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EROSION AT SANTO TOMAS RIVER DIVERSION DAMSITE

AND

SEDIMENT PROBLEMS AT DIVERSION STRUCTURES

IN THE PHILIPPINES

By

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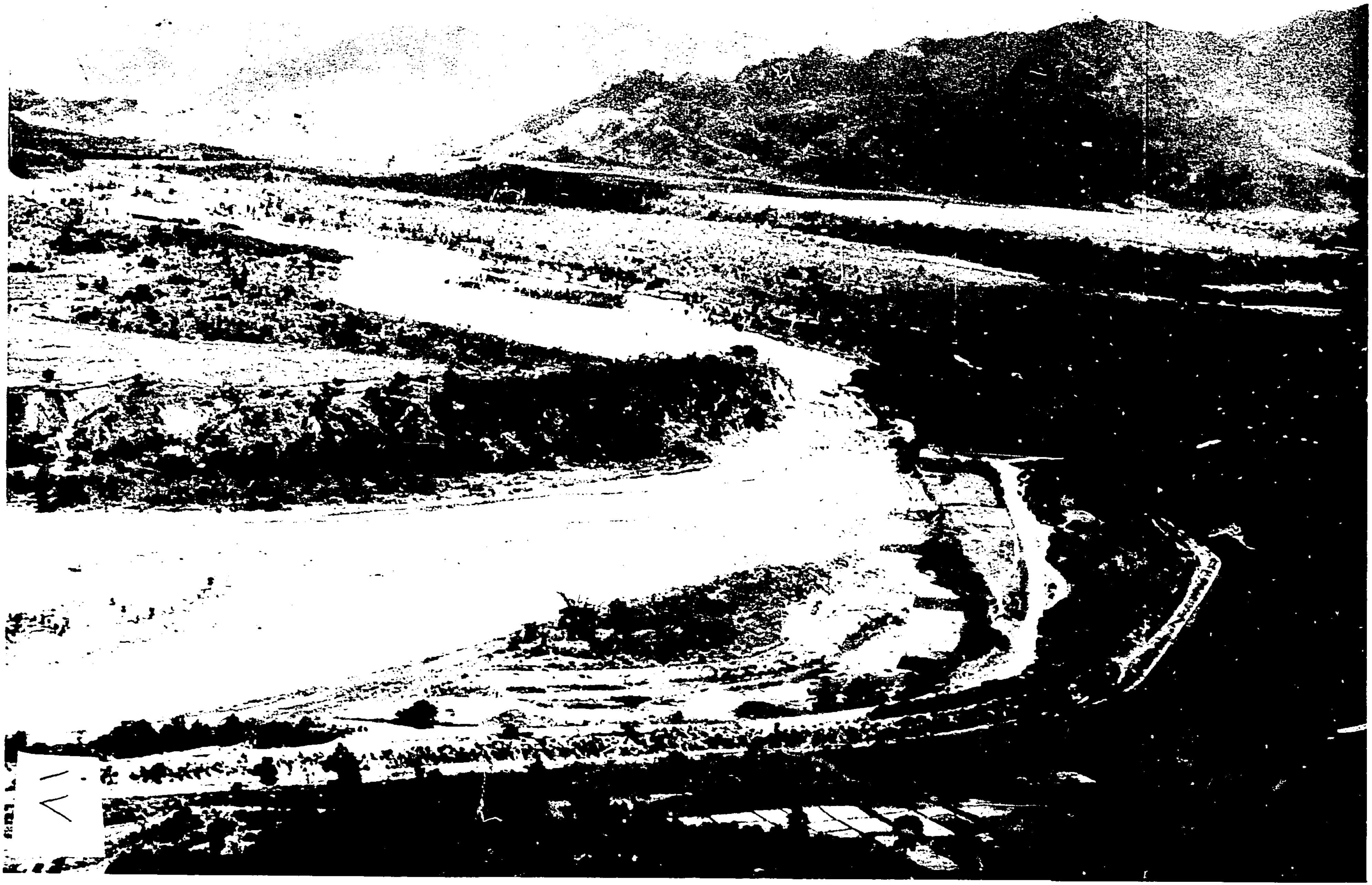
May 1975



Frontispiece: Upstream view of Santo Tomas River

Erosion at Santo Tomas River Diversion Damsite and Sediment Problems at Diversion Structures in the Philippines

by Ernest L. Pemberton

Introduction

The request for a representative from the Bureau of Reclamation to visit the Philippines and provide consultation service on deep river scour at the Santo Tomas Diversion Dam was made through the Department of State by telegram dated February 7, 1975. The diversion facility on Santo Tomas River had failed several times since construction in 1952. Mr. Richard Dangler of USAID and Mr. Adrian K. Long, the Bureau of Reclamation representative in the Philippines, were concerned about the adequacy of the structure being proposed as a replacement to the diversion dam. In addition to problems of piping underneath the proposed diversion dam, they believed erosion within the streambed channel could also contribute to the failure of a structure built at this location. I was selected as consultant on channel erosion, as well as on sediment diversion problems for excluding the sand-size material from the canal at the Santo Tomas Diversion Dam.

During my visit to the Philippines from February 26 through March 10, 1975, I was in contact with various persons in the Department of State Agency for International Development (USAID) Office, the Philippine National Irrigation Administration (NIA) Office, and Mr. Adrian K. Long, the USBR Water Resources Advisor. Attached to this report is a list of those persons contacted.

A field inspection was made at the Santo Tomas River on February 26 and again on March 5. Photographs 1 through 18 show the river at the time of the inspection. We observed the damsite from the air and also on the ground. We were looking for possible channel or hydraulic conditions that would cause failure and also at the erosion potential in the vicinity of the diversion damsite.

After a meeting on March 3 with Mr. Alfredo L. Juinio, Administrator for NIA, arrangements were made for field inspection of sediment problems at several existing diversion dams. On March 5, we visited the Agno River Diversion Dam located in NIA, Region 1, where we observed by air the diversion dam, but due to time limitation were able to observe only sediment problems in portions of the canal several miles downstream. The sediment problem on Agno River is the fine-type sediment load that is diverted into the canal where it is later carried onto the farmlands. On March 7 and 8 we visited two diversion structures on the island of Mindanao. On March 7 we inspected the Banga River Diversion Dam and observed the sediment deposition at the headworks and the sand-size sediment that is carried into the canal. At the present time, the sediments are sluiced down the river twice daily for 1 hour each sluicing period. The other structure visited was the Marbel River Diversion Dam where we stopped at the diversion point on the river and at the drop from the canal above the powerplant which is several miles downstream from the diversion point. There is some deposition of sediment in the canal upstream from the drop into the powerplant.

This report describes in more detail the observations made during the field inspection and presents the conclusions reached as a result of the inspection and review of the available data. The conclusions were verbally given to both the representatives of the NIA and USAID Offices. Keith Long, the USBR Water Resources Advisor, participated in all field inspections as well as office discussions and should be credited with his valuable input to the conclusions. Mariano Leuterio and Pablo Clutario of NIA not only aided in obtaining necessary data but made many local arrangements and participated in the joint efforts summarized in this report. The next step proposed for the Santo Tomas River Diversion would be to have a Design Engineer from the Bureau of Reclamation work with the NIA design group and help make design drawings for use in construction of a new diversion dam on the Santo Tomas River. This report also presents the results of water surface profiles that were computed by electronic computer in the Denver Office of the Bureau of Reclamation from the field data that were collected during our inspection of the Santo Tomas River.

Santo Tomas River Diversion Damsite

Santo Tomas River Diversion damsite, as shown in the frontispiece to this report, is located in a very restricted and narrow reach of the Santo Tomas River. This location necessitates a diversion dam with crest length of only about 50 meters. A flood with the magnitude similar to the 1972 flood would have a unit discharge over the crest much greater than 9.3 $m^3/s/m$ (100 $ft^3/s/ft$) normally used in design of a diversion structure. Upstream from the old diversion dam are at least two narrow sections where rock outcrops confine the river. At these restricted points, high discharges would create high scour depths whereby it is possible that material normally in the streambed would be carried in suspension or removed. It is possible that with the high river discharges the scouring condition at these restrictions above the dam could affect the streambed and erosion potential at the diversion damsite.

An examination of old photographs showed that the diversion dam built on the Santo Tomas River in 1952 had experienced several failures. Soon after initial construction, part of the upstream and downstream aprons were lost. We understand from Gregorio Farales that at one time the apron and sheet piling cutoffs located upstream as well as downstream were washed out and that you could drive a small truck or car underneath the ogee concrete section which was nothing more than a cantilevered beam across the river. This illustrates that in the past tremendous scouring action has occurred below the diversion dam.

In a review of the history of structures built at the Santo Tomas River Diversion damsite, the first was the 1952 structure having an ogee crest length of 50 meters and height of about 3.2 meters. This structure was analyzed for piping by use of the weighted creep ratio and it appears to have been adequate. There could be some question regarding the driving of sheet piling into a streambed having cobbles and boulders such as observed on the Santo Tomas River. It is extremely difficult to drive sheet piling under such conditions; therefore, it would be better to use the technique of trenching and placing a more positive concrete-type structure for controlling piping. The design for uplift pressures for the 1952 structure appears to have been satisfactory. The weak points of the 1952 structure were that there would have been a limited hydraulic jump within the downstream apron. The structure had no rubber waterstop joints to allow for any shifting. For a design discharge of 1,000 m^3/s over the weir crest, the unit discharge would be 20 $m^3/s/m$ (215 $ft^3/s/ft$). The scour depth downstream from the structure is roughly estimated at about 4.5 meters (15 feet). The type of material used for riprap to protect against this scour was inadequate. Rocks normally angular in shape would be required with maximum size of 0.76 m^3 (1 yd^3), an average weight of 1,100 kg (2,500 lb), and a nominal thickness of 0.91 m (36 in) size. In summary, the 1952 structure appeared to be adequate except that the design did not provide for controlling downstream degradation.

The 1971 structure of rubble masonry was added to what was left of the 1952 dam. The downstream apron was lowered by about 2 meters and chute and baffle blocks were added. The crest elevation of the dam and the length of the weir remained the same. Although the apron was lowered by 2 meters, proper flow and hydraulic jump conditions would have been dependent on the downstream tailwater elevations and degradation. In 1971, 200-kilogram-size boulders were placed on gravel blanket below the dam to control erosion. This riprap does not appear adequate to control downstream degradation. It should have been at least 1,100-kg size at a density of 2,640 kg/m^3 (165 lb/ft^3). Both the piping and uplift appeared adequate. In the 1971 design the energy dissipators with blocks were better than the 1952 design; however, no allowances were apparently made for downstream tailwater and degradation conditions. The unit discharge was about 20 m^3/s/m for a 1,000-m^3/s flow which is extremely high and again the riprap of only 200-kilogram size was too small.

The 1973 structure was built about 150 meters upstream from the old damsite and consisted of 20-foot steel sheet piles with a crest length across the piles of about 52 meters. Downstream and along the left bank, a row of sheet piling with a concrete cap was used to divert water into the existing canal headworks. Riprap of 300-kilogram-size boulders was used above and below the sheet piling structure and also along the guidewall paralleling the left bank of the river. The analysis shows that probably no piping occurred under this structure. However, with no grouting or mass of structure, the uplift pressures would be excessive. The weak points of this structure are inadequate design for uplift pressures and the undermining because of the inadequate riprap size.

Site selection. - After reviewing the history of failure and types of structures previously placed on the Santo Tomas River, review was made of the plan as presently proposed and which had been felt inadequate by both the USAID Office, the NIA Office, and Keith Long of the Bureau of Reclamation. This plan proposed to replace the structure lost in the 1972 typhoon at the same site as the old dam with the crest length of about 55 meters. The proposed dam is grouted stone or cobble with a compacted backfill core and series of collector drain pipes in gravel materials. Analyzing the structure by weighted creep ratio shows that piping would not occur for a design flood of around 1,000 m^3/s. The uplift pressures were adequate; however, there are many weak points. No provisions for any shifting or joints were provided in the mass of grouted stones. With the downstream apron at elevation 85.3 meters there is some question whether a hydraulic jump will occur because of no tailwater information. The unit discharge of about 18 m^3/s/m is high. The riprap of 0.30-m (12-in) minimum boulders is poorly defined and inadequate because the size of the material should be at least 0.76 m^3 maximum size, weigh 1,100 kg, and have a nominal thickness of

0.91 m (36 in). The degradation or scour hole would be at least 4.5 meters and could be up to about 9 meters deep immediately downstream from the dam. Therefore, large size riprap for protection against vertical degradation is required.

As a result of the field inspection and after reviewing the problems with structures that had been built in the past, it appears that it would be desirable to locate a dam somewhere outside of this narrow restricted reach of river. The first location appeared to be at the upper end of the restricted reach where a rock outcrop shows on both banks of the river. A dam at this location could possibly be built to a rock foundation; however, this section would have to be drilled to determine the depth to bedrock. The site is about 180 meters upstream from the old diversion dam and would require a dam with a crest length of about 70 meters. This crest length results in a unit discharge of 14 $m^3/s/m$ (153 $ft^3/s/ft$) which is still somewhat high. A critical point would be to provide protection against downstream erosion. Downstream erosion would be severe because there is still an extremely narrow restricted section of channel below the damsite. Also, the proposed headworks to this structure is critically located at a point in the river just below the junction of Santo Tomas River and Pili River. There is an advantage to a structure at this site because a series of sediment ejectors or sediment excluders could be added to the canal section now defined by the sheet pile guidewall along the left bank. The coarser sand and gravel-size materials diverted into the headworks could be sluiced back to the river. It is recognized that during this sluicing, water would be lost; however, it appears that during the low flow period there is less movement of sand-size material and at the higher flows, when bedload is being moved, there would be sufficient water for sluicing.

Another proposal that was first brought to our attention by the NIA Office was to locate a dam downstream from the narrow reach on the Santo Tomas River. This location was about 300 meters below the old dam and would require a completely new structure to divert water into the present canal. This downstream location is possible because head is available in the present canal. The location, as envisioned by NIA, would provide a crest length of about 150 meters and thereby reduce the unit discharge considerably. The one problem with this location is that the river is turning at this point. On the right bank a considerable amount of sand and gravel is being deposited on a sandbar and a good portion of the crest length might become ineffective because of the sand deposits. Also, a dike is required to high ground on the right bank to force all of the water over the overflow section. The real advantage of this location is that the river is directed into the left bank area where a headworks structure could be built and sluicing of the coarse sediments through a sluice gate would not be too difficult.

