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LABORATORY TESTS OF ARTIFICIAL CONTROL FOR THE MILK RIVER - PHASE I

by

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MANAGEMENT PERSPECTIVE

Accurate discharge data are required for stream gauge sites of the Water Survey of Canada such as the one on the Milk River at Eastern Crossing. The waters of the river are shared by Canada and the United States in accordance with the Boundary Waters Treaty. In the past it has been difficult to obtain discharge data of sufficient accuracy because of unstable stage-discharge relationships caused by shifting channel bed and migrating sand waves.

This report examines two types of submerged weir to select the one best suited for application to the Milk River. Physical model tests have shown that the three dimensional weir, known as the Flat Vee weir, provides the best choice as an artificial control. Additional model tests need to be conducted to examine the effect of changes in bed elevation in the approach channel on the performance of this weir.

Dr. J. Lawrence
Director
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PERSPECTIVE - GESTION

Des données de débit précises sont exigées pour les stations de jaugeage du débit de la Division des relevés hydrologiques du Canada, par exemple pour la station de la rivière Milk de Eastern Crossing. Le Canada et les États-Unis se partagent les eaux de la rivière conformément au Traité des eaux limitrophes. Dans le passé, il a été difficile d'obtenir des données de débit d'une exactitude suffisante en raison de relations hauteur-débit instables causées par le fond mobile du chenal et par les vagues de sable mouvant.

Ce rapport examine deux types de déversoirs submergés pour sélectionner celui qui convient le mieux à l'utilisation projetée de la rivière Milk. Les essais sur maquettes ont révélé que le déversoir tridimensionnel, qui a la forme d'un V plat, est le meilleur choix aux fins d'un contrôle artificiel. D'autres essais sur maquettes doivent être effectués pour examiner l'effet des changements dans l'élévation du lit (dans le chenal d'approche) sur le rendement de ce déversoir.

J. Lawrence

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ABSTRACT

Tests were conducted to compare two artificial control structures being considered for use on the Milk River at Eastern Crossing, Montana. The first weir tested was broad crested with a triangular cross-section and the second weir was short crested with a triangular cross-section. The tests indicated that the short crested weir, also known as the Flat Vee weir, showed more promise for use in sand bed streams such as the Milk River.

RÉSUMÉ

Des essais ont été réalisés pour comparer deux structures de contrôle artificiel à l'étude en vue d'une utilisation sur la rivère Milk, à Eastern Crossing, dans le Montana. Le premier déversoir testé était un déversoir triangulaire a seuil épais et le second, un déversoir triangulaire à seuil mince. Les essais ont révélé que le déversoir à seuil mince, aussi connu comme déversoir en V plat, était plus prometteur pour une utilisation dans des cours d'eau à lit sablonneux comme la rivière Milk.

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1.0 INTRODUCTION

Sandbed streams often present problems in determining their discharge because they are subject to changes in bed roughness and bed elevations due to scour and deposition. As a result, stage-discharge relationships are often poorly defined and corrections based on frequent water discharge measurements must be applied. The Milk River at Eastern Crossing Montana, is an example of such a stream (Figure 1).

Various methods to improve the measurement of discharge on the Milk River have been examined by Engel, Lau and Dick (1986). The most effective way of avoiding sediment deposition in the approach channel is to minimize backwater effects. To do so, a structure should be selected whose head-discharge curve coincides with the stage-discharge curve of the channel. An appropriate structure is one which has a truncated triangular control section. Such a structure also satisfies the requirements of good sensitivity at low flows, stability in the coefficient of discharge over the full flow range as well as good sediment and debris passing capability. However, there is some uncertainty regarding the effect of changes in bed levels in the approach channel on the performance of such weirs. The weir crest must be located above the channel bed by some minimum height at all flows and be effectively free of sediment in order for the coefficient of discharge to be stable. To examine the effect of the sandbed on the control structure, a limited model study was conducted.

This investigation is part of a comprehensive study to determine a suitable method to measure the discharge of the Milk River. The work was conducted at the request of the Water Survey of Canada, Calgary, Alberta. All the tests were carried out in the Hydraulics Laboratory of the Research and Applications Branch at the National Water Research Institute, Burlington, Ontario.

2.0 MODELLING CONSIDERATIONS

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 head-discharge relationships. Engel, Lau and Dick (1986) examined two types of Vee notch weirs which could be considered for further evaluation. These weirs are the broad crested Vee notch (BV) and a short crested Vee notch weir initially proposed by White (1970) referred to as the Fat Vee (FV) weir. These two weirs are shown in Figures 2 and 3 respectively. Their basic characteristics are summarized in Table 1.

2.1 Modelling of the Weirs

The elevation of the bed in the approach channel of the weir can be quantified in terms of the upstream weir height P which is defined as the distance between the lowest point on the weir crest (apex of the Vee notch) and the average elevation of the sand bed in the channel cross-section. Using dimensional analysis it can be shown that

$$P/H_b = f(h_1/H_b, D_{50}/H_b, \gamma_s D_{50}^3/\rho\nu^2) \qquad (1)$$

where h_1 = the measured head on the weir, H_b = the difference in elevation between the lowest and highest point on the weir crest, D_{50} = the median diameter of the sand bed in the approach channel, ρ = the density of the water, ν = the kinematic viscosity of the water, γ_s = the specific weight of the sediment, and f denotes a function. If one now writes $D_* = \gamma_s D_{50}^3/\rho\nu^2)$, then equation (1) becomes

$$P/H_b = f(h_1/H_b, D_{50}/H_b, D_*) \qquad (2)$$

If the fluid properties and the sediment properties are kept constant, then each grainsize represents a constant value of D_*. For the Milk River the median grainsize for the bed material is of the order of 0.1 to 0.2 mm. As a result, one can expect that the sand waves generated by the flow are ripples (Yalin, 1977). Once the sand waves have been formed, their roughness height is large enough to cause the flow to be in the rough turbulent flow regime where viscosity effects are negligible and therefore the variable D_* can be omitted from the equation (2). Therefore, one may now write

$$P/H_b = f(h_1/H_b, D_{50}/H_b) \tag{3}$$

Equation (3) states that for a bed material of a given density the elevation of the bed of the approach channel depends on the relative head on the weir and the relative size of the bed material.

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

$$\lambda\pi_1 = 1, \ \lambda\pi_2 = 1, \ \lambda\pi_3 = 1 \tag{4}$$

in which $\pi_1 = P/H_b$, $\pi_2 = h_1/H_b$, $\pi_3 = D_{50}/H_b$ and λ denotes the ratio of a property of the model to the same property of the prototype. If one now adopts a scale ratio, say λ_L, for all length dimensions, then one obtains in accordance with the conditions of equation (4)

$$\lambda\pi_1 = 1: \quad \lambda_p = \lambda_{H_b} = \lambda_L \tag{5}$$

$$\lambda\pi_2 = 1: \quad \lambda_{h_1} = \lambda_{H_b} = \lambda_L \tag{6}$$

$$\lambda\pi_3 = 1: \quad \lambda_{D_{50}} = \lambda_{H_b} = \lambda_L \tag{7}$$

The conditions required by equation (7) cannot be met in a scale model. This is due to the fact that the grainsize in the prototype stream bed is of the order of 0.15 mm which is already quite small. As a result, assuming a realistic scale factor of, say 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 D_{50}/H_b because P/H_b in equation (4) is quite sensitive to changes in D_{50}/H_b, particularly at higher flows (Engel, Lau and Dick, 1986). Therefore, a dynamically similar model which takes into account both the characteristics of the structure and the behaviour of the sandbed in the vicinity of the structure is not possible. The best that can be done is to conduct tests of relative performance of different structures for similar flow and sand bed characteristics.

2.2 Modelling of Sediment Transport

According to Yalin (1971) and Krishnappan (1987), the scale relationships for designing small scale physical models of sand bed rivers are given as

$$\lambda_D = (\lambda_s \lambda_y)^{-1/2} \tag{8}$$

$$\lambda_{\gamma_s} = (\lambda_s \lambda_y)^{-3/2} \tag{9}$$

$$\lambda_s = n \tag{10}$$

$$n = \lambda_y^{-3/7} \tag{11}$$

$$U = \lambda_y^{1/2} \tag{12}$$

where λ denotes the ratio of the model value of a property to the prototype value of the same property, y refers to all vertical length dimensions, D = the grain size of the sediment, U = the mean velocity of the flow, s = the slope of the approach channel and n = the geometric distortion given by λ_y/λ_x in which x refers to all horizontal dimensions. It is clear from equations (8), (9), (10) and (11) that the bed material of the scale model will have a lower density and a larger diameter than that of the prototype. If one considers a scale ratio of 1/10, the distortion would be 2.68 and a suitable material with the required size and density would be anthracite coal.

A scale model based on equations (8), (9), (10) and (11) would be reasonable if one were modelling the sediment transport only. However, the distortion required to model the sediment transport is not admissable for modelling of the weir. Therefore, direct modelling of the effect of the weirs on the bed of the approach channel is not possible. To overcome this dilemma, the modelling criteria were reduced to the conditions imposed by equations (8), (9) and (12).

