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Remedial Measures of Rasul Barrage

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BY

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SUMMARY

1. The Barrages under the Indus waters Treaty consisted of five numbers at Chashma (Indus), Rasul (Jhelum), Qadirabad and Marala (Chenab), Sidhnai (Ravi) and a gated siphon near Mailsi (Sutlej). These were designed to divert the waters of the Indus, Jhelum and Chenab towards the East through the Link Canals off-taking from these Barrages. The Rasul barrage was constructed on the river Jhelum about two and a half miles downstream of the old Rasul weir. This diverts Jhelum supplies from releases of Mangla reservoir into the Rasul Qadirabad Link Canal for ultimate supply to the existing Sulemanki Headworks on the river Sutlej. The barrage also diverts supplies to Lower Jhelum Canal. This barrage has 48 spans each 60 feet wide with a total length of 3209 feet. The barrage is designed for a maximum discharge of 850,000 cusecs. The contract was awarded to M/s. Cogefar Astaldi on 31st March, 1965 and the project was completed on 21st December, 1967 about 3 months ahead of schedule. M/s. Coode and Partners were the Consulting Engineers for this Project.

2. The barrage was commissioned on 8th January 1968. Later on in May 1968 an unusual occurrence of excessive uplift pressures and also the bubble phenomenon were the cause of great concern, and operational restrictions had to be introduced to ensure the safety of the barrage. As an immediate precaution it was decided to restrict the head across of the barrage to 16 feet but later on this was increased to 21 feet until the remedial measures were done. However the operational problems were overcome by releasing enough water through the barrage so as to maintain a sufficiently high downstream level and thus keeping the differential head within safe limits.

3. The bubbles were noticed at the surface of the water mainly in the areas of the down stream concrete block and stone aprons of the barrage when the tail water was low and still water conditions existed below the barrage. This phenomenon which had given rise to concern that piping of material from below the aprons due to underseepage might be connected with it, had arisen at the same time as higher pressures than design were being recorded in the pressure points located under the downstream floor of the undersluice bays of the barrage. The following conclusions were drawn after carrying out the analog study, permeability tests below the level of sheet pile cut offs, chemical analysis of bubbles samples and inspection by divers of the source and nature and emergence of bubbles :

   i) The piezometers indicate that pressures under the standard bays are well withen acceptable values and the analog studies do not indicate any excess gradients.

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Therefore the bubbles have no significance either with regard to the safety or the operational performance of the barrage.

ii) The permeability of the subsoil is normal for the type encountered, and the underseepage velocities are well below velocities which could cause piping of material from below the barrage floor.

iii) That the bubbles are not originating from some underground source such as a gas field or strata of decaying organic matter.

4. After the formation of the pond in January 1968 the recording of the readings of the pressure points were started at pier 1 in the undersluice bays and piers 8,16,27 and 38 in the standard bays. The reliable readings started in March after the piezometers were tested and purged. It was later observed that the pressures being recorded at pier 1, although acceptable for the cross-head at that time but would be excessive under the maximum design conditions of 36 feet.

A study of the geological sections indicates that a clay layer about 10 feet thick extends downstream under the concrete block and stone aprons and to the right to about Pier No. 6. This clay layer forms an almost completely impermeable blanket which confines the underseepage flow until it reaches the edge of the clay layer where it can find its exit to tailwater. A greater head of water is required to force the underseepage flow to the exit point. It is therefore concluded that the higher pressure under the undersluice bays are caused by the presence of clay layer which extends beneath the concrete block and stone aprons downstream of the undersluice bays and obstructs the flow of normal seepage.

The proposal of relief wells was finally accepted to relieve the higher pressure. Accordingly 24 relief wells, 80 feet deep with a spacing of 15 feet 6 inches were recommended for installation in the downstream concrete block aprons of the undersluice bays.

5. The construction of an earth bund cofferdam/working platform was started in September 1968 with a top elevation of 718.00 and extended from the left downstream guide bank across the block apron and gates closed from Bay No. 1 to 6 of the undersluices. At each well location, a steel sheet pile caisson was driven 12 feet in diameter, down through the work platform upto the top of the block apron and the material inside the caisson was removed by a clam shell bucket grab. The inside of the caisson was kept full of water upto top level by pumps after removal of the material from the caisson. Twenty four relief wells 80 feet deep of fibre glass pipe were installed at 15 feet 6 inches interval in a line from left abutment to pier 6 The slotted pipe screen was surrounded by an annular 7 inch thick filter pack consisting o coarse to medium sand with little fine gravel. The filter was designed to protect the scree slots from being clogged by the filter pack, which in turn was designed to prevent the escape o

the foundation sands and silts through the well to the river. A slotted metal cover was then bolted into the recess left in the new apron block, the 28 inch hole in the block having been previously filled with a graded filter of maximum size 1½ inches. The relief wells were designed to relieve pressure under the floor by permitting water to enter each well through filter pack and the slots in the screen and escape to tail water through openings in the steel cover. On completion of all the wells in November 1968 two leading cuts were excavated through the temporary earth cofferdam/platform and by passing water through the undersluice bays, the earth was slowly eroded away.

The pressures observed at pier No. 1 in the undersluice bays showed a marked reduction and were well withen acceptable limits. The relief wells were therefore functioning and relieving the pressure caused by the clay layer.

6. An examination of the Phi-values at Piers 1 and 8 indicate that the readings were even influenced during the installation of the wells. There was some increase in pressure under the undersluice floor at the start of construction period due to further restriction of underseepage flow caused by the temporary embankment but there was no danger because the floor was loaded with earth. The installation of the 24 relief wells was completed on November 16, 1968. The piezometers immediately showed a reduction of pressures under the floor when removal of the embankment was completed in December 1968. A comparison of the phi-values obtained after installation of the wells with earlier readings demonstrate the effectiveness and utility of the wells. The relief wells reduced the excessive pressures under the downstream floor of the undersluice bays to safe limits.

7. If the phi-values show a long term tendency to increase without any apparent reason, such as the effects of sediment deposits, re-trogression, or other factors, it may indicate that the screens of the wells are gradually getting clogged. If the factor of safety against floatation for the down stream floor approaches 1.1 plans should be prepared to open the relief wells and inspect them during low water period.

The Author visited Rasul Barrage in June 1985 and observed that sediment deposits were accumulated on the downstream aprons all along the barrage as sufficient discharges are not released so that these sediment could be washed away. After the increase in pressures was noticed on points 8 & 9 of piers 1 and 8, relief wells were opened during 1984. The silt was filled inside the fibre-glass pipe and the same was cleared with the help of compressed air. The crew including diver of Mangla Dam carried out the cleaning operation of the relief wells. The relief wells were cleared of silt first time during 1974 under the supervision of the Author by lowering two pipes inside the relief well, one for the entry of compressed air and the other for flushing out the silt/mud from the well.

8. The following recommendations may be followed for the barrages to be constructed in future.

i) Pressure points may be provided at every 4th bay as at present the spacing is about double this number.

ii) Daily readings may be taken on all the pressure points during first year of operation of a barrage.

iii) Piezometric readings should be taken only in stabilized conditions of upstream and downstream water levels.

iv) Accumulation of silt should not be allowed on the concrete block apron.

v) Four rows of bore holes may be drilled in every bay, one at the end of upstream concrete floor, second at the centre of the barrage, third at the end of downstream concrete floor and the fourth at the end of concrete block aprons. The data from these bore holes will help to locate the soil profile likely to block the underseepage and preventive measures could be taken at the time of detailed design.

If economically feasible, removal of clay layers and replacing with similar sandy fill material as existing under the barrage will be best alternative to avoid blocking the underseepage otherwise some other suitable measure will have to be adopted.

vi) The fibreglass pipe/screen can be safely used for the installation of relief/drainage wells.

REMEDIAL MEASURES TO RASUL BARRAGE

1. INTRODUCTION

1.1 General

The Barrages under the Indus Basin waters consisted of five numbers at Chashma (Indus), Rasul (Jhelum), Qadirabad & Marala (Chenab), Sidhnai (Ravi) and a gated siphon near Mailsi (Sutlej). These were designed to divert the waters of the Indus, Jhelum & Chenab towards the East through the Link Canals off-taking from these Barrages. The conbined length of the five barrages and Mailsi Siphon is about 3.5 miles which is about half of the total length of all existing weirs and barrages in Pakistan. The total flood discharge capacity of these six structures is about 4.4 million cusecs. The regulators at these Barrages will provide for an aggregate diversion of about 100,000 cusecs of water into the Link Canals off-taking from these projects.

1.2 Rasul Barrage

This barrage (Fig. 1–2 was constructed on the river Jhelum about two and a half miles downstream of the old Rasul weir. This diverts Jhelum supplies from releases of Mangla reservoir into the Rasul Qadirabad Link Canal for ultimate supply to the existing Sulemanki Headworks on the river Sutlej. The barrage also diverts supplies to Lower Jhelum Canal.