An alternative to the NIA location was found after our field inspection of March 5 at a site about 100 meters upstream or about 200 meters below the old diversion dam. At this point, a dam could be built with a 100-meter crest length. It could be keyed directly into rock on the right bank abutment. At this location the flow pattern over the weir crest and conditions for excluding sediment by sluicing are very good. It is recognized that at this site, the left bank headworks would have to be protected against erosion. It is necessary at this point to provide adequate downstream riprap of 0.76 m^3 maximum size, 1,100 kg in weight, and with a nominal thickness of 0.91 m (36 in). The riprap material should be high density rock so that it will not wash out. Much of the present rock in the channel is poor grade, porous, and low specific gravity which is not ideal rock for riprap. It should be massive and angular in shape in order to better maintain and control downstream degradation. As discussed with the NIA Office, this riprap could be blasted to obtain the angularity that is desired for riprap protection.

The sediment-excluding facilities at either the upstream or downstream diversion sites are extremely important for a dam built on Santo Tomas River because there are large quantities of sands and coarser materials that will be diverted into the canal. As indicated earlier, the upstream site would have the advantage that excluders could be constructed in the existing canal section by use of bottom guide vanes, vortex tubes, or some combination to force the coarser material moving as bedload on the bottom of the canal at right angles to the flow and back into the river.

For a diversion damsite located 200 meters downstream below the old diversion site, flow conditions are favorable for a very efficient sediment excluder. To develop the best design for sluicing the sand and coarser materials downstream it is advisable that the University of Philippines National Hydraulics Research Laboratory conduct model testing of this diversion structure. Dr. Angel A. Alejandrino, Director of the Laboratory, was contacted regarding such tests. One proposal to be tested would be to place the canal headworks in direct alinement with the flow down Santo Tomas River. In addition, bottom guide vanes directed out into the river and sloping downstream to force the bedload away from the canal headworks toward a sluice gate should be tested. Additional sluice gates provided in the main dam will help move the bedload down the river rather than over the main overflow ogee weir section of the diversion dam. The design first considered by NIA used some undersluices through the dam. These would appear to be subject to severe abrasion caused by the movement of the sand- and gravel-size materials down the river. It is recognized that there is always going to be some abrasion for this type of structure, even if some structural materials could be found to control abrasion within the under-sluice tunnels. The main disadvantage to the under-sluices is that they would be inadequate in size to move the total bedload. In most cases, it is much better to provide large radial-type sluice gates that can be raised out of water during the larger flood flows. These gates permit more efficient bedload movement of sands, gravels, and boulders through the diversion structure. The downstream erosion will also be reduced by maintaining the bedload transport through the dam.

Tailwater. - Cross sections of the Santo Tomas River were surveyed both upstream and downstream from the old diversion damsite during February and March 1975. These were supplemented with other cross sections that were surveyed in late 1974. The location of both the 1974 and 1975 surveyed cross sections is shown on figure 1. The alinement of these sections is approximate; however, the distance between them should be adequate since the field survey crews identified the cross sections by channel distance in meters above and below the old dam. It would be well to determine the angle or alinement of these sections in the field so that they could be more precisely located on a map similar to figure 1. Water surface profiles were computed by use of the electronic computer in the Denver Office of the Bureau of Reclamation for a range of flows from 30 m^3/s up to 1,000 m^3/s. These water surface profiles are shown on figure 2. The roughness coefficient (Mannings "n") used in the water surface profile computations was estimated from field observations and flow conditions expected within the narrow reach above the old diversion damsite. Downstream from the section 200 meters below the old dam an "n" value of 0.045 was used for the profile computations. In the reach from 100 meters below the dam up to and through 150 meters above the dam an "n" value of 0.03 was used. An "n" value of 0.045 was used for computing the water surface profiles at sections 210 and 310 meters above the dam. This difference of "n" values in the restricted section is caused by the more efficient channel geometry as compared to both the upstream and downstream reaches where the channel meanders across a much wider width. Also, the size of the material is considered to be much coarser in the downstream river although naturally there will be some coarse material in the narrower gorge section of this reach. The values selected were supported by the fact that "n" values of 0.03 were required to get the critical flow conditions appropriate for the restricted reach at and above the old diversion damsite. The beginning water surface slope of 0.007 m/m in the lower reaches of channel was the other criterion used in the tailwater studies.

The tailwater conditions defined in the water surface profiles shown in figure 2 are considered adequate for design of any diversion structure whether downstream from the old diversion, at the old diversion dam, or at the upstream location. Cross sections 210 meters and 310 meters above the old dam contain angle points because they were taken from a topographic map and were drawn to approximate a section normal to flows that would occur in this basin just upstream from the restricted portion of the Santo Tomas River. The rating curves defining tailwater conditions at a section 200 meters down from the old damsite, at the old damsite, and 180 meters above are shown in figure 3. Also shown is the rating curve for a section 400 meters below the dam which should be near the stream gage location on Santo Tomas River. In checking the gage heights at this location, it appears that the stream channel has degraded since some of the gage heights were read. It is not known how these gage heights were related to discharge because all of the higher discharge values would be estimates. They were probably computed from an assumed slope and cross section. The accuracy of the estimates would depend on the number of cross sections used and length of channel used for computing the water surface profiles. It is also possible that at the gage location 400 meters downstream, considerable pileup on the left bank could occur. Water flowing through the narrow restricted reach is turning to the right; therefore, a much higher water surface could occur on the left bank than on the right. This is not unusual, although most of the pileup should have occurred downstream from this point. The rating curves shown in figure 3 are considered representative of present channel conditions and useful for a design at these specified locations. Rating curves could be developed at other locations from the water surface profiles shown on figure 2.

Design Discharge. - In the Bureau of Reclamation the design discharge used for most diversion dams is the 50-year frequency flood peak and then checked to determine whether the design freeboard will contain the 100-year flood peak.

There is some question with regards to the actual flood peak discharge that occurred in 1972 at the Santo Tomas River Diversion Dam. With a reported water elevation immediately above the dam of 92.7 meters, the discharge could vary from 750 m^3/s to above 1,000 m^3/s. The dashed lines on figure 2 represent the computed water surface with the diversion dam in place - that is - a 50-meter dam with crest at elevation 89 meters. The starting elevation was computed by the equation $Q = CLH^{3/2}$ where the coefficient C was adjusted for submergence. Those profiles show that if the dam was intact at the time of the peak discharge, then the discharge was more nearly 750 m^3/s. Any other condition such as partial loss of dam at the time of peak flow could increase this discharge as shown by the lower curves with no dam to about 1,000 m^3/s.

The previously recorded peak discharge at the Santo Tomas River Diversion was 855 m^3/s in July 24, 1962. Luis Sosa indicated a frequency study could be made for use in design. Such a study is advisable to support the selected design.

Sediment Problems at Diversion Structures

Agno River Diversion. Field inspection at the Agno River Diversion Dam was limited to the aerial flight over the diversion dam on March 5, 1975 (photograph 19). The diversion has sluice gates, a guidewall upstream, and a headworks located at a 90° angle to the river. The dam has an ogee crest and there is a large amount of rock and sand piled up near the left bank. This is indicative of a good location for a diversion dam with the main flow on the right bank near the headworks. From observations of Agno River near Villasis downstream there appears to be sand and coarse material moving as bedload at the diversion dam. It is not known how much of a problem there is to clean the sediments from the canal in the upper reaches immediately downstream from the dam because we did not visit this area. If the canal requires cleaning, then it may be possible to provide some temporary solutions at the diversion dam whereby the bedload can be sluiced down the river. As indicated in later discussions on the Marbel River Diversion Dam, the bottom guide vanes could be used to move the bedload away from the headworks.

Another problem on Agno River is the amount of fine, whitish-type sediment load, colloidal in nature, that is being diverted into the canal system. The sediment was observed in the canal many miles downstream from the diversion structure. This fine material is primarily contributed by mining operations in the upstream river basin. A natural solution to this problem is to control it at its source, which would prevent the colloidal material from reaching the Agno River. However, even if the controls could be placed at the mines, there will probably still be material of colloidal size that would wash down the river for many years to come. If the fine colloidal material becomes a problem that the farmers can no longer tolerate, then a settling basin could be built to settle out the fine material. The settling basin would have to be large enough and with adequate detention time to get the proper settling action 1/. If this is impracticable then a smaller basin could be built and flocculating agents used to speed up the settling process. The settling basin would have to be cleaned periodically with a dredge or some other appropriate method. Also two settling basins could be built. While one is being cleaned, the other could be used; however, if a hydraulic suction dredge is used, usually only one settling basin would be needed. More information is needed on the sizes and volume of material to be deposited for determining the settling basin size, need for more than one basin, and capacity of dredge if that technique is selected for cleaning the basin.

Banga River Diversion Dam. - The field inspection of the Banga River Diversion Dam was made on March 7, 1975 (photographs 26 through 31). This inspection was late in the day and we observed the sluicing operation. At this time of the year, the sediment is sluiced away from the headworks for 1 hour in the morning and 1 hour in the evening of every day. The present diversion dam is roughly shown on the attached sketch, figure 4. In this type of design, the intent is to skim the clear water into a boxed area in front of the canal headworks. The sediment is deposited in the river channel. The problem is that if the sediment accumulates too long it starts coming through slots near the top of the guidewalls. The slots provide most of the skimming action seen on photographs 26 and 27. This technique of skimming is adequate where the water can be ponded and sediments will drop out and the top water is diverted into a canal. When the river channel is filled with sediment upstream from the diversion dam, then the skimmers usually become ineffective because of the turbulence created at the skimmers. In this case, about all that can be done is to operate exactly as Banga River Diversion Dam with the periodic sluicing of the deposited sediments.

One possible alleviation of the problem is shown on the sketch, figure 4. This technique has worked effectively in separating the water diverted into the canal from the water being sluiced. This is done by either one or a series of boxes or undersluices whereby the bottom water that carries the sediment load is sluiced down the

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1/ "A Procedure to Determine Sediment Deposition in a Settling Basin," by Ernest L. Pemberton and Joe M. Lara, Bureau of Reclamation, August, 1971 river. The top water is diverted into the canal. A temporary measure shown on figure 4 tries to duplicate this undersluice technique by placing some pipes parallel to the river along the front of the canal headworks. The slide gate at the upper end of this area would have to be removed. The downstream end of the pipes would have to be bulkheaded into the existing sluicing gate which is a slide gate. The bedload would be moved through the pipes on down the river. The amount of water lost by this action should be kept to the minimum required to move the bedload. The bulkhead at the lower end of the pipes would limit flows to the pipes and the slide gate would control the sluicing water. A problem with this type of structure would be the plugging of these pipes with trash and debris. Some kind of trashrack at the upstream end of the pipes may be required. The second phase for this temporary solution would be the addition of slots in the top of the pipes to capture the coarser suspended sediment. Another alternative would be to add another pipe to increase the capacity for bedload movement on down the river. This pipe would be shorter in length and placed beside the two shown in figure 4. The first step is to take out the stoplogs that are now in front of the headworks to the six slide gates. By removing these stoplogs the velocity of flow will be reduced thereby decreasing the transport and sediment inflow to the canal. The removal of stoplogs and installation of the two pipes should be tried before constructing any major modifications.

Marbel River Diversion Dam. - The field inspection on March 8, 1975, included the Marbel River Diversion Dam, the upper end of the main canal, and the canal at the powerplant (photographs 20 through 25). There is a significant bedload on Marbel River as indicated by the bed material of sand sizes and the sluicing schedule. A rough sketch of this diversion dam is shown on figure 5. This sketch is not to scale and shows only the major features for illustrating a possible solution to the sediment problem. The operation of Marbel River Diversion Dam requires sluicing for about 9 hours every 6 days. This sluicing water is lost to the canal but should not be much greater than that required for continued sluicing to move bedload on down the river. The proposed bottom guide vanes are shown on figure 5. These were originally estimated about 1.8 to 2 meters in height; however, the top elevation of the vanes should be no less than the sill elevation to the slide gate diverting water into the canal. These bottom guide vanes could be fabricated steel and could be placed in the channel above the headworks in order to take waterflow over the top and into the canal and keep the bedload movement towards the guidewall down to the slidegate used for sluicing. It is possible that the bottom guide vanes should be somewhat higher than 2 meters if the lower vanes are not successful. The guide vanes could be fabricated with some slight upstream curvature on the top to encourage a spiral movement of bedload away from the headworks and the canal headgate.

Another alternative to the bottom guide vanes would be either vortex tubes in the canal bottom or the combination riffle-deflector vortex tube sand trap type shown in figure 6 and exemplified by the photographs of an existing structure, figure 7. The location for these sand excluders can be in a straight reach of canal or below a bend where the sediments are moving from the outside wall towards the inside as illustrated on figure 6. The design dimensions of the vortex tube, or the riffle-deflector vortex tube sand trap, can be best obtained from articles written in (1) Journal of Irrigation and Drainage Division of the American Society of Civil Engineers by A. R. Robinson "Vortex Tube Sandtrap" published in December 1960 and (2) Transactions of the ASCE, Vol. 117, "Model and Prototype Studies of Sand Types" by R. L. Parshall published in 1952. A large settling basin in the canal would be the less likely alternative solution. Water would be lost to the river by seepage out of the basin. A fairly large settling basin would be needed and the materials deposited in the basin would have to be removed by either dredging or dragline to maintain adequate storage capacity.

Conclusions and Recommendations

In the design of Santo Tomas River Diversion Dam, the old diversion dam location does not appear to be most ideally located because of the narrow crest length and excessive channel scour occurring in this narrow restricted reach of the river. A much more desirable location for the diversion dam would be to either move upstream or downstream from the extremely narrow reach of river. The best damsite now appears to be about 200 meters below the old diversion damsite where a crest length of 100 meters could be built and the dam would have a good rock right abutment. The left abutment would be built along the present bank but would need riprap upstream and downstream to protect against bank erosion. It is important that there be adequate riprap of 0.76-m maximum size, 1,100 kg rock, and a nominal thickness of about 0.91 m (36 in) below the diversion dam. This riprap should also be of high density material and be blasted in order to obtain better angularity. The diversion dam design should include an efficient sand or bedload excluder at the headworks structure. The technique presently proposed will be model tested at the University of Philippines National Hydraulic Research Laboratory. The canal headworks would be alined in the direction of the incoming riverflows and the bedload moved by bottom guide vanes towards the sluice gate. There should be sufficient sluice gate capacity at the dam to move the large bedload during the high flood flows. To provide guidance in developing a good specification design, someone from the Design Division of the Bureau of Reclamation is needed to assist the NIA and USAID Offices in the Philippines.

