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HYDRAULIC DESIGN FEATURES OF OUTLET WORKS FOR TARBELA LEFT BANK IRRIGATION TUNNEL

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M.Aslam Rasheed[1] *& Bashir Ahmad*[2]

1.0 INTRODUCTION

1.1 Brief Background of the Project

Tarbela Dam, one of the major projects in the system of works implemented under the Indus Basin Treaty and the Indus Basin Development Fund Agreement of 1960, is located across river Indus, about 29 miles upstream of Attock where the river leaves mountainous country & enters flatter alluvial terrain.The maximum height of the Dam, above the foundation is 470 feet and the length between the abutments is 9,000 feet.The reservoir extends to about 50 miles upstream of the Dam providing a gross storage of 11.1 MAF at maximum reservoir elevation of 1550 feet with a maximum drawdown to elevation 1300 feet. The net usable capacity of the reservoir is 9.3 MAF.The Project, in many aspects establishes new precedents for hydraulic structures and hydromachinery in terms of size, capacity, design and operational details.

The Tarbela Project originally envisaged construction of four tunnels on the right bank of the river as shown on Figure-1,Layout Plan. Two of these tunnels (Tunnels 1 & 2) were used for river diversion during the closure of the river. These tunnels will be finally used for generation of electric power, each serving 4 turbine generator units. Tunnels 3 & 4 are meant for irrigation releases but ultimately Tunnel 3 is scheduled to be converted to power generation. The ultimate power generation capacity of the Project will be 2,100 MW.

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The hydrological studies based on historical data of river Indus indicated that for many years of record, the available water release capacity at Tarbela through the four right bank tunnels was short of the available river flows during the critical months of June and July. The irrigation requirements warranted that all the river flows at Tarbela be passed on downstream during these months. The limited release capacity would have resulted in reduction in the available irrigation supplies which would have affected badly the Kharif crops. Accordingly, the Pakistan Government decided to construct the Left Bank Irrigation Tunnel (LBIT) or the 5th Tunnel, through the rock on the left bank between Service Spillways and Auxiliary Dam No.2, for augmenting the low level outlet capacity during June and July when the reservoir level would be low, between 1,300 & 1,370.

The status on construction of the Tarbela Dam Project required that the Intake Structure and about 700 feet length of the Tunnel be constructed immediately so that these were completed before commissioning of the Tarbela Project in 1974. Accordingly, the Intake Structure and about 700 feet long 45 feet diameter concrete lined portion of the Tunnel (called the Stub Tunnel) were constructed under the main Tarbela construction contract through the Tarbela Joint Venture (TJV). The engineering services for these structures were provided by M/s. Tippets Abbett McCarthy Stratton International Corporation (TAMS) of New York.

The Stub Tunnel was to be followed by a 36 feet diameter steel-Lined Tunnel and its appurtenant works comprising Gate Passage and Transitions, Gate Shaft, Service and Bulkhead Gates, Outlet Control Structure, Outlet Control Radial Gates and arrangements for energy dissipation at the Tunnel outlet.

The Pakistan Government in line with its policy of self-reliance entrusted the complete engineering services for these works to National Engineering Services (Pakistan) Ltd., (NESPAK), a firm of consulting engineers established by the Government of Pakistan, as a limited company in the private sector. The contract for the construction of these works of the Left Bank Irrigation

Tunnel was awarded, after negotiations, to Pakistan Tarbela Consortium (PTC). The PTC comprised; National Construction Company (NCC), established by the Government of Pakistan as a limited company in the private sector like NESPAK and the Tarbela Joint Venture (TJV), the main contractor for the Tarbela Dam Project.

The engineering services for the remaining portion of the Tunnel included investigations, design, supervision of construction and equipment installation and administration of a large number of supply and construction contracts. The design work for the Tunnel was carried out at the Central Design Offices of NESPAK, Lahore. Various Speciality Divisions were assigned the work on different aspects pertaining to their specialities. The authors in their capacity as engineers in Hydraulics Division of NESPAK, were closely involved in the hydraulic designs of the various components of the Tunnel specially with the hydraulic design of the outlet works.

This paper describes the hydraulic considerations in the design of various components of the outlet works of LBIT including the chute profile, the bucket, side wall flare, central pier and the plunge pool. The results of the model studies which were conducted at Irrigation Research Institute Lahore and Nandipur, and their application to the design are also included.

1.2 Description of the Tunnel

The Left Bank Irrigation Tunnel (LBIT) is located on the Left Abutment of the Tarbela Dam between the Service Spillway and the Auxiliary Dam No.2. In its first 923 feet length the Tunnel is 45 feet in diameter and is concrete lined. This includes the 223 feet long Intake Structure. The concrete lined portion is designated as "Stub Tunnel". This is followed by a Gate Shaft including the Gate Passage and Transitions. It comprises an 80 feet transition on the upstream and a 100 feet transition on the downstream of the 45 feet long Gate Passage. Two Service Gates each 45 feet by 13.5 feet, separated by a central pier, close the two rectangular sections. A pair of Bulkhead Gates has been provided immediately upstream of the Service Gates. The same pair of Bulkhead Gates can be used at the Tunnel Intake as Stoplogs to close the Tunnel whenever necessary.

The Tunnel downstream of the Gate Passage and Transitions is steel-lined and is 36 feet in diameter and 1912 feet in length. It is lined with one inch thick steel plate backed by reinforced concrete filling about 4 feet thick. For this reason the excavated tunnel diameter was approximately 44 feet. The Steel-Lined Tunnel bifurcates into two 24 feet diameter branches with transitions which terminate into two rectangular sections each 16 feet wide and 24 feet high. Radial gates have been provided at the end of these sections for controlling the flow from the Tunnels.

Downstream of the Outlet Gates, there are two 62 feet long and 18'-2" wide steel lined channel sections. These are followed by a concrete lined chute which terminates into a Flip bucket that leads the water away into the spillway channel. A plan and profile of the Tunnel and its outlet works is shown on Figure-2.

The Intake of the Tunnel is an ungated structure about 113 feet high. Grooves for insertion of stoplogs have, however, been provided so that the Tunnel downstream of the Intake Structure may be dewatered whenever required. The invert of the Tunnel at the Intake is at RL 1,190. From here the Tunnel slopes to elevation 1,187.35 upto the Gate Passage and Transition. Beyond this point the Tunnel is horizontal in a length of 1912 feet where it bifurcates into two 24 feet diameter branches. At the Outlet Gates the floor elevation is RL 1193.35. The bed of the steel-lined channel section is at RL 1192.10 for 62 feet length. The concrete lined chute follows a curve $-y = x^2/2000$ and terminates at the invert of the 75 feet radius Flip Bucket. The invert is at an elevation RL 1173. From the invert the Bucket rises to elevation 1190 where the water jet leaves the Bucket. The jet rises about 80 feet from the Bucket Lip and falls some 300 to 500 feet away from the Flip Bucket in the spillway channel.

The construction of the Tunnel and its related structures were started in May 1973 and were completed in April 1976. The Tunnel started its test operation on April 7, 1976. The regular operation commenced from May 1,1976.