The Contract was awarded to M/s. Cogefar Astaldi on 31st March 1965 and the project was completed on 21st December, 1967 about 3 months ahead of schedule. M/s. Coode and Partners were the Consulting Engineers for this Pronect. The main data of this project is as follows :–

Length	3,209 feet
Openings	48 spans each 60 feet wide.
Normal Pond Level	719.00
High Flood Level upstream	722.75
Road level	729.00
Maximum discharge	850,000 cusecs

2. COMMISSIONING OF THE BARRAGE

The ponding commenced at Rasul Barrage on the 8th January, 1968, and the following rates of raising of the pond were adopted for the first wetting of the works :–

1st Day	(i) Close all barrage and regular gates and allow pond to rise to 707.00
	(ii) Then allow pond to rise to 712.00 and keep steady for 12 hours.
2nd Day	Raise pond to 714.00 at the rate of 2 inches per hour and keep steady for 12 hours.
3rd Day	Raise pond from 714.00 to 716.00 at the rate of 1 inch per hour and keep steady for 12 hours.
4th Day	Raise pond from 716.00 to 717.00 at the rate of 1 inch per hour and keep steady for 12 hours.
5th Day	Raise pond from 717.00 to 718.00 at the rated of 1 inch per hour and keep steady for 12 hours.
6th Day	Raise pond from 718.00 to 718.50 at the rate of 1/2 inch per hour and keep steady for 12 hours.
7th Day	Raise pond from 718.50 to 719.00 at the rate of 1/2 inch per hour and keep steady until further notice.

Continuous watch was kept during the ponding operations on all earth bunds, pitched slopes and concrete structures. A longitudinal crack developed on the unprotected length of the left upstream guide bank at elevation 716.50 and the bank slipped away along this face on 3rd day when the pod level was 714.50. Immediately the ponding operations were stopped and the pond reduced to 707.00 gradually over a few days with a rate of drop of 2 inches per hour.

The damage was confined to a 500 feet length and in the location of an old river channe over which the bund had been constructed. An earth bank was formed some 200 feet wide, ou from the existing toe of the bund, and on this the bund reconstructed in layers to specifie compaction and re-trimmed to section.

After completing this work, the commissioning programme was recommended from th beginning and successfully completed with no further problem.

3.0 OPERATIONAL LIMITATIONS

Later on in May 1968 an unusual occurrence of excessive uplit pressures and also the bubl phenomenon were the cause of great concern, and operational restrictions had to be introduc

to ensure the safety of the barrage. As an immediate precaution it was decided to restrict the head across of the barrage to 16 feet but later on this was increased to 21 feet until the remedial measures were done.

The limitation of head created complications for the operation of the link canal system. These were overcome by releasing enough water through the barrage so as to maintain a sufficiently high downstream level and thus keeping the differential head within safe limits.

4.0 BUBBLE PHENOMENON

4.1 After the formation of the pond and with water both upstream and down-stream of the barrage, bubbles were noticed at the surface of the water mainly in the areas of the down stream concrete block and stone aprons of the barrage (Fig. 3) when the tail water was low and still water conditions existed below the barrage.

The following initial site investigations were carried out into this phenomenon.

i) Inspection at the source of the bubbles by divers which indicated no perceptible upward flow of water or material but that cone depressions in the silt overlaying the area were formed.

ii) Collection of samples of the bubbles for chemical analysis.

iii) The survey of locations of the bubbles.

4.2 This phenomenon which had given rise to concern that piping of material from below the aprons due to underseepage might be connected with it, had arisen at the same time as higher pressures than design were being recorded in the pressure points located under the downstream floor of the undersluice bays of the barrage (Fig. 4). A high level meeting of all concerned agencies was called to discuss both points and to decide what measures should be taken in connection with them and the future operation of the barrage.

4.3 It was deicded to obtain more precise data on both the bubbles and the possibility of under seepage of sufficient magnitude to cause piping of material from below the floor of the barrage :–

i) Tests should be carried out to determine the relative permeability of the "sand with gravel" strata below the level of the sheet pile cut-offs.

ii) Samples of bubbles should be taken under strict control for chemical analysis.

iii) Inspection

iii) Inspection by divers of the source and nature of emergence of the bubbles.

iv) Introduction of dye into the water both upstream of the barrage and the right closure bund in an attempt to trace the origin and time of travel of the water coming up at the downstream of the barrage.

4.4 The tests carried out and the conclusions reached are described below :–

i) (a) The carefully observed permeability tests wre carried out in the areas behind the left and right abutments of the barrage. At each location three pumping tests and two recovery tests were carried out. At the left abutment area, the pump well was 150 feet deep and the 29 observation wells varied from 40 to 137 feet deep, while at the right abutment, the pump well was 150 feet deep and the 13 observation wells varied from 150 feet to 250 feet deep.

(c) The results of the tests gave a coefficient of permeability in the aquifers as under :–

(i) Left Abutment area : 6×10^{-2} centimeter per second.
(ii) Right Abutment area 4×10^{-2} centimeter per second.

These are normal for such a "sand-gravel" strata as encountered under the barrage. For the permeability of 6×10^{-2} centimeters per second and head of 36 feet, 20 days will be required for a particle of water to move a distance of 350 feet. It will require 32 days to move a particle of water 350 feet with the same head and permeability of 4×10^{-2} centimeter per second. The corresponding average velocities will be in the order of 0.7 foot per hour and 0.5 foot per hour. For piping, the velocity will have to be atleast 5 feet per hour. Therefore the factor of safety is about 8 against piping.

ii) (a) A total of 35 further samples of the bubbles were collected under the control of a qualified chemist ; Orsat apparatus for chemical testing of gas bubbles was set up and the samples tested for composition. The bubbles were composed of Oxygen and Nitrogen in varying proportions but with no measurable amounts of carbon dioxide or combustible gases.

(b) The conclusions drawn from the tests were that the source of the gas making up the bubbles was air from the atmosphere and that this had entered the subsoil underlying the barrage during dewatering (Fig. 4.1). It was subsequently trapped under the barrage floor during impounding of the water and was gradually being forced out with the crosshead of water rising to the water surface as bubbles.

iii) Further careful investigation by divers did not reveal any more significant information.

iv) Although dye was introduced into the upstream pond, by placement of bags of dye into the stone apron by the divers, no indication was ever observed down-stream even with the use of a black lamp detector.

4.5 The following final conclusions were drawn after carrying out the above mentioned investigations:

i) The piezometers indicate that pressures under the standard bays are well within acceptable values and the analog studies do not indicate any excess gradients. Therefore the bubbles have no significance either with regards to the safety or the operational performance of the barrage.

ii) The permeability of the subsoil is normal for the type encountered, and the underseepage velocities are well below veolocities which could cause piping of material from below the barrage floor.

iii) That the bubbles are not originating from some underground source such as a gas field or strata of decaying organic matter.

5. HIGH PRESSURE UNDER UNDERSLUICE BAYS AND REMEDIAL MEASURES

5.1 High Pressure

After the formation of the pond in January 1968 the recording of the readings of the pressure points were started at pier 1 in the undersluice bays and piers 8, 16, 27 and 38 (10 points each as given in Fig. 5) in the standard bays. The reliable readings started in March after the piezometers were tested and purged. It was later observed that the pressures being recorded at pier 1, although acceptable for the cross-head at that time but would be excessive under the maximum design conditions of 36 feet. The pressures being recorded in the standard bays were well within the acceptable limits.

The presence of higher pressures in the undersluice bays gave great concern as the barrage could not be operated to its maximum limits. The project was the key to the R-Q-B–S Canal system and for the distribution of water from Mangla, any enforced closure or curtailment in the operation might have severe economic effects.

A high level meeting of all concerned agencies was called to discuss and suggest remedial measures to counteract these higher pressures.

5.2 INVESTIGATIONS AND PROPOSAL FOR REMEDIAL MEASURES

i) Cause of High Pressures in the Undersluice Bays

A study of the geological sections (Fig. 6) indicates that a clay layer about 10 feet thick extends downstream under the concrete block and stone aprons and to the right to about Pier No. 6. This clay layer forms an almost completely impermeable blanket which confines the underseepage flow until it reaches the edge of the clay layer where it can find its exit to tailwater. A head of water is required to force the underseepage flow to the exit point. The excess pressure below the under-sluice downstream floor reflects the additional head required to force the underseepage flow to the exit point at the edge of the clay layer over that which would be required to force the under seepage flow to exit through the adjacent concrete block apron if the clay layer obstruction was absent. It is therefore concluded that the higher pressure under the undersluice bays are caused by the presence of clay layer which extends beneath the concrete block and stone aprons downstream of the undersluice bays and obstructs the flow of normal seepage.

ii) Two proposals of remedial measures were discussed to deal with this problem (a) weighting of the floor with additional concrete to counteract the higher pressures and (b) relieve the pressure by installing relief wells into the sand gravel strata below the clay layer. The proposal of relief wells was finally accepted, as being more efficient and positive one.

