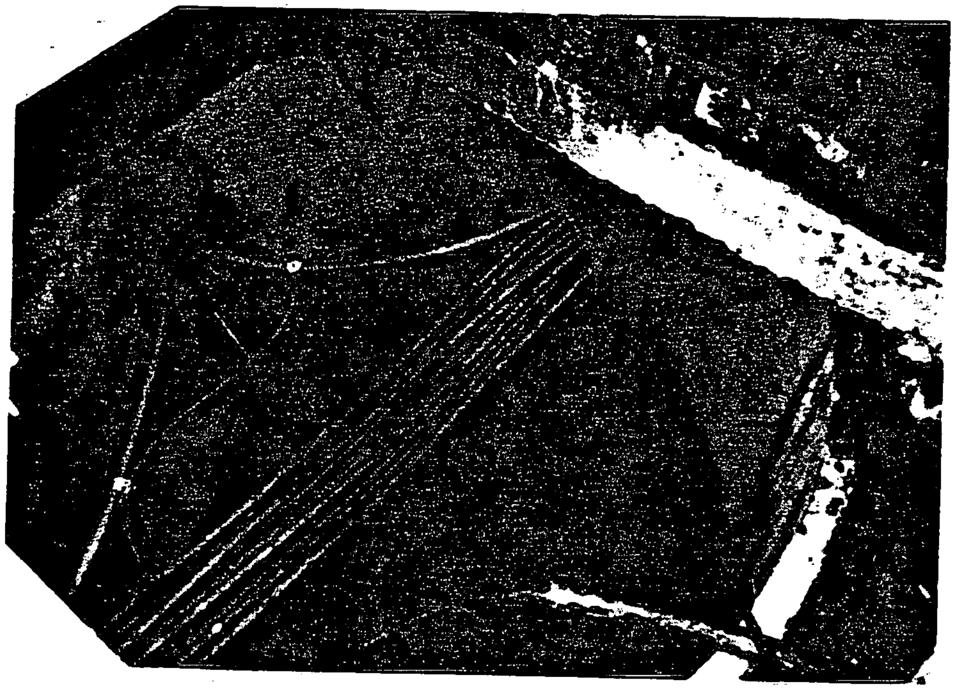

Fig. 18 River Soar Gauge. Coil conductors containing 11 conductors. There are 10 cables and conductors are connected to give 110 turns with a current 10 amperes.

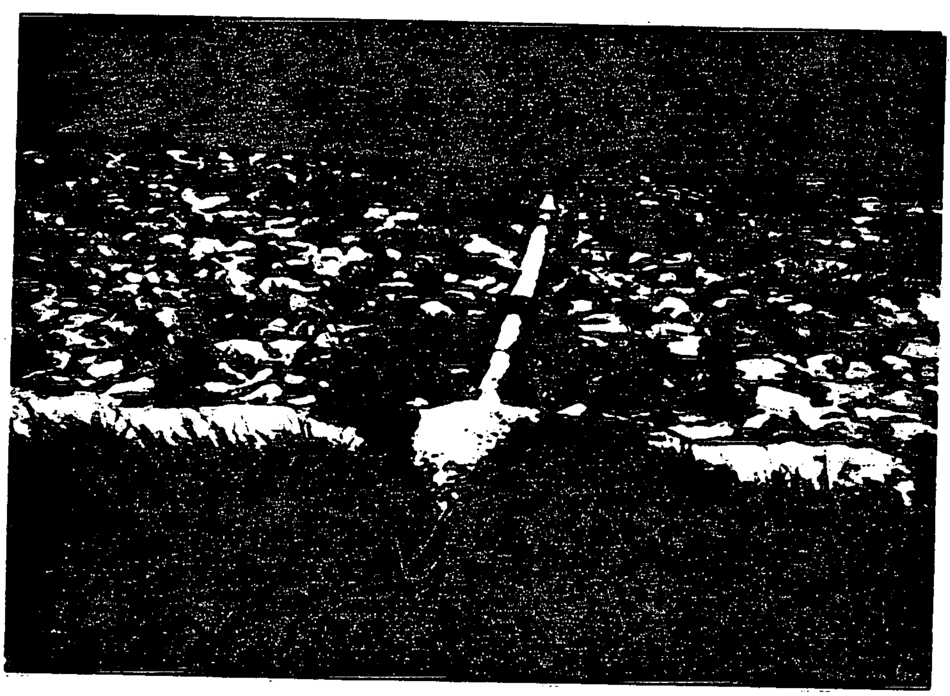

Fig. 19 Gauge site on River Sence where banks are stabilized with Rip Rap. Membrane and coils lie under the rip rap.

Fig. 20 Electrode is placed within rip rap layer to minimize damage in River Sence gauge.