The three diversion dams visited were the Agno River Diversion Dam, Banga River Diversion Dam, and Marbel River Diversion Dam which were all experiencing sediment diversion problems. The temporary techniques proposed are shown on the sketches, figures 4 and 5. These proposals to exclude the sand-size material are not major or too expensive but should be field tested before any major modifications are made to these structures. The Banga River Diversion Dam proposal provides the undersluice technique which in this case would be pipes placed in the bottom of the channel. The Marbel River Diversion Dam proposal is to use fabricated steel guide vanes placed on the streambed immediately upstream from the diversion point. Some type of a cofferdam would be required to hold back the water, while the guide vanes were being anchored to the existing structure. The proposed modification to Marbel River Diversion Dam (figure 5) should be adequately field tested before any modifications shown on figure 6 are made in the canal. The canal changes involve more expensive construction. The existing canal section would be removed and a new enlarged section constructed to provide an ejecting system.

Persons Contacted During Field Trip to Philippines
February 23 to March 11, 1975

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Filipe Nilo
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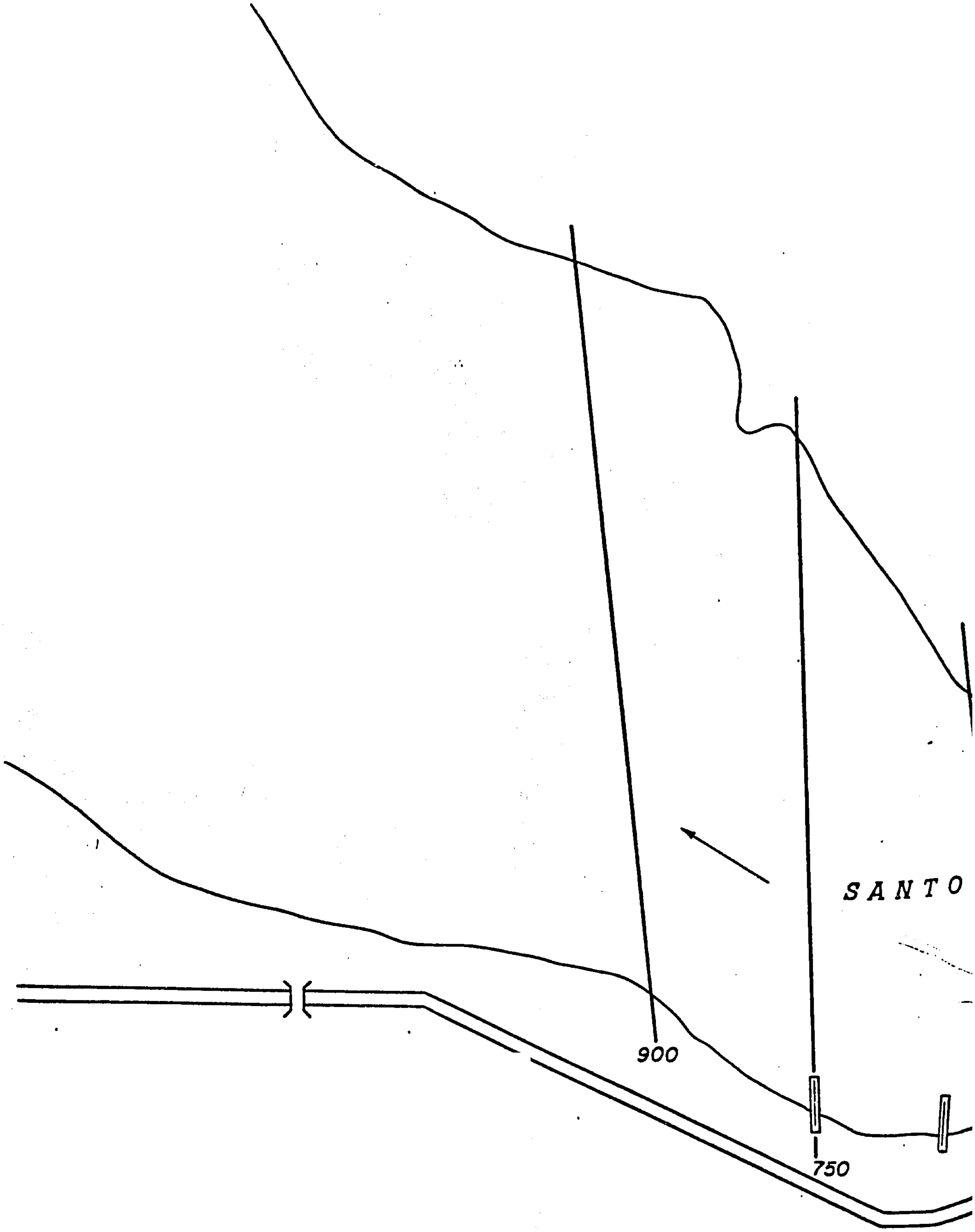
Regional Director of Irrigation, Region 8
Jose H. Hipe
Regional Director of Irrigation, Region 8-A
Agustin Cordoba
Chief, Operations Division, Region 8-A
Basilio Demafeliz
Chief, Engineering Division, Region 8-A
Dr. Angel A. Alejandrino
Director, National Hydraulics Research Laboratory, University of Philippines
Acting Director, National Water Resources Council



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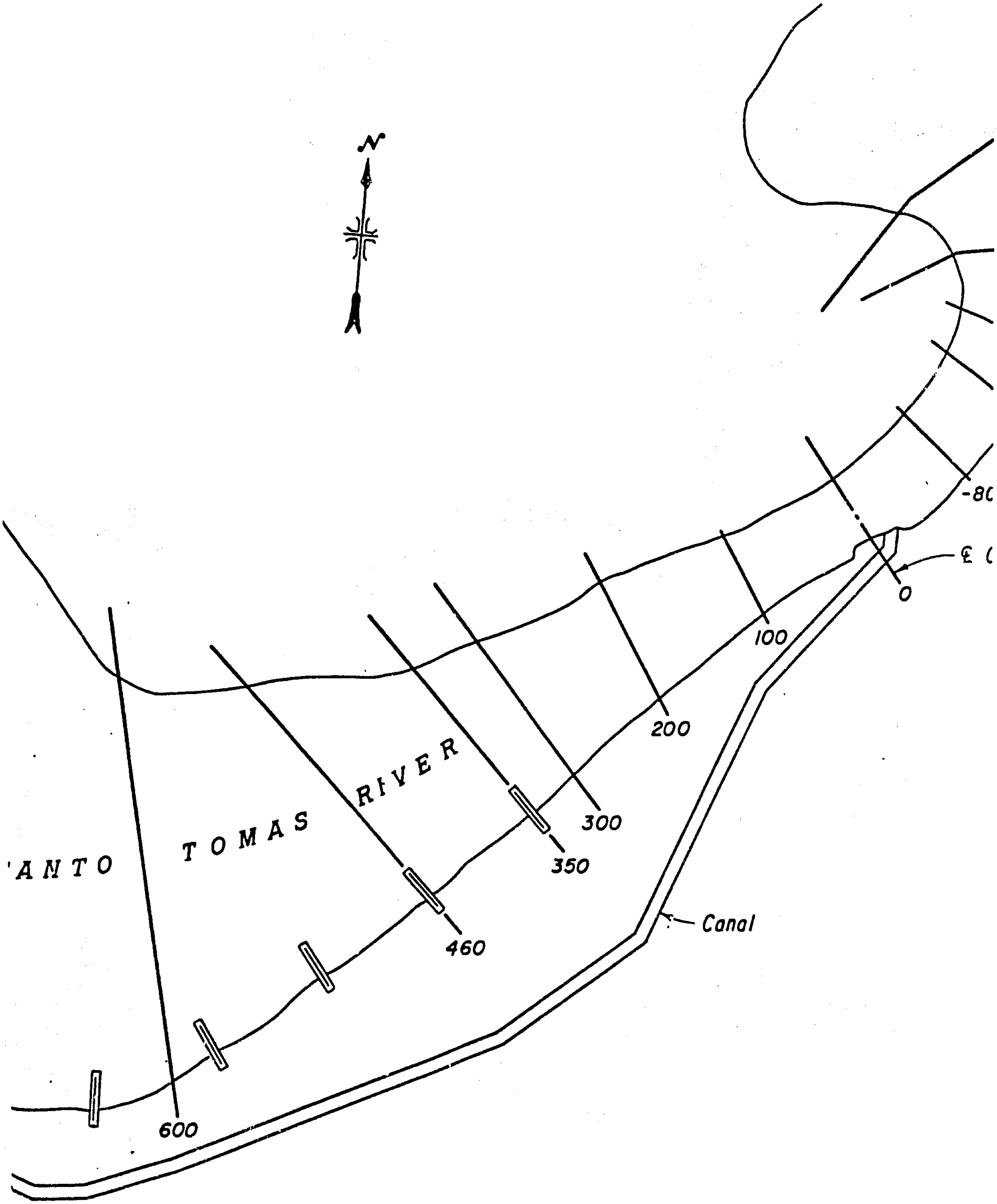

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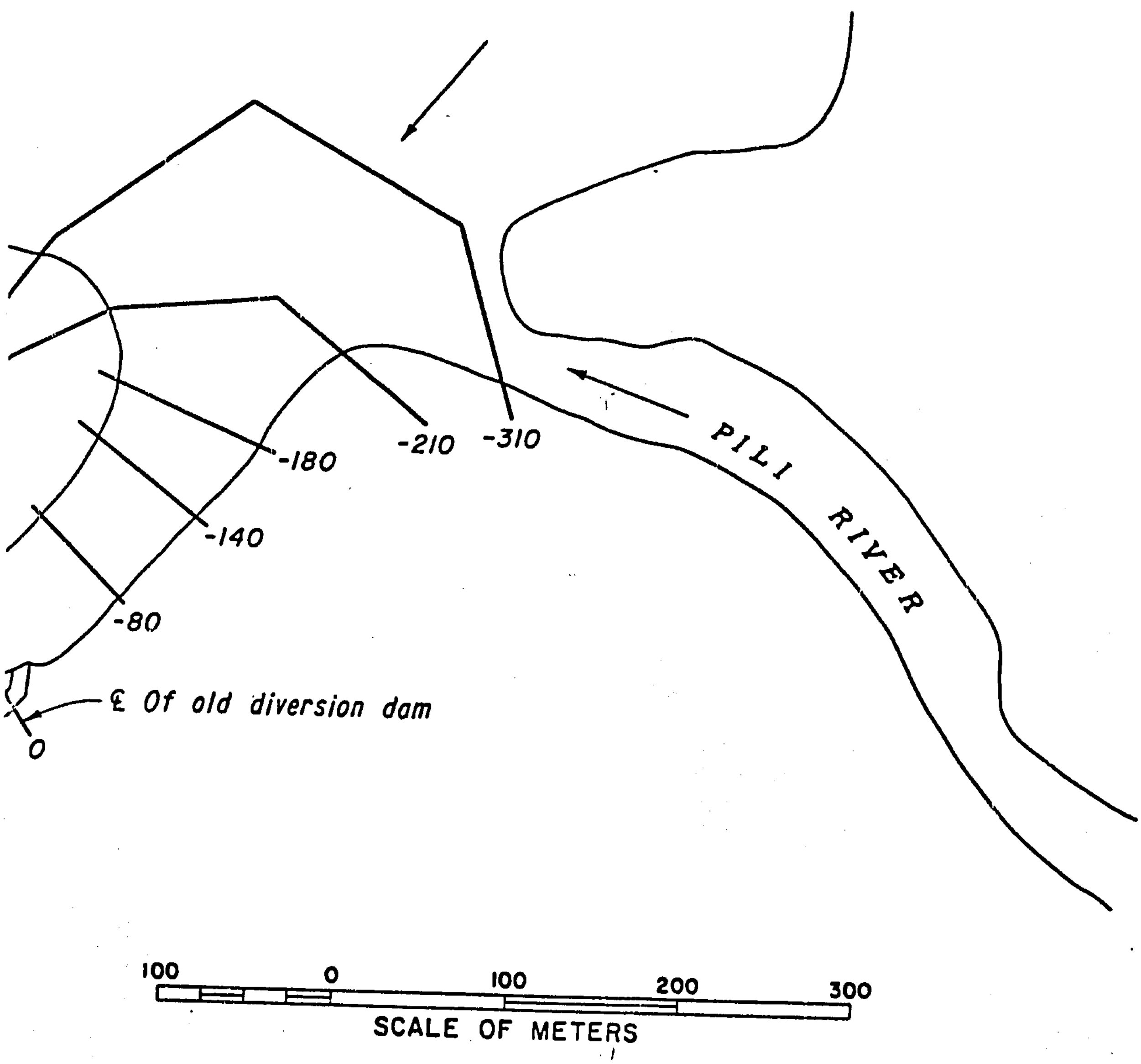
'ANTO TOMAS RIVER
Canal