2.0 DESIGN CONSIDERATIONS

Originally the LBIT was proposed to be used only when the reservoir level was low between elevation 1370 and 1300. After the damages to the right bank tunnels, it was indicated that the LBIT may have to be operated at any time under any head according to the requirements. The Tunnel, as designed, is now capable of operation upto reservoir elevation 1500. Operation beyond this reservoir elevation is not proposed as at this elevation the two spillways can release the desired discharges. Possibility of operation of the Tunnel beyond reservoir elevation 1500 upto 1550 could be extremely remote as it could only be conceived in the event of both spillways being unable to discharge.

The Tunnel, besides meeting the primary aim of augmenting the low level releases from the Tarbela reservoir is being used at higher reservoir elevations as a substitute for Tunnels 3 and 4 when these have to be shut down for inspection and/or maintenance. This has provided complete flexibility to the system.

At reservoir elevation 1300, which is the minimum level to which the Tarbela reservoir is designed to be depleted every year, the Tunnel is capable of discharging about 49,000 cfs with an outlet velocity of 63 feet per second. The outlet velocity at this elevation may increase to about 81 feet per second at a gate opening of 6 feet. At the maximum head of 295 feet corresponding to reservoir elevation 1500, the Tunnel is capable of discharging about 86,000 cfs. The velocity of flow as it emerges from Outlet Gates is about 112 feet per second. At 6 feet gate opening the velocity corresponding to a discharge of about 19,000 cfs is about 139 feet per second. These rarely paralleled high discharges and velocities required special considerations in the design of various components of the Tunnel specially the Outlet works which are required to dissipate an enormous amount of energy. It may be observed that at maximum discharge about 2.1 million horse power are to be dissipated.

3.0 SELECTION OF OUTLET WORKS

For dissipation of hydraulic energy at outlet works, generally two concepts are employed, which include the stilling basin concept and the flip bucket concept. In case of a stilling basin, energy is dissipated within the boundaries of the structure, while in case of a flip bucket, dissipation is achieved partly in the air due to air friction and mainly in a plunge pool by formation of return eddies. The selection of the method of dissipation depends on characteristics of the site as well as hydraulic design requirements.

At the Left Bank Irrigation Tunnel Outlet, a flip bucket has been provided for leading the water away into the spillway channel. The bucket lifts the water about 70 to 80 feet into the atmosphere and the jet of water, after spreading out, falls at a distance of about 300 to 500 feet from the structure forming a deep plunge pool. A flip bucket type structure was adopted as it offered several advantages over a hydraulic jump type basin and also was more economical.

Preliminary calculations of a stilling basin for LBIT indicated that at maximum discharge, the Froude Number of flow would be near 10 which would require about 80 feet deep basin. The overall length of the chute and the basin would have extended to the centre of the spillway channel obstructing the flow from Auxiliary Spillway. Though it would have been possible to obtain an efficient jump at this Froude Number, the water surface would have been very rough and a pulsating jump would have been formed because of the high velocity jet penetrating the jump to a varying degree with respect to time. This would have resulted in excessive pounding pressures which could in time, damage the floor because of high dynamic forces. The studies of pounding pressures in the stilling basins have not as yet progressed to a degree where they could be effectively applied for design of basin floors. Also a stilling basin would have had a large surface exposed to high velocity flow and thus subjected to potential cavitation. The inspection and maintenance of a deep stilling basin would also have been difficult.

The selected design of flip bucket had none of these problems. The structure was shorter and shallower than a stilling basin and was thus more economical to construct, both in terms of time and money. The surfaces subjected to high velocity were less as compared to a stilling basin. Also the bucket structure is well above the tailwater level and the surfaces exposed to high velocity are very easy to inspect and maintain. Any minor damage is repaired easily and quickly before it may assume an alarming proportion. Also there is no problem of excessive pounding pressures and consequently the structure is relatively safer.The major problems related to this type of structure are created in the plunge pool area because of deep scour and return eddies, which were studied on a hydraulic model and were found to be manageable.

Considering the advantages of the flip bucket type of structure, it was adopted for the outlet works of LBIT.

4.0 MODEL STUDIES

Many of the hydraulic flow phenomena in hydraulic structures of this magnitude are not amenable to mathematical analysis due to non-linear character of equations of motion as well as the complexity of hydraulic parameters. In such cases, it is much more convenient to predict the behaviour of the prototype by conducting experimental studies on models. Model studies are helpful in the study of such phenomena where the knowledge about the constituting parameters and their functional relationship is limited. Besides, theoretical computations may also be verified by hydraulic model tests.

The hydraulic designs of the outlet works for the Left Bank Irrigation Tunnel were subjected to extensive model tests. The model studies for the Tunnel were carried out by the Irrigation Research Institute at their laboratories both at Lahore and Nandipur.

In all, five different models at different scales were constructed from time to time to observe the behaviour of flow in different parts of the Tunnel.These included:

i. 1:60 Geometrical scale part model of the portion beyond Bifurcation at Lahore.

ii. 1:78.6 Geometrical scale full length model at Lahore.

iii. 1:36 Geometrical scale part model of the Central Pier and Gate Shaft at Lahore.

iv. 1:36 Geometrical scale model of the Outlet Gates and the Flip Bucket at Lahore.

v. 1:80 Geometrical scale full length model at Nandipur. This model of LBIT was incorporated in the existing complete model of the Tarbela Dam Project.

Of these models, the 1:60 and the 1:36 scale models of the outlet gates and the flip bucket were utilised for studying the pressures along the chute and side walls of the flip bucket. The 1:80 scale model at Nandipur was utilized for the study of the scour in the plunge pool and the effect of return eddies.

The model tests have greatly helped in predicting the flow pattern and the hydraulic behaviour of various components. Based on these tests, several improvements in the design of various components were incorporated.

5.0 MAIN FEATURES OF HYDRAULIC DESIGN OF OUTLET WORKS

The outlet works of LBIT beyond the Outlet Gates essentially comprise the following main components:

1. Aeration system downstream of Gates.

2. Steel lined channel section.
3. Concrete lined chute slab.
4. Cylindrical bucket.
5. Flaring side walls.
6. Central Pier.
7. Plunge pool.

These components are shown on Figure-3.

The hydraulic considerations in the design of these components are briefly described in the following sections:

5.1 Aeration System Downstream of Gates

The flow through the Tunnel is regulated by two radial gates, each 16 feet wide and 24 feet high. A 1.25 feet step has been provided in the floor of the outlet control structure to accommodate bottom sealing arrangements for outlet gates. Because of the high velocity of flow under the gates, a jet trajectory is formed downstream of the step in the floor. This trajectory strikes the floor downstream of the step, (the distance of the impact point depending upon the reservoir level and the gate opening) creating a cavity between the jet and the floor. Introduction of air is necessary below the jet trajectory to keep the jet supported and avoid occurrence of negative pressures which would result in cavitation and cavitation damage. For this purpose two aeration systems have been provided just downstream of the Outlet Gates as shown on Figure-4. One of these systems known as primary ventilation system ventilates the underside of the jet trajectory. A secondary system located along the slope of the step is for ventilating the jet as it comes out when the gates are just opened or approach their closing position. The primary system provides air from each side of the bay through a two feet diameter pipe in an open trench downstream of the step. The secondary system comprises eight 3" diameter nozzles on the slope of the step. The two systems draw air from separate vent pipes exposed to atmosphere.