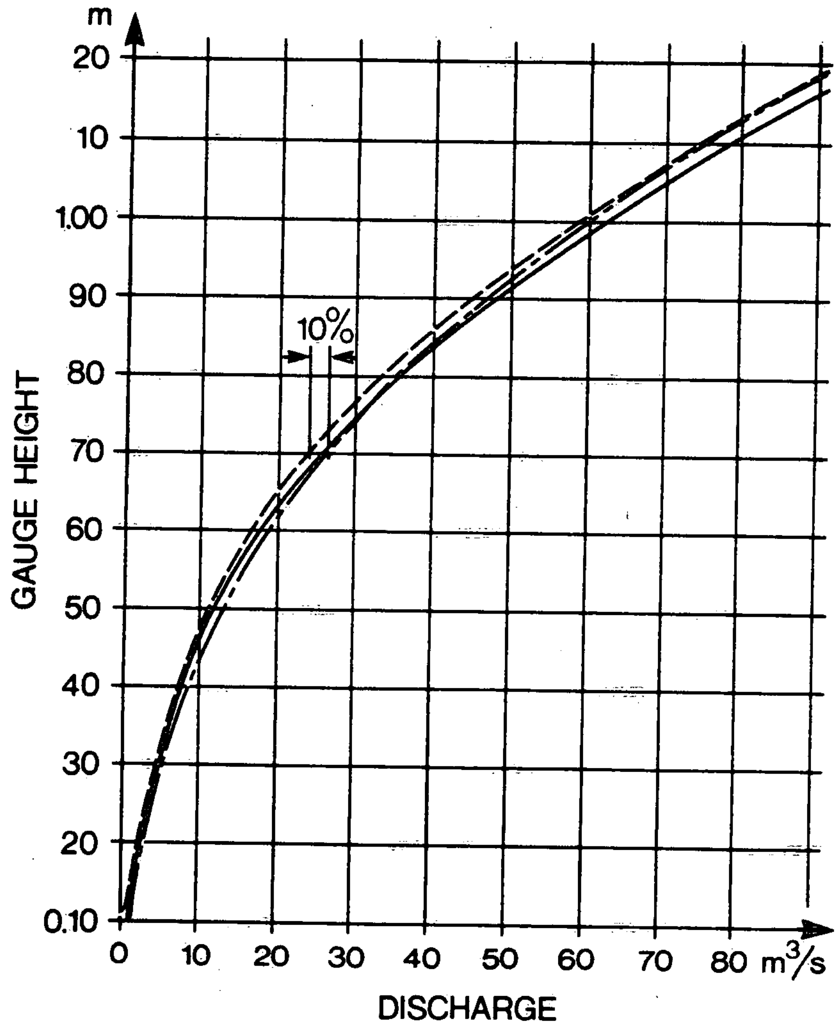

FIGURE 21. Effect of sediment deposition behind broad crested weir on stage discharge curve. (WALSER, 1970).