Equations (8) and (9) make it possible to construct a limited scale model using a bed material similar to that in the prototype. This means that $\lambda_D = 1$ and $\lambda_{\gamma s} = 1$, and consequently, $\lambda_s = 1/\lambda_y$. The model is limited because as a result of eliminating the scale distortion the bed slope had to be exaggerated. This means that, although the conditions imposed by equations (8) and (9) are met, the flow resistance due to grain roughness of the sand and form roughness of the sand waves is not modelled. However, in the approach channel near the weir, this limitation may not be too serious because the weir acts as the local flow control and may largely over-ride the depth effects due to the resistance of the channel bed itself. Certainly such a model can be used for the relative comparison of the two weirs being considered. Practical considerations dictate that a scale ratio of $\lambda_y = 1/10$ should be used, which means that the bed slope of the approach channel in the model must be ten times greater than that of the prototype.

The width of the measuring section at Eastern Crossing is about 48 m and a scale ratio of 1/10 requires that the model should have a width of 4.8 m. However, for the purpose of relative comparisons, it was decided that a model width of 2 m would be sufficient, particularly in view of the fact that the primary concern is the sedimentation in the central portion of the weirs where the crests have the lowest elevations. Accordingly, a suitable temporary flume to test these models was built.

3.0 EXPERIMENT

3.1 Experimental Set Up

The experiments were conducted in a flume especially constructed for this purpose, 20 m in length, 2.0 m in width and about 0.6 m in depth as shown in Figure 4. The flume was built directly on the laboratory floor and therefore its slope could not be adjusted. The flow depth was controlled by a set of vertical louvres located at the downstream end of the flume. Water was supplied to the flume from a large constant head tank through a head box. The flow was conditioned prior to entering the test section by a series of baffles and flow straighteners. The water temperature was kept at about 18°C.

The weir to be tested was placed into the flume 15 m from the entrance on an elevated false floor about 0.30 m above the flume bed. The false floor was extended upstream from the crest of the weir for 1 m thereby providing a rigid section equivalent to a stabilized bed in the prototype. It was reasoned that such a stabilized bed would be resistant to scour and thus velocities of the flow passing over this section would be large enough to ensure the passing of the bed load approaching the weir during high stages. Building the weir on the raised false floor provided a recess into which the sediment was placed for a length of 8 m upstream from the weir. The sand had properties similar to that of Milk River at Eastern Crossing. A typical grain size distribution of the test sand is given in Figure 5.

The intake of the stilling well for each weir was placed at a distance upstream in the side wall of the flume in accordance with the recommendations given in Bos (1976).

3.2 Measurement of Water Levels

The water levels for the determination of the head on the weir were measured using a stilling well fabricated from a 5 cm diameter acrylic cylinder fitted with a Mitutoyo point gauge mounted as shown in Figure 6. The point gauge had a resolution of 0.05 mm. The point gauge was set to read zero at the elevation of the lowest point on the weir crest.

3.3 Measurement of Discharge

Prior to an experiment, the pump being used to provide the flow was tested to obtain the true delivery rate. Once the desired flow had been established in the flume, the discharge could be determined. This was done by diverting the overflow from the constant head tank into a calibrated volumetric tank in which it was measured by observing the change in water level over a sufficiently long period of time. The discharge through the flume was then simply the difference in the flow rate supplied by the pump and the overflow rate of the constant head tank. This procedure was repeated twice for each experiment and the results averaged, providing an accuracy of about 2%.

3.4 Test Procedure

At the start of an experiment the sand was levelled to obtain a smooth bed with a surface slope equal to 0.0068 which is 10 times that of the prototype (Engel, Lau and Dick, 1986). The surface elevation of the bed was coincident with the surface of the stabilized section near the weir. The downstream louvres were closed and water

was gradually introduced upstream, taking care that no sediment was transported through the test section at this time. This process was continued until the tailwater was well above the lowest point on the weir crest. At this depth the flow rate could be increased without creating any scouring of the sediment in the approach channel. The discharge was increased and the louvres adjusted until the desired flow rate was obtained. The tailwater level was always set so that the final flow conditions at the weir were modular (i.e., unsubmerged). Once the desired flow was established, sand was added manually to the flow at the upstream end of the flume to approximately compensate for the removal of sediment due to the downstream transport.

Each experiment was allowed to continue until there was no apparent change in the bed conditions within the last two metres upstream of the weir. The head on the weir was then measured, the discharge was determined and the water was slowly drained from the flume, taking care that the sediment bed was not disturbed. Once the bed had been completely drained, photographs were taken for comparison of the two weir models under similar flow conditions.

4.0 DATA ANALYSIS

4.1 Sedimentation in the Approach Channel

The transport of sediments is usually divided for the sake of convenience into two components, namely, bed load and suspended load. In the present tests only bedload transport has been considered because it is a good indicator of the effects of sediment transport on the weirs in question. Bed load is the transport of sediment particles sliding, rolling, over the channel bed and transport en masse is generally in the form of moving bed forms such as dunes and ripples.

The rate of transport of the bed load and the height of the sand waves are dependent on the intensity of the flow which is often expressed as the mobility number given by

$$Y = \rho U_*^2/\gamma_s D_{50} \tag{13}$$

where Y denotes the dimensionless mobility number, U_* = the shear velocity, and all other terms have been defined in a previous section. Therefore it is important to compare the interaction of the sediment with the two control structures for similar flow conditions.

4.1.1 The BV weir

Tests on the BV weir were conducted for values of h_1/H_b = 2.24 and 2.90 to ensure satisfactory bed load transport. The sediment depositions after completion of the tests are shown in Figures 7 for the lower flow and Figure 8 for the higher flow.

Figure 7 shows that the sand bed in the vicinity of the weir has taken on the characteristic pattern of sand waves for the sand size used. Deposition of sediment in the approach channel is very even over its entire width even though the flow structure is three dimensional due to the triangular shape of the control structure. The average level of the sand bed is slightly below the vertex of the triangular crest and therefore the clearance of the crest increases with distance towards each end of the weir. The view upstream from the weir shows that the pattern of sand waves is on average quite uniform, further indicating that the effect of the triangular shape of the weir does not seem to have a significant effect on the sediment transport distribution in the appraoch channel when h_1/H_b = 2.24.

Figure 8 also shows the characteristic sand wave pattern. On the approach apron the sediment deposition is quite sparse compared to that observed for the lower flow in Figure 7. This is due to the

high velocities on the apron which ensured that the bed in the immediate vicinity of the weir was clear of significant sediment deposition. It is also clear from Figure 8 that the sediment deposition is slightly greater toward the centre of the approach apron. This indicates that for large discharges the three dimensional geometry of the weir has some effect on the flow distribution near the weir. Once again the average level of the sand bed is slightly below the vertex of the triangular crest indicating that for the BV weir the sediment builds up close to the crest regardless of the flow rate. This is not a good feature because it may affect the discharge coefficient.

4.1.2 The FV weir

Tests with the FV weir were conducted for values of h_1/H_b = 2.0 and 2.60. These values are slightly different than those used for the BV weir but represent approximately the same discharge. The sediment depositions after completion of the tests are shown in Figure 9 for the lower flow and Figure 10 for the higher flow.

Figure 9 again reveals the sand wave pattern and the even distribution of the deposition across the width of the channel. However, it is also quite clear that the average level of the bed does not encroach on the weir crest as much as in the case of the BV weir. The clearance at the vertex is greater and more of the weir is exposed near the walls of the flume. In view of this improvement it was felt that tests at the higher flow with the approach apron used for the BV weir would not be necessary. It was quite apparent that the FV weir would perform equally well or better. Instead, a set of tests was conducted with the approach apron removed, thus permitting free mobility of the sand bed in the immediate vicinity of the weir. The effect on the sediment bed is shown in Figure 10.

It can be seen in Figure 10 that the sand waves have again migrated right up to the weir where the sediment is then washed over the crest. The average surface of the sand bed at the centre of the

weir has been eroded to a level slightly below the elevation at which the approach apron had been placed in the previous tests. In the immediate vicinity of the weir deep scour holes have developed progressing from the highest point at the centre of the weir to a maximum at each end, indicating some influence of the three dimensional geometry of the weir. It is quite clear from these observations that at the high flow rates the FV weir will be free of sediment. In the approach channel the average bed level is approximately the same over the full width, showing that the effect of the weir on the upstream flow is limited to the immediate vicinity of the weir.

4.2 The Head-Discharge Relationship

In general, the discharge passing over a weir should depend on the geometric properties of the weir, the approach channel, the downstream channel and the fluid properties. Given that fluid properties are constant, then for a site-specific weir the variable properties are reflected in the head-discharge relationship. In order for the control structure to be effective it is important that the head-discharge relationship is stable in the presence of the moving bed load throughout the full operating range of the weir. Therefore, it is necessary to compare the head over the weir crest for the same flows with and without sediment at the same discharges.

Values of the measured head h_1, were plotted versus the corresponding discharge Q in Figure 11 for flows with sediment and for flows without sediment for the BV weir. In the case of the tests without sediment values of P/H_b were fixed at 1.2 The plot shows that there is no significant effect created by the presence of the moving sediment on the head-discharge curve. Similar data were plotted for the FV weir in Figure 12. The plot shows that the FV weir is also not significantly affected by the sediment in the approach channel and the bed load passing over the crest. This shows that for the conditions tested, the two weirs are similarly unaffected by the

sediment. Examination of the sediment deposition on the upstream side of the two weirs in Figures 7, 8, 9 and 10 show that the weirs were clear enough of sediment at the flows indicated to be equivalent to values of $P/H_b = 1.2$. It appears that for such conditions both weirs were equally unaffected by the presence of the sediment.