The design of the ventilation system was based on

1/12 scale model at Colorado State University and 1/36 scale model at Irrigation Research Institute, Lahore. Several shapes of the primary ventilation system were tested and finally the above described configuration was adopted as it was considered to be more efficient than the other tested systems.

5.2 Steel Lined Channel Section

In areas where effective boundary layer has not been established, an extremely smooth surface finish is required to avoid damage from possible cavitation. It is, therefore, usual practice to steel-line areas downstream of gates and steps in floor. The steel liner also provides protection against decay of vortices originating at the corners of the gates during the part-open operation. Accordingly a 62 feet length of the floor downstream of the step has been steel-lined. The steel-lined floor is horizontal and is tangential at its downstream end to the concrete-lined chute. In this area the side walls of the channel are also steel-lined for similar reasons. The steel-lined channel section has a constant width of 18'-2" throughout. The side wall flare starts from the end of the steel liner.

5.3 Chute Profile

In the preliminary design, the chute slab was designed to follow the profile of $-y = x^2/1440$. This curve approximated the theoretical shape of a free jet having an initial velocity corresponding to full reservoir head at elevation 1550 and the gates barely open. In this case the head loss was completely neglected and the entire head was assumed to be converted to velocity head. As the tunnel was not proposed to be operated beyond reservoir elevation 1500 it was considered that due to 50 feet reduction in head and also due to the head losses through the Tunnel the trajectory would be safe at reservoir elevation 1500 and below.

The chute profile was tested on the 1:60 scale part model at IRI, Lahore. Presence of negative pressures along the chute indicated the inadequacy of the chute profile at various combinations of reservoir elevations and gate openings. Accordingly the trajectory of the chute was revised and it was finally designed in accordance with the criterion laid down by the U.S.Bureau of Reclamation, which recommends that to avoid the tendency of water to spring away from the floor and thereby reduce the surface contact pressure, the floor shape for convex curvatures should be made substantially flatter than the trajectory of the free discharging jet under a head equal to the specific energy of flow as it enters the curve. The curvature should approximate the shape defined by the equation

$$-Y = x\tan\theta + \frac{x^2}{4\ K(d+h_v)\cos^2_\theta}$$

where θ is the slope angle of the floor upstream of curve

d is the depth of flow in feet

v is the velocity of flow in feet/sec.

K is a factor such that for

$k = 1$, the equation is that of a free discharging jet.

h_v=Velocity head = $v^2/2g$

In case of LBIT, the upstream end of the chute profile is tangential to the horizontal flow which gives a value of $\theta = 0$. The equation for the chute curvature in this case is accordingly reduced to

$$-y = \frac{x^2}{4k\ (d+h_v)}$$

where $(d+h_v)$ represents specific energy at the beginning of chute profile.

To ensure positive pressures along the entire contact surface of the curve, USBR recommends that the value of K should be equal to or greater than 1.5. Considering the high heads and velocities in case of LBIT where the tendency for separation would be high, a conservative value of K as 1.7 was adopted.

Calculations for chute profiles corresponding to different reservoir elevations and gate openings showed that the most critical situation occurred at reservoir elevation 1500 and gate opening of 12 feet where the equation for the chute profile using K = 1.7 becomes $-y=x^2/1933$ Accordingly the chute profile was designed to follow a curve $-y=x^2/2000$

When subjected to model tests on the 1:60 and later 1:36 scale models, this profile gave positive pressures all along the chute floor confirming its adequacy. This profile was, therefore, adopted in the final design with its origin at elevation 1192.10 where it is tangential to the horizontal floor. On the other end the chute parabola is tangential to the 75 feet radius bucket.

5.4 Bucket

The chute terminates in a bucket which is a cylindrical surface, tangential at its upstream end to the parabolic base slab profile. The bucket turns the direction of the incoming flow and flips it into the atmosphere at an angle of 35°.

The bucket has been designed on the basis of USBR criterion which specifies that the radius 'R' of the bucket should not be less than 10d where d is the average depth of flow. In case of LBIT, the average depth of flow at the bucket is about 6.25 feet which gives a limiting value of 'R' as 62.5 feet. A radius of 75 feet has, therefore, been adopted. The invert of the Bucket is at Elevation 1173 while the lip elevation is 1190.

5.5 Sidewall Flare

The sidewalls of the Flip Bucket were originally constructed with a flare of 1 in 8 with respect to centre lines of the two flow passages. This design was based on the satisfactory model test results on the 1:60 and 1:78.6 scale models, and was considered adequate for reservoir elevations 1370 to 1300 at which the Tunnel was originally required to be operated.

When it was indicated that the Tunnel may be required to operate regularly at higher heads, a large scale model of the Outlet Works at 1:36 scale was built to verify the adequacy of hydraulic design of various components of Outlet works. Tests on this model indicated presence of negative pressures of the order of -20 feet along the sidewalls, at the sides of the pier and on the chute slab at the start of the chute. Various alternatives for eliminating these negative pressures were tested on the 1:36 scale model and finally it was recommended to alter the sidewall flare from 1 in 8 to 1 in 12 as it resulted in complete elimination of negative pressures and improved the throw of the jet at reservoir elevation 1500 by 70 feet. The recommended flare also conformed to the USBR criterion, according to which the sidewall flare should not exceed the angular variation given by the equation:

$$\tan \alpha = 1/3F$$

where

F = Froude Number for average velocity and depth at the start and end of the transition.

It was, however, not possible to change the flare of the sidewalls of the prototype to 1 in 12 in its entire length, because this required construction of additional thickness of the sidewalls over a contraction joint in the chute slab which ran parallel to the sidewall at about 5 feet from its toe. This difficulty was overcome by changing the flare in the first 93 feet length to 1 in 12 and making the additional wall parallel to the existing sidewall with a flare of 1 in 8, so that the additional wall was clear of the contraction joint.At the transition from 1 in 12 to 1 in 8 flare, a curve of 16 feet length and 388 feet radius was provided to avoid any kink. Another curve of 192 feet radius was provided in a 16 feet length at the start of 1 in 12 flare to avoid separation due to formation of the kink.

The end of the additional sidewall was made perpendicular to the edge of the lip of the flip bucket and was also inset by 16 inches for improving the aeration of the jet as it leaves the lip of the bucket and also to increase the throw distance of the jet.

Implementation of these modifications have resulted in improved hydraulic conditions and complete elimination of negative pressure from the sidewall, face of the pier and from the chute. A plan and profile of the proposed changes is shown on Figure-3.

