"the reproduction and observation of sediment movement in a model is the most important single factor in a particular model study."

Sediment Control In Irrigation Canal Intakes*

by

Clarito C. Gonzales**

Abstract

The control of sediments at irrigation canal intakes is a universal problem in the design of irrigation diversion systems. To facilitate the solution of the problem, the sediment load is first predicted using one of the numerous analytical equations. From the computed estimate, a preliminary design of the sediment control scheme is made. However, this initial design is not necessarily the optimum scheme. In this regard, model studies serve as an aid in the evaluation of the sediment control scheme. Several model studies conducted and presently being conducted at the National Hydraulic Research Center are thus discussed to illustrate the evaluation procedure involved.

Introduction

The location of arable land and the location of a suitable source of irrigation water are seldom in proper juxtaposition. In this regard it is necessary to conceptualize the delivery of irrigation water by means of man-made conveyance structures. The design of these structures should be such that the sediment of the water is reduced to the minimum allowable possible. This is to prevent the deposition of sediments on the conveyance system which may result in the shortening of its service life, and on the irrigation fields which in turn may result in the reduction of the productivity of the soil.

Sediment control may be achieved at the watershed area which comprises all the land and water surfaces within the confines of the drainage divide. Watershed management includes land treatment measures, flood detention reservoirs, stream channel improvement and stabilization, and debris basins. These sediment control schemes are concentrated on the areas at a considerable distance from the canal intake. Hence, they will not be included in the discussion.

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At the vicinity of the canal intake, the sediment load is first estimated by means of stream sampling or by using one of the numerous sediment discharge equations. Notable among these equations are those of H.A. Einstein for bed load and suspended load, respectively, as well as those of Meyer-Peter for bed load. These equations will be discussed in the next section.

After the estimate of the sediment discharge is made, the preliminary design of the sediment control scheme is prepared. This may be done by the proper location of the point of diversion and selection of the angle diversion, and by the use of training structures, pocket and divider walls, sand screens, guide vanes, and tunnel-type sediment diverters. Such sediment control schemes are described in this study.

Regardless of the type of sediment control scheme used at the canal headworks, some sediment load will enter the canal. This load is removed by the use of tunnel type sediment ejectors, vortex tube ejectors, and settling basins. These control methods will be discussed accordingly.

The optimum design of a sediment control scheme at the intake may be arrived at with the aid of model studies. This will be illustrated by the model tests conducted at the National Hydraulic Research Center. Discussions of the model studies will be presented.

An Overview of the Transport of Sediments

List of symbols

τ	mean tractive force
τ_c	critical tractive force
γ	specific weight of water
R_H	hydraulic radius
S	slope of the channel bed
K	friction coefficient
G	specific gravity of solid sediment particles
Ø	particle diameter
N'	Manning's coefficient for a plane bed
N	actual value of Manning's coefficient for a rippled bed
λ_s	specific weight of solid sediment particles
g	gravitational acceleration

gs	bed load discharge per unit width
R’	hydraulic mean depth of unrippled channel
R_b	hydraulic mean depth of bed with frictionless sides
g’s	suspended load discharge
V_*	shear velocity equal to gRHS
C_a	known suspended sediment concentration at a known depth
Δ	apparent roughness of the bed surface
δ	thickness of boundary layer
ν	kinematic viscosity
K_s	mean diameter of bed material

Sediment transport depends on the physical properties of the sediment and the hydraulic characteristics of the flow. Among the physical properties, the weight, shape, and terminal velocity of the sediment particles directly govern particle movement. These properties can be expressed in terms of the size of the sediment particle of which the mean diameter is the most commonly used parameter.

Where cohesion is not present, the movement of material on the banks and on the bed of a channel is dependent on the velocity and turbulence near the banks and the bed, and on the steepness of side slopes. For uniform flow the mean tractive force per unit area, which is exerted on the channel bed in the flow direction is given by

$$\tau = \gamma R_H S$$

In a wide open channel, the hydraulic radius is taken equal to depth of flow d such that the equation for the tractive force becomes

$$\tau = \gamma d S$$

When the tractive force is greater than the frictional resistance between sand particles, the latter is set in motion. The resistance of the sediment to motion is proportional to the particle diameter and to the submerged weight of the sediment in water. Accordingly, the equation of the critical tractive force is

$$\tau_c = K\ (G-1)\ \emptyset, \text{ K: friction coefficient}$$

An alternate equation was given by E.W. Lane and is given below

$$\tau_c \ (\mathrm{Kg/m^2}) = 0.078 \ \emptyset, \quad \emptyset \text{ in mm units}$$

Values of the critical tractive force for common soil types are given in Table 1.

The sediment load is classified in two categories namely the bed load and the suspended load. These loads are estimated by either stream sampling or by analytical methods.

Bed load is the material in the bottom layer of the flow and consists mainly of the coarser particles. Motion of these particles is one of rolling, sliding, and jumping which is termed saltation. Although the amount of bed load is small comparatively to that of suspended load, it is important because it shapes the bed and influences the stability of the channel, the grain roughness, the form of bed roughness, and other factors.

Numerous bed load equations relating bed load discharge, flow conditions, and bed material composition, have been proposed. The two more widely used equations are those given by Meyer-Peter and by H.A. Einstein.

The bed load formula of Meyer-Peter is as follows:

$$\left(\frac{N'}{N}\right)^{3/2} \gamma S_d = 0.047 \ (\gamma_s - \gamma)(\emptyset) + 0.25 \frac{\gamma^{1/3}}{g} \ (g_s)^{2/3}$$

The bed load formula of Einstein is as follows:

$$\Theta = f\,(\psi),$$

where

$$\Theta = \frac{g_s}{\gamma_s} \left(\frac{1}{s-1}\right)^{1/2} \left(\frac{1}{g\emptyset^3}\right)^{1/2}$$

$$\psi = (\gamma\text{-}1) \frac{\emptyset}{R's}$$

$$R' = R_b \left(\frac{N'}{N}\right)^{3/2}$$

Suspended load usually consists of the finer particles with turbulence as the important factor in its suspension. The suspended load formulas are numerous but the equation by H.A. Einstein is one of the most widely used. Accordingly, the Einstein equation for suspended load is as follows:

$$g's = 11.6 \ V_* \ Ca \ a \left[2.303 \ \log \left(\frac{30.2\emptyset}{\Delta}\right) I_1 + I_2\right]$$

where I_1 and I_2 are functions given by Einstein, some values of which are given in Table 2. Δ is the apparent roughness of the surface, the value of which is obtained from Table 3 for values of the corrective factor x and of the ratio $\frac{K_s}{\delta}$. δ is the thickness of the boundary layer and is given as

$$\delta = \frac{11.6}{V_*}$$

Values of x for typical values of $\frac{k_s}{\delta}$ are given in Table 4.

Sediment Control Methods at the Canal Intake

Sediment Control Methods Upstream of the Canal Intake

The control of sediments at the canal intake is first approached by the proper location of the point of diversion. Careful selection of the point where the water is to be diverted from a stream is an important factor in the reduction of the quantity of sediment taken into the canal. In general, the concave side of the stream curve has proven to be the best location. The sediment concentration at this point is lower than at other points in the stream. As the flow passes the curved channel, spiral flow is induced as shown in Fig. 1. This helicoidal flow pattern sweeps the bed load to the inside of the curve, and is sometimes referred to as the principle of bed load sweep.

After the point of diversion is located, the angle of diversion is selected. The angle of diversion is the angle of deflection between the direction of flow in the parent channel and the direction of flow in the diversion channel (Fig. 2). From the discussion on the location of the point of diversion, it is easily seen that any diversion at an angle with the flow in the parent stream channel becomes in effect a curve with the curvature opposite to that of the parent channel. The higher velocity surface water requires a greater force for flow diversion than the lower velocity water near the bed. As a consequence, the surface water tends, by virtue of its higher momentum, to continue with the parent stream. The slower moving water near the bed, carrying the greater concentration of sediment, tends to flow into the diversion channel.