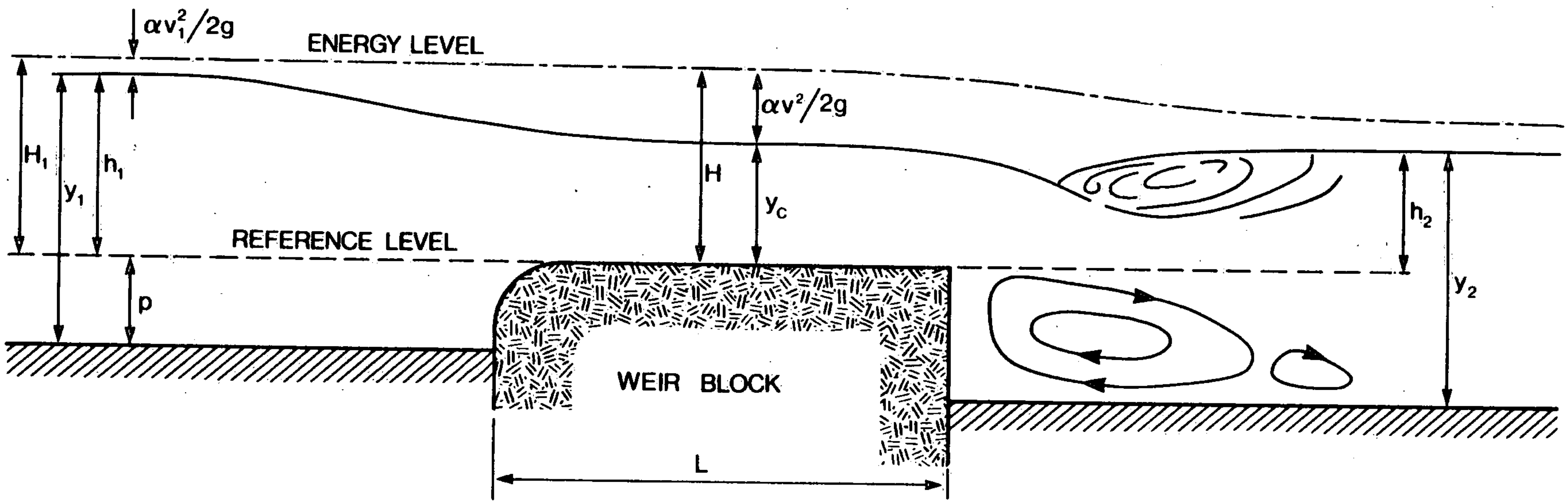

FIGURE 22. Flow pattern over a broad crested weir

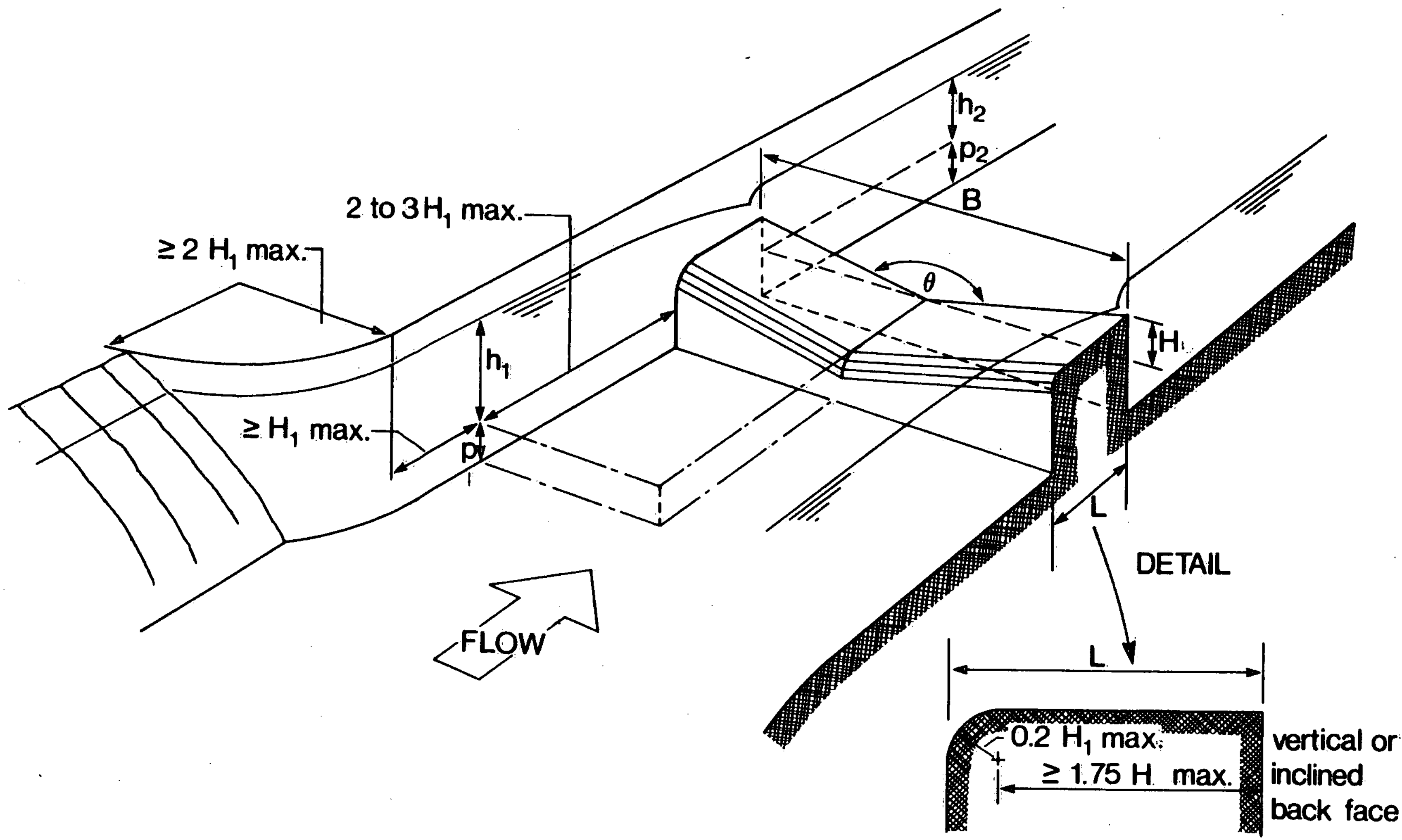

FIGURE 23. Definition sketch for triangular broad-crested weir.