Later on the electric analog studies confirmed that the clay layer was the cause of the higher pressures and that the proposed installation of 24 relief wells would relieve the pressure Fig. 6.1).

5.3 INSTALLATION OF RELIEF WELLS

i) Design of Relief Wells

The bottom of clay layer under downstream block aprons is located at approximately EL.665. Bottom of 80 feet long, 8-inch diameter slotted fiberglas s well screen will be set at approximately EL.600. The well screen will be surrounded with 7-inch thick filter pack.

The effective diameter of well is taken as 1.8 feet and area of slotted openings in wells screen through pervious aquifer taken as 12 percent of well screen surface.

Total area of well screen opening

$$\frac{12 \times 65 \times 3.14 \times 8}{100 \times 12} = 16.3 \text{ Sq.ft.}$$

Seepage will feed to well system from upstream pond through pervious aquifer under barrage and around right abutment.

Average distance from upstream cutoff to middle of relief well system along shortest path under barrage is about 400 feet. Average distance from reservoir and around abutment to middle of relief well system is approximately 1,000 feet along shortest path.

For the seepage flow under the barrage the flow net indicates about 2.5 flow channels per 10 equipotential drops. This gives a seepage shape factor of 0.25 which is also in close agreement with the approximate ratio D/(L+O.4D), suggested by H.R. Cedergren. (page 247 of his book "Seepage, Drainage and Flow Nets" published by John Wiley and Sons, Inc., 1967). In this ratio (L) is average distance from upstream cutoff to well system along shortest path and (D) is depth of aquifer :

$$100/(400+0.4\times100) = 0.23 \text{ say } 0.25$$

The seepage shape factor for the seepage flow around the abutment is about 0.10 on the basis of the ratio D/(L + 0.4D):

$$100 / (1000+ 0.4 \times 100) = 0.096 \text{ say } 0.10$$

From the pumping tests carried out on the left bank of the barrage during July 1968 an average coefficient of transmissibility of 133,000 gallons per day per foot of aquifer was obtained for the aquifer tapped by the relief well system. For an aquifer thickness of 100 feet the coefficient of permeability (K) will be 0.002 feet per second.

$$K = 133{,}000/7.5 \times 86\ 400 \times 100 = 0.002 \text{ ft/sec}$$

For the design head of 36 feet across the barrage, the quantity of seepage per lineal foot of aquifer to the relief well system in the downstream block apron of the undersluice bays is as follows :—

$q = K\ S\ h$

Where q = flow per lineal foot of aquifer in cfs.

S = seepage shape factor, (ratio of number flow channels to number of equipotential drops of the flow net, 0.25 under barrage and 0.10 around abutment)

K = coefficient of permeability (0.002 feet/sec.)

h = design head (36 feet)

$q = q_1 + q_2$

q_1 (seepage under barrage)

$0.002 \times 0.25 \times 36 = 0.018$ cfs/lin. ft

q_2 $.002 \times 0.10 \times 36 = 0.007$ cfs/lin.ft.

$0.018 + 0.007 = 0.025$ cfs per lin. ft of aquifer

The wells will be installed in the block aprons of the undersluice bays. Since each block is nominally five feet square the well spacings will also be increments of five feet.

In his book, "Seepage, Drainage and Flow Nets", Mr. H.R Cedargren has developed a method for computing (H_m) the head difference between the edge of the relief well and midpoints between the relief wells for a 100 percent penetrating well in an aquifer overlain by a relatively impervious layer, such as the clay layer, which underlies the filter of the concrete block aprons at Rasul Barrage undersluice bays. In this method, H_m is computed by applying a well formula for flow to an artesian well modified for 180 degrees, on the assumption that flow occurs only in the half-circle upstream of the wells. The modified formula is as follows :

$$H_m = \frac{Q \log_{10} (R/rw)}{1.37\ KD}$$

where

H_m = head difference between edge of well and midpoints between relief wells.
Q = discharge of well in cfs.
R = radius of influence, 1/2 well spacing, in feet.
r_w = radius of well in feet
K = permeability of aquifer in feet per second
D = depth of aquifer in feet

For the undersluice bays the piezometric levels in the aquifer acting on the base of the overlying clay layer would be as follows :–

(i) At edge of well, tailwater level plus H_w (the well losses)

(ii) at midpoints between wells, tailwater level plus H_w plus H_m

The aquifer under the downstream block and stone aprons of the undersluice bays is a confined aquifer (artesian) since it is overlain by the clay layer located underneath the block aprons and is under pressure. Model studies of reliefe wells for levees along the Mississippi River, United States of America, carried out by the U.S. Corps of Engineers are reported in ASCE Paper No. 2701 'Relief Well Systemfor Dams and Levees" by Messrs W.J. Turnbull and C.I. Mansur. These model studies show that for conditions of pervious substratum with relatively impervious top layer (such as exist at Rasul Barrage undersluice bays downstream block and stone apron areas) the difference in well flow is less than 3 percent as between 50 percent penetrating and 100 percent penetrating wells when the well spacing is less than 25 ft. Since the relief wells at Rasul Barrage will penetrate at least 65 percent of the aquifer and the well spacing will be less than 25 feet, the method of computation suggested by Mr. H.R. Cedargren for computing head midway between the relief wells is considered applicable for the relief well installation in the Rasul. Barrage underlsuice bays and will be used.

Try 20-feet spacing :

R (1/2 well spacing) = 10 feet

Q (discharge through one well) = 20 x 0.025 = 0.50 cfs

Allows 1.50 feet for all well losses (H_w)

Well radius r_w = 0.9 feet

Depth of acquifer D = 100 feet

Permeability K = 0.002 feet per second.

$$H_m = \frac{Q \log_{10}(R/r_w)}{1.37\,KD} = \frac{0.50 \times 1.05}{1.37 \times 0.002 \times 100} = 1.93 \text{ feet}$$

For the design cross-head of 36 feet the pressure at the edge of well will be 1.50/36, or 4.2% of crosshead and the pressure at the midpoint will be (1.50 + 1.90)/36, or 9.5% of crosshead. Assuming parabolic distribution for H_m the average pressure above tailwater will be approximately 7.7 percent, or about 1.7 percent above the khosla theoretical phi-value of 6 percent at piezometer point 10 (located just below the filter under the block apron) which would be the expected pressure there if the clay layer were absent).

Try 15-feet spacing for the relief wells :

R (1/2 well spacing) = 7.5 feet

Q (discharge through one well) = 15 x 0.025 = 0.38 cfs

H_w (well losses) = 1.15 feet, (0.38 cfs x 1.50 ft = 1.15 ft)

0.50 cfs

$$H_m = \frac{Q \log_{10}(R/R_w)}{1.37\,KD} = \frac{0.38 \times 0.92}{1.37 \times 0.002 \times 100} = 1.28 \text{ feet}$$

For the design crosshead of 36 feet the pressure at edge of well will be 1.15/36 or 3.2% of crosshead, and the pressure at the midpoint will be (1.15 + 1.25) /36, or 6.7% of the crosshead. Assuming parabolic distribution, the average pressure above tailwater will be approximately 5.5% of crosshead.

The 15 foot spacing is recommended. For this spacing under 36 feet crosshead the uplift factors under the floor should be slightly below the Khosla theoretical uplift factors that would be expected if the clay layer did not exist. Although the pressures under the 20 foot spacing can probably be safely tolerated the 15-foot spacing gives reserve capacity which is prudent in installations of this type. Therefore 24 relief wells, 80 feet deep and spacing of 15 feet 6 inches are recommended for installation in the downstream concrete block aprons of the undersluice bays.

ii) Construction

The installation of Relief Wells was done through M/s. Cogefar Astaldi as a variation order of Rasul Barrage Project and was divided into two parts : Temporary and permanent works. The location plan, geology and main steps of construction are shown on figures 7 to 12.

a) Temporary Works

The contractor started construction of the relief wells in September 1968 by placing an earth fill embankment across the undersluice bays on the downstream side.

The relief wells were positioned (Fig. 7) through the concrete blocks in the 1 on 7 downstream slope of the block aprons, some 25 ft 6 inches downstream of the edge of the concrete floor and evenly spaced between the left abutment of the barrage and the left divide wall, that is 15 feet 6 inches apart.

An earth bund cofferdam/working platform was constructed with a top elevation of 718.00 and extended from the left downstream guide bank across the block apron and closed with the gates from Bay No. 1 to 6 of the barrage undersluices.