Further comparison of the two curves in Figures 11 and 12 shows that the FV weir will pass a given discharge at a lower head than that required with the BV weir. This feature of the FV weir is of great advantage in minimizing the back water effect upstream of the prototype weir installation. In addition, because the FV weir operates with lower heads it can be expected that it will have better sediment passing capability than the BV weir. Typical test conditions for high flows with non-submerged control in the sediment flume are shown in Figure 13 and 14 for the BV and FV weirs respectively.

4.3 Other Considerations

The tests have clearly shown that for the conditions tested, the FV weir is superior. The FV weir is better at passing the encroaching bed load, while at the same time keeping a stable head-discharge relationship and passing a given discharge at a lower head than that required by the BV weir. However, the tests were restricted to steady state flow conditions without suspended sediment. Such tests, even within the constraints imposed on the physical model scaling requirements, are very expensive and were therefore outside the scope of the present investigation. Nevertheless, some observations regarding the effects of suspended sediment on artificial controls such as the FV weir can be made. The flows in sand bed rivers do not always obtain their total sediment load through tractive bed erosion alone. At high flows, quite often large amounts of the suspended load originate from bank slumping, slides and sediments brought in from erosion in the drainage basin.

At such times suspended sediment concentrations can be extremely high. As long as the rate of flow of the river is great enough to carry such sediment loads, sediment build up at the FV weir should not be a problem. However, when high flows recede, the suspended load may drop out abruptly, depositing sediments in the approach channel of the weir at a rate faster than it can be removed as bedload. Such a rapid drop out of the suspended sediment is partially controlled by the suspended load particle concentration, grain-size distribution, clay content, particle to particle interaction and the availability of sediment (Lowe, 1988). At such times there may be an intermittent effect on the head-discharge relationship, depending on the excess of suspended sediment when hydrographs are in recession.

The FV weir is the best suited artificial control structure for these conditions. To reduce the deposition between the head measuring station (the gauge) and the weir, a structure that has a sloping upstream face should be used (Bos, 1976). The FV weir has such a feature and that is a major reason why it performs so much better than the BV weir. However, the uncertainty regarding the removability of deposited suspended sediment is still a problem. In order to better understand the relationship between the weir and the bed levels immediately upstream of the weir, further tests should be conducted. These tests should reveal the change in the average level of a sand bed having an initial elevation equal to that of the lowest point on the crest of the FV weir. The tests should be repeated for different flows over the weir, each time beginning with the same bed elevation. Such tests should reveal if sediments deposited during the recession of high flows will affect the function of the weir during lower flows.

5.0 CONCLUSIONS AND RECOMMENDATIONS

1. Tests were conducted on two weir models in the sediment transport flume constructed especially for these tests. It was not possible to obtain complete similitude for the weirs and sediment transport simultaneously and therefore a limited series of tests was conducted to make relative comparisons between a triangular broad crested (BV) weir and a triangular short crested (FV) weir.
2. The tests have shown that the FV weir has better sediment clearing capabilities than the BV weir. Examination of head-discharge plots have shown that, for the flow conditions tested, both the BV weir and FV weir were relatively unaffected by sediment deposition.
3. The tests have shown that a given discharge can be passed by the FV weir with a lower head than that required with the BV weir. This feature is of great advantage in minimizing the backwater effect upstream of the prototype weir installation.
4. It is recommended that the FV weir be considered for further testing as outlined in section 4.3.

ACKNOWLEDGEMENTS

The writer wishes to thank Doug Doede for his assistance in conducting the required experiments.

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TABLE 1

Significant Operational Characteristics of Test Weirs

Structure	$\frac{Q_{max}}{Q_{min}}$	H_2/H_1	Accuracy E%	Debris Passing Ability	Sediment Passing Ability
BV weir	830	0.8	E > 10% for low flows E > 5% for high flows	good	fair
FV weir	17,500	0.7	"	very good	good

H_1 = the total energy head on the upstream side of the weir

H_2 = the total energy head on the downstream side of the weir

Q = the discharge passing over the weir

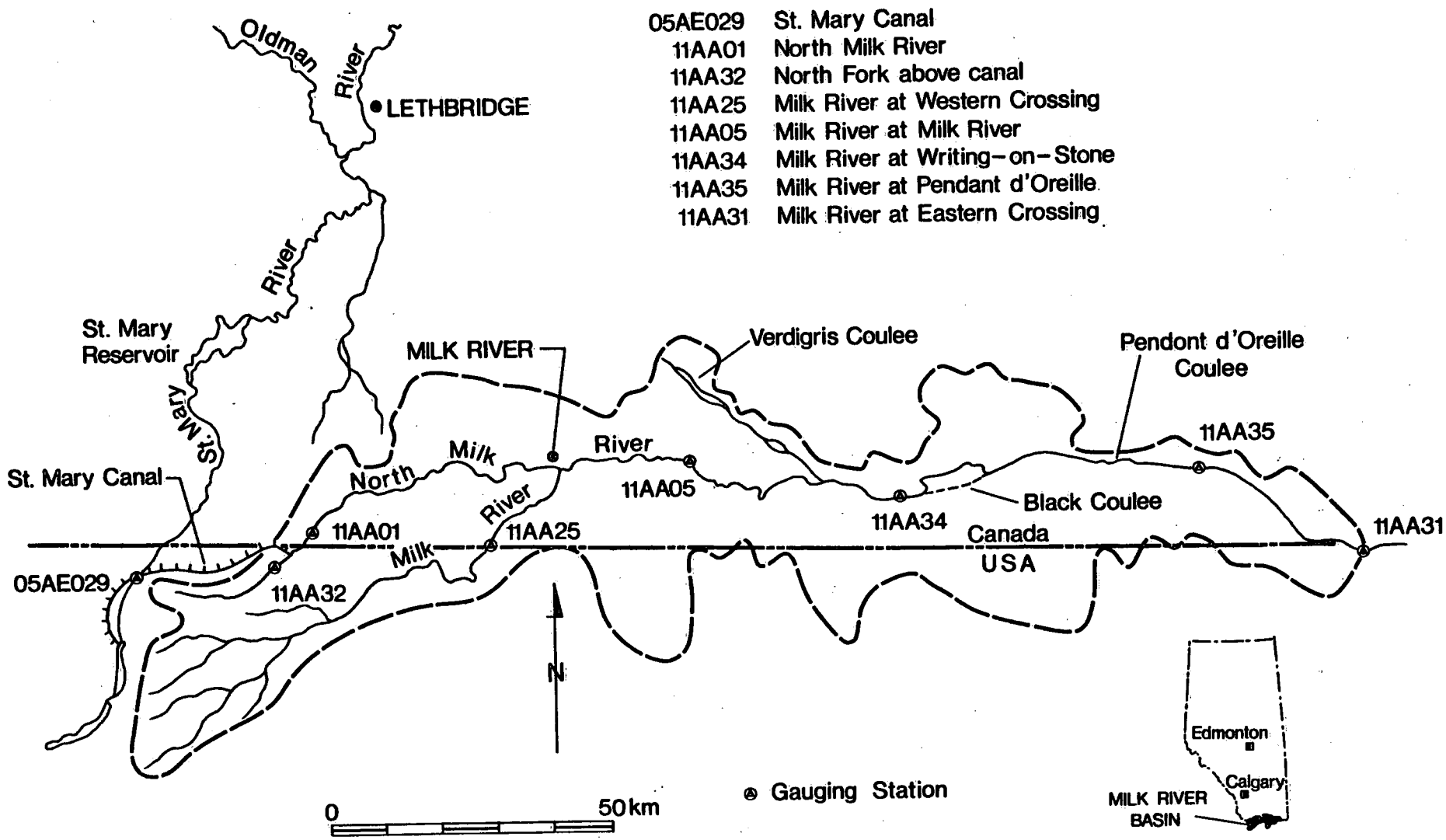


FIGURE 1. Milk River basin (from MacLean & Beckstead 1981)