It was found that the optimum angle of diversion varies with the diversion ratio, i.e., the ratio of the diversion discharge to that of the stream discharge. The optimum diversion angle increases as the diversion ratio decreases. By model study the optimum diversion angle corresponding to the dominant diversion ratio is selected.

A sediment diverter is a device or structure arrangement at the canal headworks. It is designed to prevent the greater part of the stream sediment from entering the canal. The use of training walls and guide banks is an example of this sediment control device. Training walls create an artificial curve in the flow with the canal intake located at the outside of the curve. The resulting helicoidal currents sweep the bed load to the inside of the curve and consequently away from the headgates.

With an elevated intake sill, the flow velocity sweeps the bed load at an elevation below that of the sill and out through the sluiceway as shown in Fig. 3. Guide banks (Fig.4) are artificial curved banks with or without straight portions that are designed to perform the same function as the training walls, i.e., to divert sediment away from the canal intake.

The structure arrangement to reduce the amount of sediment taken into the canal headgate is the divider wall which is located immediately upstream of a dam (Fig. 5). This wall induces the formation of a pocket in front of the canal intake. The wall divides the streamflow upstream of the dam such that part of the flow is directed to the sluiceway and part through the dam. A ponding area of 1 low velocity is thus produced in which the sediment will deposit. This deposited material is subsequently scoured out through the sluiceway.

Sediment sluicing can be accomplished by intermittent sluicing called the still pond method, or by continuous sluicing if an adequate amount of streamflow is available. For the still pond method it is advantageous to close the canal headgates during the sluicing procedure because the induced turbulent flow conditions put the sediments into suspension. Where the diversion dam is gated, the amount of opening of each gate can affect the sediment movement into the canal headworks. The gates farthest from the diversion point are opened the greatest amount and the gates nearest the pocket are opened the least amount. Thus, the increased discharge through the far gates creates an artificial curved flow which pulls the sediment away from the diversion intake.

San screens or skimming weirs (Fig. 6) are low barricade walls which allow the overflow of relatively sediment-free water into the canal diversion headworks. To adapt the walls to changes in stage of the streamflow, flashboards are often provided on top of the walls. The efficiency of the sand screen is greatest when the sediment are transported as bed load. For the walls to be effective, there must be sufficient flow past the diversion point to carry the bed load that tends to deposit at the upstream face of the walls. Otherwise the sediment deposits will form ramps which deflects the stream current up and over the walls resulting in the inflow of sediments into the canal headworks.

Guide vanes are sediment control structures which produce localized helicoidal flow patterns similar to those generated in the flow around a curve discussed earlier. These vanes are classified either as bottom guide vanes or surface guide vanes. Bottom guide vanes direct the flow in the lower part of the stream prism away from the canal headgates. Since most of the heavy bed load is concentrated in the lower stream prism, the bottom vanes also deflect the bed load away from the intake. Surface vanes are generally supported by a raft arrangement from which the vanes project into the water far enough to influence the flow direction of the surface water. Typical guide vanes are shown in Fig. 7.

Tunnel type sediment diverters are basically composed of an upper and a lower chamber through which the water for diversion is routed (Fig. 8). The clearer water flows through the upper chamber into the canal, while the sediment-laden water flows into the lower chamber and is passed back into the stream. The openings or entrances for water into the tunnel diverters are limited at the upstream end of the tunnel although some attempts have been made to provide entrances for water along the tunnel sides. Pressure flow should be maintained in the tunnel during the design discharge.

Sediment Control Methods Downstream of the Canal Intake

Sediment ejectors employ the same general principles of sediment removal as sediment diverters except that they are designed to remove sediments from the canal flow and are located downstream of the canal headgates. Diversion of a major part of the bed load back into the stream is all that can be expected of a diverter of any type. Thus, some material, often in suspension due to eddies formed upstream of the headgate, will eventually deposit in the canal prism.

A tunnel-type ejector (Fig. 9) is similar to the tunnel type diverter in principle and operation. The height of the tunnel at its entrance is generally 25 percent of the canal depth of flow; the roof usually extends upstream of the entrance to guide the sediments that are lifted as a result of eddy formation at the entrance. The tunnels lead to the side of the canal and through the bank to the outfall channel back to the stream.

Sediments entering the canal are also removed by vortex tube ejector. This type of ejector consists of an open-top tube or channel with a cross-section as shown in Fig. 10. The tube is placed at an angle to the canal flow direction (usually 30° or greater), with the edge at the same elevation as the bottom grade of the canal. Bed load is picked up by the spiral flow in the tube and is carried along the tube towards the bank and to the outlet section back into the stream.

The removal of suspended small particles is facilitated by a settling basin which is placed downstream of the headworks (Fig. 11). This sediment control scheme consists of an oversized section of the canal which results in the reduction of the flow velocity, and consequently in the settling of the suspended particles. Accumulation of sediments in the basin is prevented by mechanical removal of the deposits.

The Use of Hydraulic Models in the Design Evaluation of Sediment Control Schemes

The most reliable way to evaluate a diversion structure design which includes a device for the control of sediment entry into the canal is by model study. Known principles of the physical movement of sediments at the vicinity of the canal diversion are used as the basis of model studies. The reproduction and observation of sediment movement in a model is the most important single factor in a particular model study. This model should be geometrically and dynamically similar to the proposed prototype structure in order for the model to give accurate information about the hydraulic design of the sediment control scheme under model study. Because of the inherent difficulties in modelling, model studies become a meams of qualitative or comparative evaluation of the different sediment control schemes in the proposed diversion project.

At the National Hydraulic Research Center, several irrigation projects have been modelled for the hydraulic evaluation of the various sediment control schemes of the proposed structures. Among these projects are the Sto. Tomas Diversion Dam, the Banga River Irrigation Project, the Allah River Irrigation Project (Surallah Dam and Norala Dam), and the Mag-Asawang Tubig Irrigation Project.

For the Sto. Tomas Diversion, the model has a hydraulic scale of 1:15 and consists of the ogee dam, the sluiceways, the diversion structure, the sediment exclusion device, and the movable bed with fixed banks. (See Fig. 12). The model study delved into the performance of the four sediment exclusion schemes.

The first scheme involves a tunnel-type sediment diverter which consists of eight entrance bays leading to four barrels downstream. Fig. 8 shows the diverter as proposed for the model. Test runs on the model showed that the average ratio of sediment sluiced to that entering the canal has a value of 1.35. The relatively low ratio is attributed to the eddy formation at the tunnel entrance. The eddies caused the sediment to be in suspension thus facilitating the entrance of sediments into the canal headworks.

Four bottom guide vanes were tested in the model corresponding to the second sediment control scheme. The arrangement of the guide vanes in the model is shown in Fig. 13. Model tests revealed that the proposed guide vanes are inefficient in the sluicing of sediment deposits at the downstream side of each vane. Secondary currents produced by the vanes resulted in the deposition which are eventually carried by the flow to the headworks.

As shown in Fig. 14, the third scheme consists of a tunnel-type diverter. For this scheme, the sediment ratio was increased to 2.60 which led to additional tests to determine qualitatively the sluicing efficiency of this scheme when sediment accumulation has taken place in the tunnels. Because of the low velocity in the tunnel which resulted from the large flow area, a considerable amount of sediment deposits was not sluiced.

From the third scheme, it was observed that the sluicing problem may be solved by reducing the flow area and by introducing a bell-mouthed entrance and slits at the upstream end of the tunnel. The area reduction was done by introducing barrels with bell-mouthed entrances. The slits were introduced to sluice the sediments at the vicinity of the slits. (See Fig. 15). Tests on this scheme resulted with the same sediment ratio as the third scheme. Additional tests on the model showed a comparatively effective sluicing capacity. This is the result of the relatively high velocities in each of the three tunnels. With the satisfactory performance of this scheme, it was recommended for adoption in the design of the prototype.