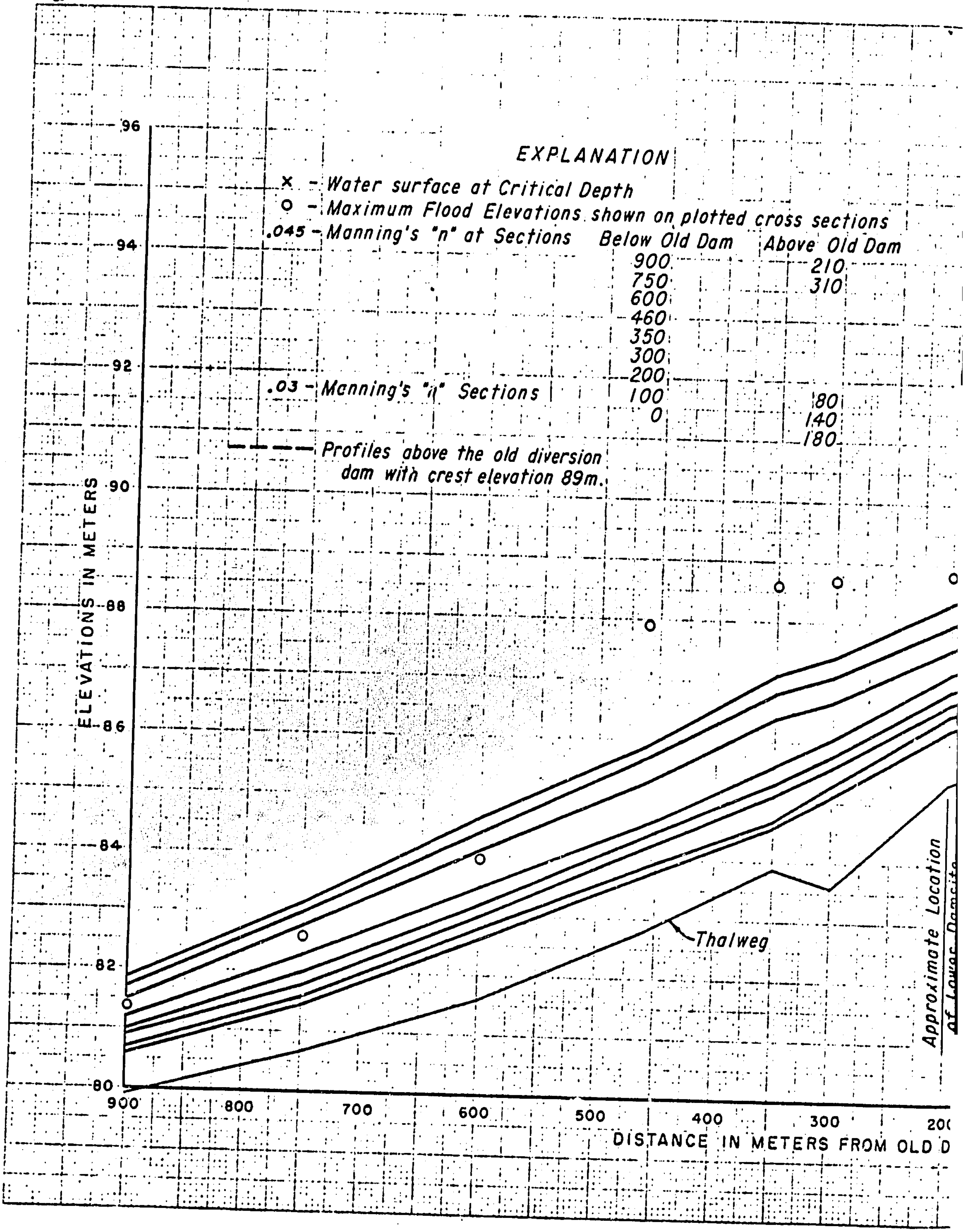


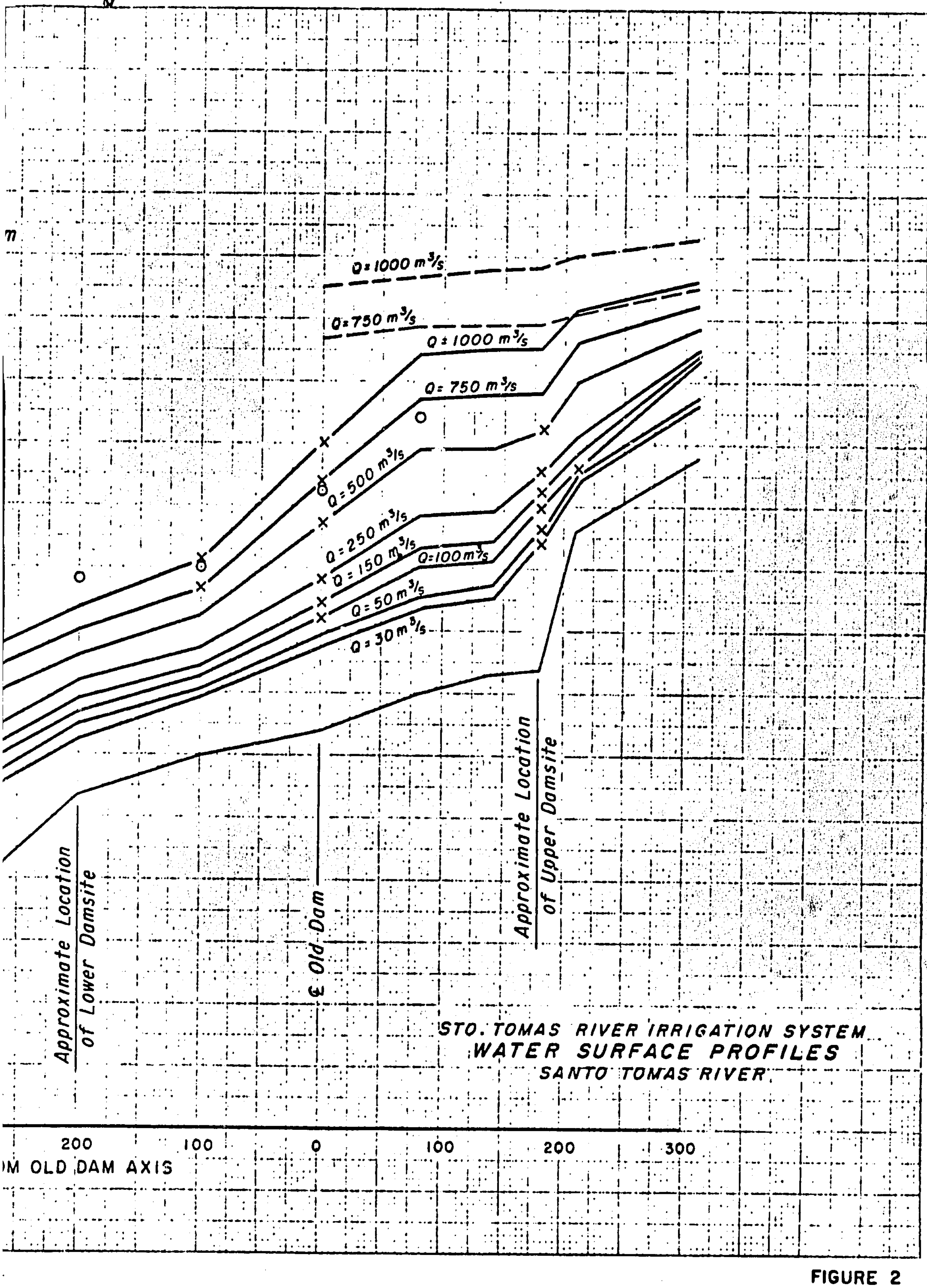
SANTO TOMAS RIVER IRRIGATION SYSTEM
LOCATION MAP

TAILWATER CROSS SECTIONS

SANTO TOMAS RIVER

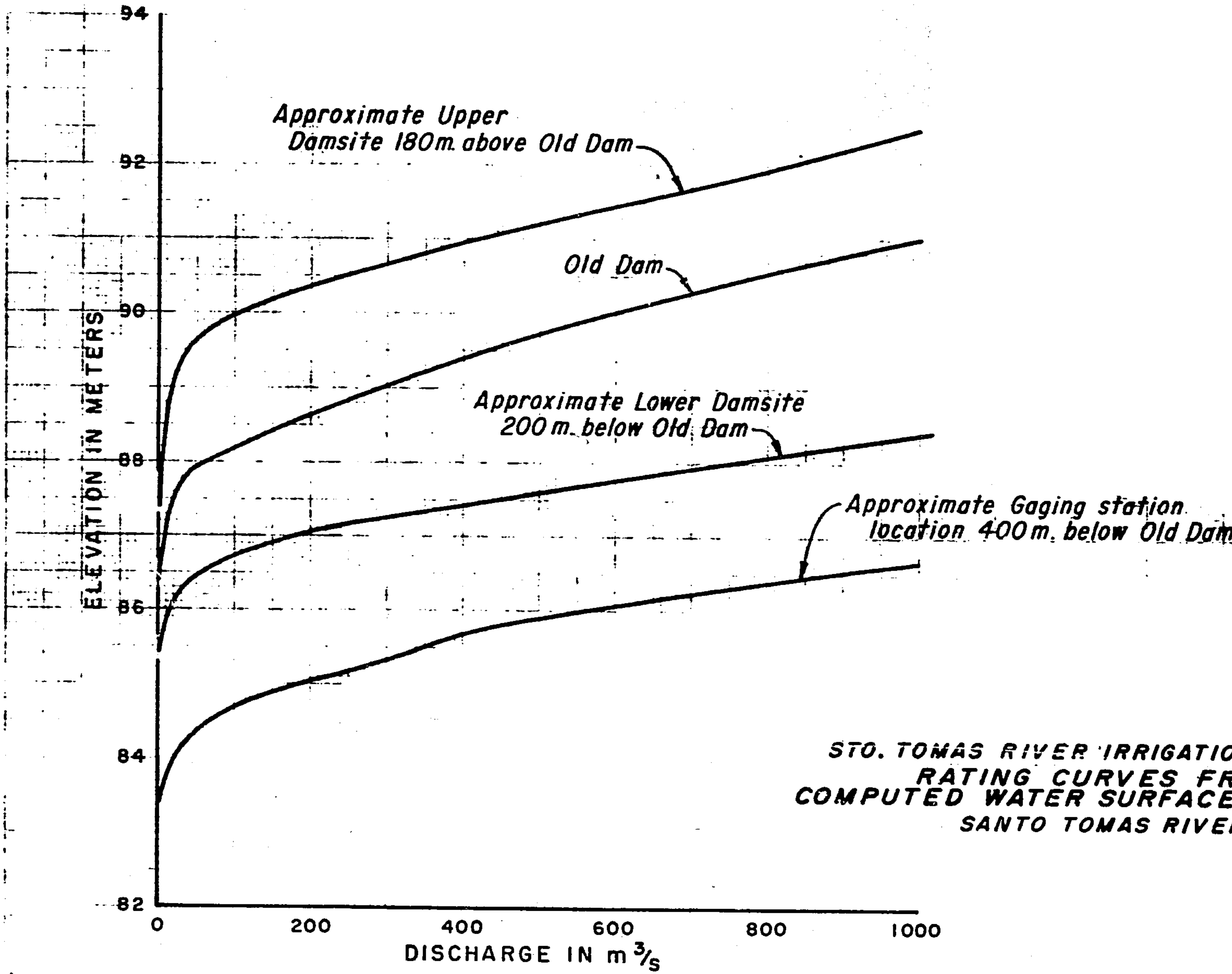
FIGURE 1


EXPLANATION
× - Water surface at Critical Depth
o - Maximum Flood Elevations shown on plotted cross sections
.045 - Manning's "n" at Sections Below Old Dam Above Old Dam
900 750 600 460 350 300 200 100 0
210 310
.03 - Manning's "n" Sections
80 140 180
Profiles above the old diversion dam with crest elevation 89m.
ELEVATIONS IN METERS
96 94 92 90 88 86 84 82 80
Thalweg
Approximate Location
900 800 700 600 500 400 300 200
DISTANCE IN METERS FROM OLD D



STO. TOMAS RIVER IRRIGATION SYSTEM
WATER SURFACE PROFILES
SANTO TOMAS RIVER

FIGURE 2



STO. TOMAS RIVER IRRIGATION SYSTEM
RATING CURVES FROM COMPUTED WATER SURFACE PROFILES
SANTO TOMAS RIVER

FIGURE 3

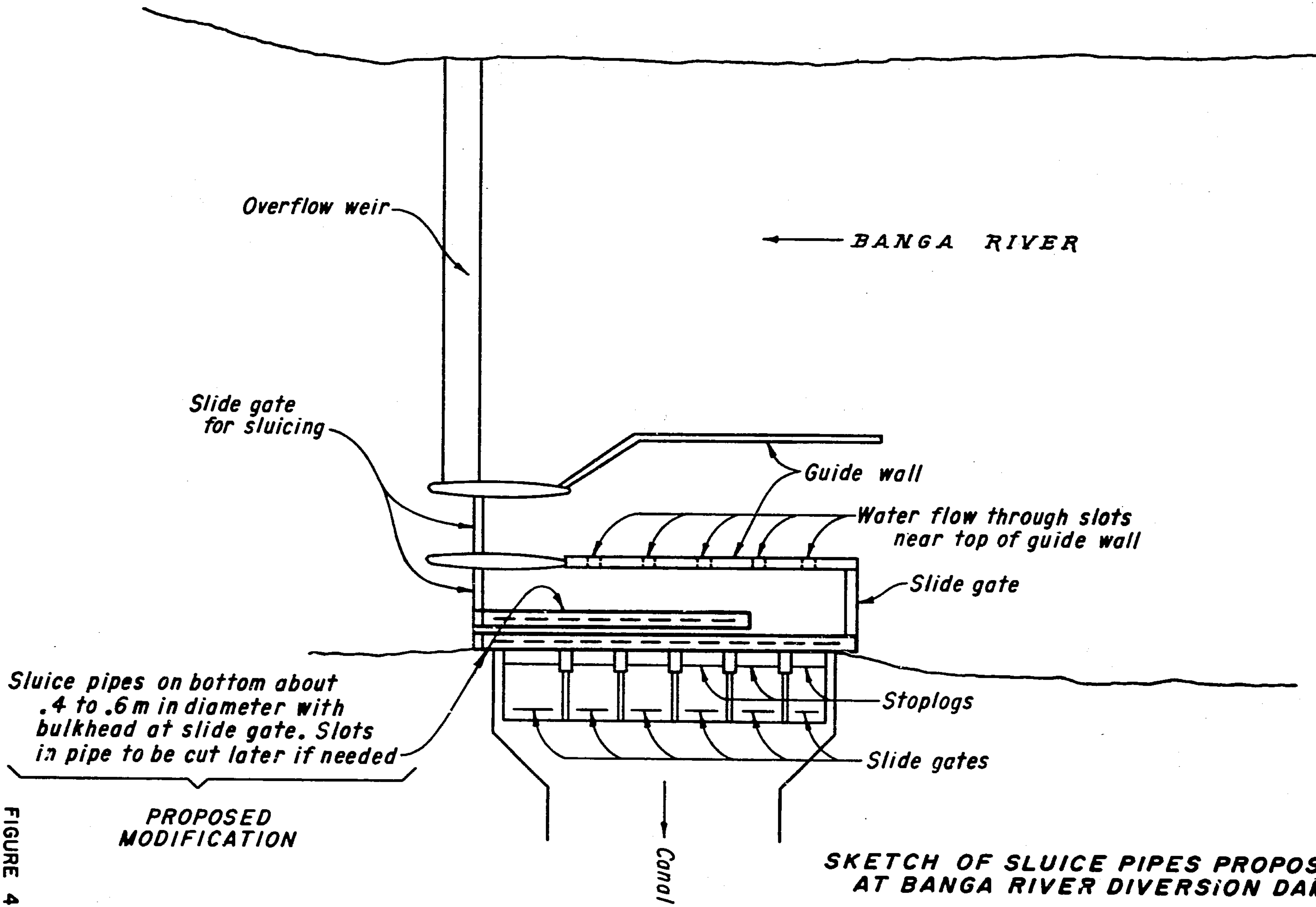
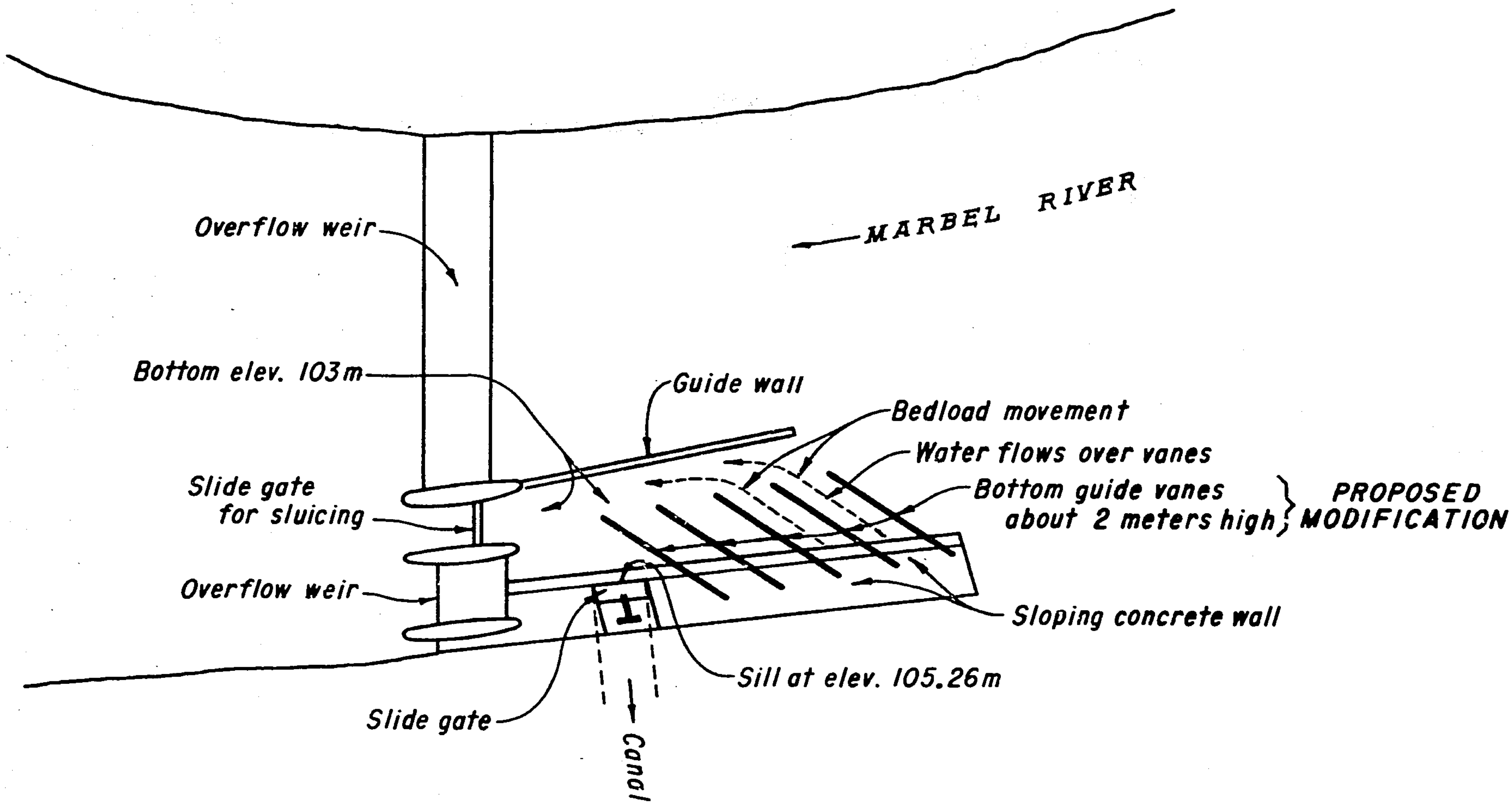