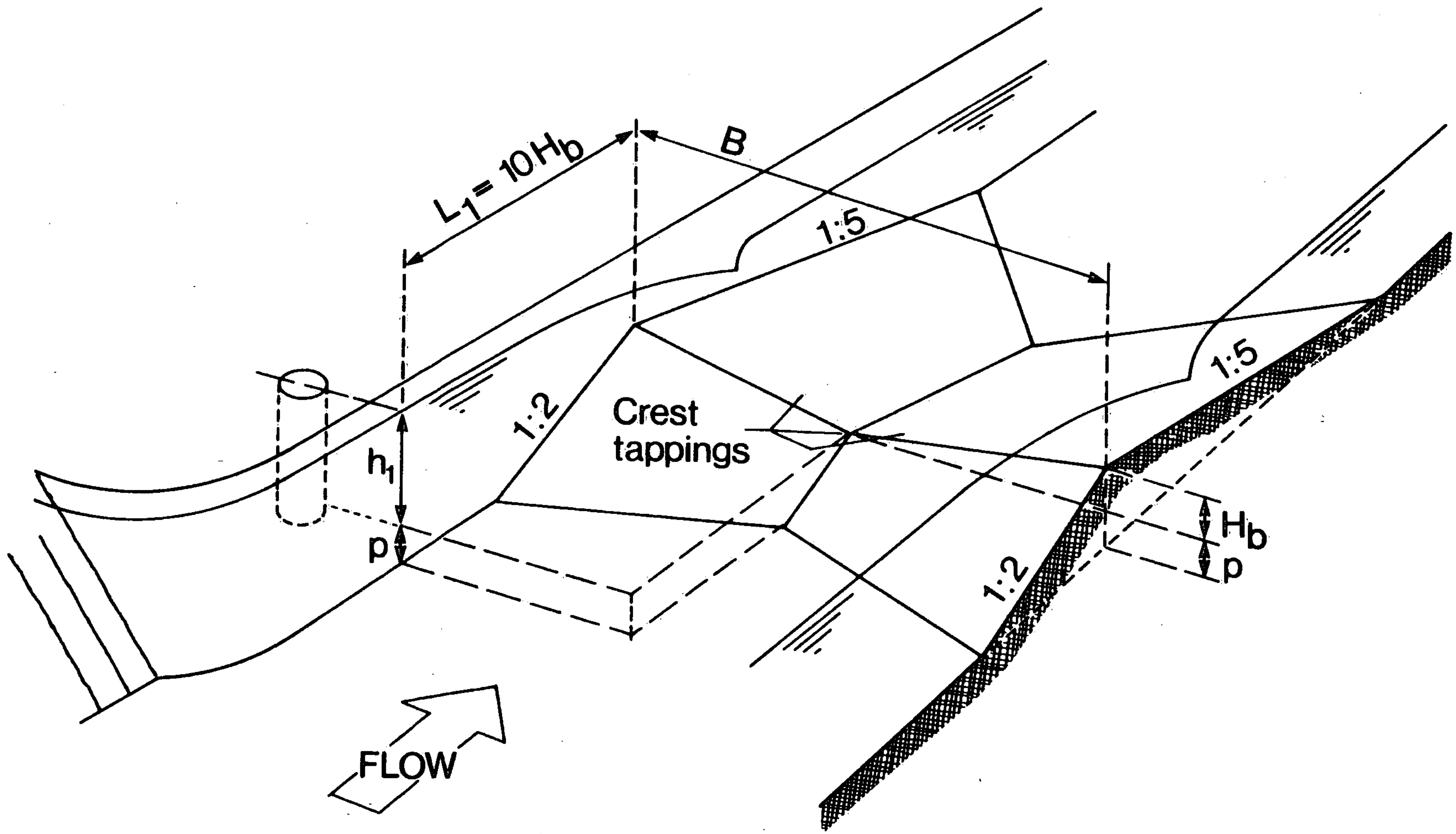

FIGURE 24 Triangular profile flat – V weir.

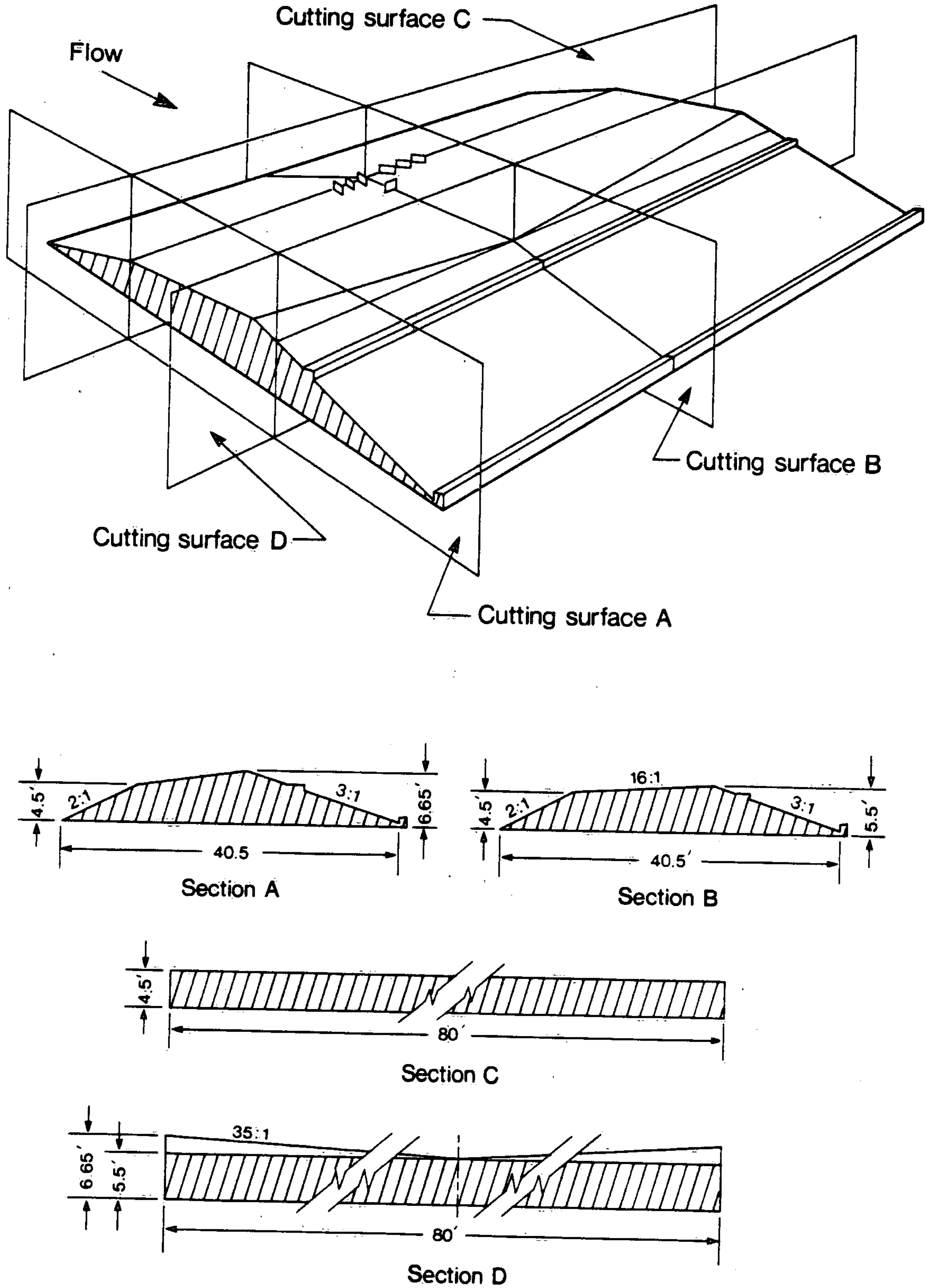

FIGURE 25. Details of the control as proposed by Harris and Richardson (1964)