Further qualitative tests were conducted on the recommended scheme in order to determine operating procedures for the prototype. These include the sluicing procedures which are the intermittent type and the continuous type. For intermittent sluicing the diversion headgates should be closed and the sluicegates fully opened. When the river discharge exceeds the irrigation requirements, continuous sluicing should be employed. The pool elevation in the reservoir should be maintained at an elevation just enough to give the required head for the canal discharge.

The Banga River Irrigation System used a settiing basin for the control of sediment flow into the canal system. The model study for this project considered the sediment deposition pattern as well as the flow pattern at the main canal intake, in the basin proper, and in the return channel. A 1:10 scale hydraulic model was contructed and tested for the preliminary design and alternative schemes for the basin.

Fig. 16 shows the preliminary design which has proven unsatisfactory. An asymmetrical velocity distribution in the barrel structure resulted in an asymmetrical deposition in that area. Within the basin proper, a bar formation occurred extending from the right bank warping transition diagonally to the left bank. This deposition pattern is the result of the sudden change of flow direction from the main canal into the basin proper. Such deposition pattern may lower the discharge capacity of the basin. Likewise, the bar formation may create backwater effects at the barrel structure which will worsen the sediment deposition therein.

To improve the deposition pattern resulting from the preliminary design, several modifications were tested in the model. The final scheme, shown in Fig. 11, extended the barrel structure by 10.0 m to allow for the occurrence of uniform flow and provided for a sloping channel to induce an increased velocity. These changes resulted in a minimal relatively uniform deposition at the intake. In the settling basin, the resulting pattern of deposition is slightly asymmetrical with no distinct bar formation. This is due to a better velocity distribution through the basin.

Based on the favorable flow conditions and deposition pattern exhibited by the final scheme, it was recommended for adoption in the prototype design.

The model study for the Norala Diversion Works of the Allah River Irrigation Project includes the location of the point of diversion, the selection of the angle of diversion, and the model evaluation of the sediment excluder schemes. A 1:30 scale model is constructed to evaluate the proposed location of the point of diversion which was provided with a 90° angle of diversion and a tunnel-type sediment diverter as shown in Fig. 17. Tests showed that the proposed location of the diversion point was prematurely set on the curvature of the river. This resulted in a poor bed load sweep because the helicoidal pattern of flow has not fully developed. Also, with the proposed diversion angle, the water flowing into the headgate formed a vortex at the radial junction of the right wall of the barrage and the left wall of the headgate. Such a vortex formation will tend to lift sediments into suspension as well as introduce the possibility of structural damage.

In this regard, the structure was moved downstream and the angle of diversion was decreased to 75° (Fig. 18). As a consequence of these revisions, a second tunnel-type sediment diverter was introduced which was compatible with the new structural arrangement. (See Fig. 19). Tests on this scheme indicated that the location of the barrage structure was such that effective sluicing of the sediments in the tunnels does not take place. This problem was solved by moving the barrage 10.0 m to the left along the dam axis. In this manner the flow velocity in the tunnels is increased to facilitate effective sluicing of the sediments.

The second tunnel design (Fig. 20) induced to vortex formation of the right side of the tunnel roof. This was due to the poor transition design of the upstream roof edge which resulted in a sudden change of flow direction and consequently the vortex formation. An alternative roof design was tested and proven satisfactory because the vortex formation was eliminated.

To improve the efficiency of the sediment excluder, the location and the length of the divider wall were varied. The final scheme is shown in Fig. 21.

At the site of the Surallah diversion works, the sediment load is composed of particles recognizable as bed load. Thus, the sediment control scheme consists of the proper location of the diversion point and selection of the angle of diversion, and the use of the appropriate guide banks and divider walls.

Tests on the first scheme, shown in Fig. 22, have shown that the angle of diversion is not sufficient to induce the artificial helicoidal flow desired. An alternate alignment of the right guide bank was proposed to improve the flow situation (Fig. 23). Model tests indicated an improved flow pattern. However, the possibility of further improving the flow situation is considered by increasing the angle of diversion by 5° (Fig. 24). Tests on this scheme are still being undertaken.

The Mag-Asawang Tubig River Irrigation Project is a departure from the previous discussions in that an intake is present at each side of the main structure which is an ogee-type dam. Proper location of the main structure is, therefore, the main concern of the model tests, aside from the evaluation of the sediment exclusion schemes.

Initial tests on the 1:40 scale model showed that the proposed location (Fig. 25) of the diversion works is not advisable because of the non-uniformity of flow across the dam due to a cross-slope of the river upstream of the main structure. Such an occurrence results in an uneven distribution of flow across the structure leading to considerable scour upstream of the left sluiceway and to deposition at the right pocket area. This deposition indicates insufficient sluicing because of the low flow across that section.

An alternative location of the structure is 265 m downstream of the previous dam line, with 12° rotation of the axis in the counterclockwise direction (Fig. 26). This location placed the axis of the structure approximately normal to the general flow direction. The structural arrangement of the previous scheme is retained.

It was observed that the flow concentration shifted to the central ogee section. As a consequence, the central area upstream of the ogee section was scoured while those upstream of the right and left sluiceways were deposited with sediments. Also the main bulk of deposition downstream of the structure occurred at the apron resulting in a reduced discharge capacity.

From the results of the tests on the model, it was concluded that the ogee-sluiceway structure be replaced by a barrage type dam. This diversion structure will be tested at the second location in a manner similar to those of the previous tests.

Conclusion

A wide variety of literature has been written on the various sediment control schemes at irrigation canal intakes. However, experience has shown that there is no set method of designing sediment control schemes of diversion systems for irrigation purposes. In view of this, it may be said that the optimum design for a given irrigation set-up cannot be arrived at by the singular approach of mathematical computations based on the above-mentioned literature. This is due to the fact that one particular irrigation diversion structure has its inherent features collectively distinct from those of projects of similar structural set-ups and settings.

Today, model studies are accepted as a useful aid in the sound evaluation of preliminary hydraulic designs of diversion systems. These model studies take cognizance of the many related physical and hydraulic factors involved that analytical methods cannot totally consider. In the light of all these, it may be said that the determination of the relatively most suitable sediment control scheme at the irrigation canal intake is achieved by model studies. The experience at the National Hydraulic Research Center has shown this.

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TABLE 1. Values of Critical Tractive Force for Different Soils

Type of Soil	(kg/m^2)	Type of soil	(kg./m^2)
Medium sand	0.17	Fine gravel	0.37
Sandy loam	0.20	Volcanic ash	0.37
Alluvial silt	0.25	Stiff clay	1.22
Silt loam	0.25	Coarse gravel	1.47
Coarse sand	0.25	Shales; hard pan	3.18

$$Z^* = \frac{V_s}{0.4V_*} \quad , \quad V_s = \text{particle settling velocity}$$

TABLE 2. Values of I_1 for Values of $\frac{a}{\phi}$

Z*	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}
0.2	2300	360	60	9	1.5
0.4	290	70	19	4	0.8
0.6	44	17	6.5	2.2	0.6
0.8	8.2	4.7	2.7	113	0.44
1.0	2.3	1.8	1.3	0.8	0.33
5.0	0.052	0.052	0.052	0.051	0.046

Values of I_2 (–)