FIGURE 4

SKETCH OF SLUICE PIPES PROPOSED AT BANGA RIVER DIVERSION DAM



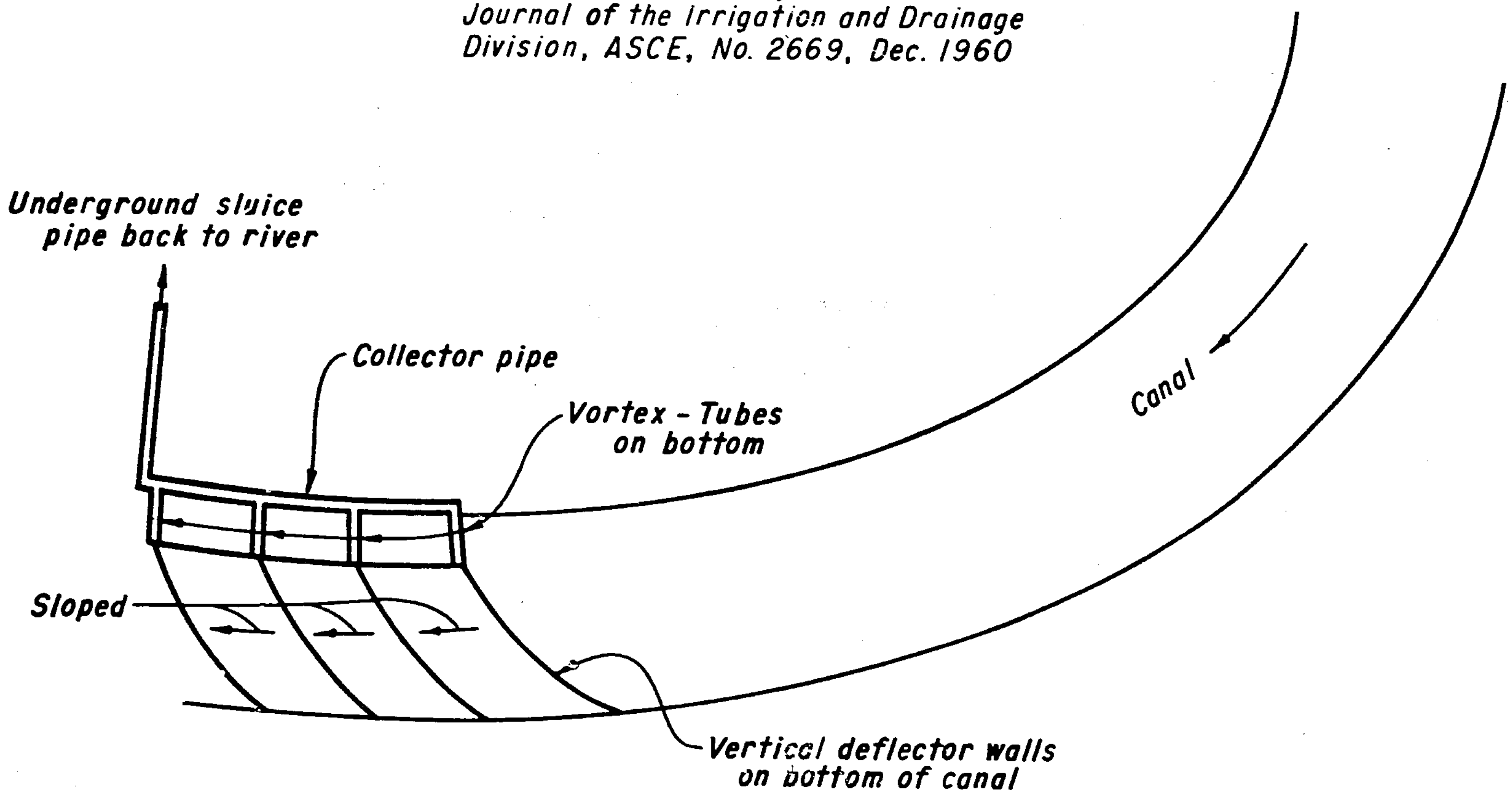
SKETCH OF BOTTOM GUIDE VANES
PROPOSED AT MARBEL RIVER DIVERSION DAM

FIGURE 5

REFERENCE

"Model and Prototype Studies of Sand Traps" by R.L. Parshall, Transactions of the ASCE, Vol. 117, 1952 p. 204

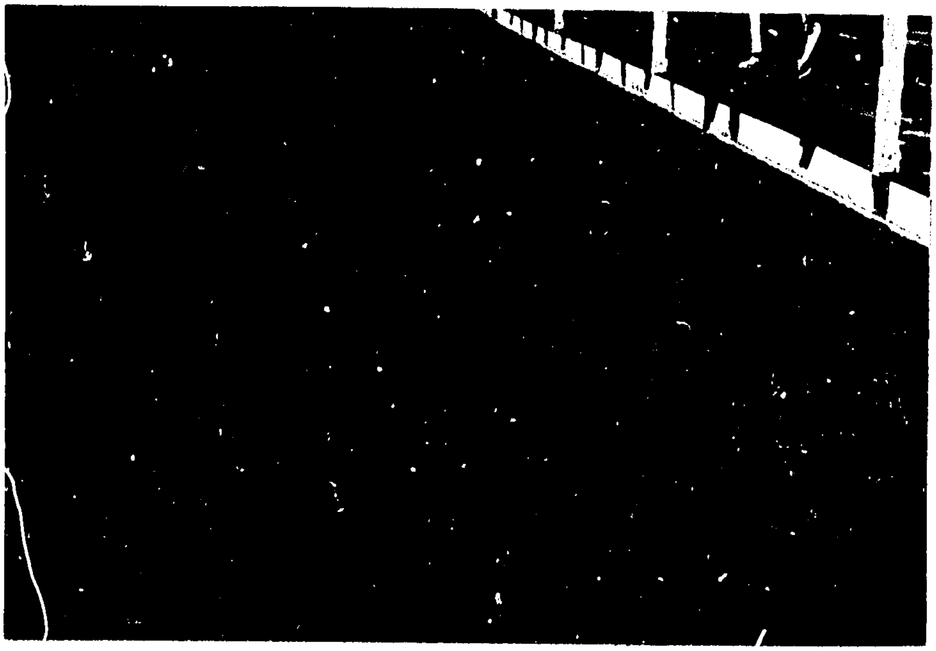
"Vortex Tube Sand Trap" by A.R. Robinson Journal of the Irrigation and Drainage Division, ASCE, No. 2669, Dec. 1960



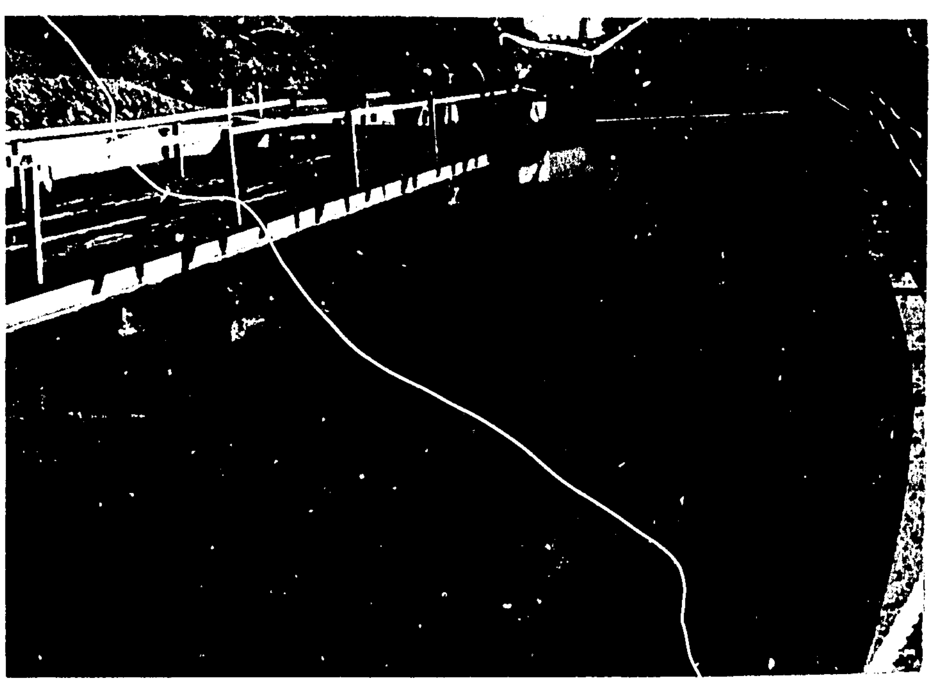
SKETCH OF RIFFLE-DEFLECTOR, VORTEX-TUBE SAND TRAP IN SUPPLY CANAL

FIGURE 6

23



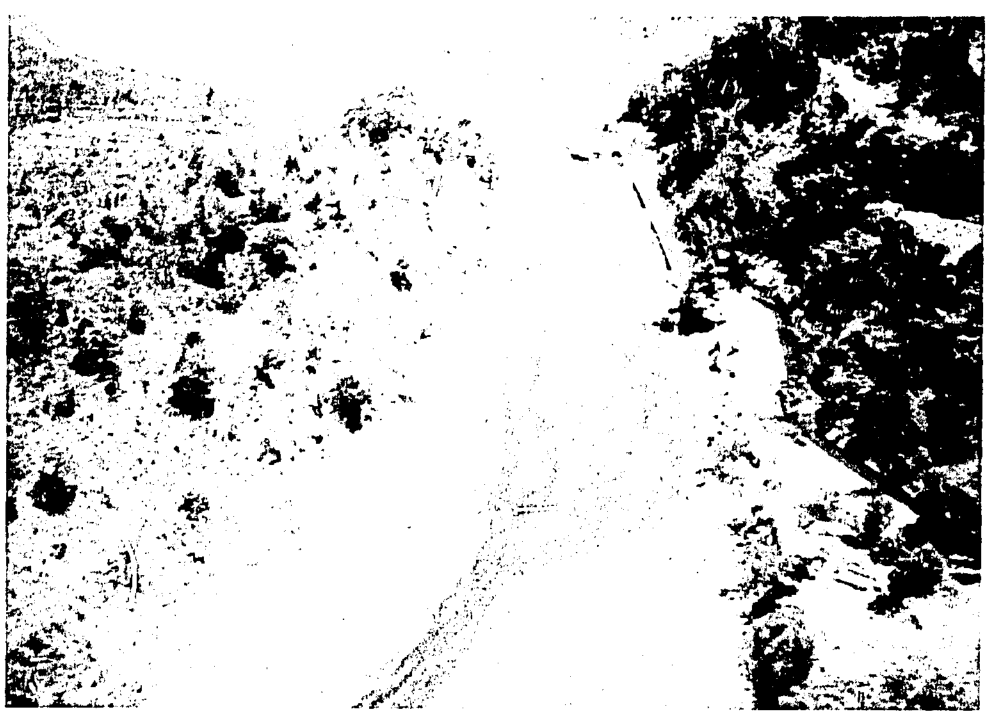
Upstream view of riffle-deflector vortex-tube sand trap installation in supply canal for Colorado Fuel and Iron Co. (Pueblo, Colorado). Capacity equals 250 ft^3/s.