5.6 Central Pier

The flow from the two outlet gates is separated by a central pier which terminates at the point where the chute slab becomes tangential to the cylindrical bucket. Several lengths of the pier were tested which included extension upto the end of the bucket and it was observed that 188.5 feet length extending upto the end of parabola gave a better flipping action. The shape of the downstream face was semicircular. When tested on 1:36 scale model, extreme negative pressures were indicated downstream of the pier nose. To avoid damage to the pier, the shape of the downstream nose of the pier was changed and the circular portion was replaced by a straight cut at the nose. This shape was tested on the model and was found satisfactory.The final shape of the pier, as constructed, is shown on Figure-5.

5.7 Plunge Pool

The jet of water as it leaves the flip bucket drops into the spillway channel where it excavates a plunge pool. The throw of the jet varies between about 500 feet from the lip of the bucket at reservoir elevation 1500 to about 200 feet at reservoir elevation 1300, at full opening of outlet gates. The maximum throw of the jet, which is about 580 feet is, however, obtained at half gate opening and reservoir elevation of 1500.

The model tests for determining the scour pattern in the plunge pool and formation of return eddies, were conducted on 1:80 scale model at Nandipur. A full length model of LBIT was incorporated in the existing model of Tarbela Dam. This model was also used to study the effect of operation of Auxiliary and Service Spillways on LBIT as the three structures discharge into the same outlet channel.

Originally the spillway channel bed was at elevation 1180. The initial model tests were conducted with this bed level. The spillway channel had about 25 thick overburden which was eroded when the Auxiliary Spillway was operated in 1974-75. Additional tests were then performed by removing the overburden and lowering the bed level to elevation 1155.

The flow patterns under the two conditions were generally similar. The jet of water after striking the channel bed impinged on the left of the spillway channel with eddies forming on the two flanks of the impingement point. The tests were conducted for different reservoir elevations and the cumulative scour produced by these tests was recorded. The bed of the spillway channel was moulded from Lawrencepur sand representing completely erodible bed conditions . The maximum scour in any sequence of tests went down to elevation 1050 i.e. about 105 feet below the channel bed. The plunge pool was found to remain at a safe distance from the flip bucket structure. Downstream of the flip bucket there exists a 150 feet wide dyke of basic rock. This rock is fairly competent and was considered adequate to act as a natural cutoff between the plunge pool and the bucket. The rock has been strengthened and protected by rockbolting and protective concrete covering. This method of treatment has avoided heavy expense of providing a deep cut off wall for erosion protection. A watch is, however, being kept on this rock so as to provide additional protection as and when required.

For providing a clear impingement area and proper aeration to the trajectory issuing from the flip bucket, the basic rock just downstream of the bucket was cut down in a mild slope to a level of about 1170 upto a distance of 120 feet from the lip of the bucket measured along the centre-line of the bucket as shown in Figure-6.

The model tests further indicated that on the left side the return eddy was rather strong and attained a maximum velocity upto 32 fps. This eroded the left flank of the bucket and as this flank receded, deep erosion was caused just downstream of it. If this action were allowed to continue unchecked it would have undermined the bucket. The maximum scour downstream of the bucket caused by the return eddy was of the same order or even deeper than that at the plunge pool area.

Further model tests for ensuring the safety of the bucket were conducted. These tests indicated that if the left flank is allowed to protrude and is held in position, the return eddy is directed away from the bucket foundation and the structure remains safe.

The geological surveys indicated presence of a band of basic rock downstream of the flip bucket and on the left flank. The tests were repeated by incorporating the basic rock and indicated that the return eddy of the left flank was deflected away and the deep erosion remained well beyond the bucket foundation as long as the basic rock protrusion on the left flank was held in position. It was, therefore, recommended to strengthen the left flank by rockbolting and shotcrete treatment against the erosive action of eddies.Complete protection, to begin with, required deep excavation. It was, therefore, proposed to initially protect the exposed parts of this rock and extend the protection as additional rock got exposed. A watch is, therefore, being kept on this rock to provide additional protection as and when needed.

6.0 TEST OPERATION

The test operation of the Left Bank Irrigation Tunnel was started on April 7,1976, .The test operation commenced with 6 feet opening of outlet gates and reservoir elevation 1501.65. During the test operation, the Tunnel was operated at various gate openings varying from 6 to 24 feet. As the Tunnel is designed to operate upto reservoir elevation 1500, the test operations were conducted under most severe conditions.

After each test operation the areas downstream of the outlet gates were inspected. The Tunnel downstream of services gates was also dewatered and inspected twice. The test operations were completed on April 30,1976, upto which date the Tunnel operated at different gate openings for a total 138 hours. Regular operation of the Tunnel commenced from May 1,1976.

During test operation, the Tunnel and its appurtenant structures all performed satisfactorily. There were two minor problems which included joint spalling and cavitation at the face of central pier. Remedial measures for both were provided and since then no major problem has been experienced with the outlet works.

6.1 Joint Spalling

Joints in concrete surfaces are found to be the trickiest places as these remain weak spots due to constructional difficulties and when exposed to high velocity flow the joint is spalled. Often the edges of forms for concrete placement may lack stiffness and thus cause the joint to be raised or protrude into the flow which may result in cavitation damage downstream of the joints.

Joint spalling was noticed during the test operation of LBIT. The spalling was caused mainly due to irregular surfaces across the joints and also due to cracking of epoxy that over-rode the joints. This epoxy was used to remove the surface irregularities at joints between adjacent chute slabs. Spalling less than 1/2 inch deep was ground smooth. Spalling less than 1 inch deep was filled with Epoxy P-103 and spalling larger than 1 inch was filled with Epoxy S-2 mortar. The overriding epoxy across the joints whenever cracked was removed by grinding.The treatment was repeated whenever needed and since then withstood reasonably well. Minor repairs of spalled areas are needed from time to time and are carried out in either of the above described manners. The repair operation takes about 10 to 14 hours.

6.2 Pier Cavitation

After the first operation of LBIT at 6 feet gate opening,which lasted two hours,cavitation erosion of concrete was noticed on both faces of the central pier, downstream of the steel liner. The eroded concrete surfaces were repaired with Epoxy P-103 and the tests operations were continued. The damage re-occurred after every operation and was repaired every time before commencing next operation.

Cavitation occurred in the shape of an arc which increased with increasing gate openings. The maximum depth of erosion was upto 2 inches. The maximum height of erosion was about 8 feet from the floor and extended to about 7 feet in the direction of flow. To counteract this cavitation damage two remedial measures were incorporated which included:

i. Improvement of pier profile
ii. Aeration of the area

Precise survey of the damaged area indicated slight undulations in this area. Accordingly the pier profile downstream of the steel liner was improved by introducing a curve made with Epoxy P-103 over a length of 5 feet and a height of 8 feet on both faces of the pier. The surface was painted with Epoxy S-2 to give a smooth surface.

In addition, an aeration system was provided on the right face of the pier in the first instance. A six inches diameter hole was drilled through the pier down to the floor level about 30 inches.behind the pier face and 9 inches downstream of the steel liner. 13 holes each of two inch diameter were drilled from the face of the pier to the vertical 6 inches diameter hole.