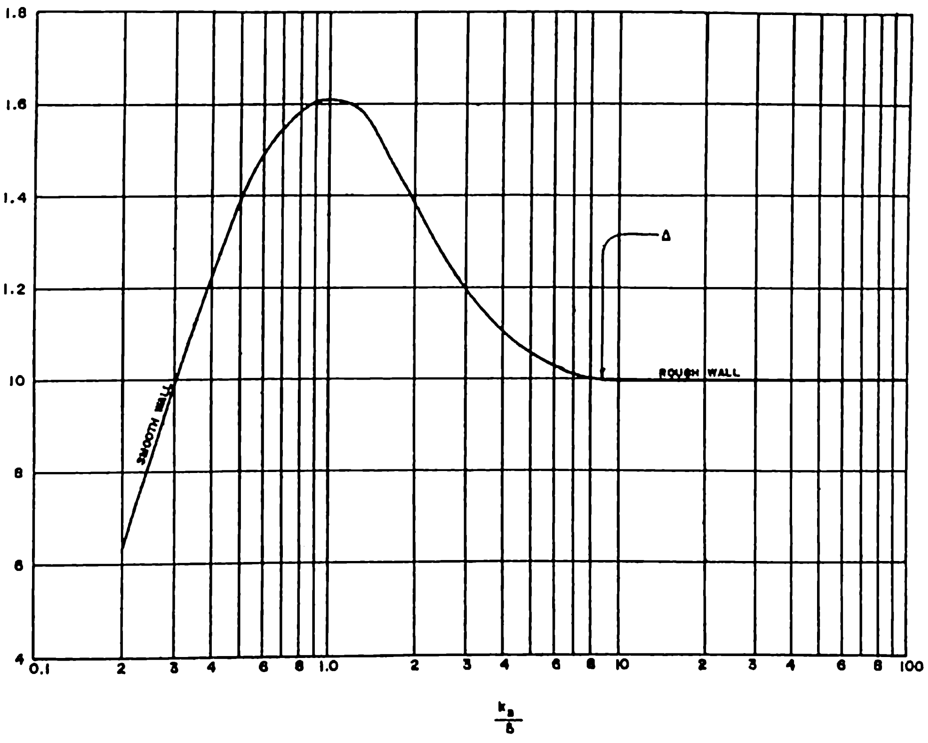
0.2	3200	500	80	12	1.1
0.4	550	150	33	6.5	0.9
0.6	120	46	17	4.2	0.7
0.8	35	18	8	3.0	0.6
1.0	15	9	5	2.1	0.5
5.0	0.6	0.5	0.37	0.23	0.1

TABLE 3. Values of corrective factor x for different values of k_s/δ

k_s/δ	x	k_s/δ	x	k/δ	x
1.0	1.62	4.0	1.1	50.0	1.0
1.5	1.60	8.0	1.0	100.0	1.0
2.0	1.40	18.0	1.0		

TABLE 4.