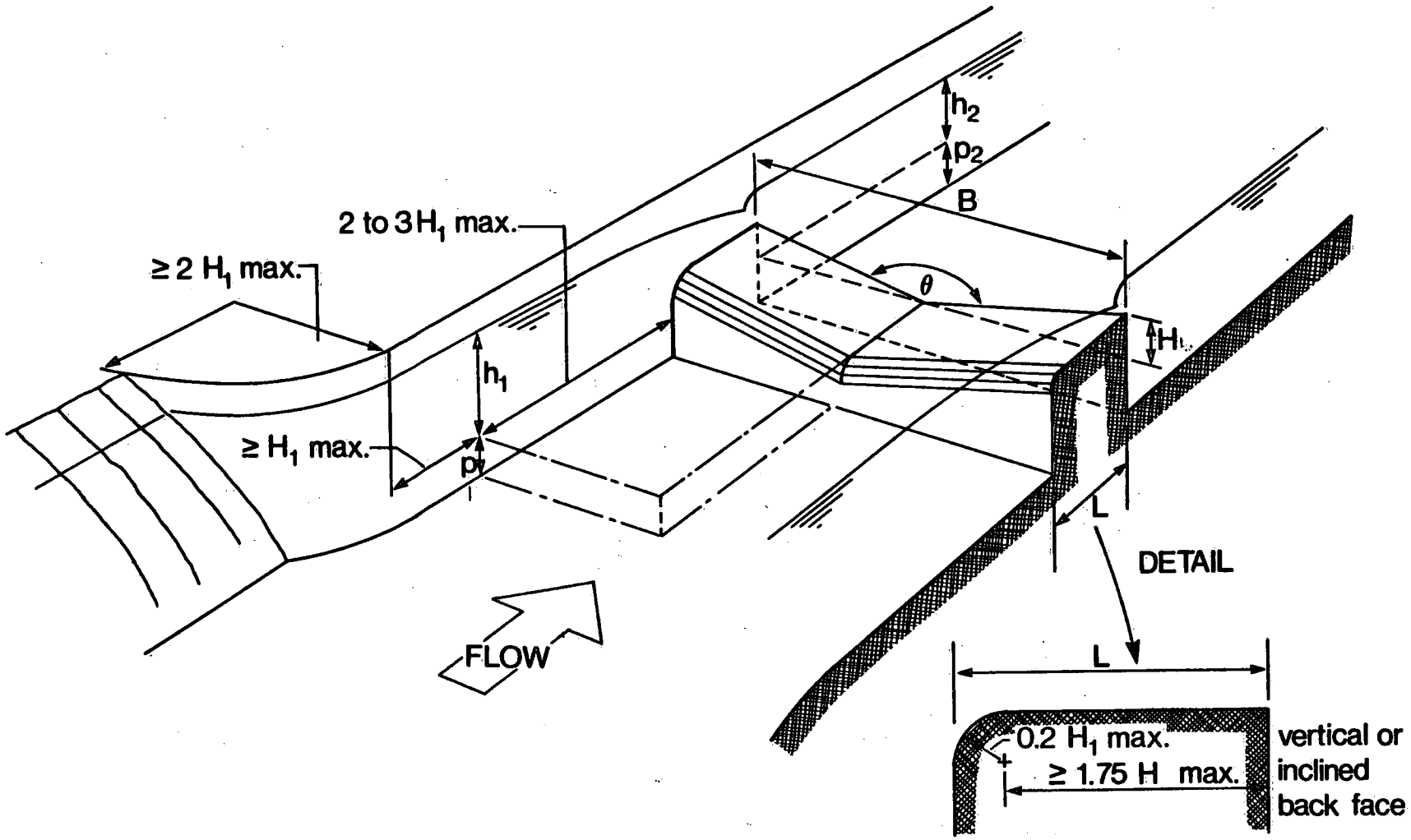


FIGURE 2. Definition sketch for triangular broad-crested weir.

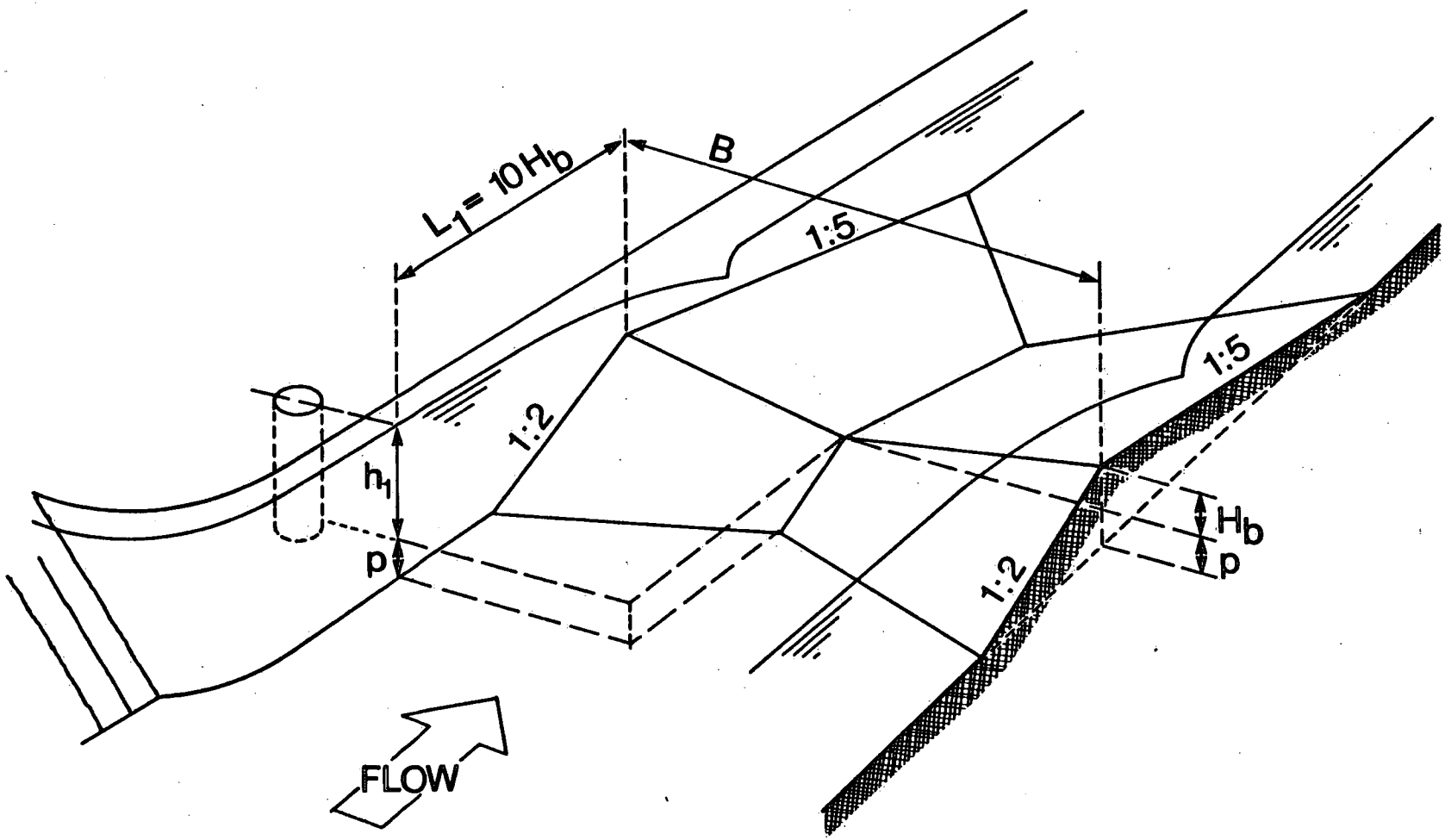


FIGURE 3. Triangular profile flat – V weir.