At each well location, a steel sheet pile ciasson was driven (Fig. 9) 12 feet in diameter, down through the work platform upto the top of the block apron and the material inside the ciasson was removed by a clam shell bucket grab. The inside of the ciasson was kept full of water upto top level by pumps after removal of the material from the caisson.

After cleaning two numbers, 2 inch expanding shackle bolts were drilled into the existing apron block with the help of a diver and the block lifted by the crane.

b) Permanent works

The new concrete apron blocks were casted each having a 28 inch diameter hole through the block, so set to be vertical when the block was placed in the 1 on 7 apron and a recess for fitting a M.S. cover plate to the well. Each block was lowered down into psition on a special cradle set at 1 on 7 slope to ease insertion in the block apron by the diver.

By using the idrosonda drilling machine for the drilling of the relief wells, 80 feet deep wells were quickly completed. The elevation of the top and bottom of the clay layer being noted, by the change in wash water.

Twenty four relief wells were installed at 15 feet 6 inches interval in a line from left abutment to pier 6. The well screen was fibreglass 80 feet along 8 inch internal diameter, with 2 inch long by 3/32 inch wide slots cut in eight rows around the circumference at 15 degrees from the horizontal. Each row has 23 slots per foot length of pipe. Thus each foot length of well screen has 14.5 square inches of openings, which equals 11.5 percent of the surface area of the screen. The bottom 4 feet was blanked off, as were any lengths which were located in the clay layers. Annular rings, one every 20 feet were provided to keep the fibreglass screen in the centre of the drilled hole. The screen was then lowered in one completely assembled length by the Marion 101 crane guided by the diver standing on the blocks within the caisson. A brass cap was provided at the top of the screen to stop entry of material.

The slotted pipe screen is surrounded by an annular 7 inch thick filter pack consisting of coarse to medium sand with a little fine gravel was placed around the fibreglass well screen and the steel borehole casing was then withdrawn. The filter was designed to protect the screen slots from being clogged by the filter pack, which in turn was designed to prevent the escape of the foundation sands and silts through the well to the river. The gradation curves of the sands and silts of the foundation and of the filter material are given in Fig. 12.

A slotted metal cover was then bolted into the recess left in the new apron block, the 28 inch hole in the block having been previously filled with a graded filter of maximum size 1½ inches. The relief wells were designed to relieve pressure under the floor by permitting water to enter each well through filter pack and the slots in the screen and escape to tail water through openings in the steel cover.

On completion of all the wells, two leading cuts were excavated through the temporary earth cofferdam platform and by passing water through the undersluice bays, the earth was slowly erorded away.

The work of relief wells was commenced in September 1968 and completed in November 1968. In March 1969 there was sufficient water to scour out the remains of the

temporary cofferdam and also material deposited on the floor and aprons by barrage operation. The pressures observed at pier No. 1 in the undersluice bays showed a marked reduction and were well within accetable limits. The relief wells were therefore functioning and relieving the pressure caused by the clay layer.

6.0 PIEZOMETRIC READINGS

The daily readings of piezometric levels at the barrage have been observed since March 1968. Plots of theoretical Phi values and readings taken at Piers 1, 8, 16, 27 and 38 during March, May, June, August and September 1968 are shown on figure 13. Pier 1 is located in the under-sluice - bay area whereas Piers 8, 16, 27 and 38 lie to the right in the standard-bay area.

An examination of the Phi values (Fig. 14) at Piers 1 and 8 indicate that the readings were even influenced during the installation of the wells. There are some increase in pressure under the undersluice floor at the start of construction period due to further restriction of under-seepage flow caused by the contractor's temporary embankment but there was no danger because the floor was loaded with earth.

The installation of the 24 relief wells was completed on November 16, 1968 and clearing of the earth embankment from the well area then began. The piezometers immediately showed a reduction of pressures under the floor. When removal of the embankment was completed in December 1968, the free discharge from the wells then caused a marked reduction in the pressures and in the corresponding phi-values computed for points 8 and 9 as given in Figure 14. A comparison of the phi-values obtained after installation of the wells with earlier readings demonstrate the effectiveness and utility of the wells.

Subsequently, the pond was dewatered during the two weeks annual closure of the barrage from 25th December 1968. This dewatering of the pond caused an accumulation of sediment deposits varying from 2 to 16 feet in depth on all downstream floors and aprons. These sediment deposits caused a general rise in piezometric pressures as indicated by the readings taken at Piers 1, 8, 16, 27 and 38 during February 1969 (Fig. 14). During March there was sufficient flow in the river to permit releases below the barrage, the sediment deposits were flushed away and the readings of April 1969 show rapid decrease of piezometric pressures again.

The phi-values upto July 1969 based on piezometric readings for pressure points 8 and 9 were upto 75 percent at Pier 1, of the Khosla theoractical phi-values.

The figure 14 gives comparison of phi-values for various representative periods of time from May 1968 to December 1984 for pressure points 8 and 9 at each of the five piers. The response at all the piezometer points of such an event as the accumulation of sediment on the block aprons and cleaning of relief wells is apparent from this data.

7.0 PERFORMANCE OF RELIEF WELLS

After the cofferdam/platform were washed away the relief wells started normal functioning and their effectiveness was immediately demonstrated by the significant reductions noted in the uplift pressures and phi-values of the piezometers at piers 1 & 8.

It was not feasible to install any type of device in the wells which could demonstrate their rate of discharge, but their performance is indicated in a relative manner by the absence of sediment in a conical area above each well and by the response of the piezometers to changes relating to the wells. When the sediment deposits were sluiced from the pond to the downstream apron area around the wells in late December 1968, the effect upon the phi-values was evident until that sediment was washed away in late March 1969.

The wells have been designed to release relatively large flows of water continously with a minimal velocity of flow through the slots of the screens. Since the wells are designed for gravity flow and are not pumped, the head differential from the sand stratum to the intirior of the well is relatively low. Consequently the physical conditions which might lead to deposition of calcium and magnesium carbonates on the screens are virtually absent.

The chemical factors affecting the clogging of well screens were also evaluated to determine the likelihood that incrustations will develop. The chemical analyses of three samples of water taken from a test well at the left abutment of the barrage in July 1968 was carried out. This well was within 100 feet of the location where the nearest of the relief wells were subsequently installed.

Since the test well is within the same zone of seepage flow under the barrage as the relief wells, the water pumped from the test well is considered to be representative of the effluent from the relief wells. The chemical analyses of river water samples taken upstream and downstream of the barrage was also done. The comparison of the analyses of both sets of samples show that the groundwater is approximately the same as the river water but has about 20 percent more Ca and Mg ions. It is the carbonates of these mineral ions which cause incrustations, particularly when super-saturated solutions of these minerals are present in the well waters or when there is a high iron content in the water.

The iron bacteria live in groundwater which has an appreciable iron content or they may be inadvertently introduced into a well during its construction or during maintenance. These bacteria will thrive only if the water contains appreciable amounts of iron or if steel pipe is present in the well. Iron bacteria produce a slime which can cause stoppage around a well screen.

It has been found that incrustations on well screens are not likely to occur if the concentration of Ca plus Mg ions is less than 3.0 parts per million (ppm). Similarly, iron

concentrations in the water of less than 0.5 ppm will cause no incrustation problems, either by (a) oxidation of the iron or (b) by supporting the growth of iron bacteria.

In view of these favourable chemistry factors of the well waters, the clogging of screen is not likely to occur due to deposition of calcium, magnesium, or iron minerals.

As a consequence of the careful design and construction of the 24 relief wells, it is anticipated that these wells will operate for several years without specific need for the wells to be opened for inspection or maintenance.

8.0 SURVEILLANCE

In order to make sure that the relief wells are performing their design function adequately, it is recommended that the following observations be performed at regular intervals.

i) The piezometer readings at Piers 1 and 8 should be taken for atleast two consecutive days each week. The correspondence phi-values should be determined, plotted, and compared to the past record and to the design values.

ii) Sediment deposit measurement should also be made at monthly intervals upstream of the barrage and, particularly, downstream of the barrage in the vicinity of the relief wells.

iii) Occasional observations should be made with the help of diver to verify that the discharge from the wells is able to escape upward through any sediment blanket which is deposited above the wells.

Efforts should be made to avoid frequent sluicing of sediment deposits from the pocket/ reservoir onto the apron area around the relief wells. If sediment deposits do occur in the area of the wells, efforts should be made to flush these away before they become unduly compacted.