After test operation at 11.5 feet and 24 feet gate openings it was found that no cavitation damage occurred on the right face where aeration was provided while on the left face where only the profile was improved, some pitting was observed. Accordingly an aeration system similar to the one on the right face of pier was provided on the Left face also.

Later operations indicated that there was some air requirement close to the steel liner also. To meet this demand, additional holes of one and a half inches diameter were provided on the upstream side of the existing holes as considered necessary from time to time. The aeration system is shown on Figure-7.

With the adoption of these measures the problem of cavitation on the pier face has been solved and no cavitation damage has occurred at the pier faces since then.

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FIGURE - I
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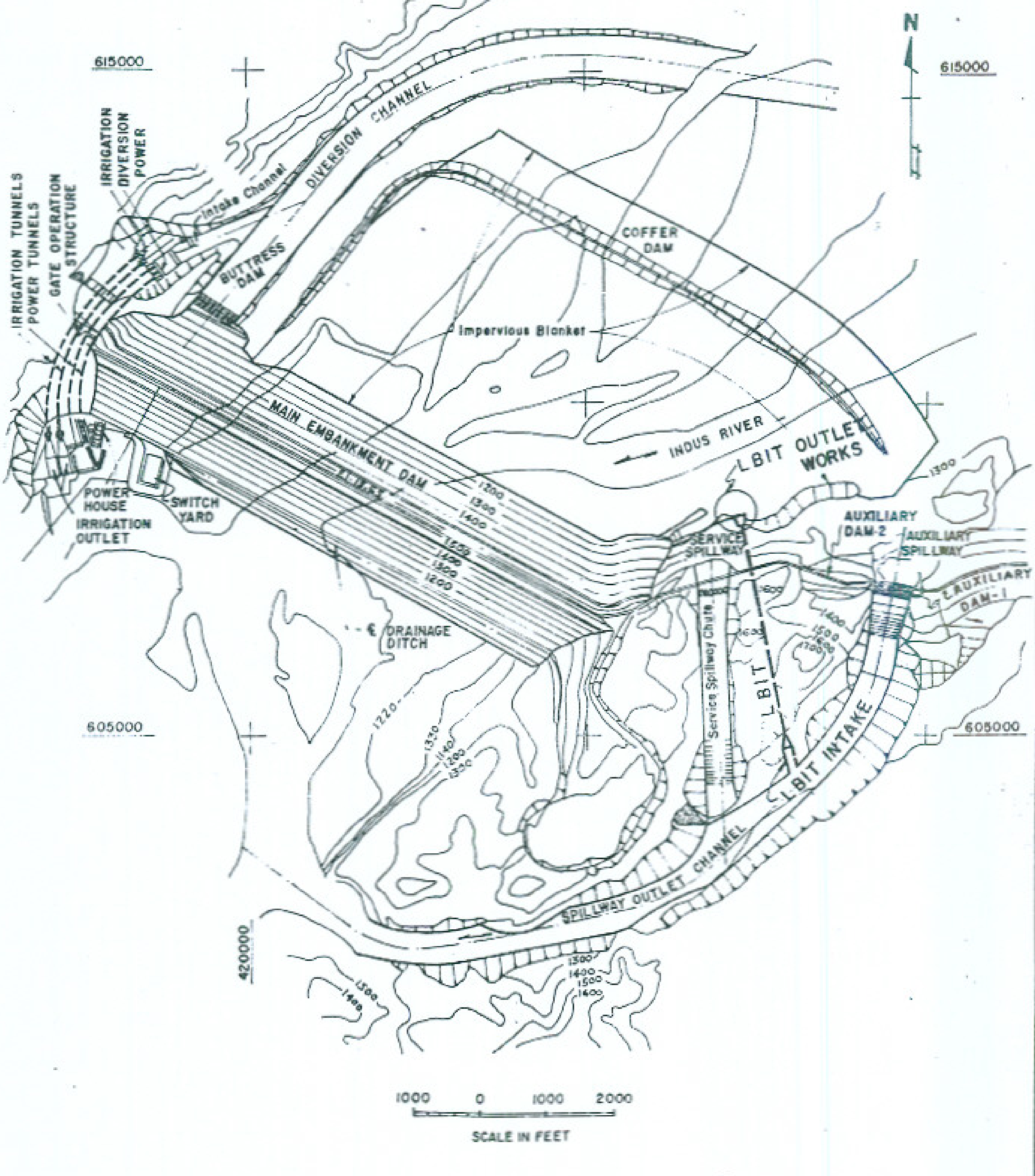
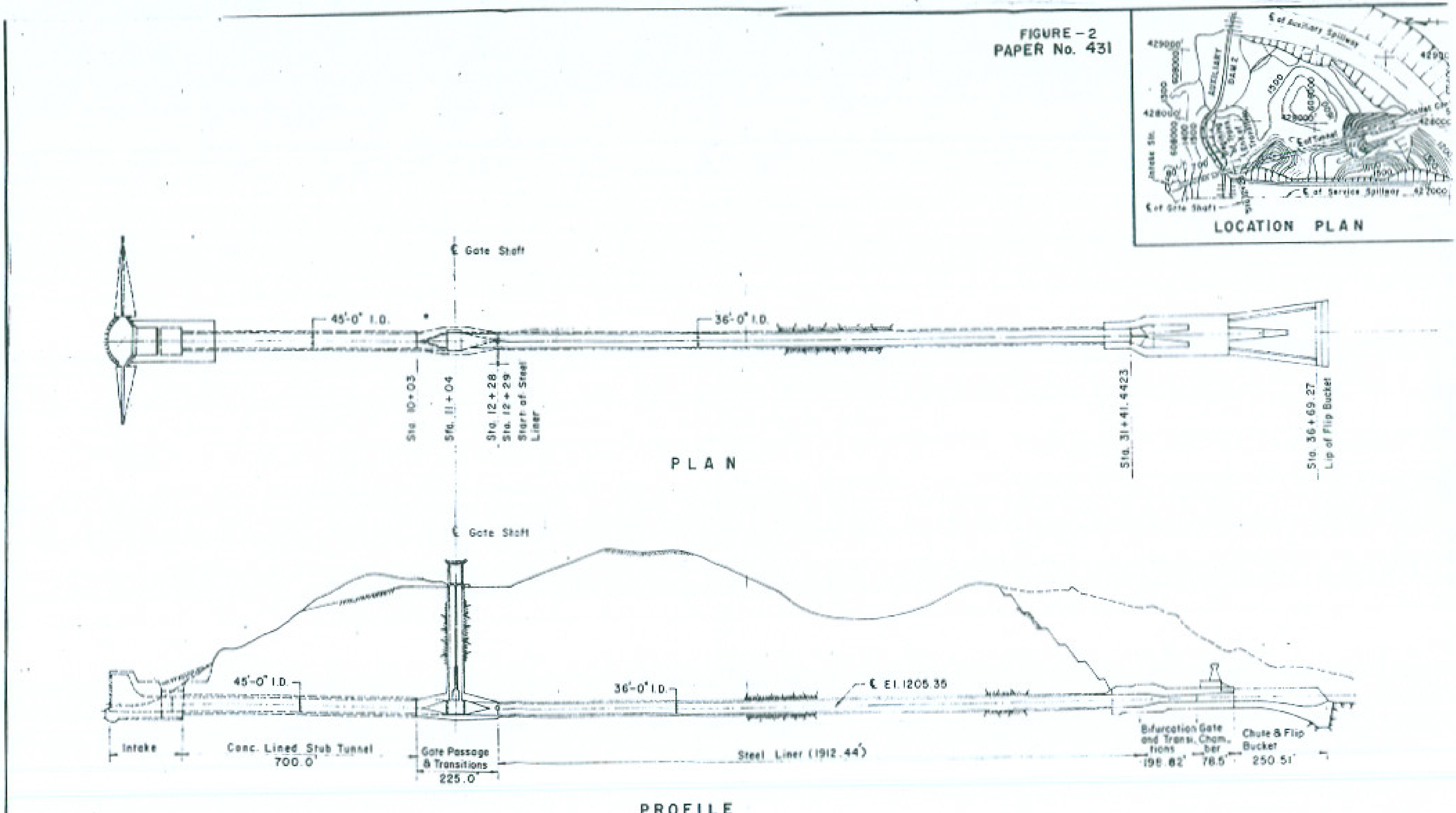