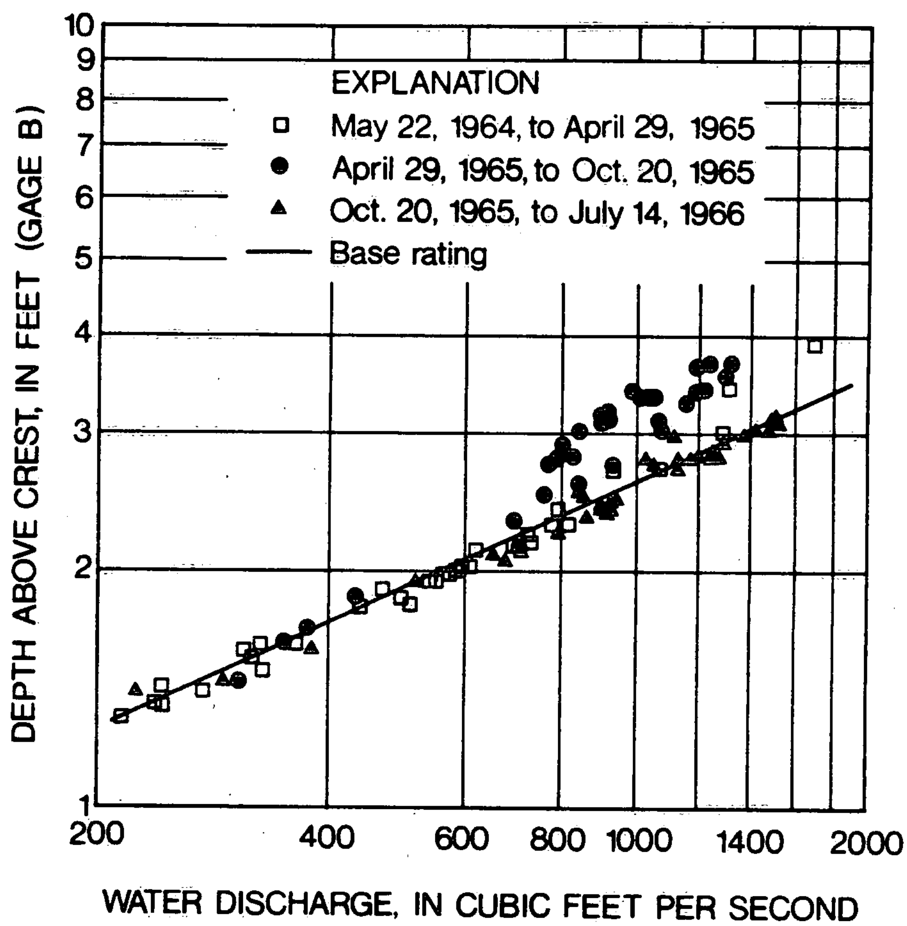

FIGURE 26. Stage discharge relation for conveyance channel, gage B, at control.