9.0 MAINTENANCE OF RELIEF WELLS

If the phi-values show a long term tendency to increase without any apparent reason, such as the effects of sediment deposits, re-trogression, or other factors, it may indicate that the screens of the wells are gradually getting clogged. If the factor of safety against floatation for the downstream floor approaches 1.1 plans should be prepared to open the relief wells and inspect them during low water period, preferably during the annual closure of the barrage. The retrogression of the downstream river channel may permit ready access to the covers of the wells to allow their removal otherwise divers will be required to remove the covers. Efforts should be made to get samples of incrustations, if these exist, or of the water, whereby the nature of the trouble can

be determined. The water samples should be analyzed and also examined under the microscope by skilled technicians to determine the presence of iron bacteria. If the screens have become clogged by carbonate deposits or the action of iron bacteria, conventional cleaning procedures can be used.

The Author visited the Rasul Barrage in June 1985 and observed that sediment deposits were accumulated on the downstream aprons all along the barrage as sufficient discharges are not released so that these sediment could be washed away. Accretion has taken place and there is no possibility of retrogression because of limited discharges passing occasionally through the barrage even during flood season. It was found that increase in pressures was noticed on points 8 & 9 of piers 1 and 8 and relief wells were opened during 1984. The silt was filled inside the fibre glass pipe and the same was cleared with the help of compressed air. The crew including diver of Mangla Dam were deputed to carry out the cleaning operation of the relief wells. The relief wells were cleared of silt first time during 1974 under the supevision of the Author by lowering two pipes inside the relief well, one for the entry of compressed air and the other for flushing out the silt/mud from the well.

10.0 CONCLUSIONS

The installation of the relief wells in the downstream block aprons of the undersluice bays of the Barrage was completed in November 1968 and the relief wells were in full operation in early December 1968. Based on their performance to date, according to the readings of pressure points at Piers 1 and 8, the final conclusions are as follows :–

i) The system of 24 relief wells has reduced the excessive pressures under the downstream floor of the undersluice bays to safe limits.

ii) The performance of the relief well system confirms that the well spacing and well design are satisfactory.

iii) As practical operating requirements permit, the gates in the undersluice bays should be regulated so as to minimize deposit of sediments on the downstream aprons, and if such deposits occur they should be removed by flushing at the earliest opportunity to minimise the possibility of their consolidation and blockage of the wells.

iv) The operation of relief wells should be monitored by taking readings and recording for atleast two consecutive days each week the piezometric levels in the Piers 1 and 8, the sediment levels on the upstream and downstream aprons should be measured and recorded monthly. The piezometer and corresponding phi-values should be plotted and reviewed so that any trends in the pressures can be studied as they

develop and corrective action taken whenever it becomes necessary.

v) The relief wells are expected to operate satisfactorily. Incrustation due to calcification or blockage due to accumulation of any type of deposits on the screens is not likely to occur because of the conservative design of the well, the excellent quality of the water, and the low differential head across the well screen required by the gravity flow design. However, if some incrustation occurs after a long period of operation this will probably be indicated by the gradual increase in the phi-values determined from observations of piezometers at Piers 1 and 8 with due allowance for temporary changes caused by sediment deposits on the aprons or other factors. If the phi-values increase to the level where the factor of safety against floatation of the downstream floor approaches 1.1, the wells should be inspected to determine the nature of the problem and appropriate remedial action taken immediately.

vi) An inspection of the relief wells screen with TV Camera during 1974 and 1984 indicated that no incrustations were found and no clogging of the screen occured. This has proved that the presence of Ca plus Mg ions and iron concentration in the water remained within prescribed limits.

11. RECOMMENDATIONS

The following recommendations may be followed for the barrages to be constructed in future.

i) Pressure points may be provided at every 4th bay as at present the spacing is about double this number.

ii) Daily readings may be taken on all the pressure points during first year of operation of a barrage.

iii) Piezometric readings should be taken only in stabilized conditions of upstream and downstream water levels.

iv) Accumulation of silt should not be allowed on the concrete block apron.

v) Four rows of bore holes may be drilled in every bay, one at the end of upstream concrete floor ; second at the centre of the barrage ; third at the end of downstream concrete floor and the fourth at the end of concrete block apron. The data from these bore holes will help to locate the soil profile likely to block the underseepage and preventive measures could be taken at the time of detailed

design.

If economically feasible, removal of clay layers and replacing with similar sandy fill material as existing under the barrage will be best alternative to avoid blocking the underseepage otherwise some other suitable measure will have to be adopted.

vi) The fibreglass pipe/screen can be safely used for the installation of relief/drainage wells.

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2. Report on under seepage at Rasul Barrage Volume I, II & III by Harza Engineering Company International (October 1968)

3. Performance of relief Wells at Rasul Barrage by Harza Engineering Company International (October 1969)

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2.	General Plan of Barrage and Allied works
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5.	Location of Pressue Points.
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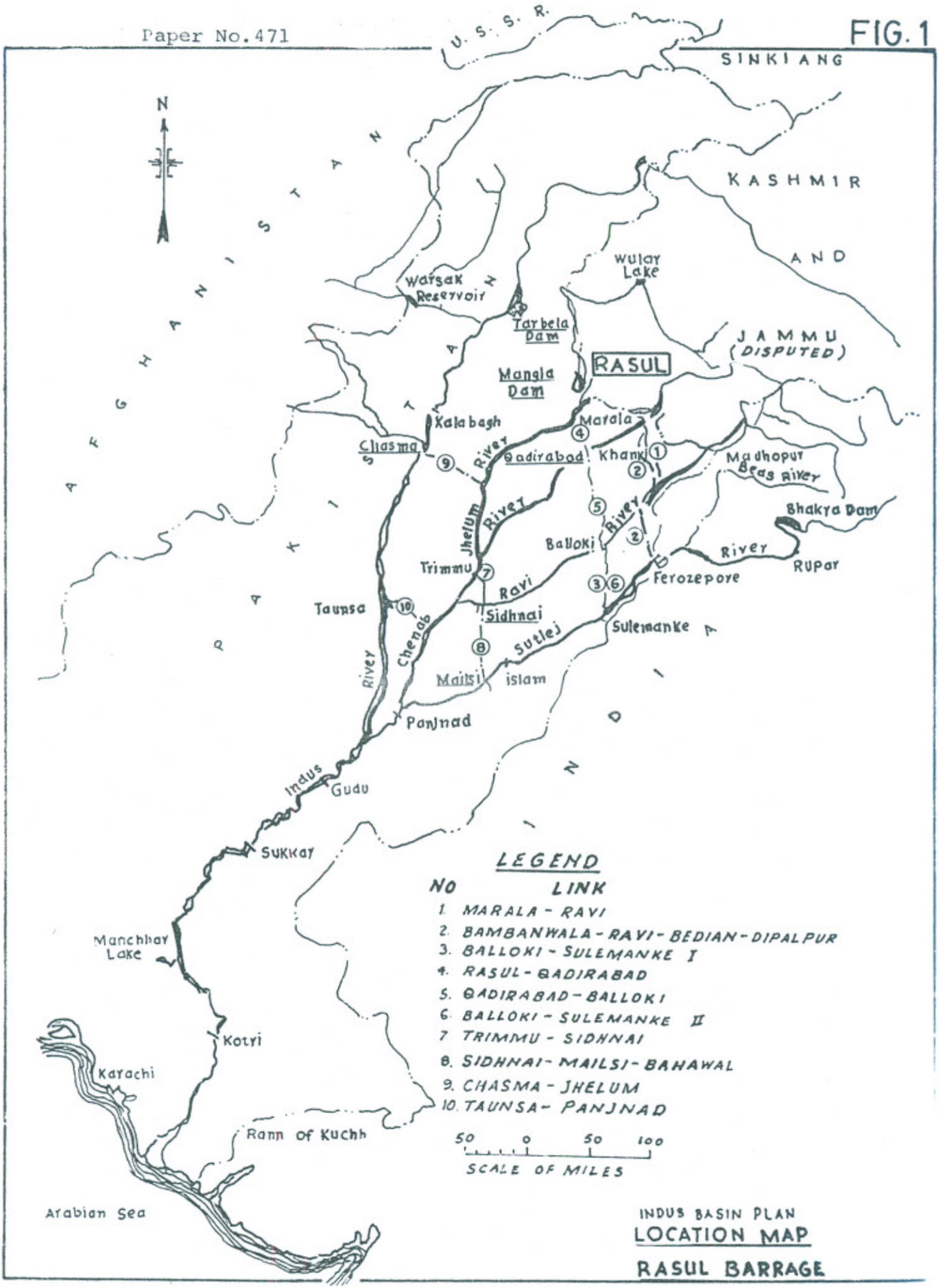
10.	Relief wells construction steps
11.	Steel cover plate details
12.	Filter details for Relief wells
13.	Comparison of Phi valves
14.	Phi valves for downstream floor (points 8 & 9)