CORRECTION FACTOR IN THE LOGARITHMIC VELOCITY DISTRIBUTION



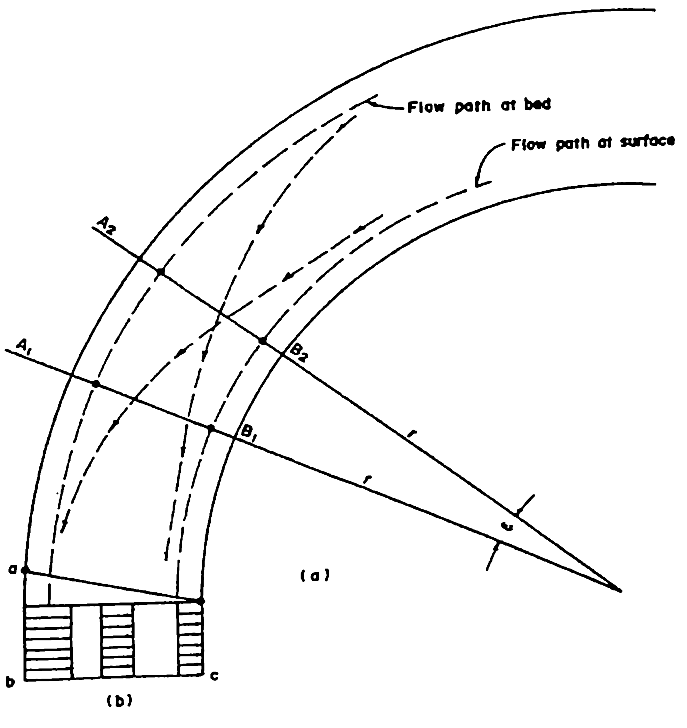


Figure 1. Flow in a Curved Channel

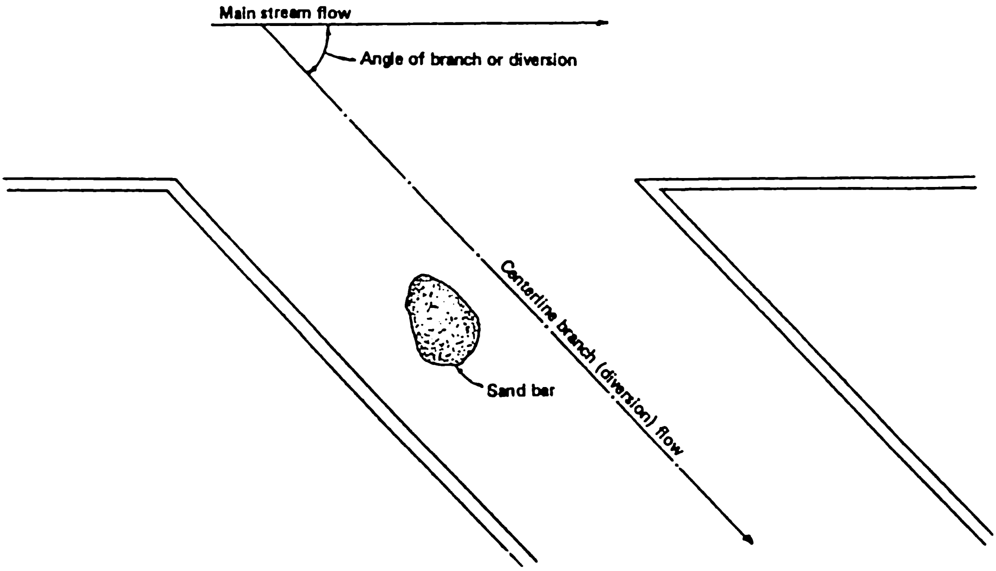


Figure 2. Angle of Diversion

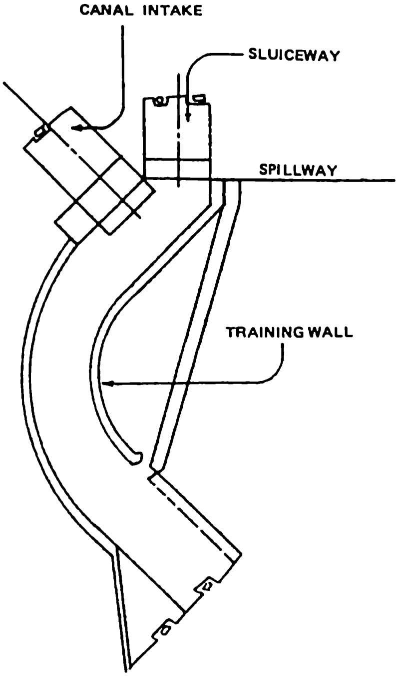


Figure 3. Training Wall

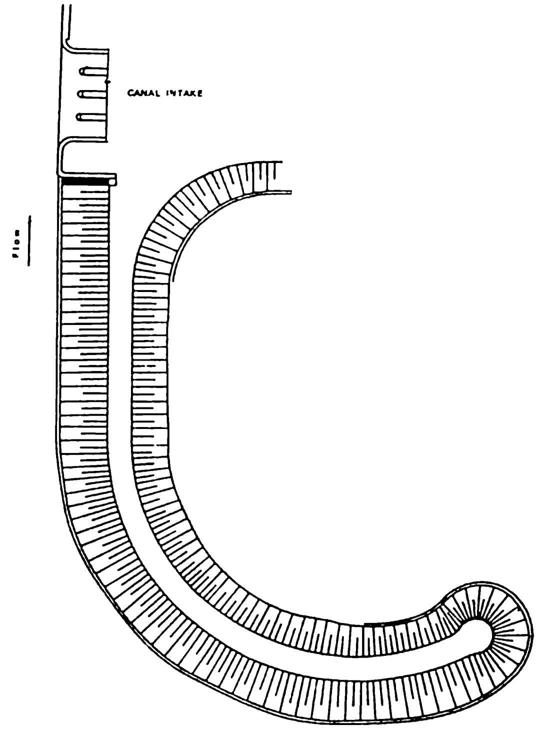


Figure 4. Guide bank of the Norala Diversion Works

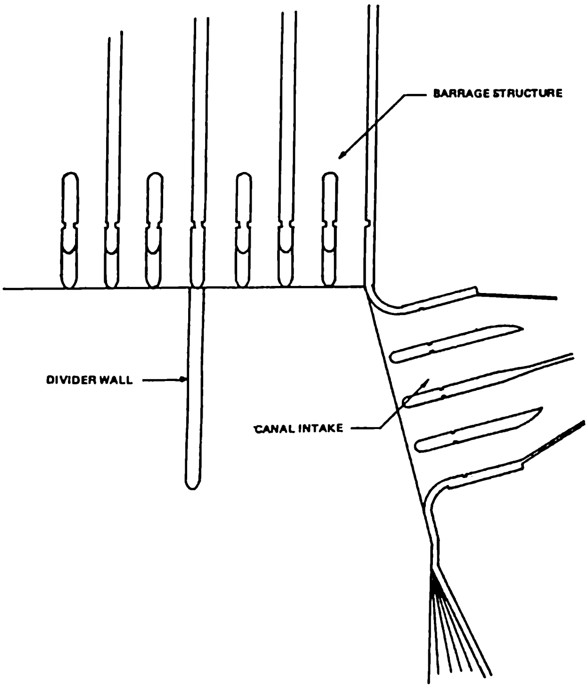
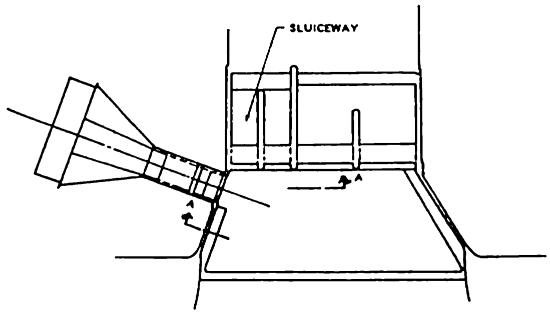
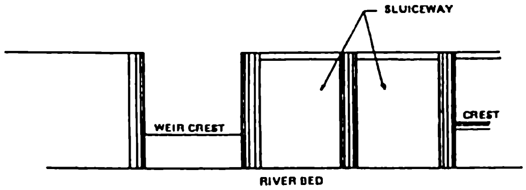


Figure 5. Divider Wall

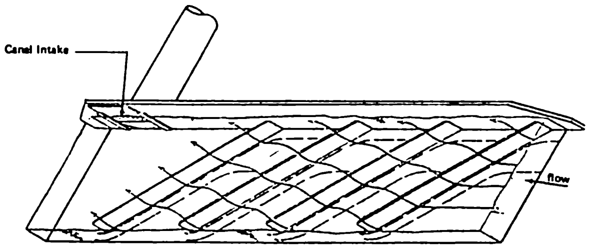


Plan

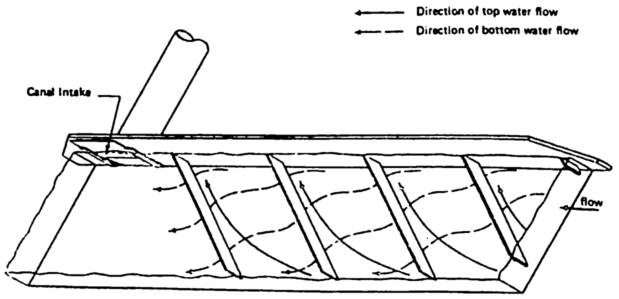


Section 'A – A'

Figure 6. Skimming Weir



–Bottom Guide Vanes



–Surface Guide Vanes

Figure 7. Typical Guide Vanes

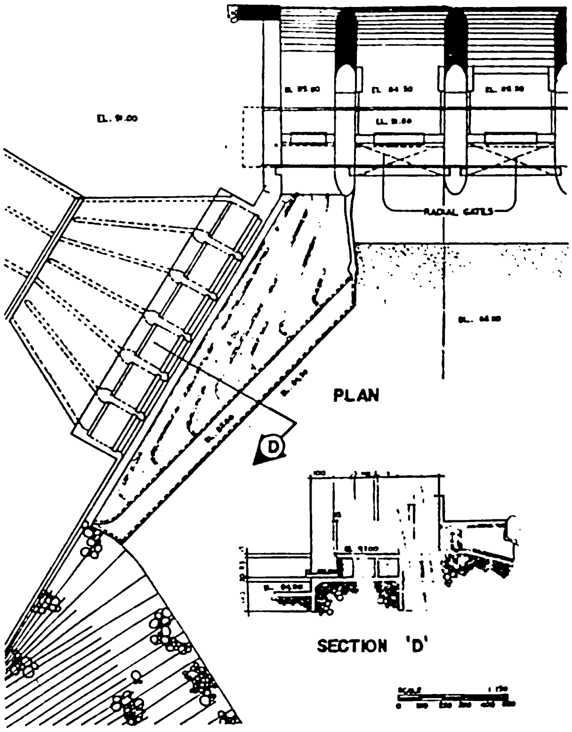


Figure 8. Proposed Tunnel-Type Sediment Diverter for the Sto. Tomas Diversion Works