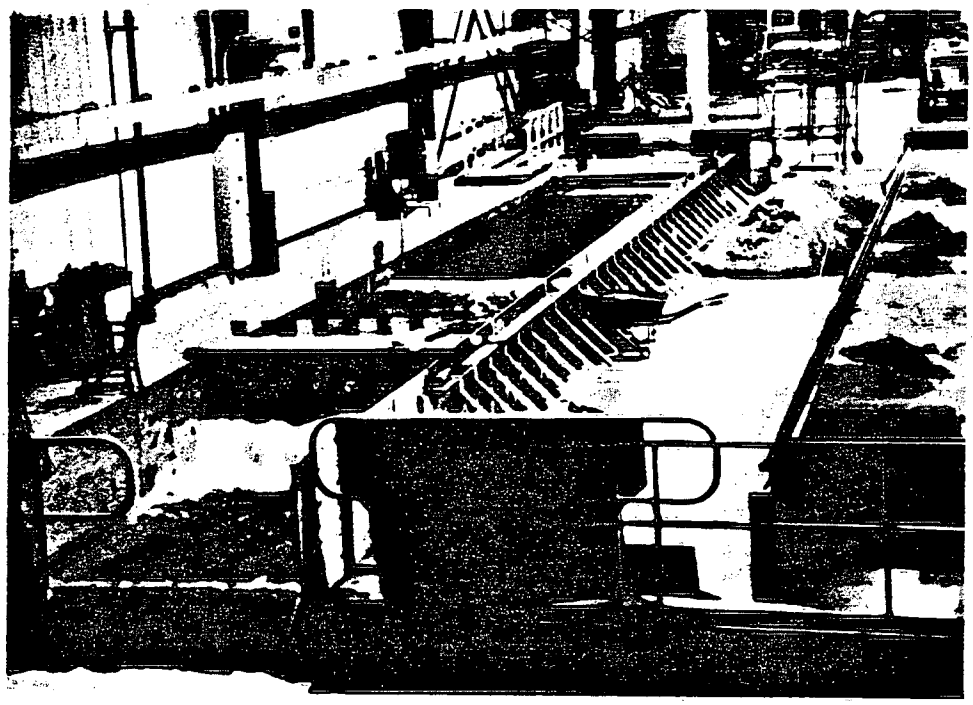


FIGURE 4. MODELLING FLUME

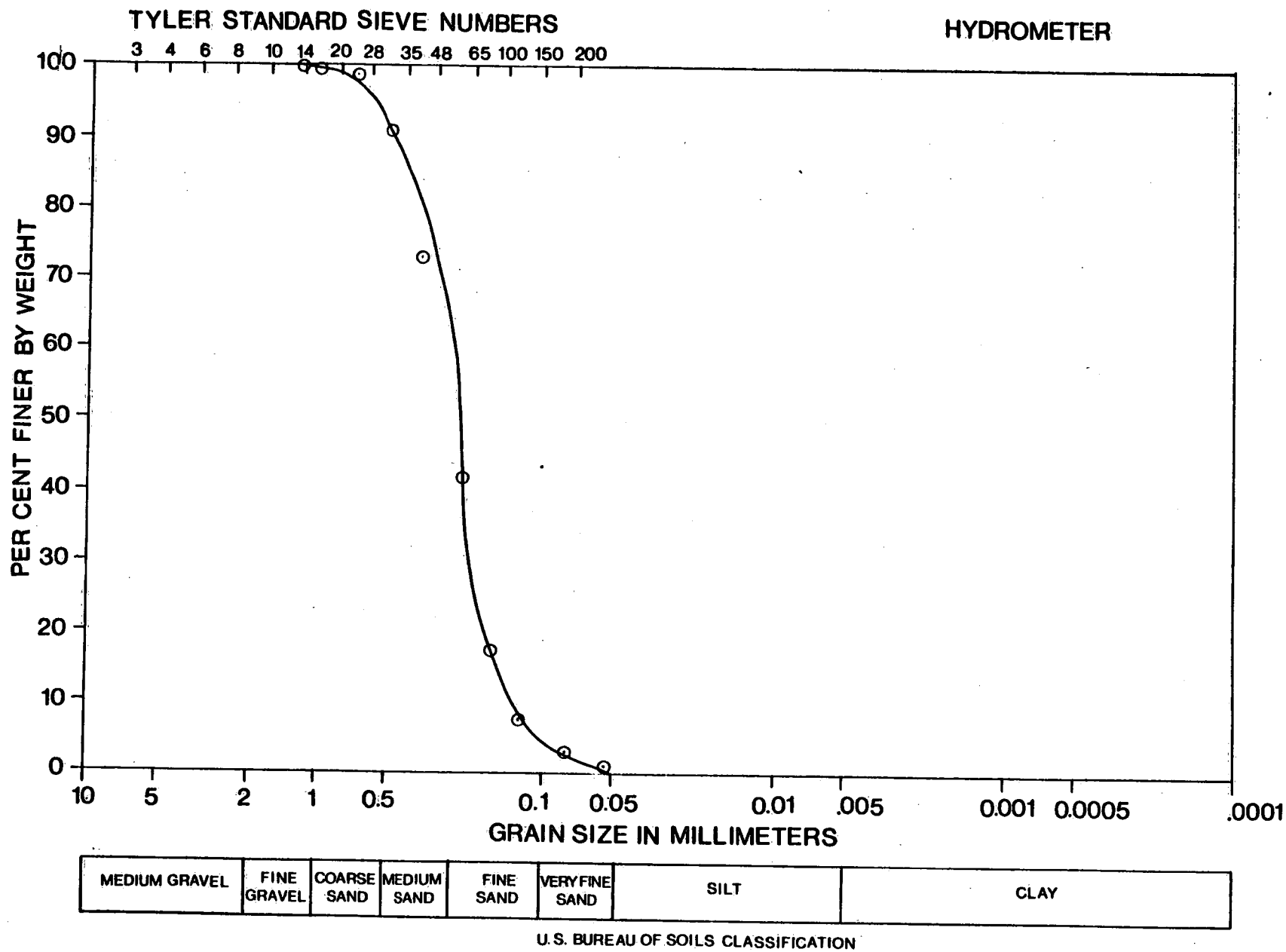


FIGURE 5. Grainsize Distribution for Test Sand

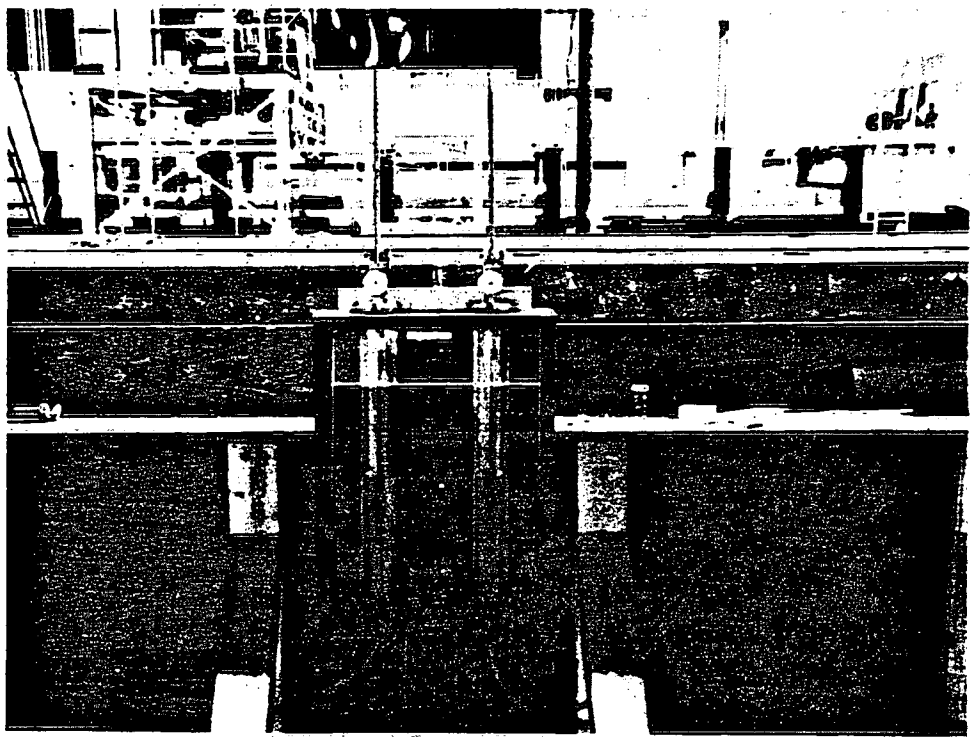
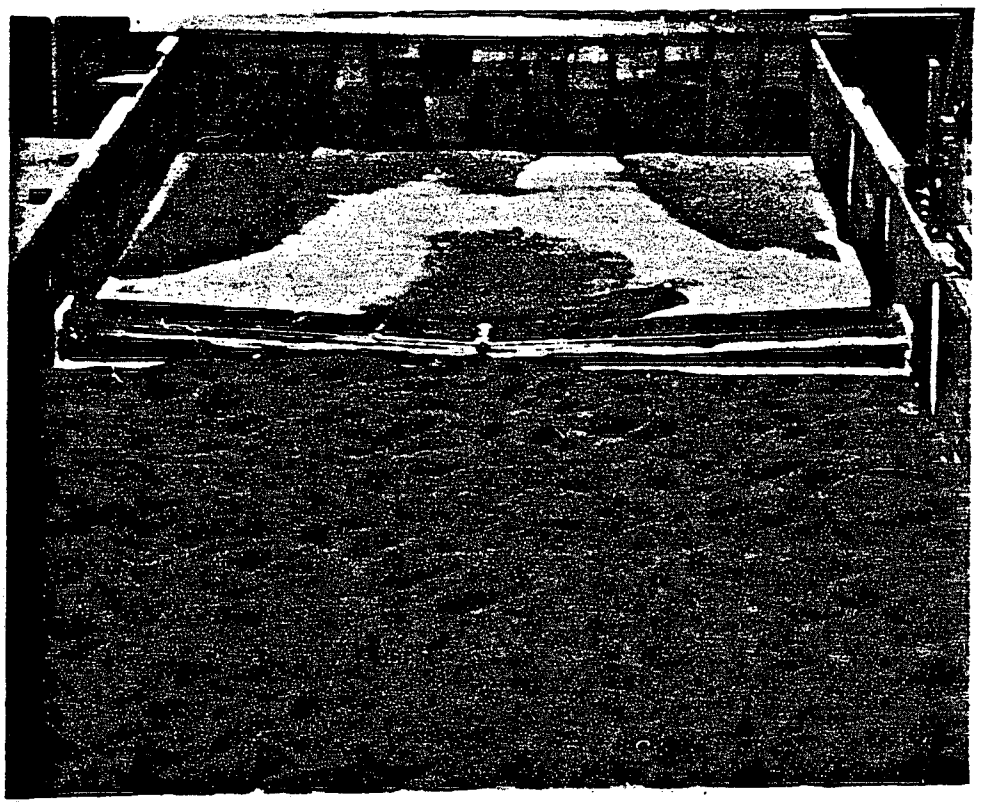
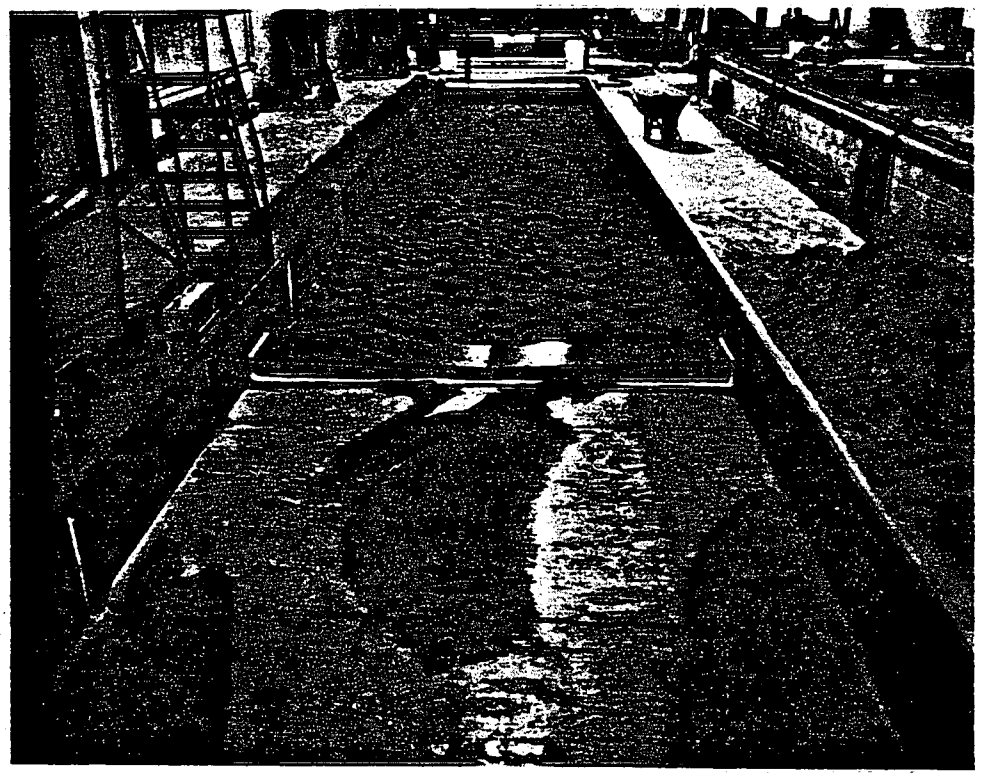