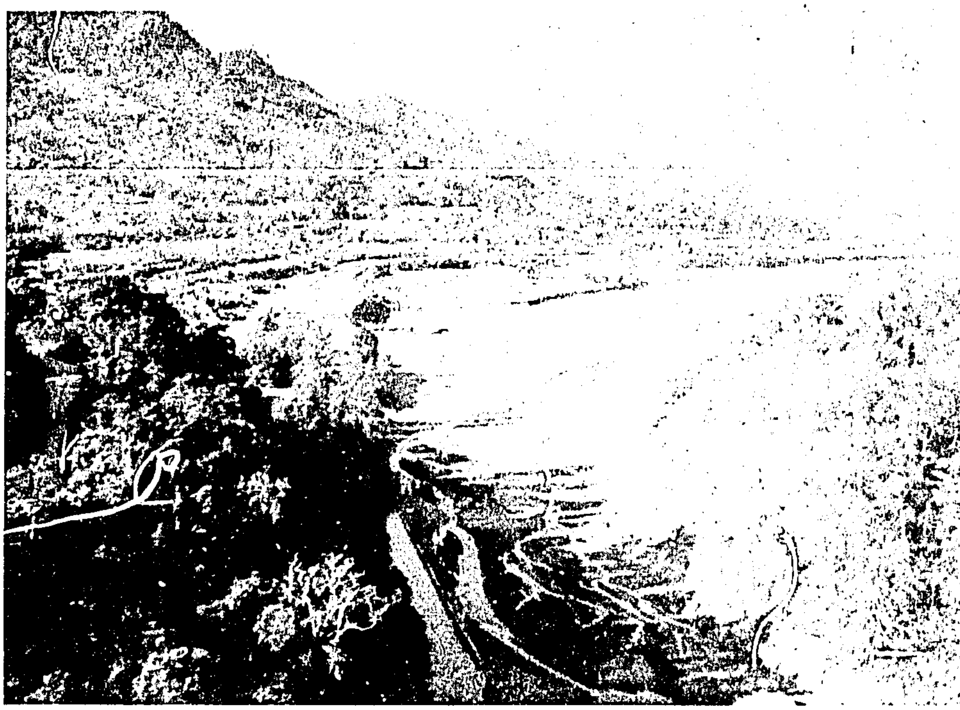
Downstream view of riffle-deflector vortex-tube sand trap installation in supply canal for Colorado Fuel and Iron Co. (Pueblo, Colorado). Capacity equals 250 ft^3/s.

Figure 7

Photograph 1. - Aerial view looking downstream showing narrow river section at old Santo Tomas River Diversion damsite.

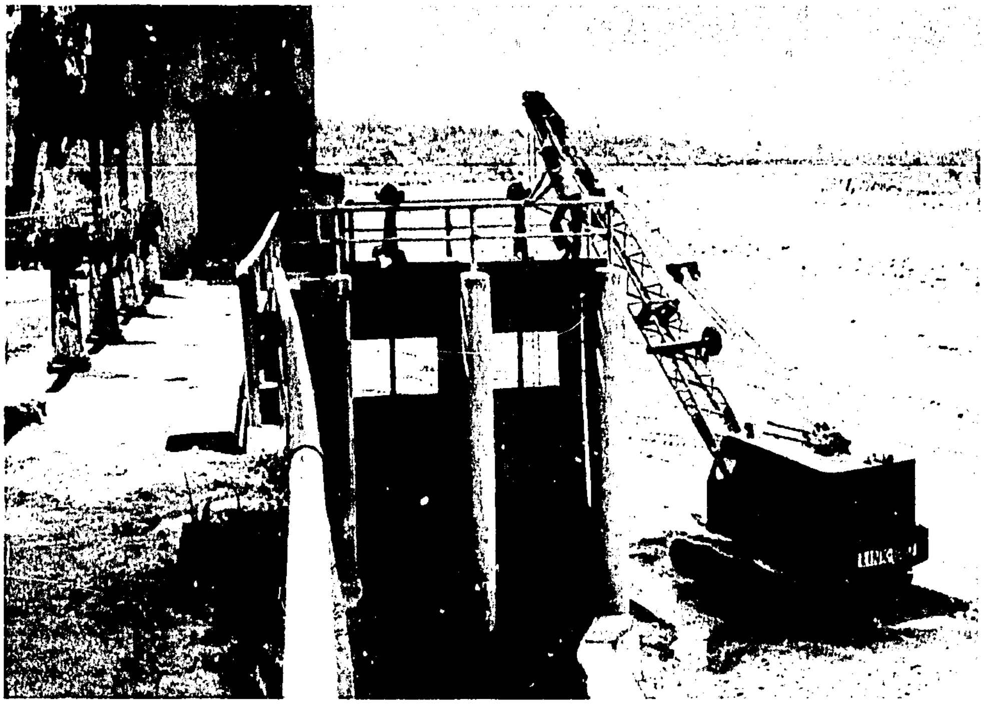


Photograph 2. - Aerial view looking upstream showing narrow river section at old Santo Tomas River Diversion damsite.

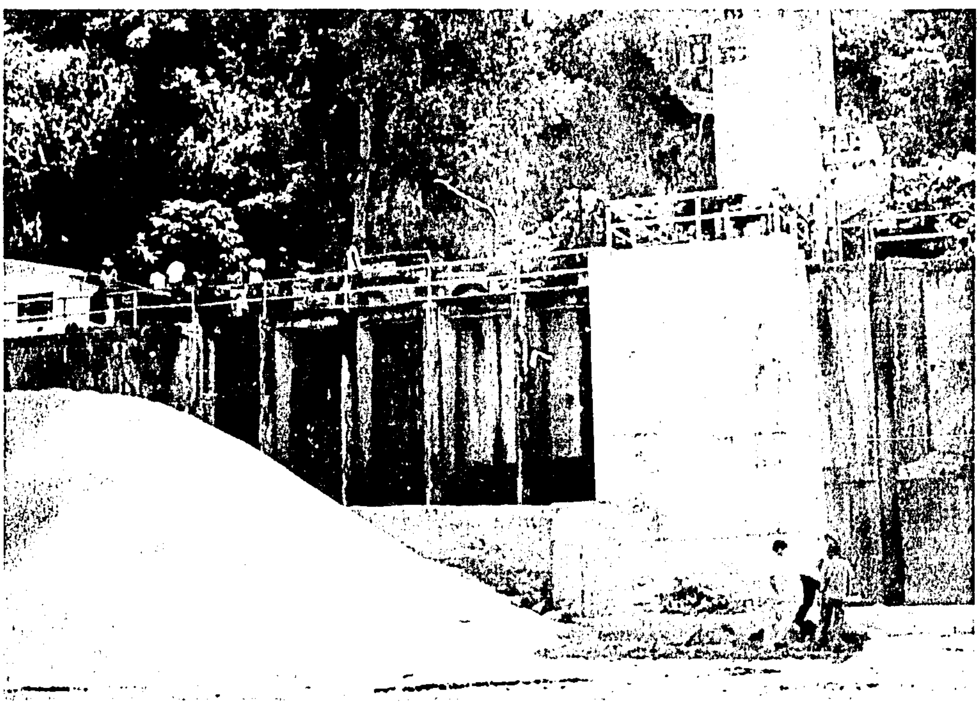


Photograph 3. - Downstream view of Santo Tomas River with old Santo Tomas River Diversion damsite in foreground.

Photograph 4. - Typical view of Santo Tomas River about 1 mile below old Santo Tomas River Diversion damsite.



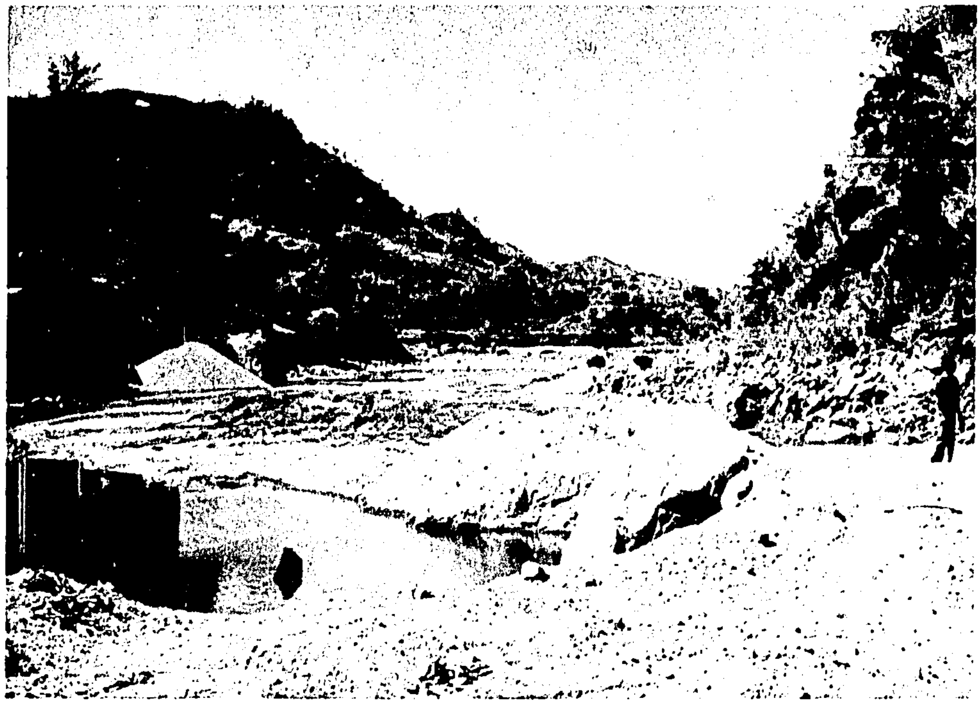
Photograph 5. - Headworks remaining at Santo Tomas River Diversion damsite.



Photograph 6. - Headworks remaining at Santo Tomas River Diversion damsite.

Photograph 7. - Upstream view from near headworks at Santo Tomas River Diversion damsite showing removal of sheet piling.

Photograph 8. - Upstream view from near headworks at Santo Tomas River Diversion damsite showing type of rock used for riprap along diversion wall.

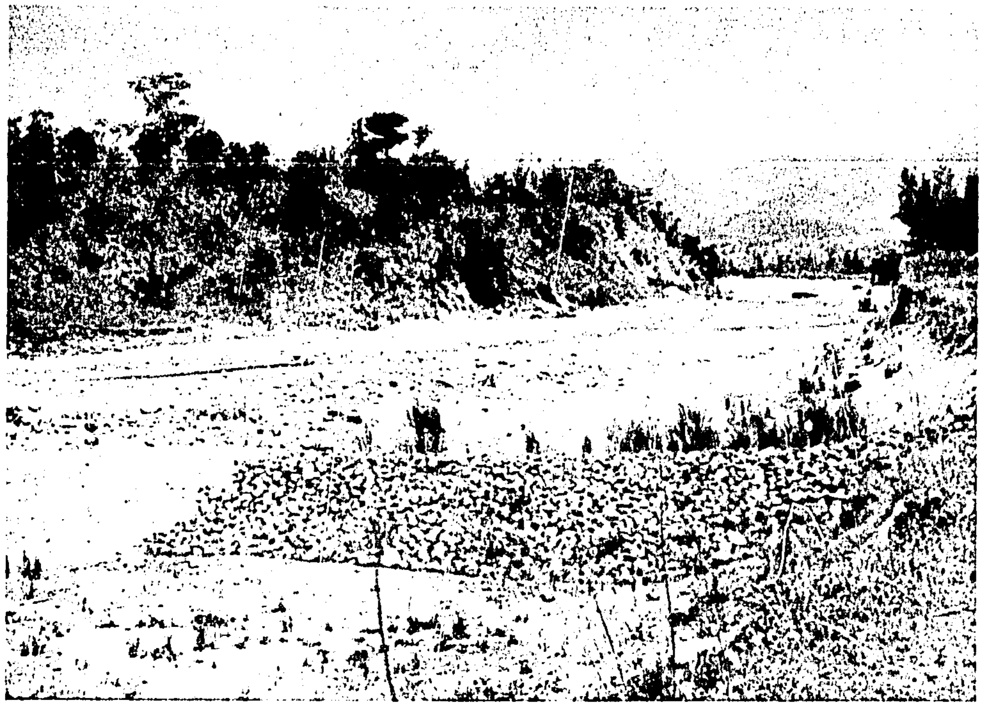


Photograph 9. - Looking downstream on Santo Tomas River from above sheet piling used the past year for diversion.

Photograph 10. - Downstream view on Santo Tomas River showing right abutment to old Santo Tomas River Diversion damsite.

Photograph 11. - Looking upstream on Santo Tomas River showing rock in channel below old diversion damsite.

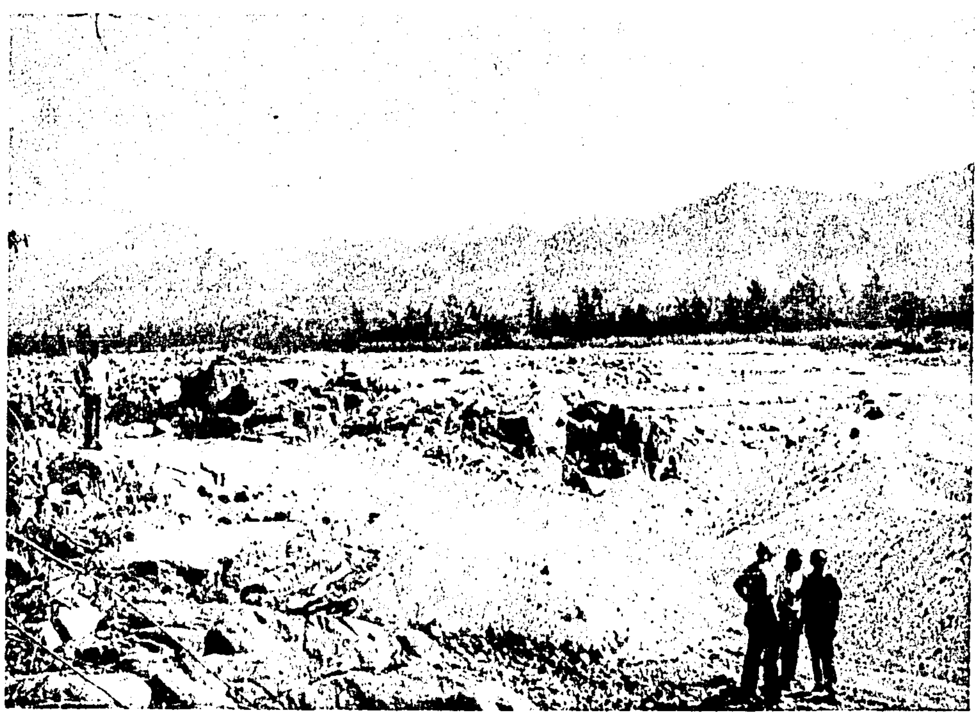
Photograph 12. - Upstream view on Santo Tomas River showing rock in streambed above narrow river section at old diversion damsite.