FIG. 1

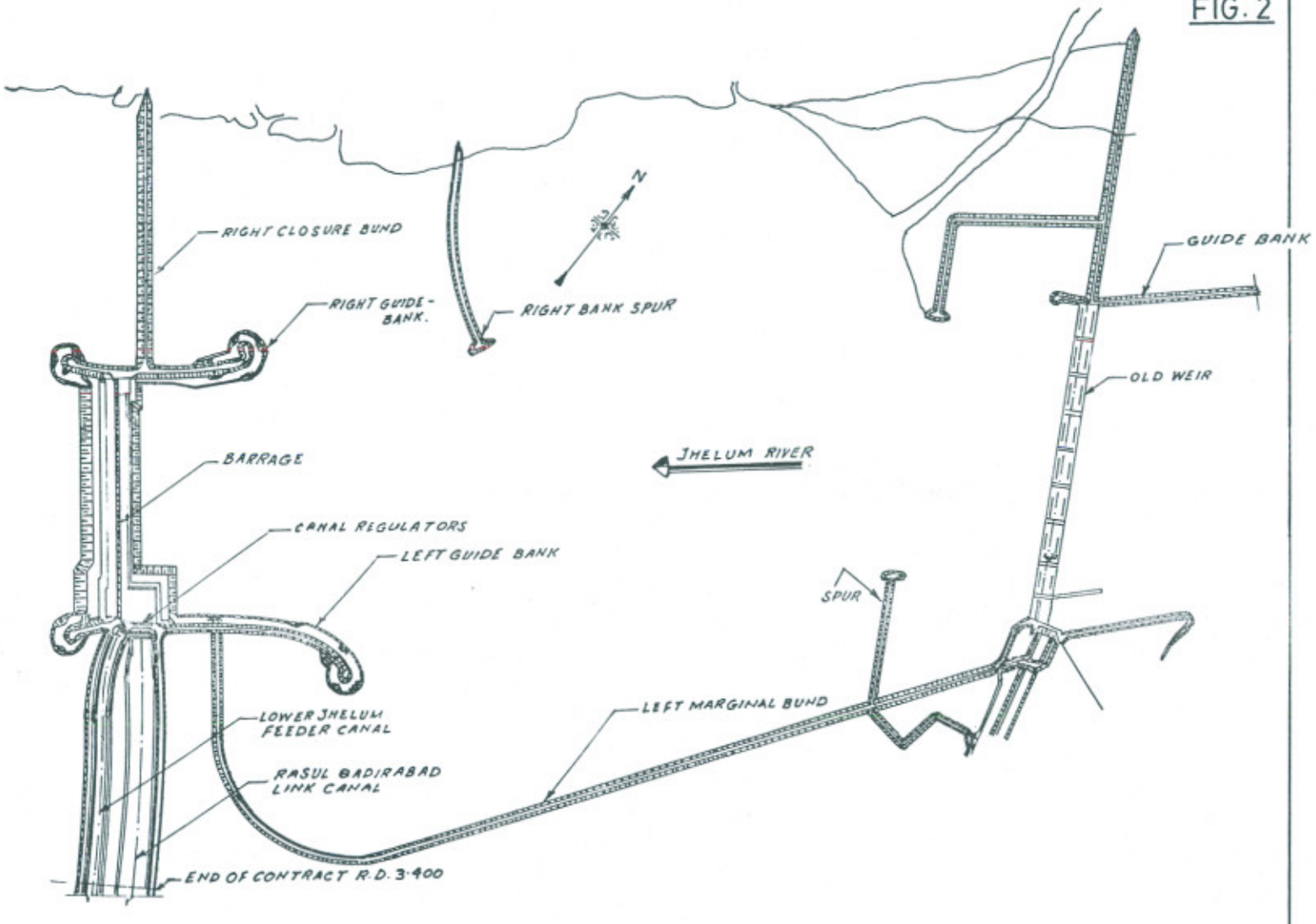


INDUS BASIN PLAN

LOCATION MAP

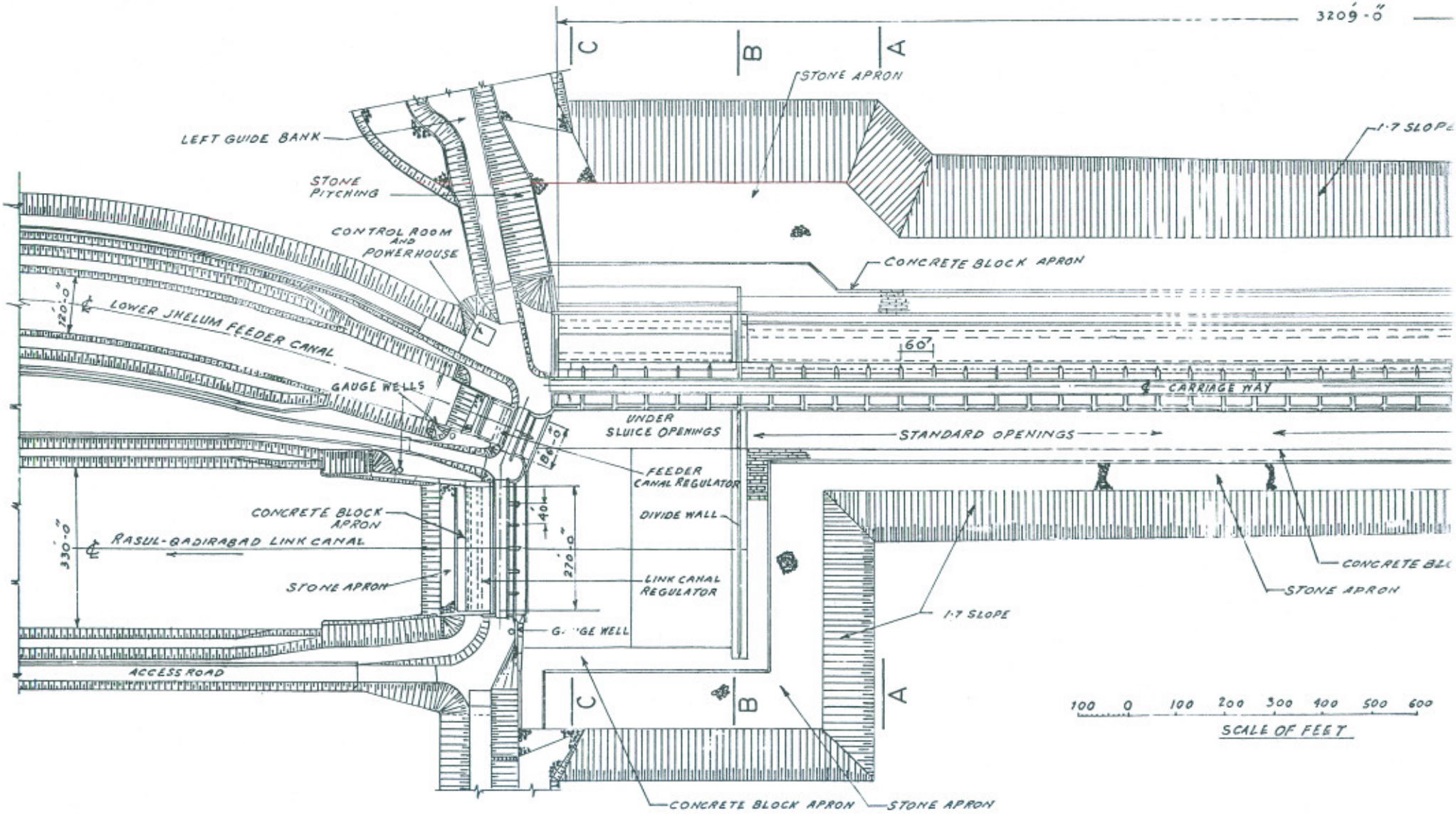
RASUL BARRAGE

FIG. 2



GENERAL PLAN OF RASUL BARRAGE WORKS.