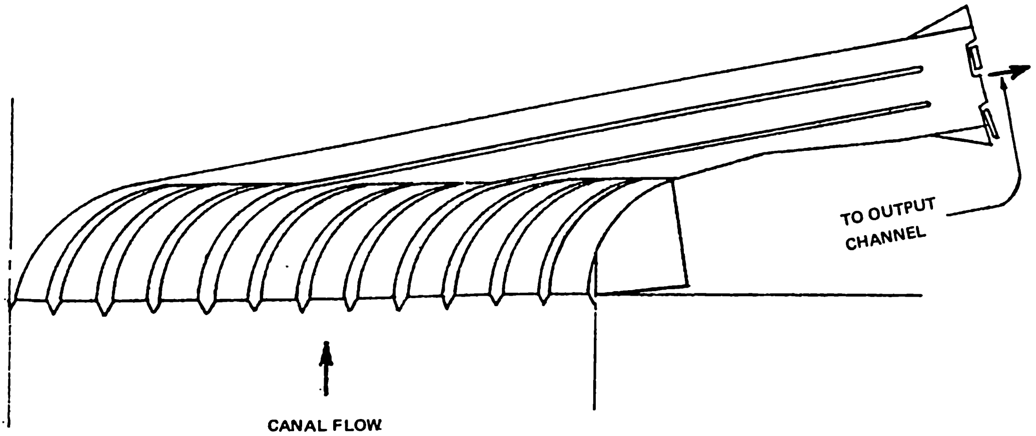


Figure 9. Tunnel-Type Ejector

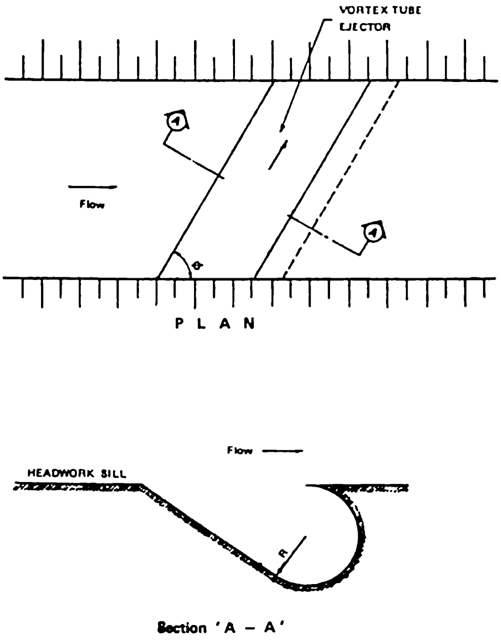


Figure 10. Vortex Tube Ejector

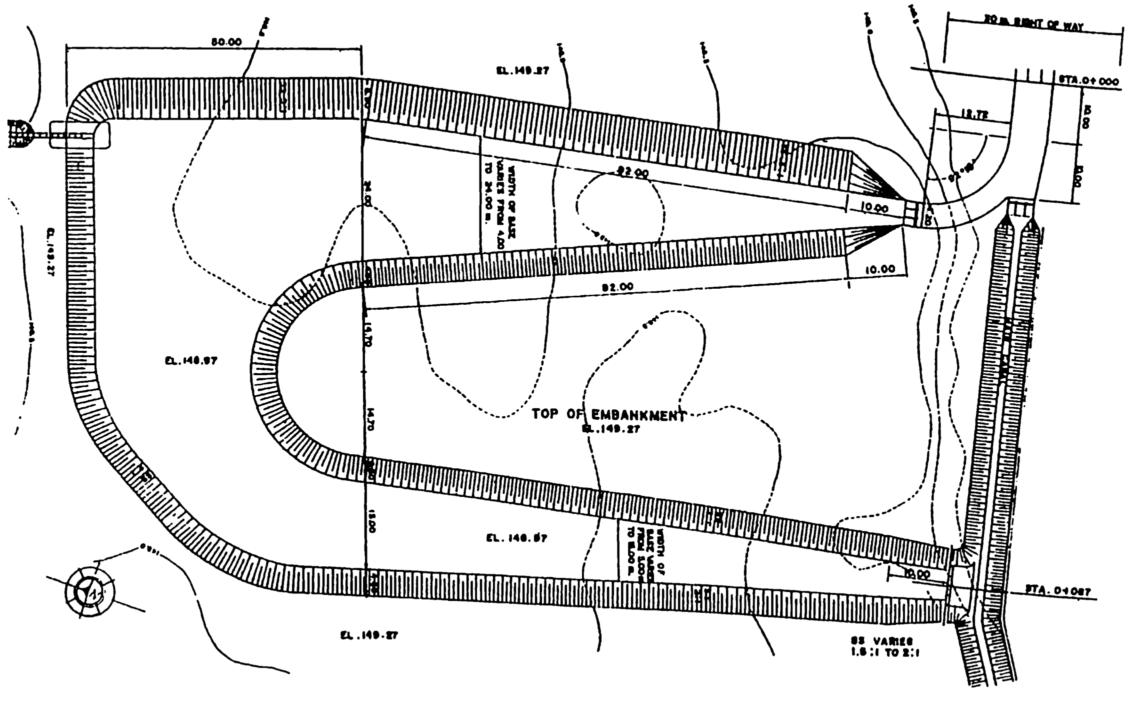


Figure 11. Suggested Settling Basin Scheme for the Banga River Irrigation Works

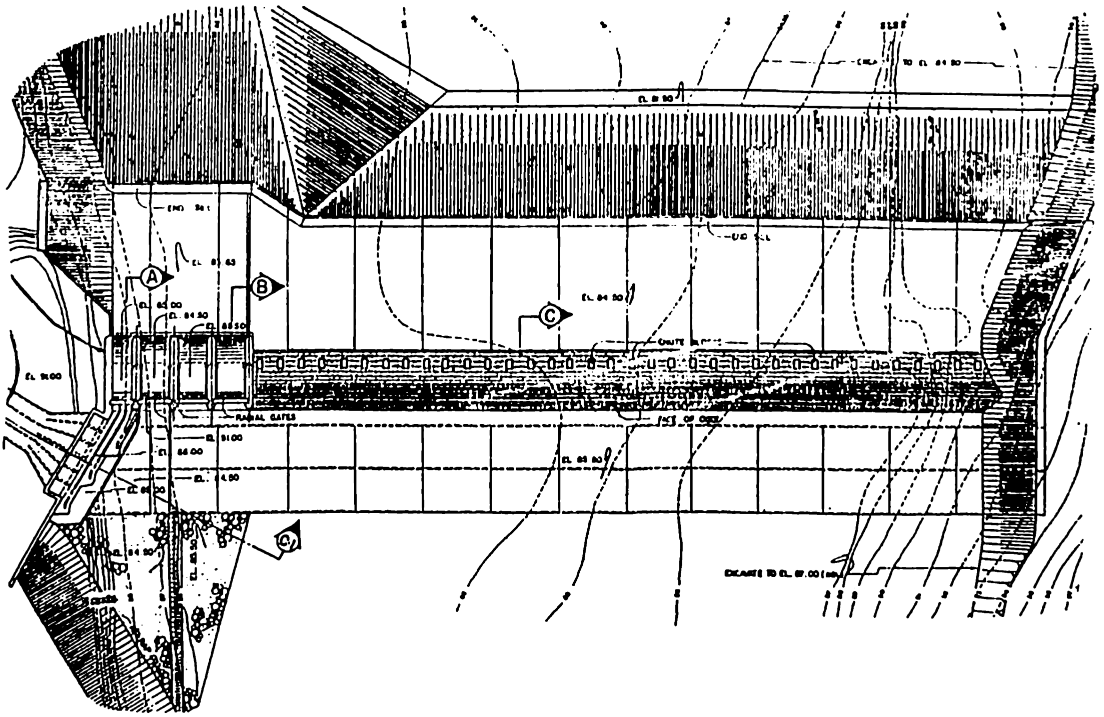


Figure 12. Sto. Tomas Diversion Dam

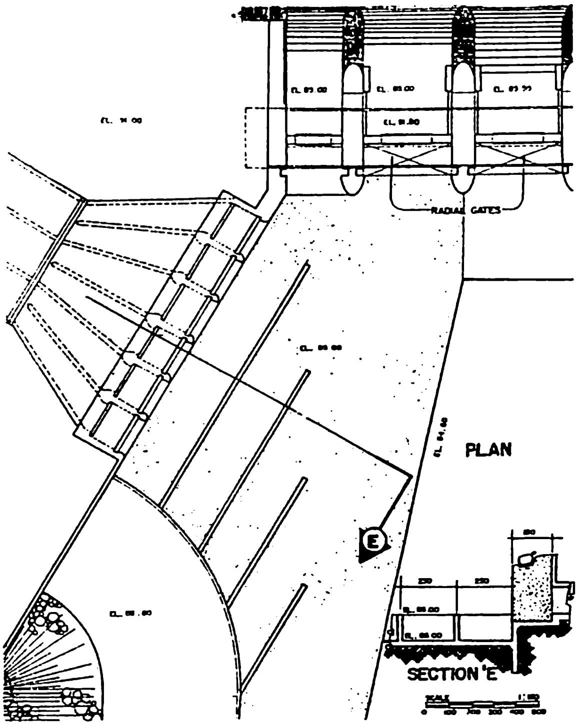


Figure 13. Guide Vanes for the Sto. Tomas Diversion Works

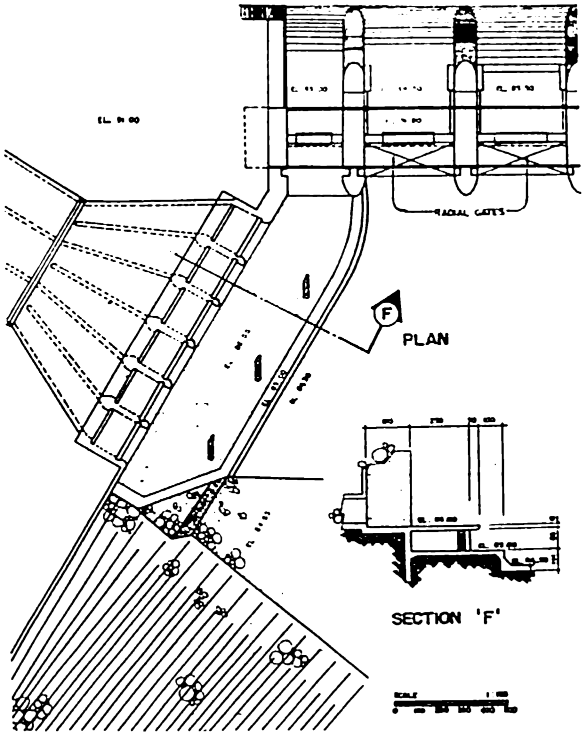


Figure 14. Alternate Tunnel-type Diverter for the Sto. Tomas Diversion Works

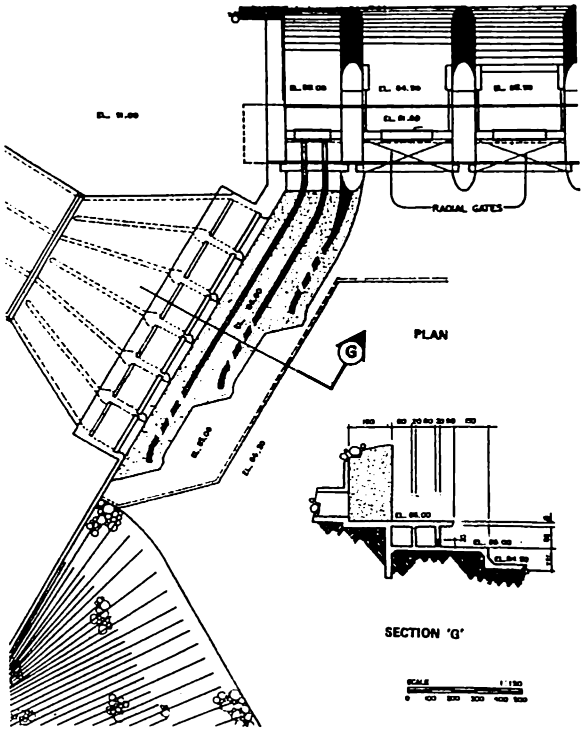