FIGURE 6. STILLING WELLS FOR WEIR MODEL

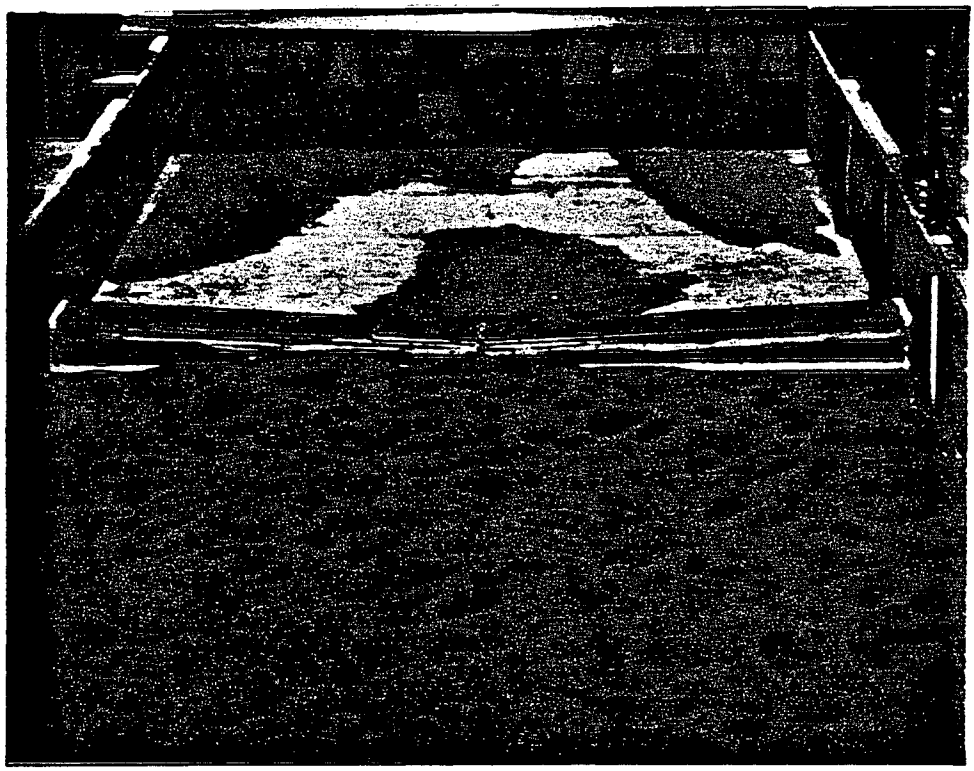


(a) SANDBED AT THE UPSTREAM SIDE OF WEIR

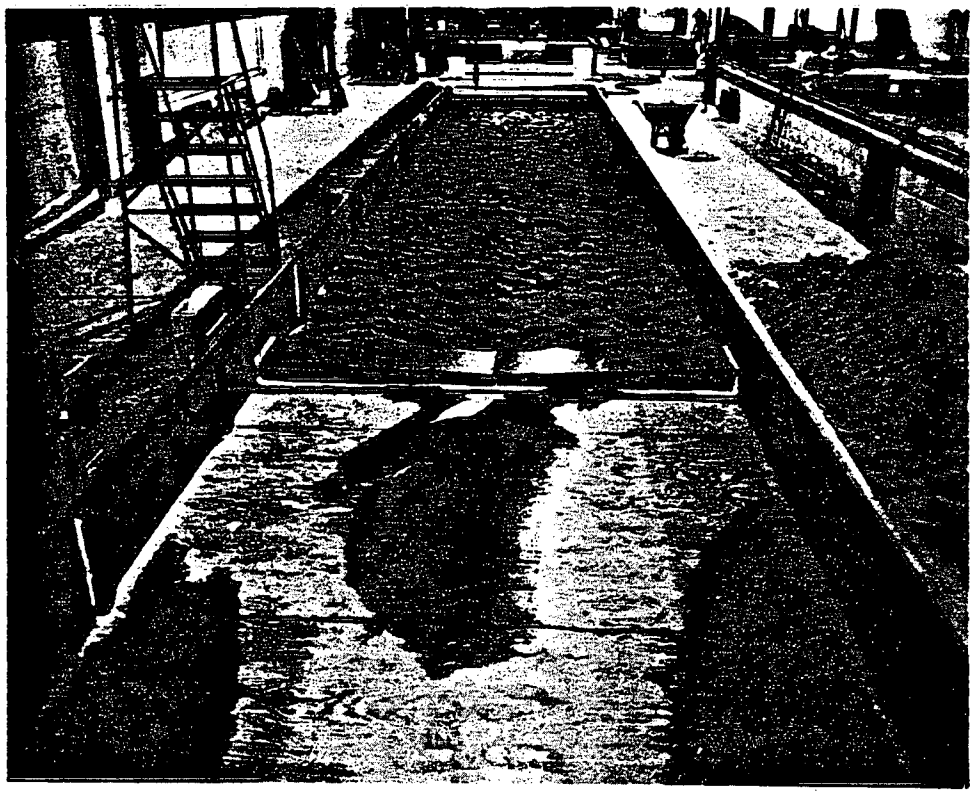


(b) APPROACH CHANNEL LOOKING UPSTREAM

FIGURE 7. THE BV WEIR AFTER TESTS FOR $h_1/H_b = 2.24$

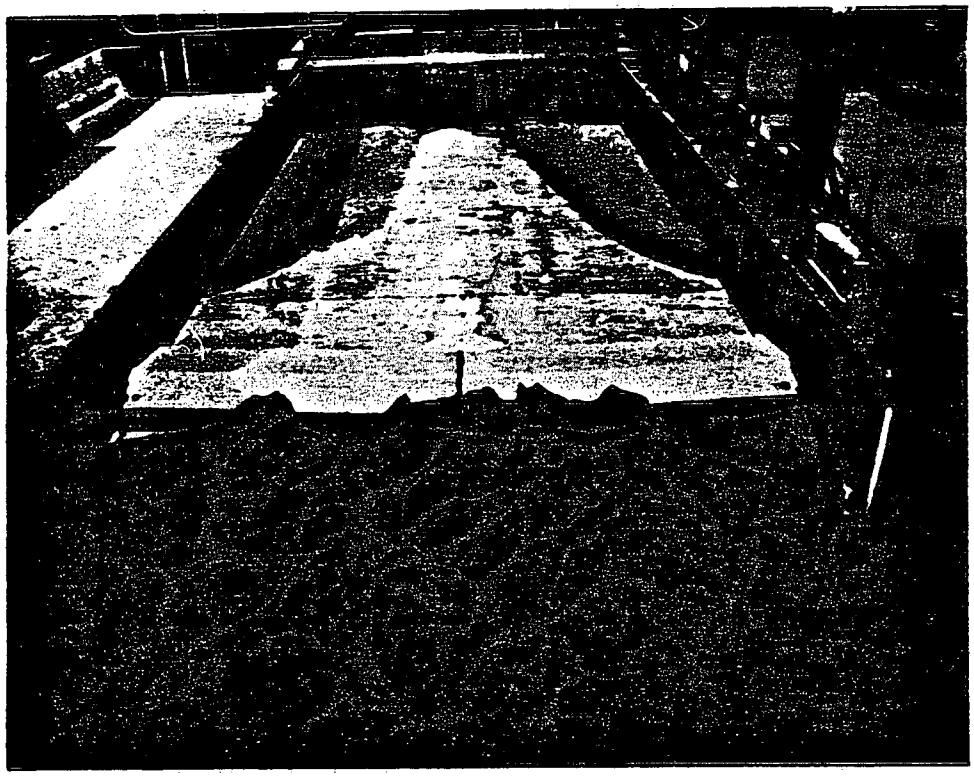


(a) SANDBED ON UPSTREAM SIDE OF WEIR

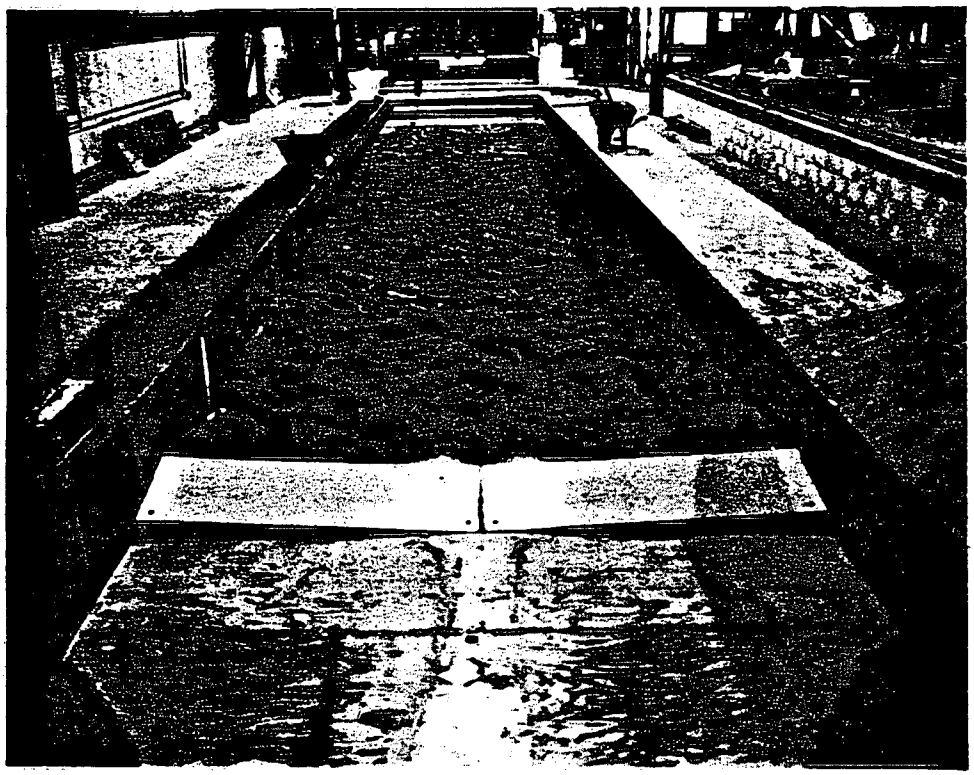