Photograph 13. - Upstream view on Santo Tomas River from left bank near No. 1 Spur Dike about 350 meters below old diversion damsite.

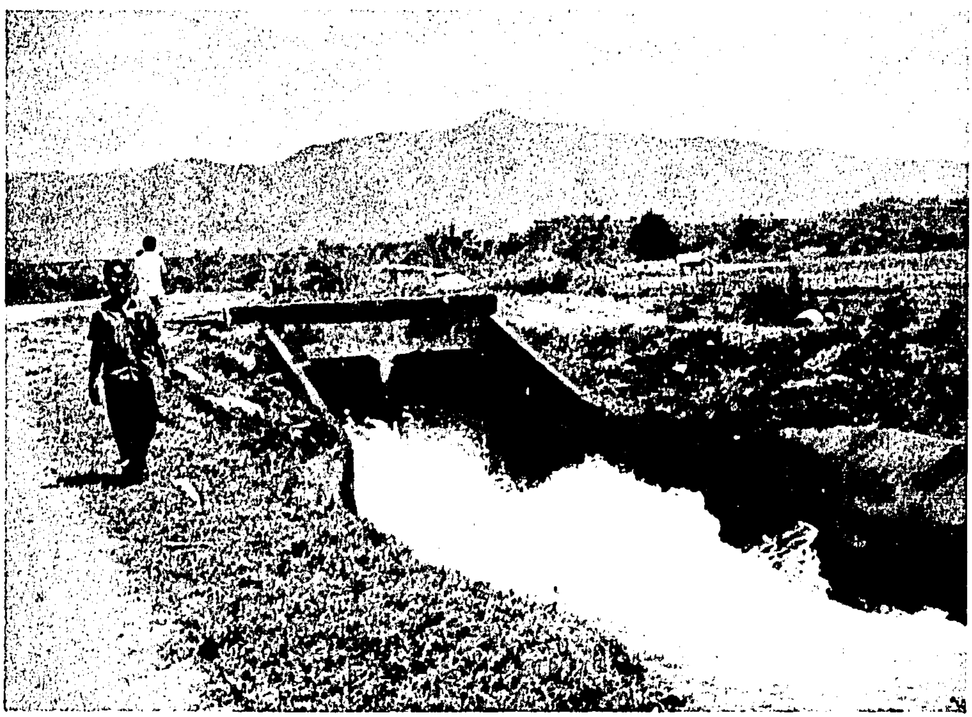
Photograph 14. - Looking across Santo Tomas River from left bank showing right bank abutment to a proposed diversion damsite about 200 meters below old diversion damsite.

Photograph 15. - Looking across Santo Tomas River from right bank showing left bank abutment to a proposed diversion damsite about 180 meters above the old diversion damsite.

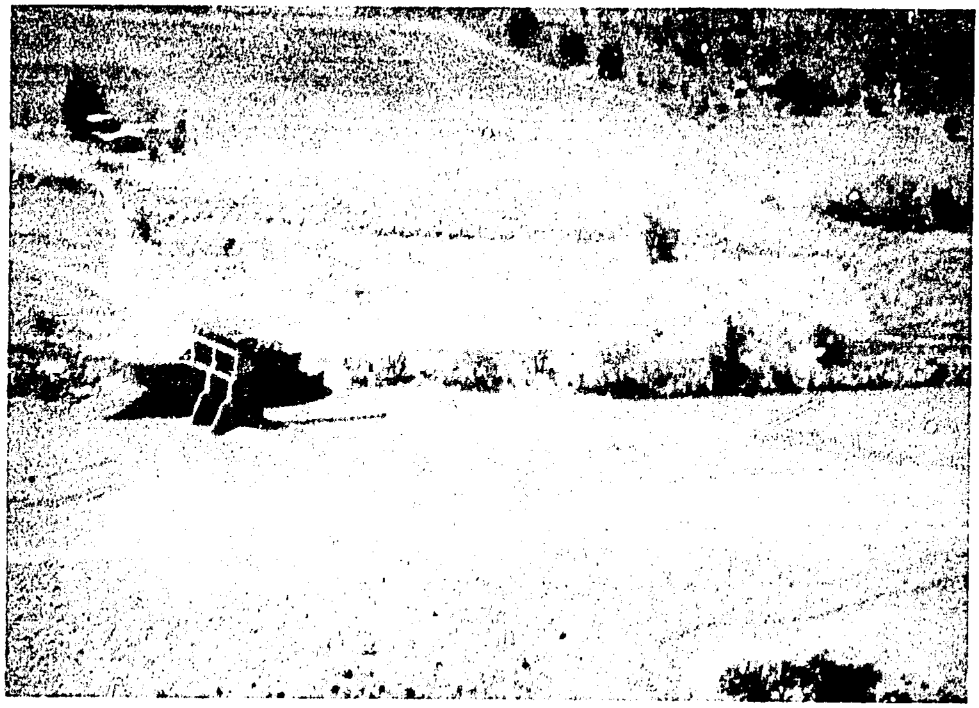


Photograph 16. - Upstream view of rock on right bank abutment to a proposed diversion damsite about 180 meters above the old diversion damsite.

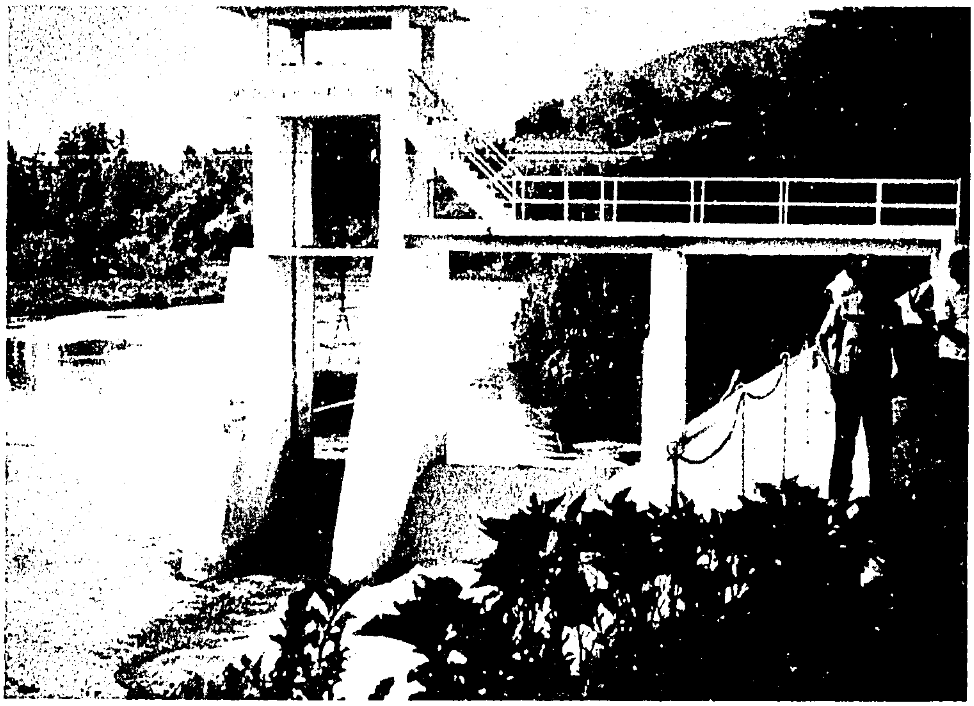
Photograph 17. - Canal below Santo Tomas River Diversion showing type of material cleaned from concrete-lined canal section.



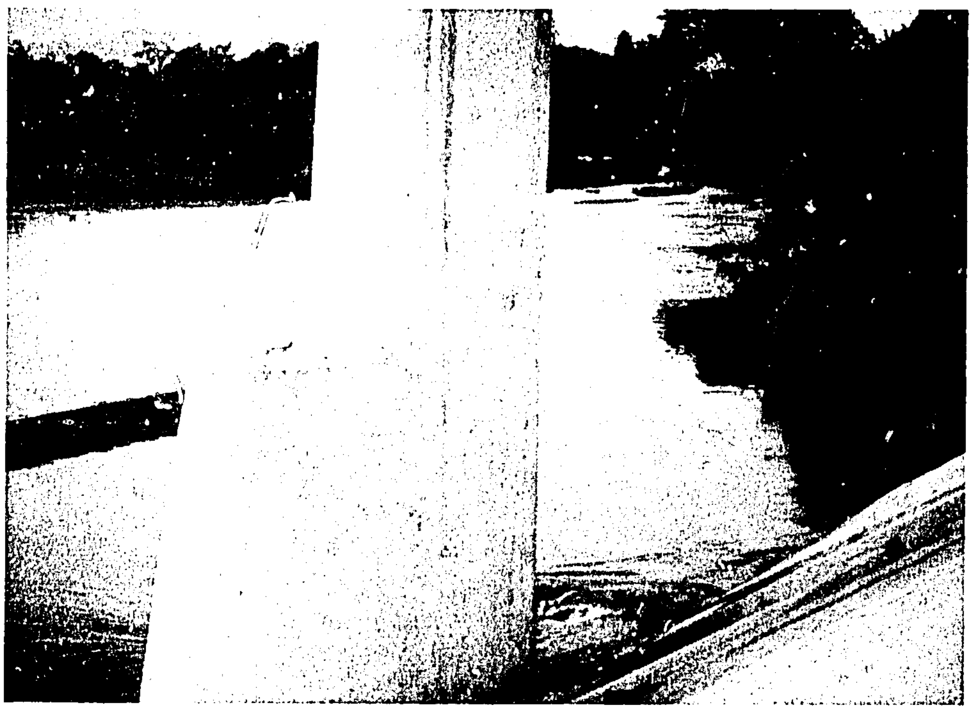
Photograph 18. Drop structure on canal below Santo Tomas River Diversion damsite.



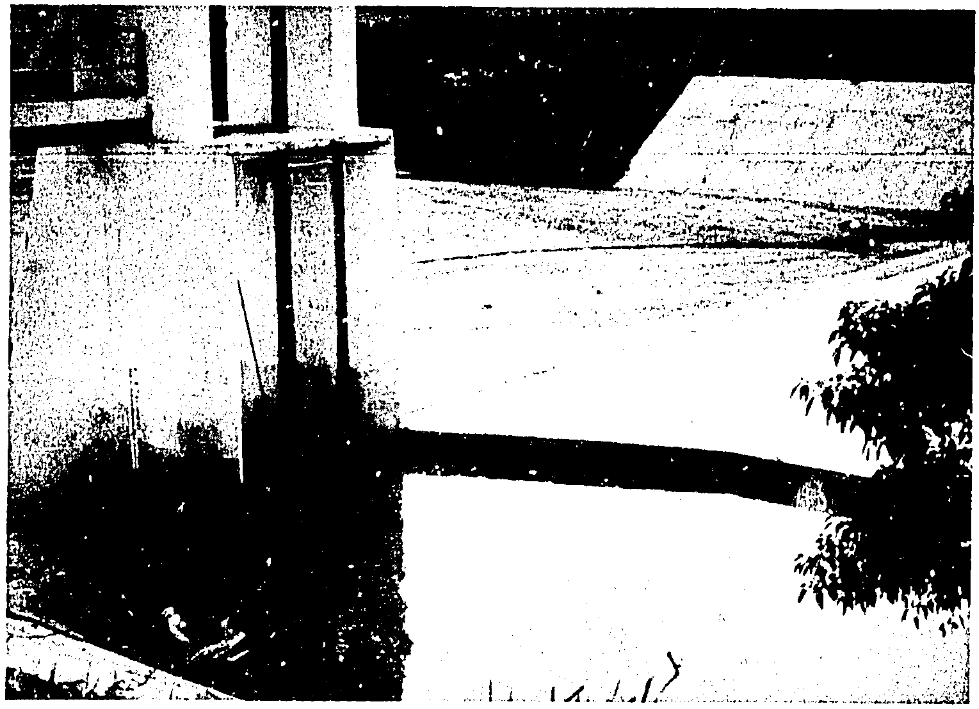
Photograph 19. - View of Diversion Dam and headworks on Agno River near San Manuel.



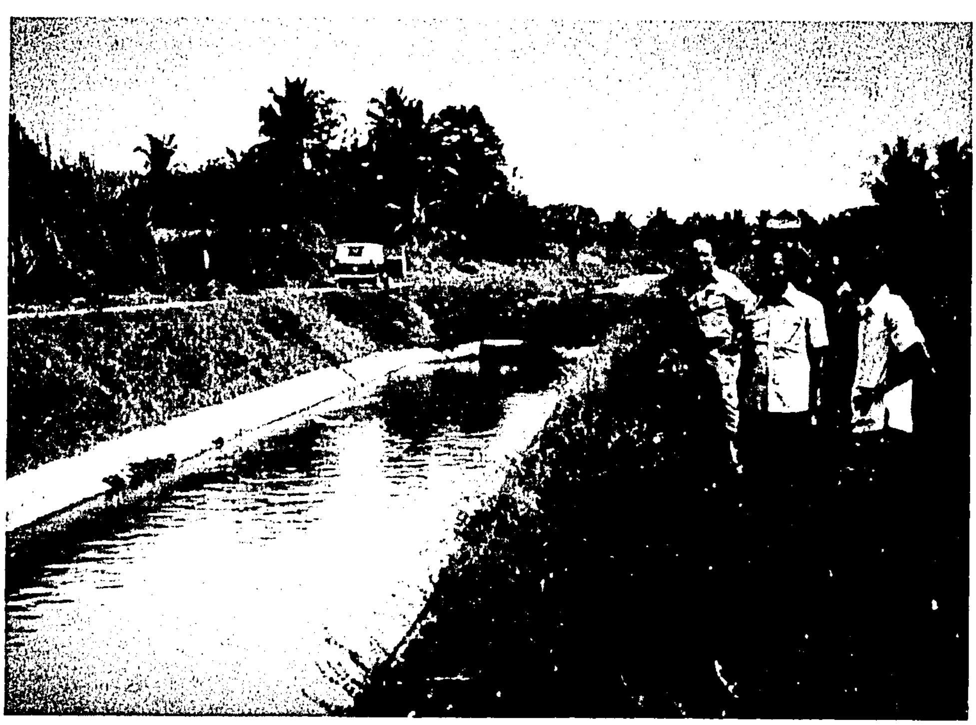
Photograph 20. - Looking upstream at diversion dam. Headworks and sluice gate on Marbel River.