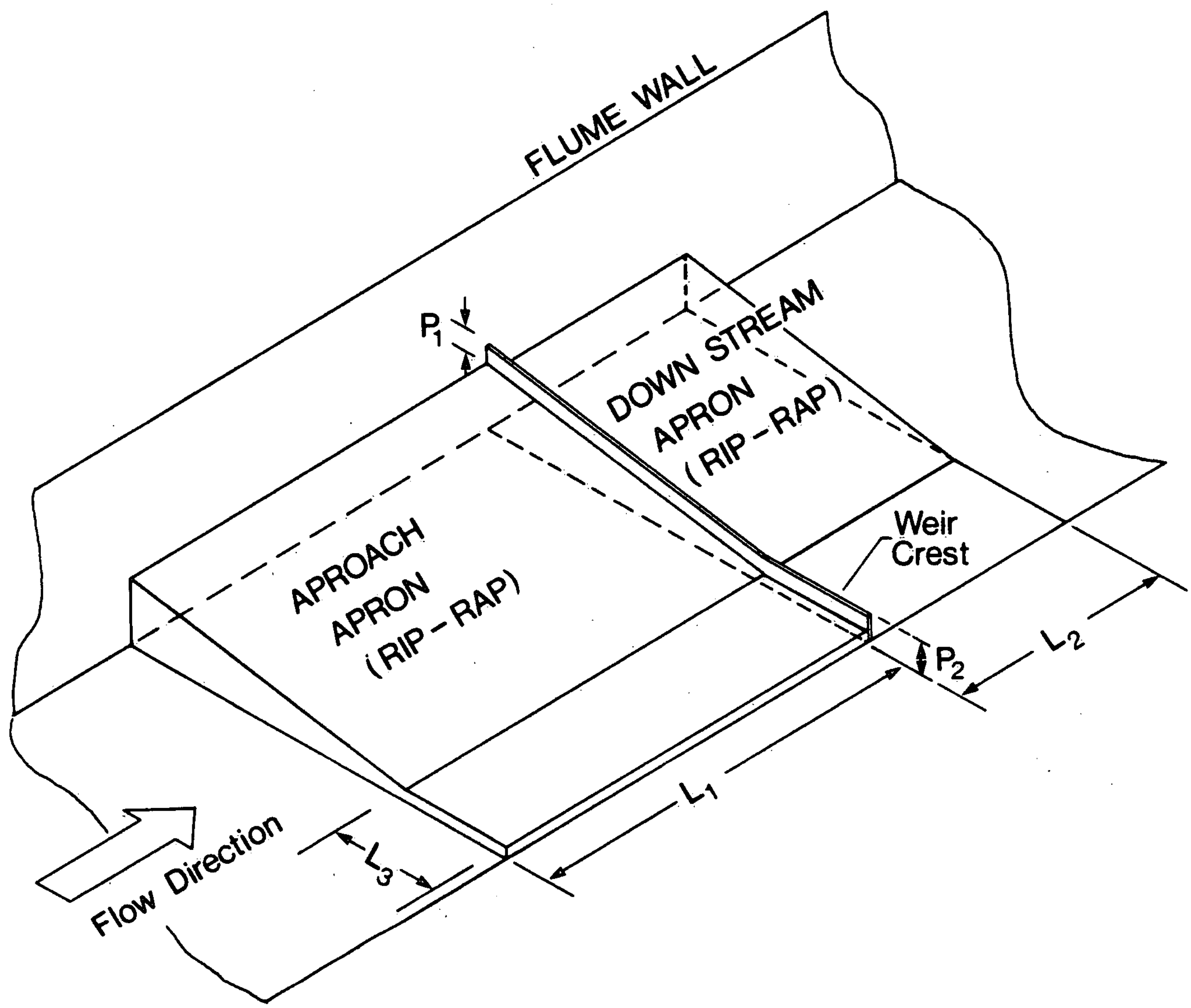

FIGURE 27. U.S. design No. 2.

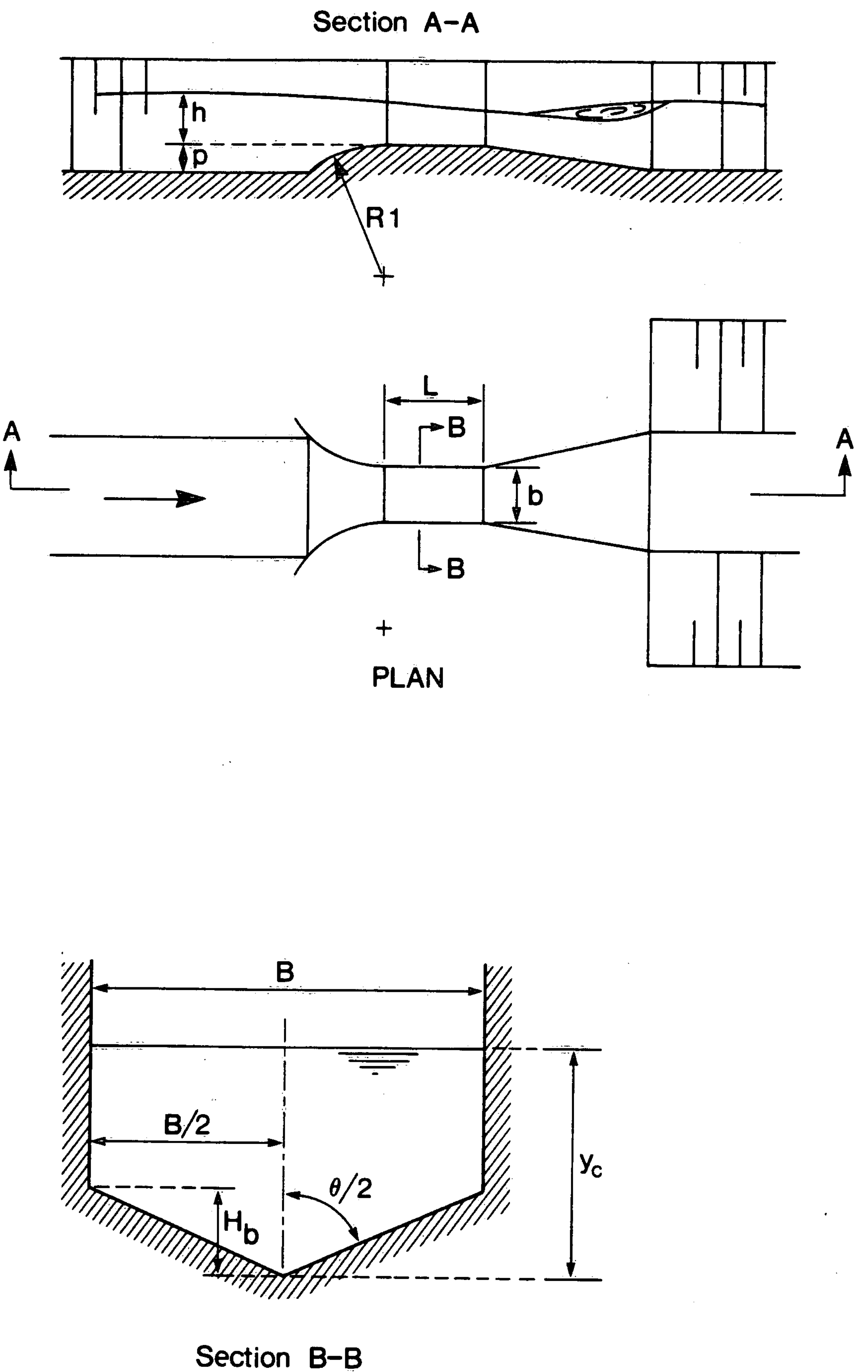

FIGURE 28. Long throated critical depth flume.