Figure 15. Alternate Tunnel-type Diverter for the Sto. Tomas Diversion Works

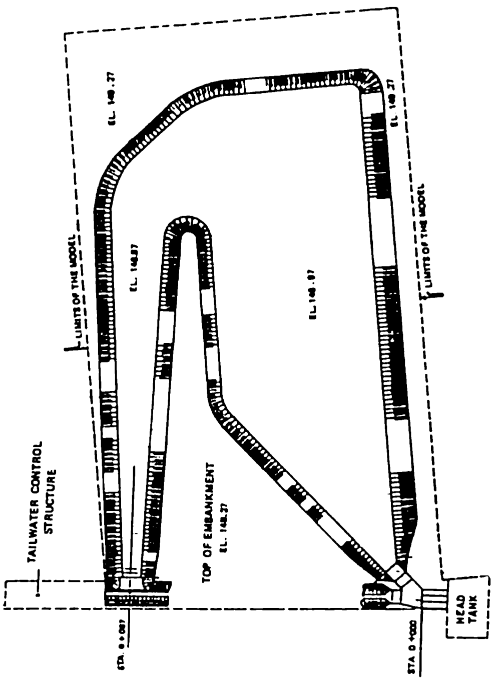


Figure 16. Preliminary Design of the Settling Basin for the Banga River Irrigation Works

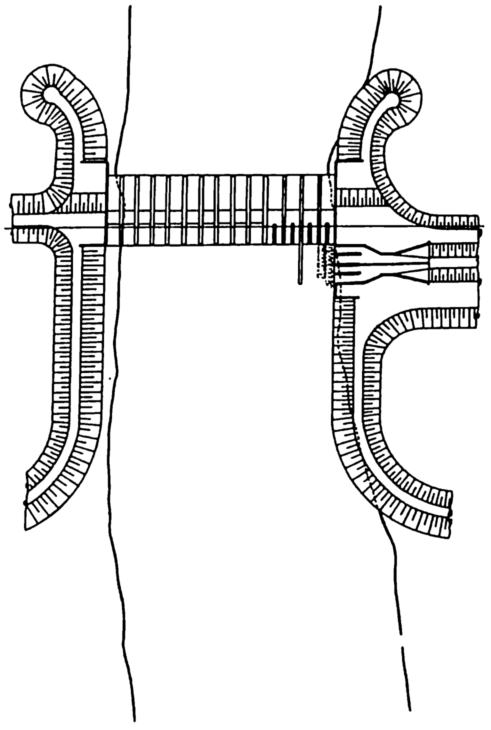


Figure 17. Initial Layout of Norala Diversion Works

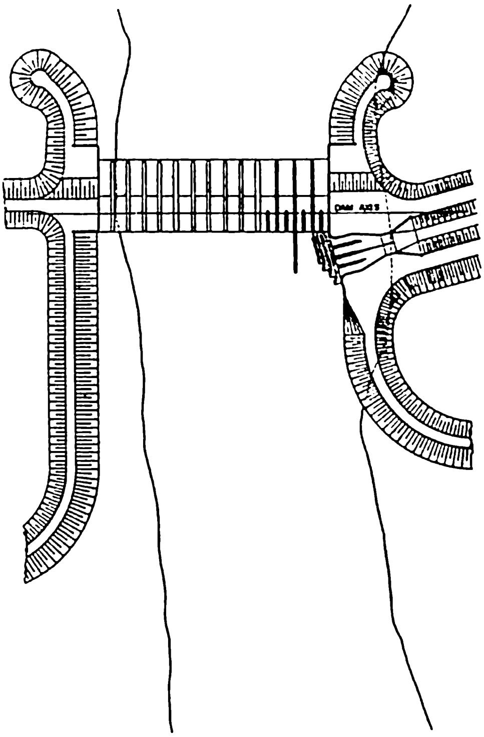


Figure 18. Relocated Norala Diversion Works Showing Reduced Diversion Angle

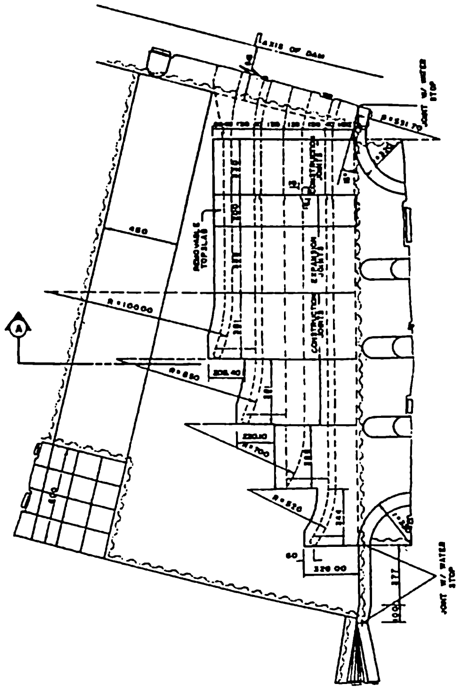


Figure 19. Tunnel-Type Sediment Diverter for the Relocated Diversion Works

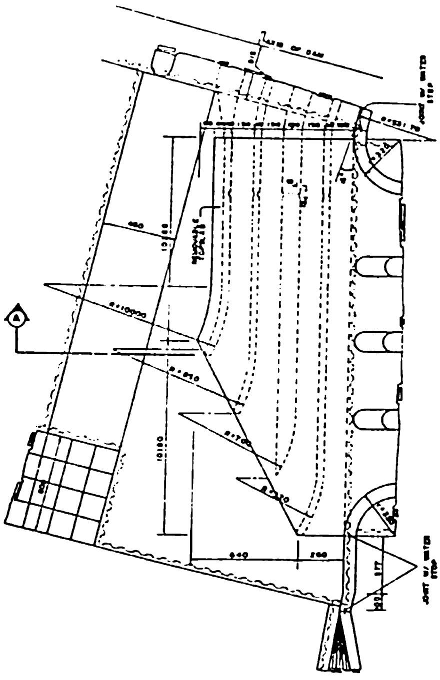


Figure 20. Alternate Tunnel-type Sediment Diverter for Relocated Norala Diversion Works