Photograph 21. - Headworks at Marbel River Diversion Dam showing vortex in flow pattern above sloped slide gate diverting water into canal.

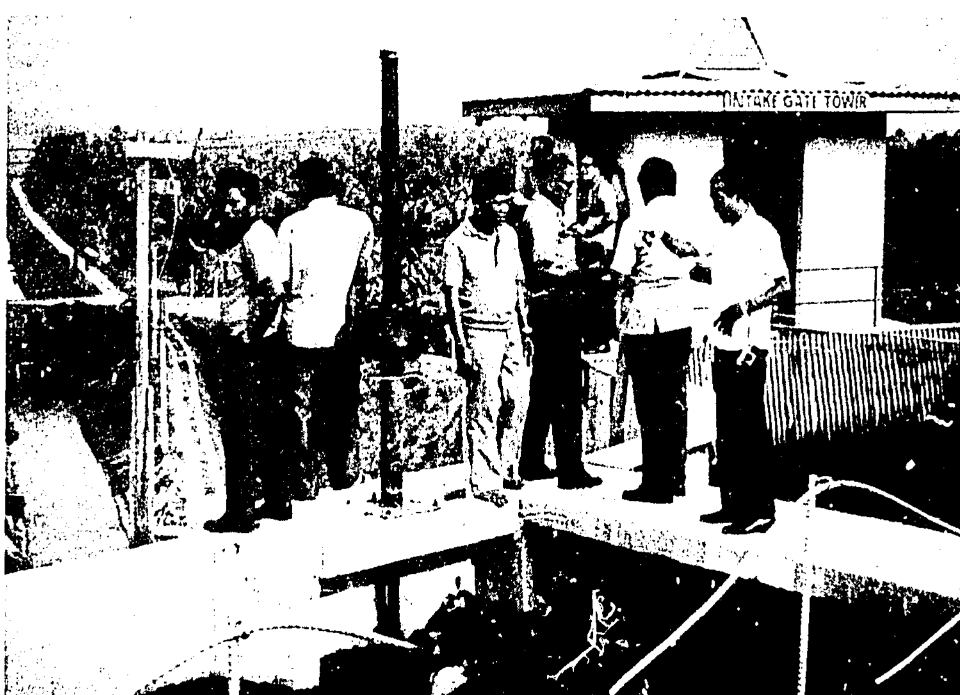


Photograph 22. - Marbel River Diversion Dam and headworks.

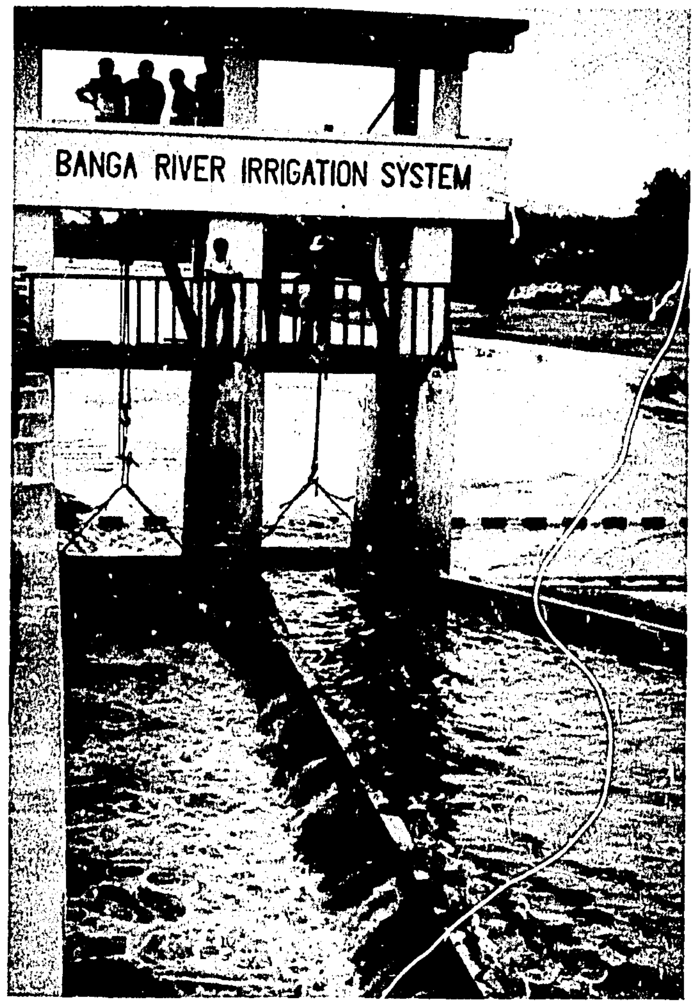


Photograph 23. - Looking down the canal below Marbel River Diversion Dam where bend in canal could be utilized to sluice bottom sediments back to the river.

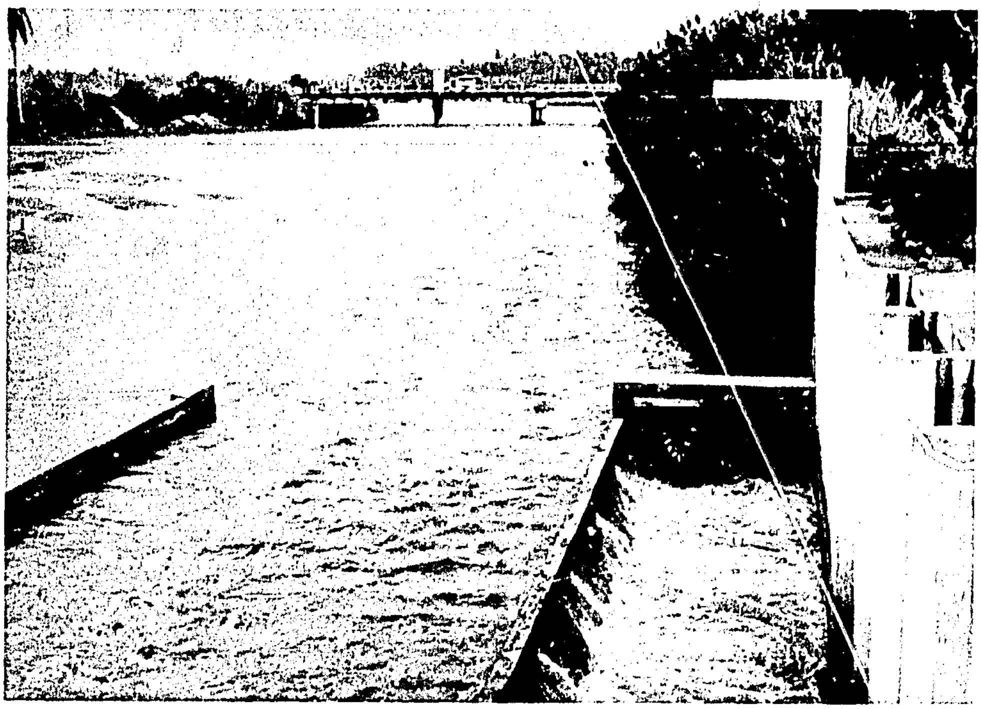
Photograph 24. - Looking up the canal several miles below Marbel River Diversion Dam at the powerplant intake structure showing the water being skimmed used for cooling purposes.



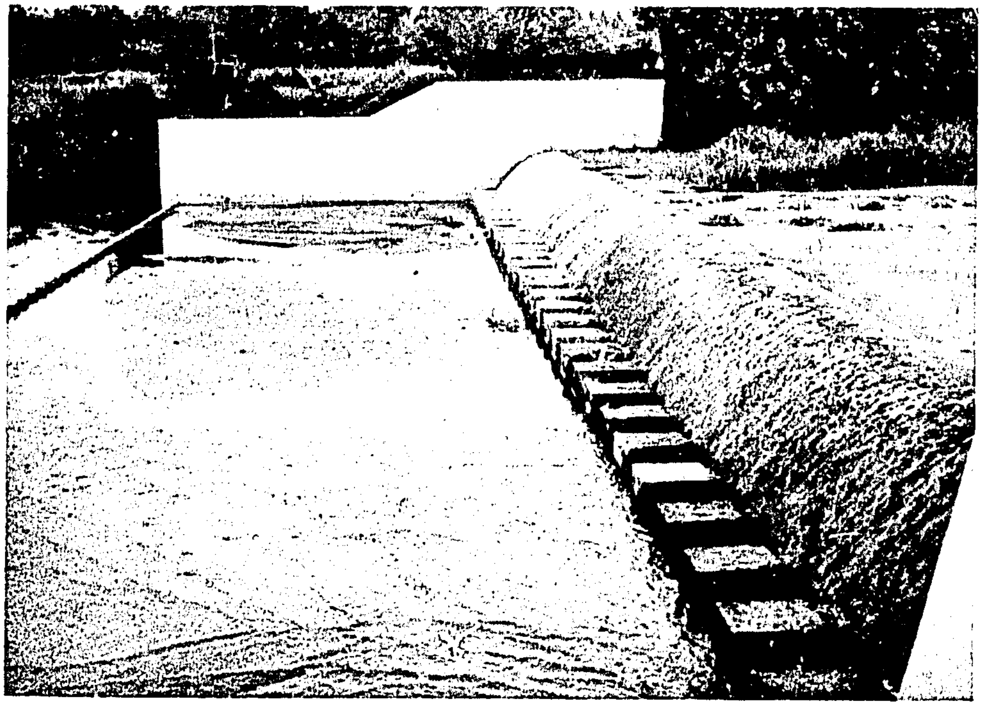
Photograph 25. - Intake to powerplant below Marbel River Diversion Dam.



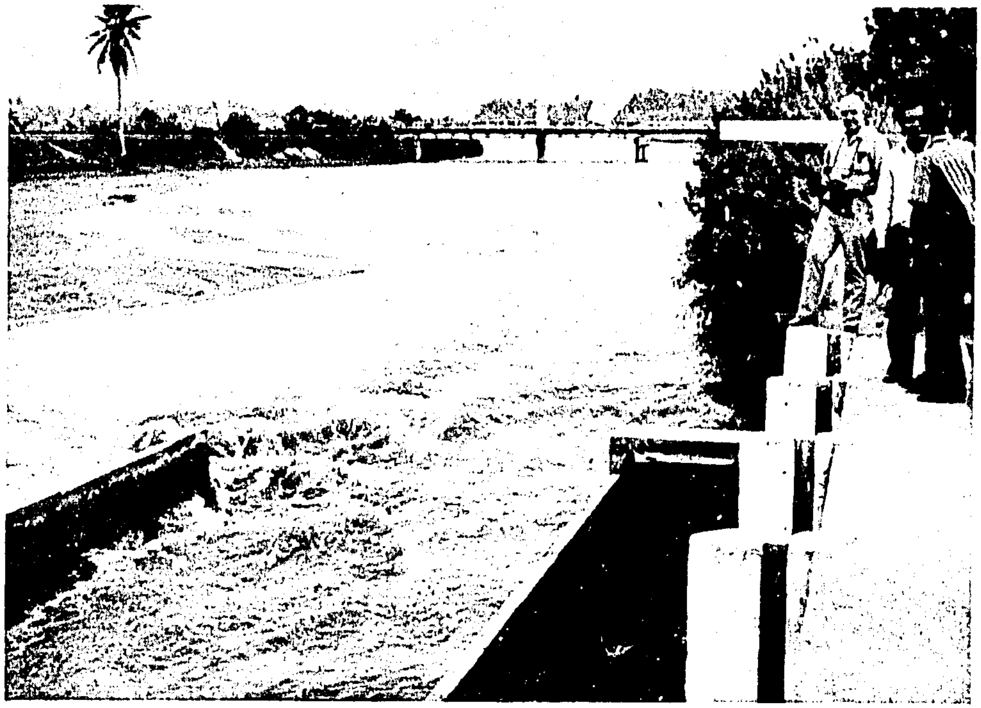
Photograph 26. - Headworks at Banga River Irrigation System Diversion Dam.



Photograph 27. - Looking upstream from Banga River Diversion Dam and headworks.

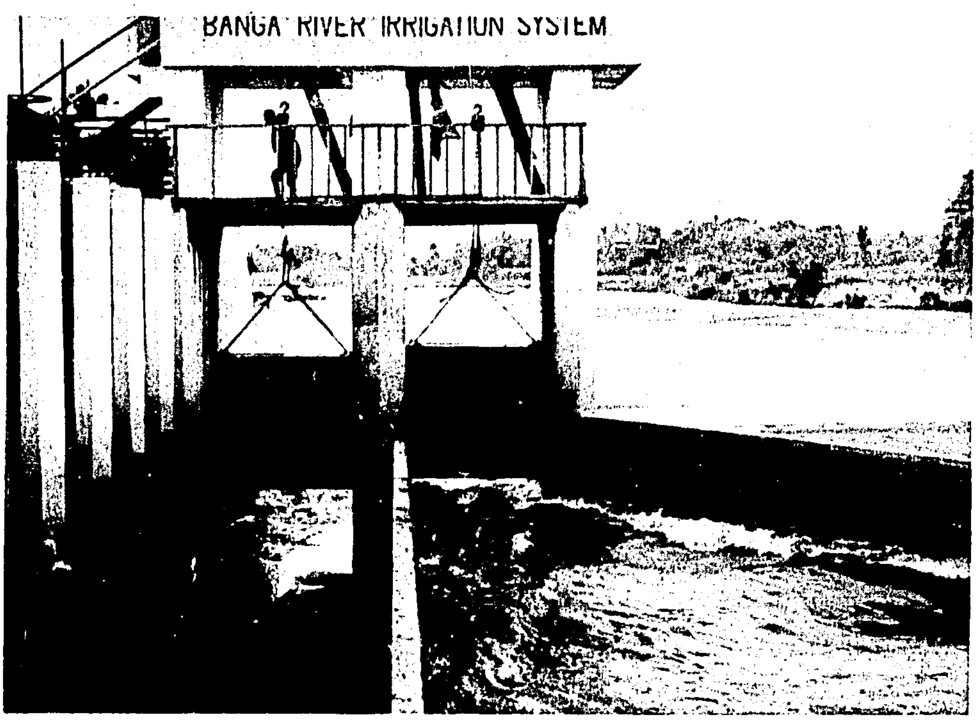


Photograph 28. - Banga River Diversion Dam.



Photograph 29. - Upstream view during sluicing operations at the Banga River Diversion Dam.

Photograph 30. - Looking upstream from below the sluice gates at Banga River Diversion Dam during a sluicing operation.



Photograph 31. - Downstream view of headworks and sluice gates at Banga River Diversion Dam during a sluicing operation.