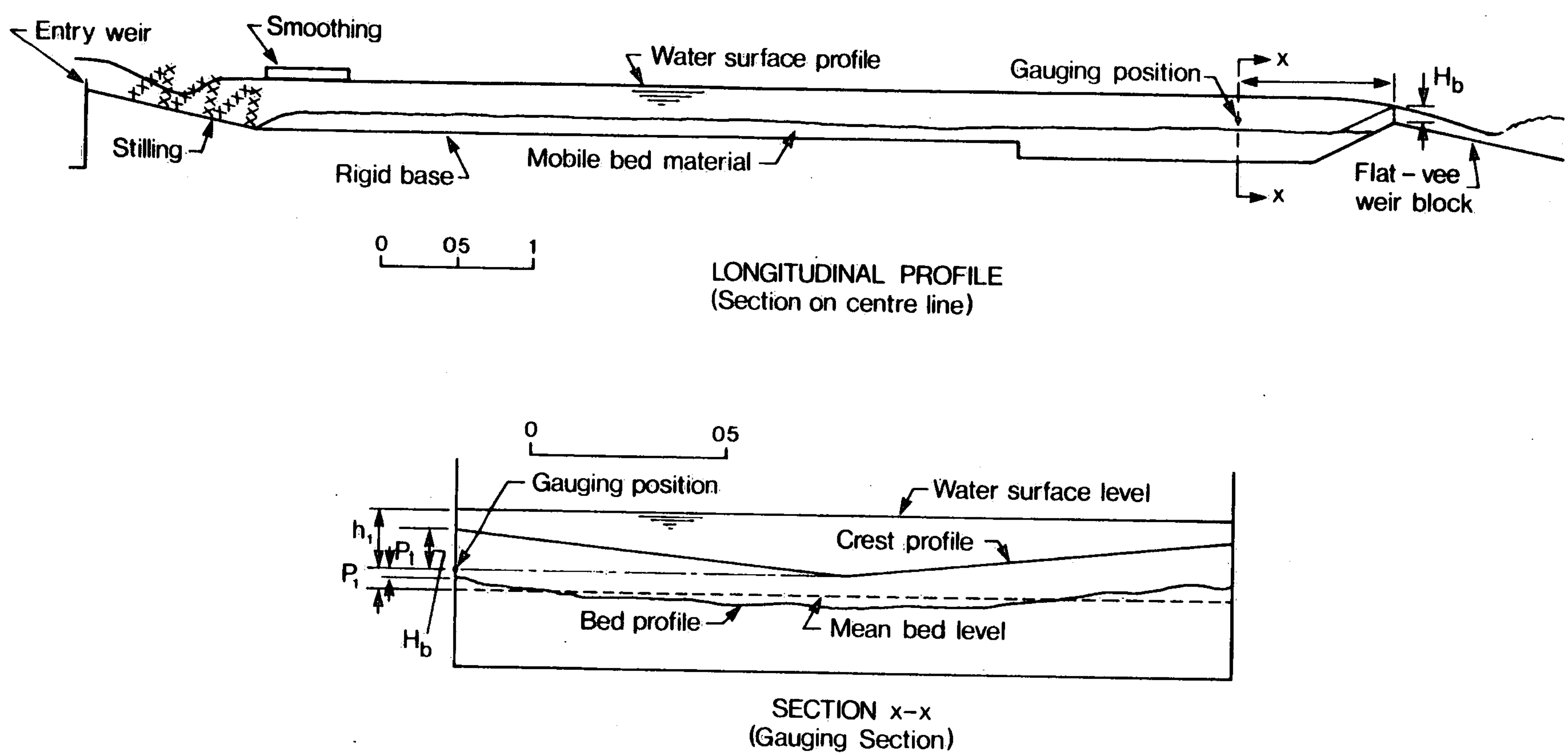

FIGURE 29. Typical model set up

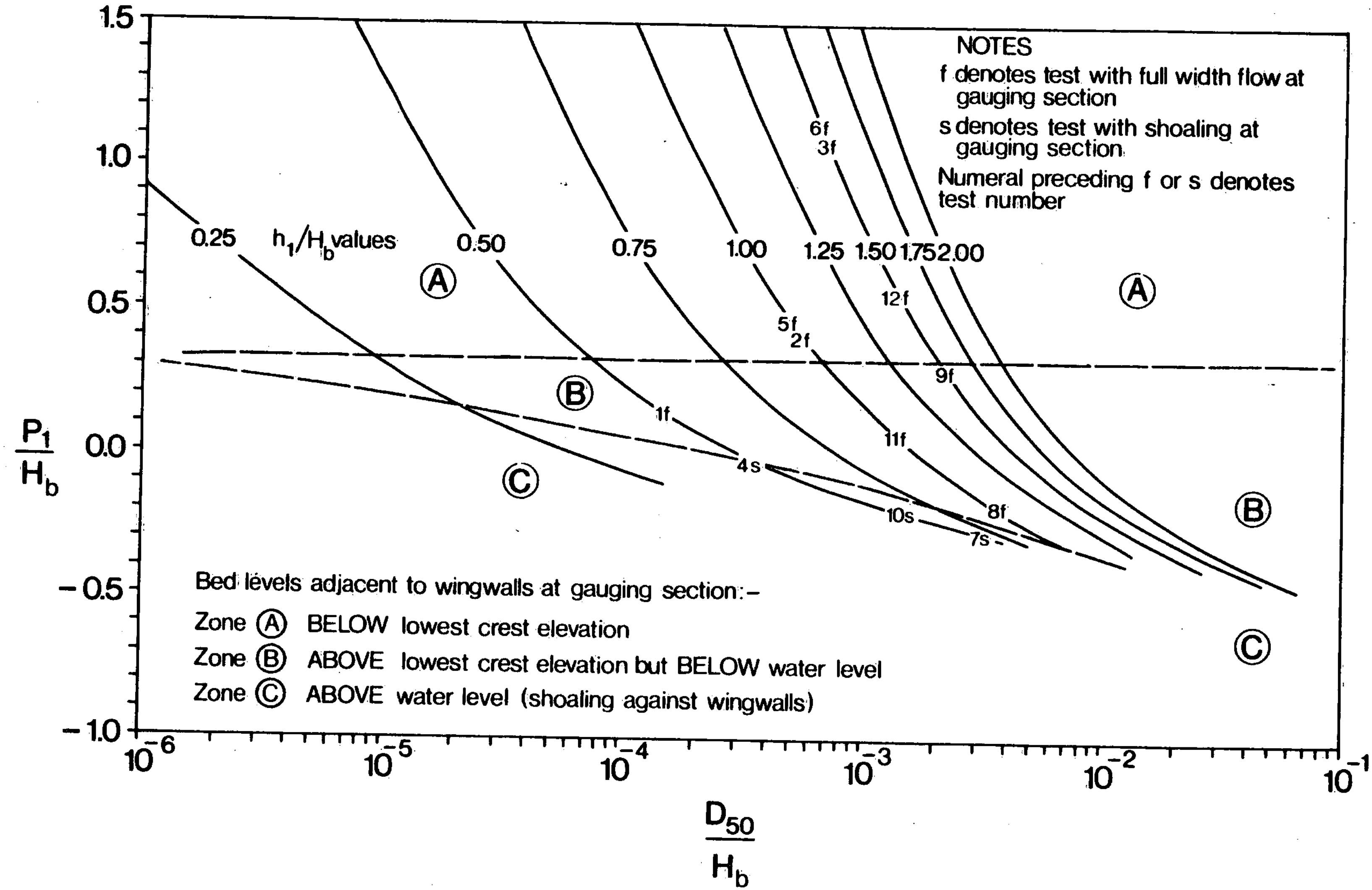

FIGURE 30 Effect of D_{50} on