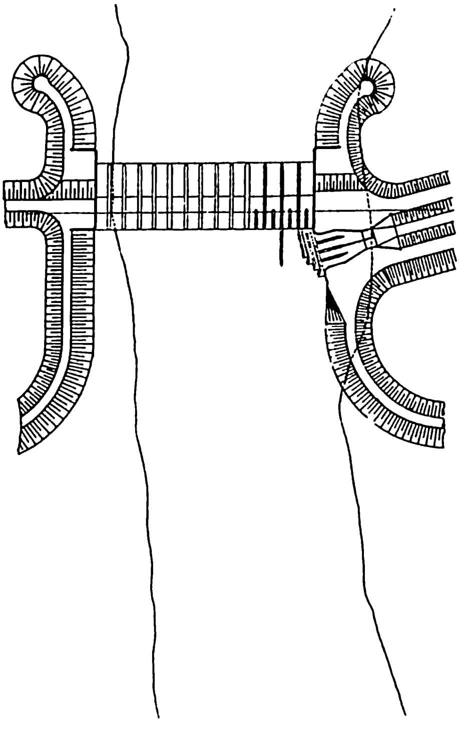


Figure 21. Final Layout of Norala Diversion Works

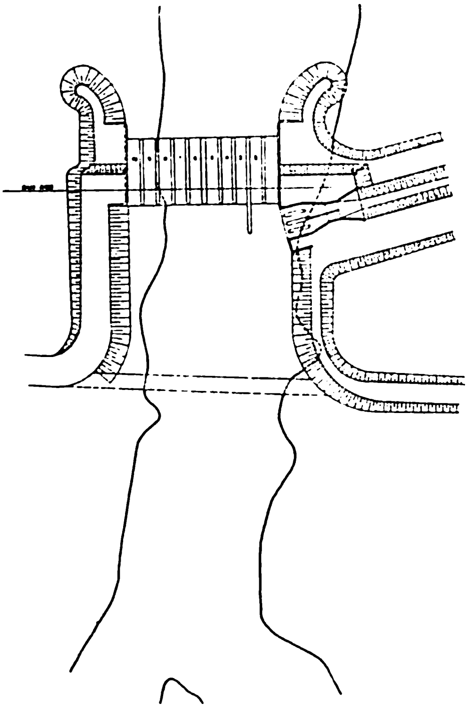


Figure 22. Initial Layout of Surallah Diversion Works

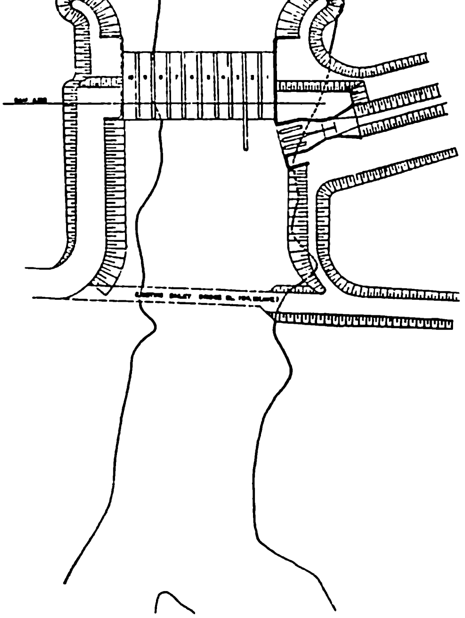


Figure 23. Alternate Alignment of the Right Guide Bank of the Surallah Diversion Works

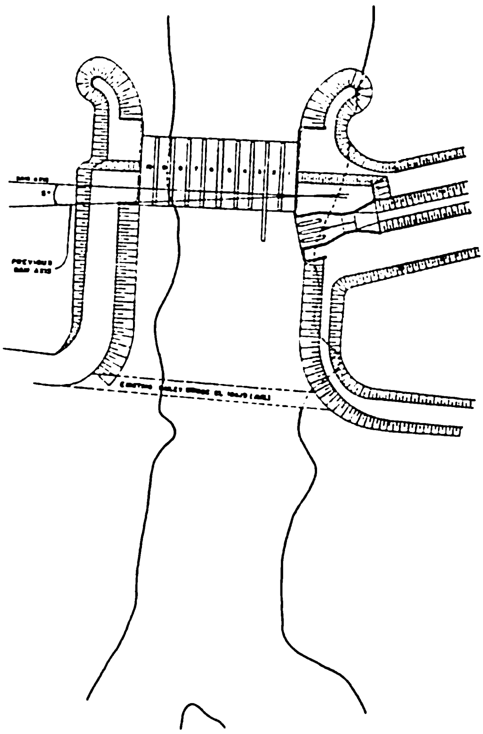


Figure 24. Rotated Headworks and Barrage of the Surallah Diversion Works

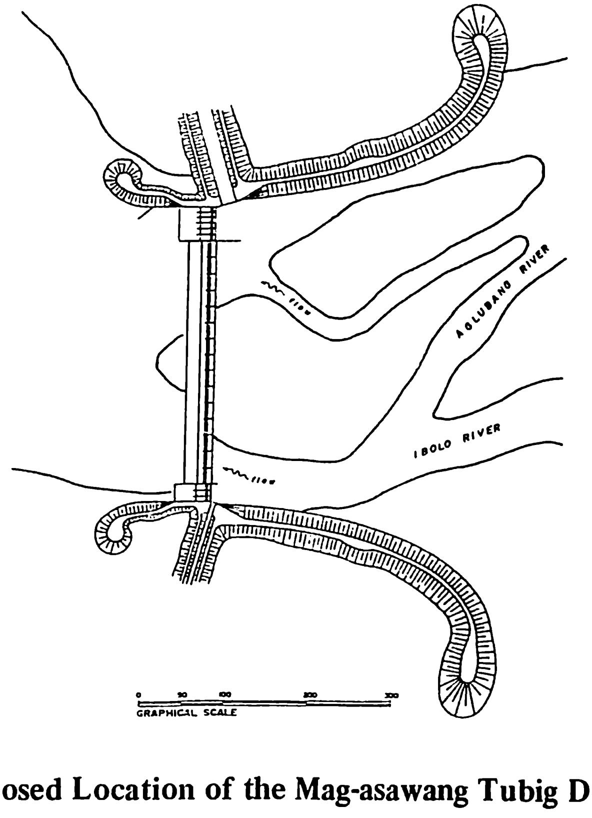


Figure 25. Proposed Location of the Mag-asawang Tubig Diversion System

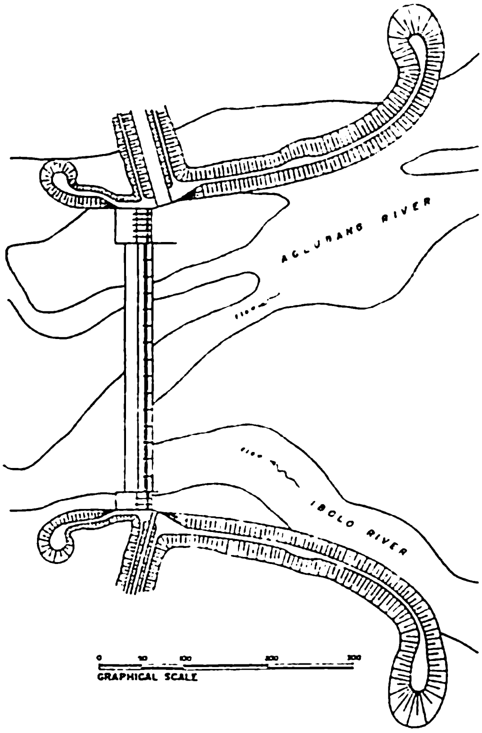


Figure 26. Alternative Location of the Mag-asawang Tubig Diversion System