GENERAL ARRANGEMENT OF RASUL BARRAGE AND REGULATORS



OF RASUL BARRAGE AND REGULATORS

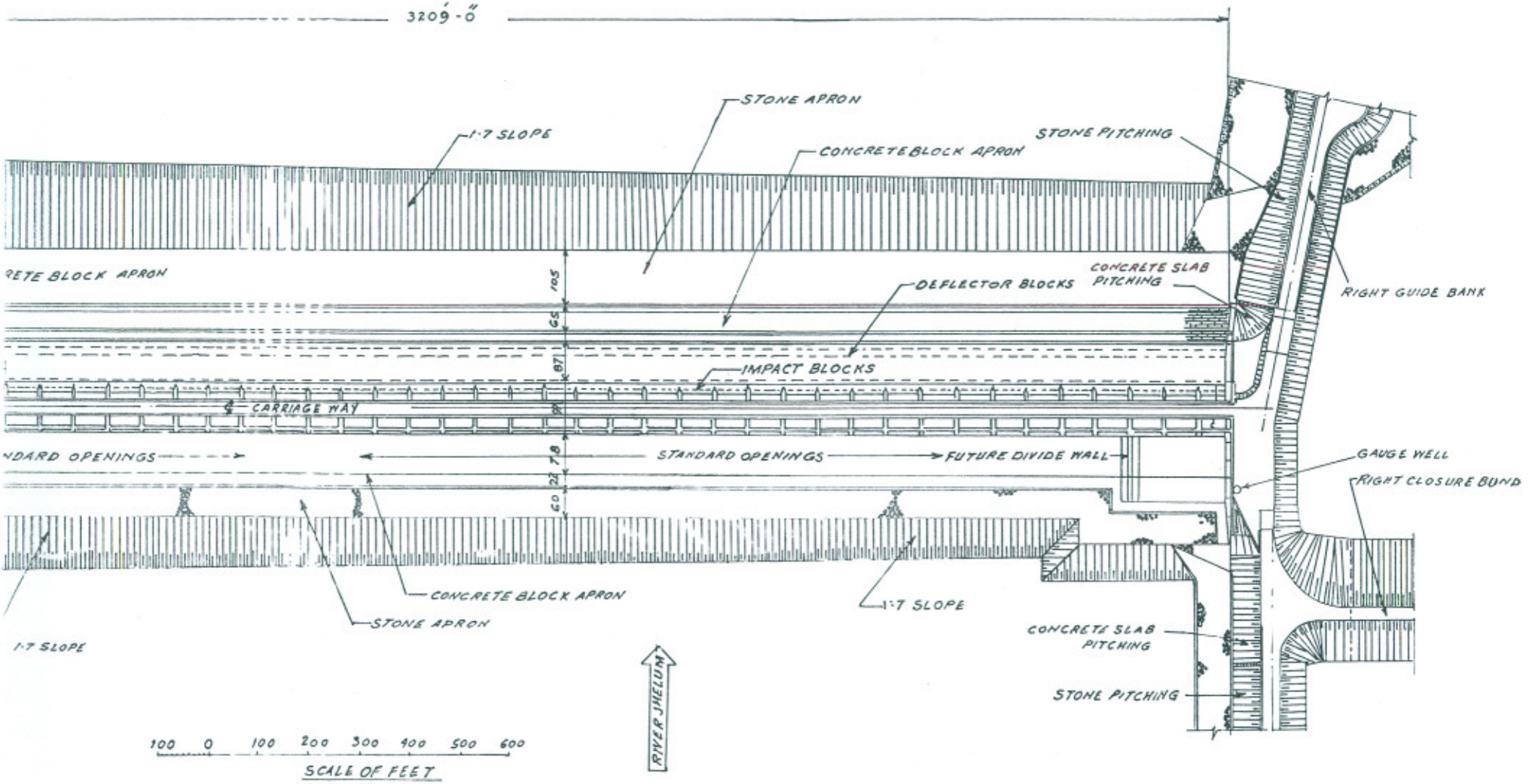
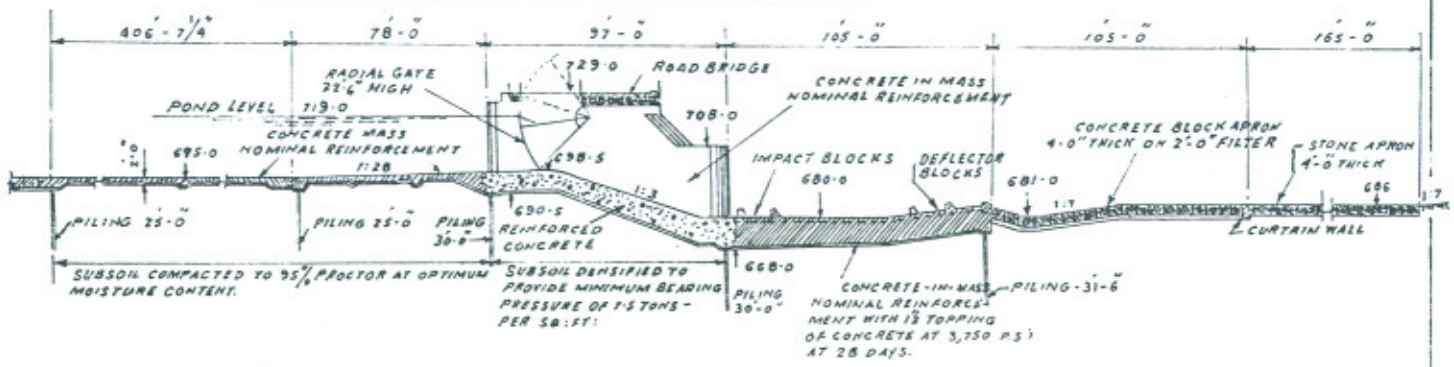