(b) APPROACH CHANNEL LOOKING UPSTREAM

FIGURE 8. THE BV WEIR AFTER TESTS FOR $h_1/H_b = 2.90$

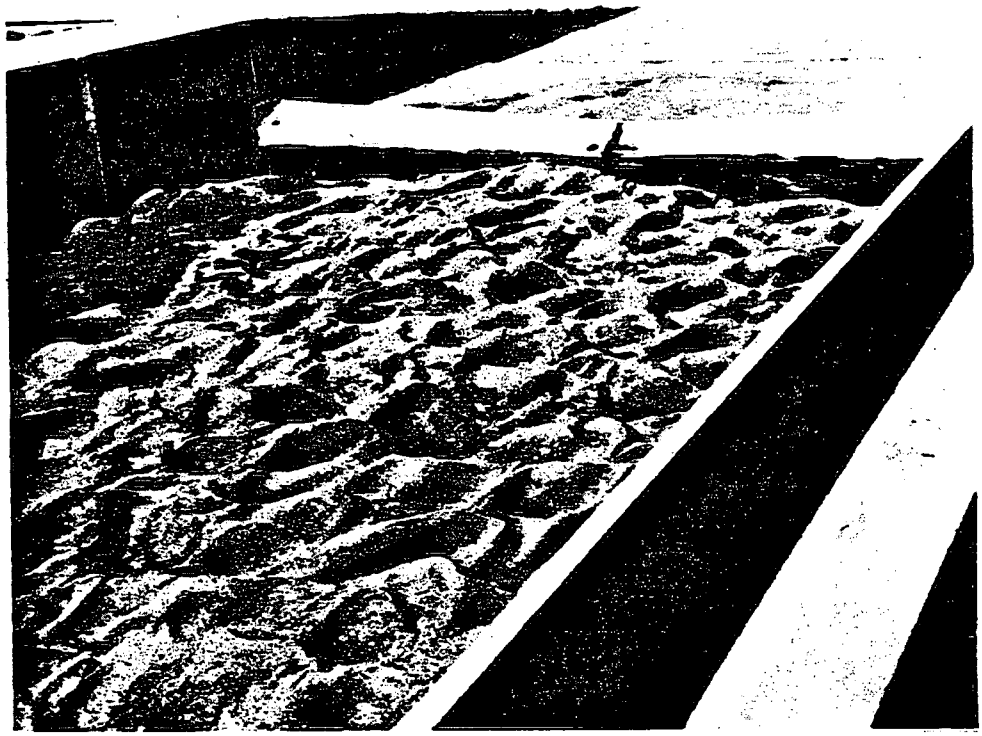


(a) SANDBED ON UPSTREAM SIDE OF WEIR



(b) APPROACH CHANNEL LOOKING UPSTREAM

FIGURE 9. THE FV WEIR AFTER TEST FOR $h_1/H_b = 2.0$



(a) SANDBED ON UPSTREAM SIDE OF WEIR

(b) APPROACH CHANNEL LOOKING UPSTREAM

FIGURE 10. THE FV WEIR AFTER TESTS FOR $h_1/H_b = 2.60$