TARBELA DAM PROJECT

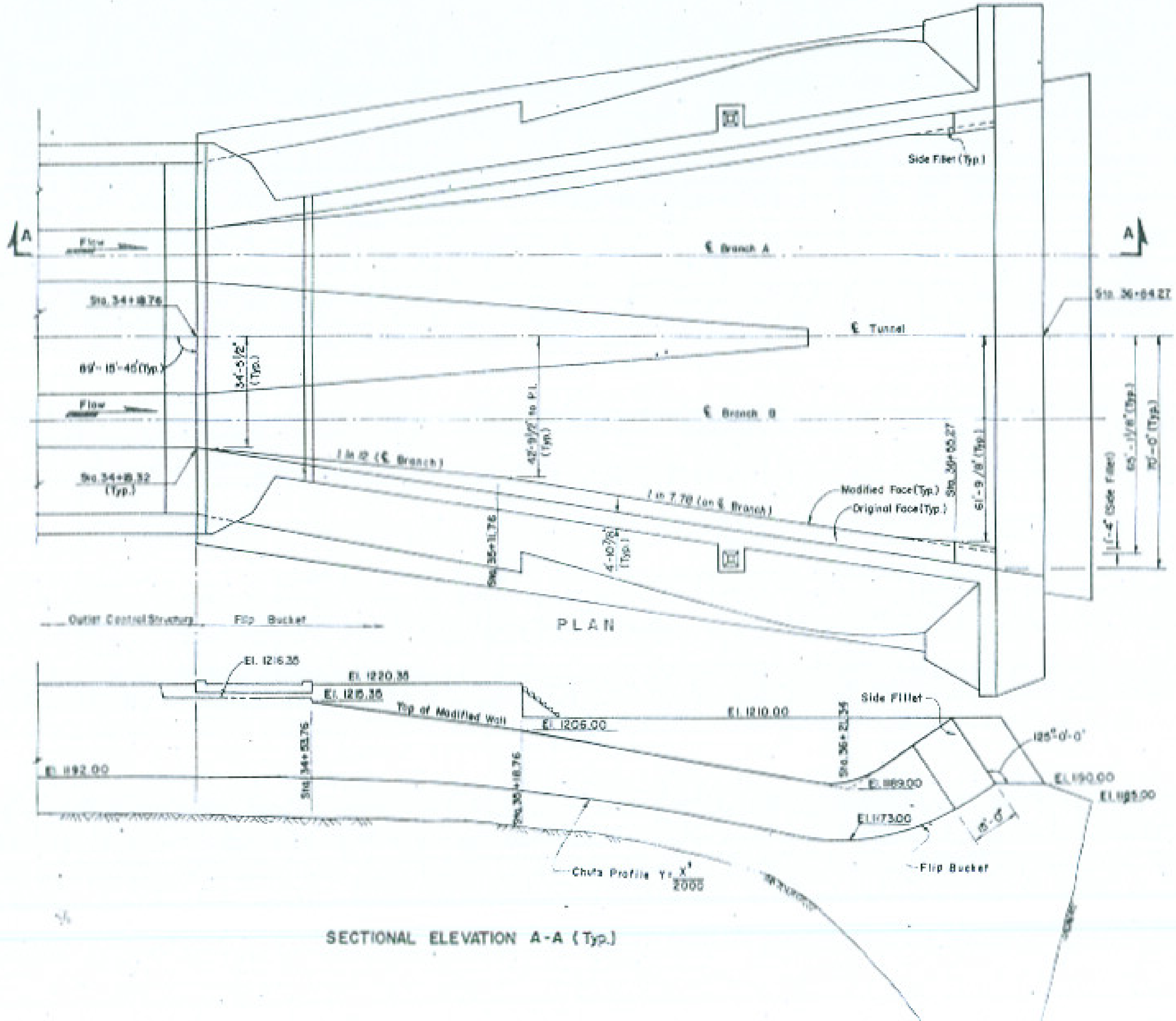
SITE PLAN

FIGURE – 2
PAPER No. 431



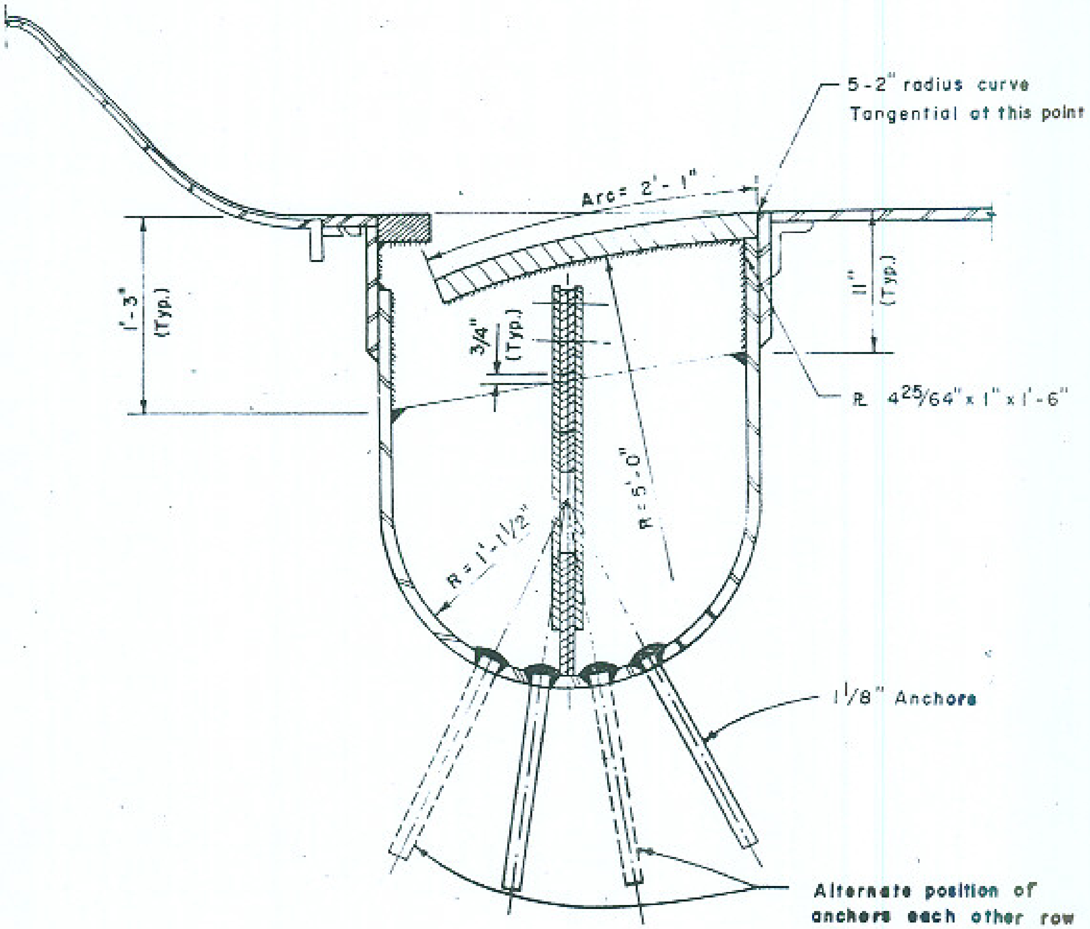
LEFT BANK IRRIGATION TUNNEL
PLAN AND PROFILE

FIGURE - 3
PAPER No. 431