effective height of weir P_1 (White, 1970)

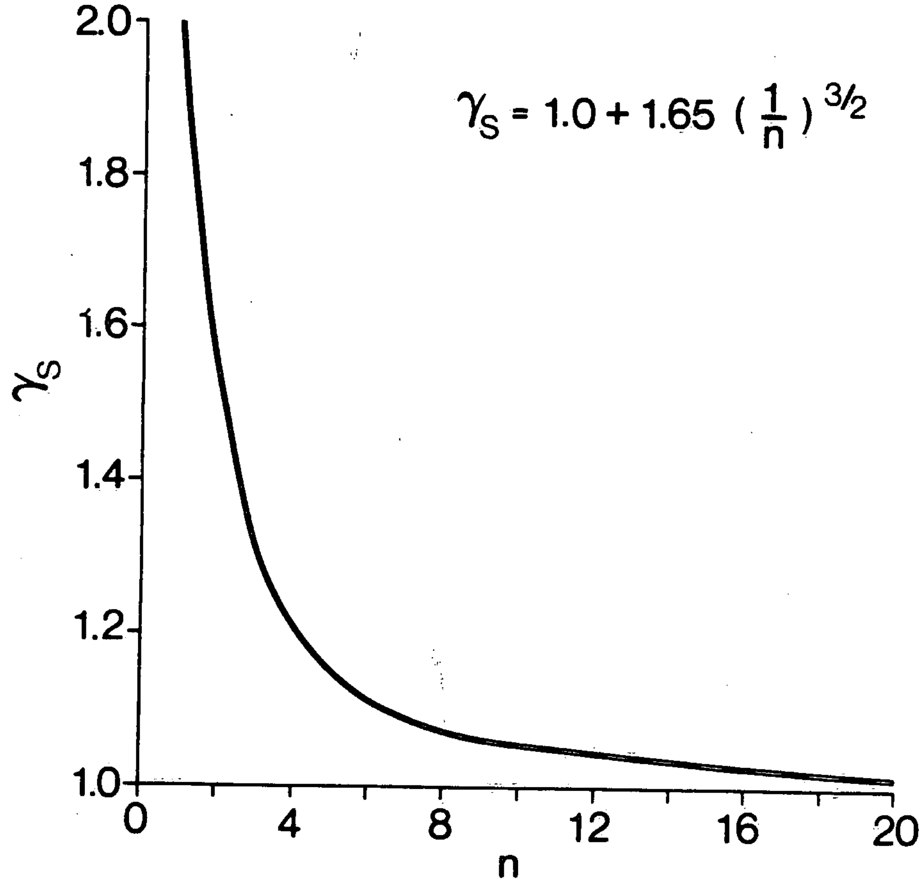

FIGURE 31. Effect of scale ratio on $\lambda\gamma_S$.