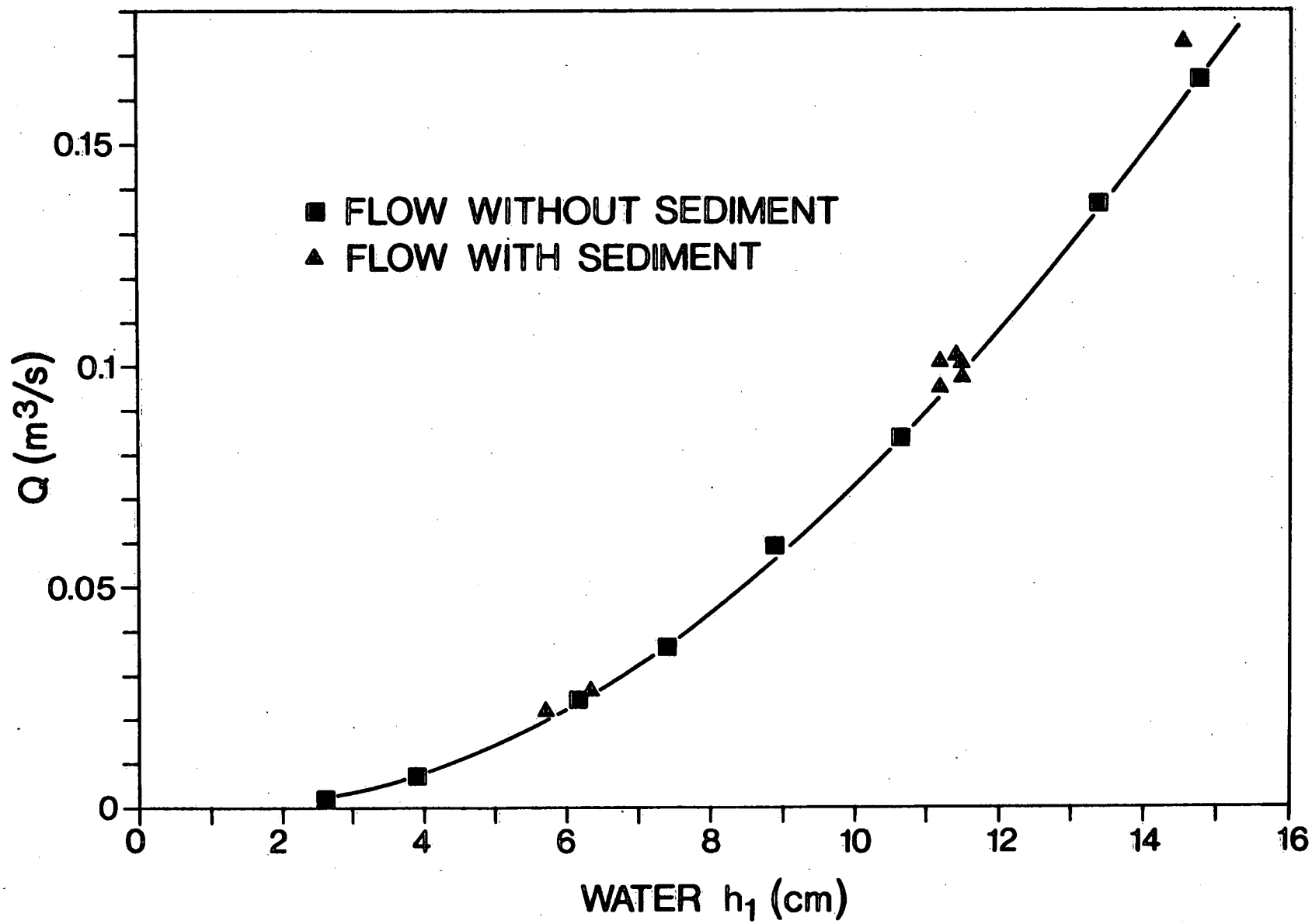


FIGURE 11. STAGE-DISCHARGE CURVE FOR BV WEIR

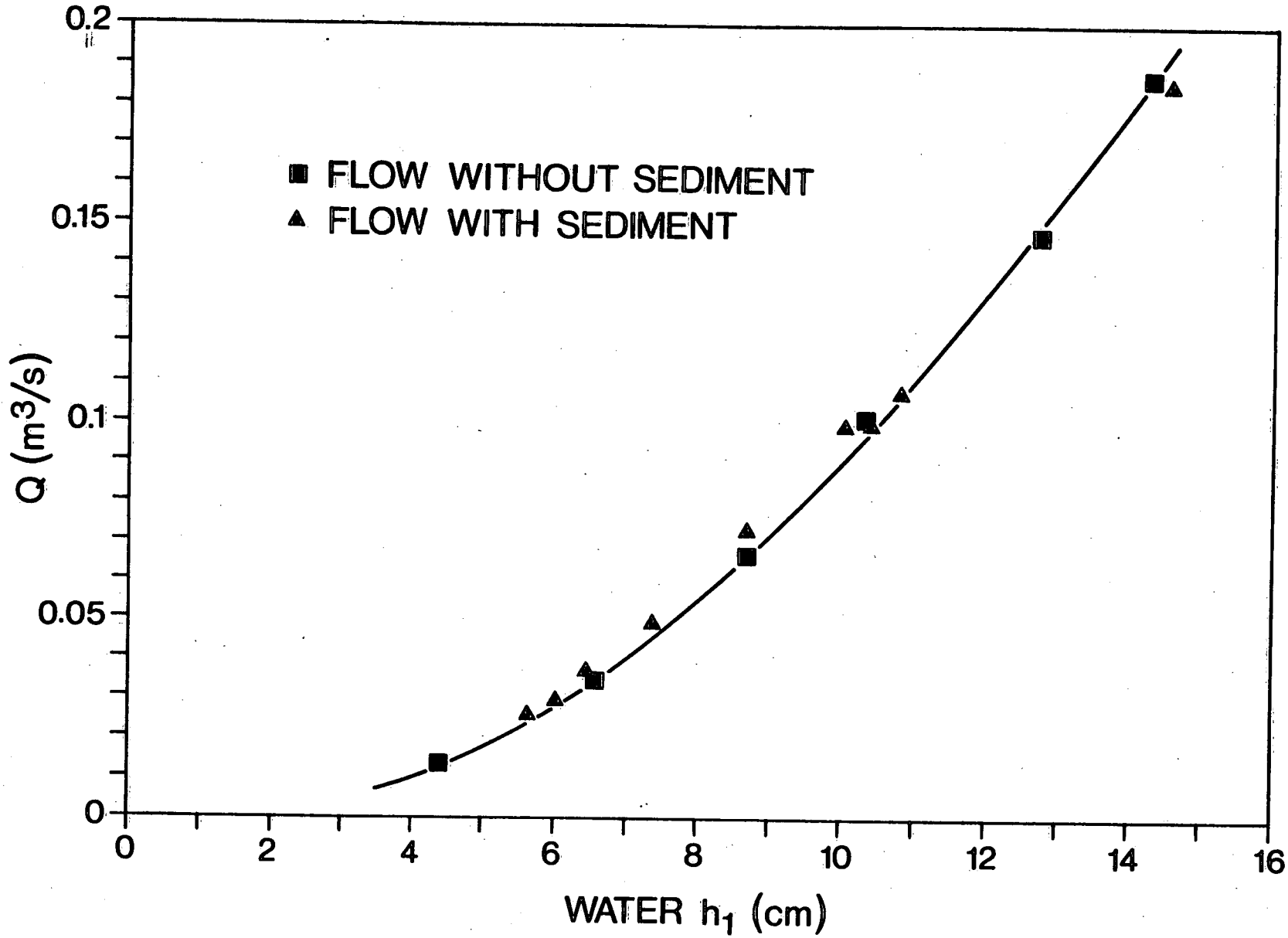


FIGURE 12. STAGE-DISCHARGE CURVE FOR FV WEIR

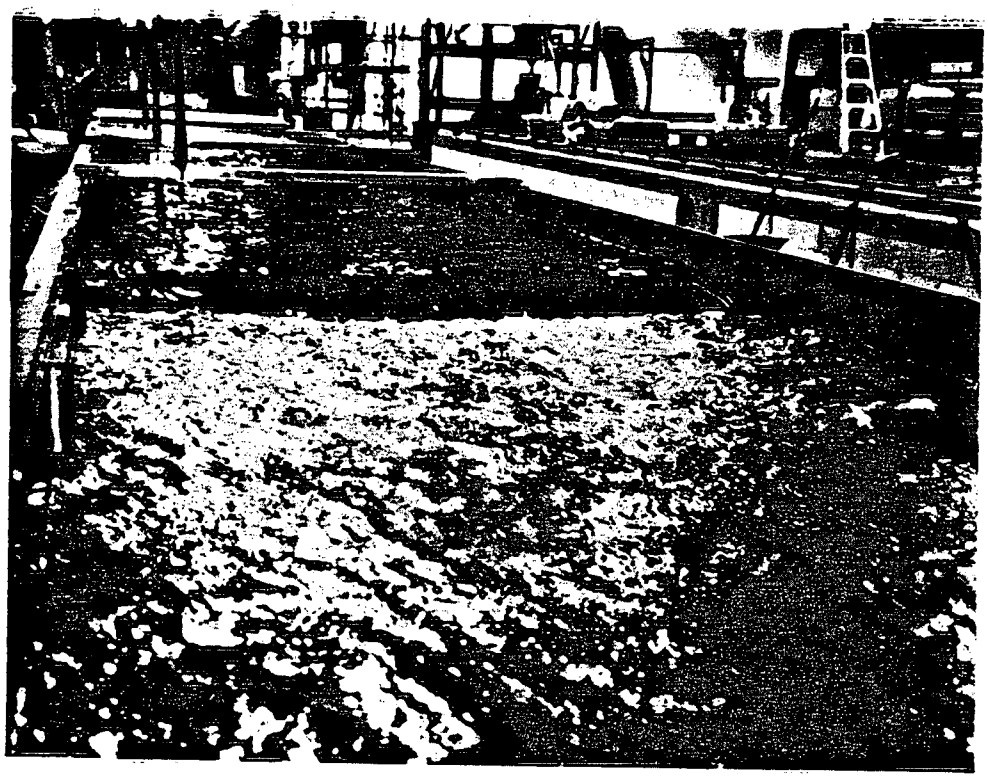


FIGURE 13. FLOW OVER BV WEIR FOR $h_1/H_b = 2.90$

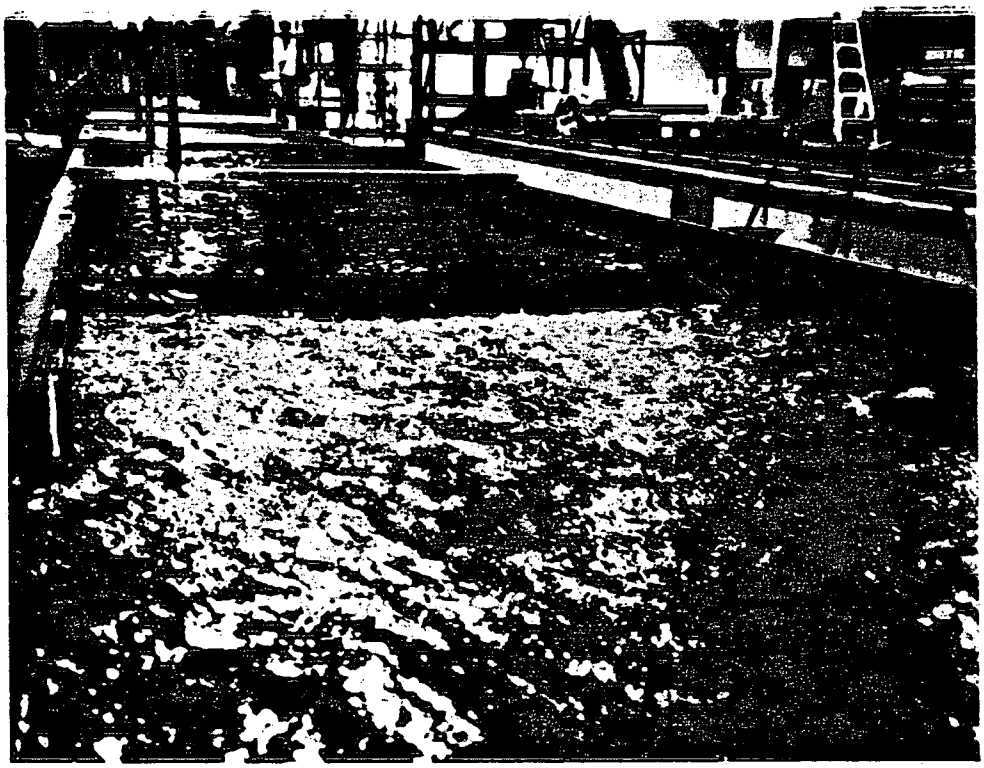


FIGURE 13. FLOW OVER BV WEIR FOR $h_1/H_b = 2.90$