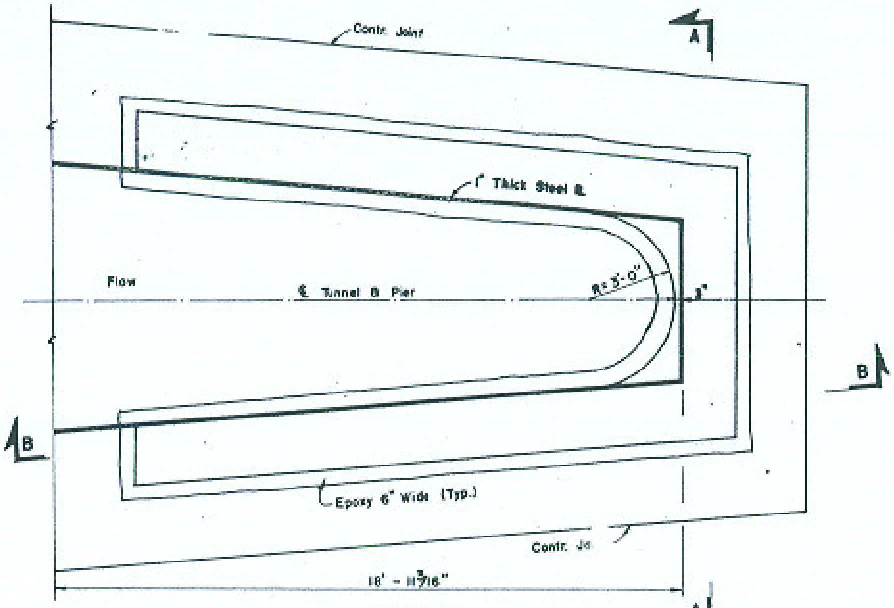
LEFT BANK IRRIGATION TUNNEL
FLIP BUCKET
SCALE 1" = 30'

FIGURE – 4
PAPER No. 431

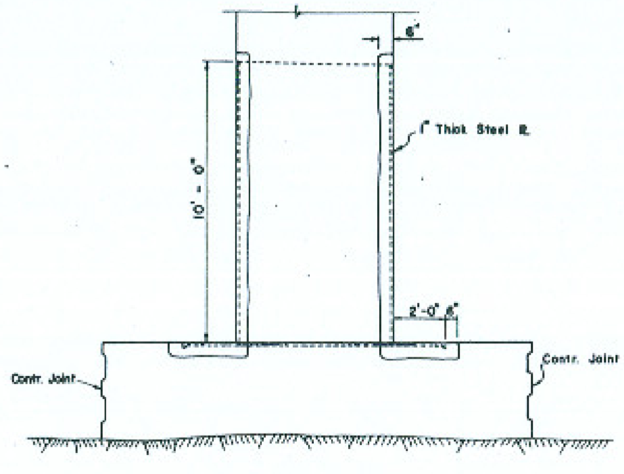


LEFT BANK IRRIGATION TUNNEL
AERATION TROUGH
SCALE :- 1"= 1'- 0"

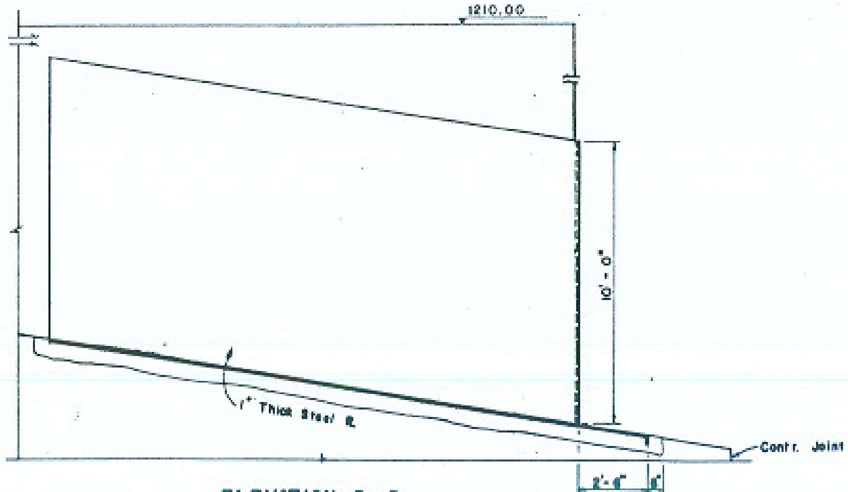
FIGURE – 5
PAPER No. 431



PLAN



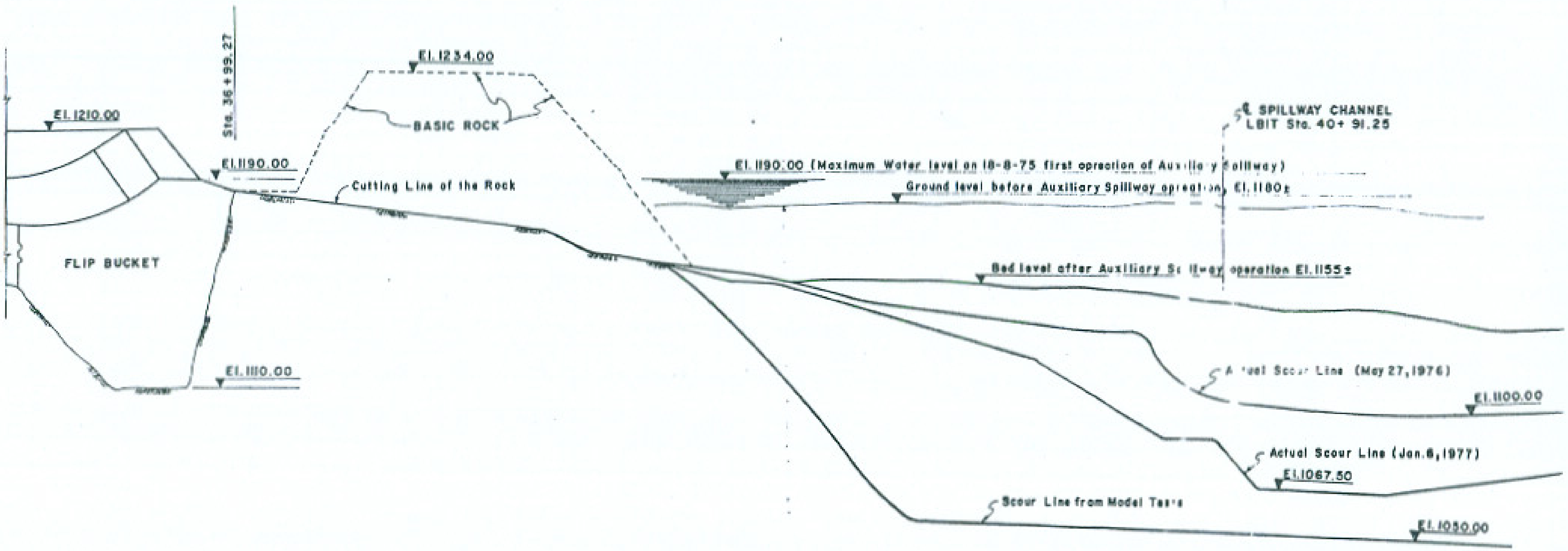
SECTION A - A



ELEVATION B - B

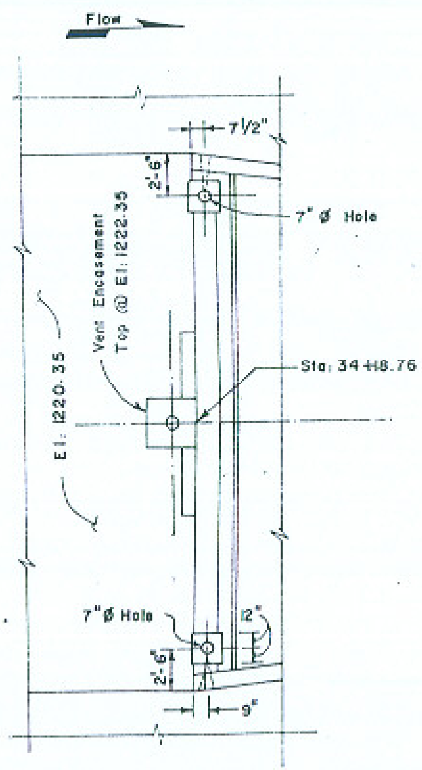
LEFT BANK IRRIGATION TUNNEL
CENTRAL PIER NOSE
SCALE:- 1"=4'-0"

FIGURE - 6
PAPER No. 431

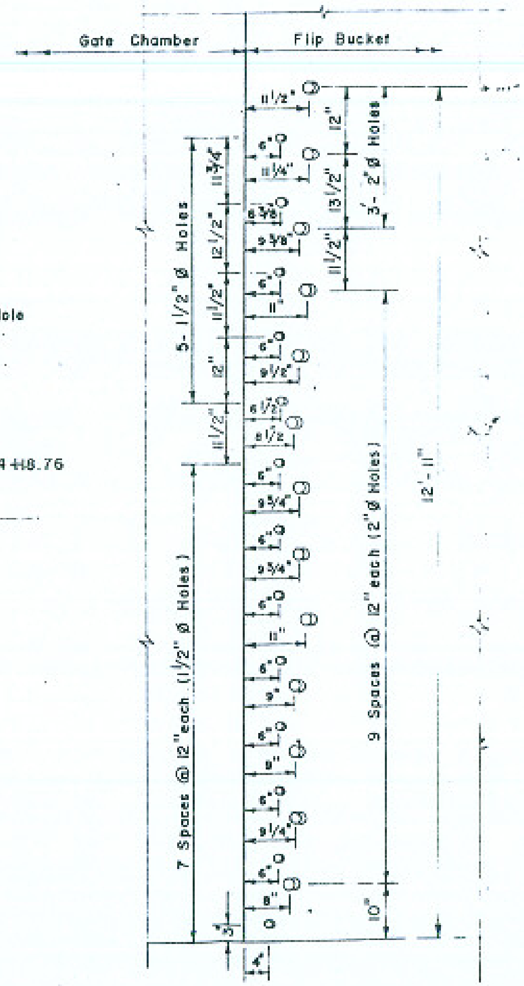


LEFT BANK IRRIGATION TUNNEL
PLUNGE POOL DEVELOPMENT

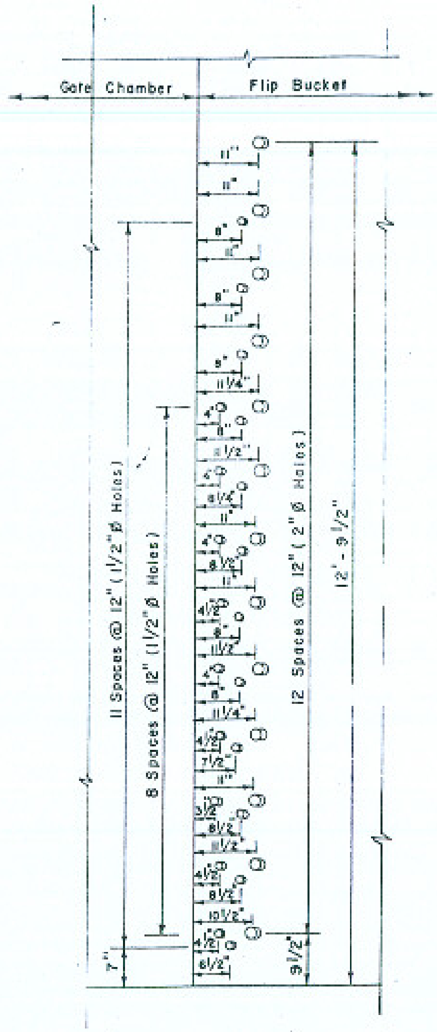
FIGURE - 7
PAPER No. 431



PLAN
Scale: 1" = 8'-0"



ELEVATION BRANCH 'A'
(SHOWN OPPOSITE SIDE)
Scale 1" = 2'-0"



ELEVATION BRANCH 'B'
(AS SHOWN)
Scale: 1" = 2'-0"

LEFT BANK IRRIGATION TUNNEL
AERATION HOLES ON
SIDEWALLS