FIG. 4

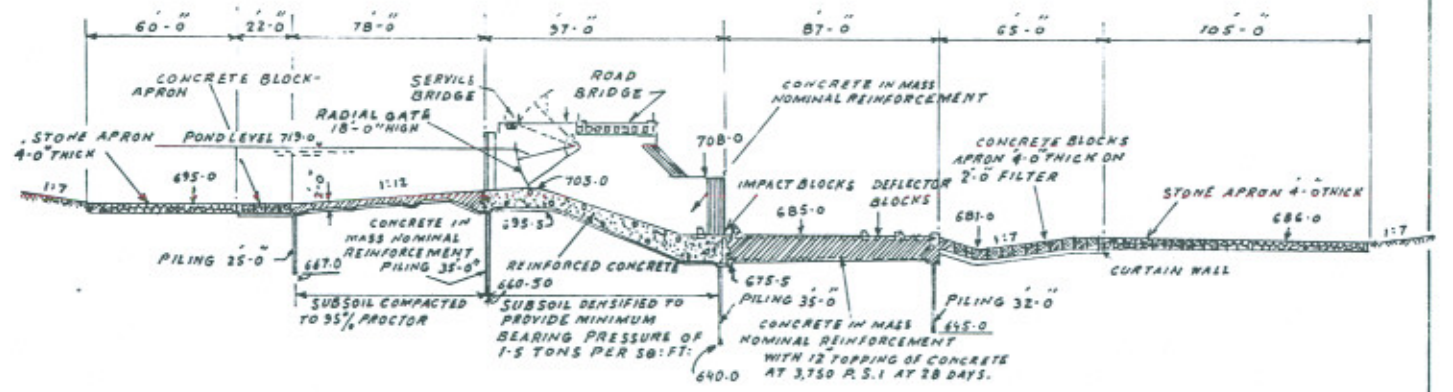
Paper No. 471

RASUL BARRAGE

TYPICAL CROSS SECTION - UNDERSLUICE BAY

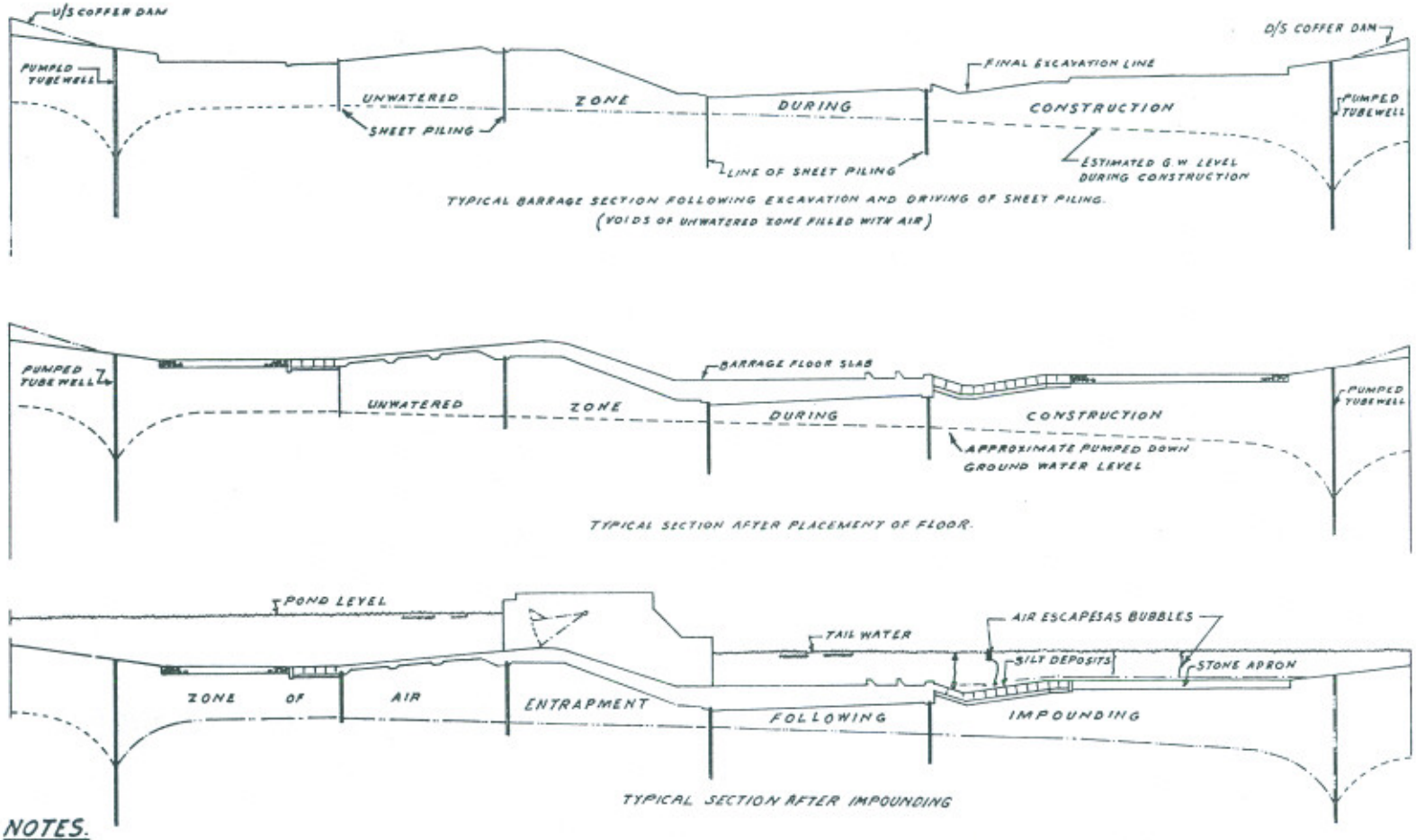


TYPICAL CROSS SECTION - STANDARD BAY



TYPICAL CROSS SECTIONS

Fig. 4.1

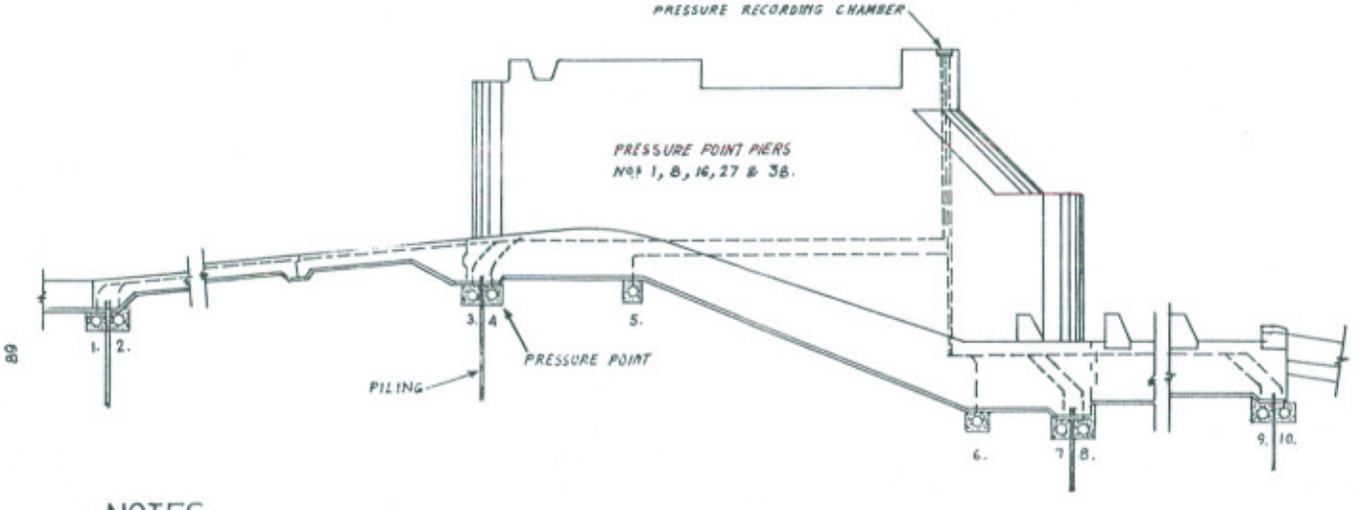


NOTES.

1. A PORTION OF AIR IN VOIDS DURING DEWATERING REMAINS TRAPPED DUE TO SURFACE TENSION AS WATER RISES IN THE GROUND.
2. VOLUME OF TRAPPED AIR COMPUTED ON ASSUMPTION THAT ORIGINAL UNWATERED ZONE CONTAINED 30 PERCENT VOIDS, 10 PERCENT OF WHICH IS TRAPPED AIR.
3. ESTIMATED VOLUME TRAPPED AIR UNDER FLEXIBLE APRONS OF BARRAGE IS 500,000 C.FT.
4. ESTIMATED VOLUME OF TRAPPED AIR UNDER D/S FLOOR OF BARRAGE IS 100,000 C.FT.

AIR ENTRAPMENT SINCE CONSTRUCTION.

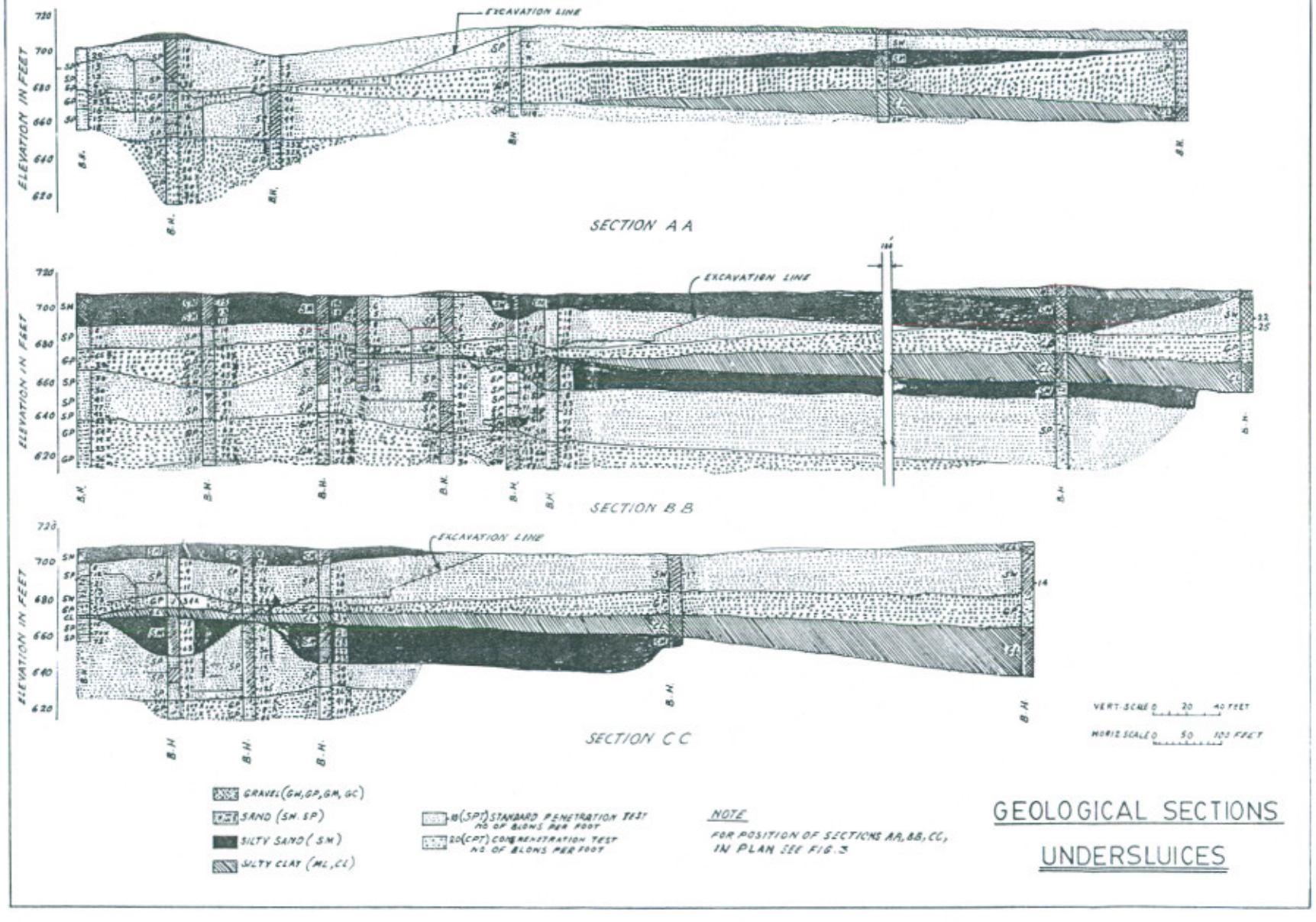
FIG. 5



NOTES

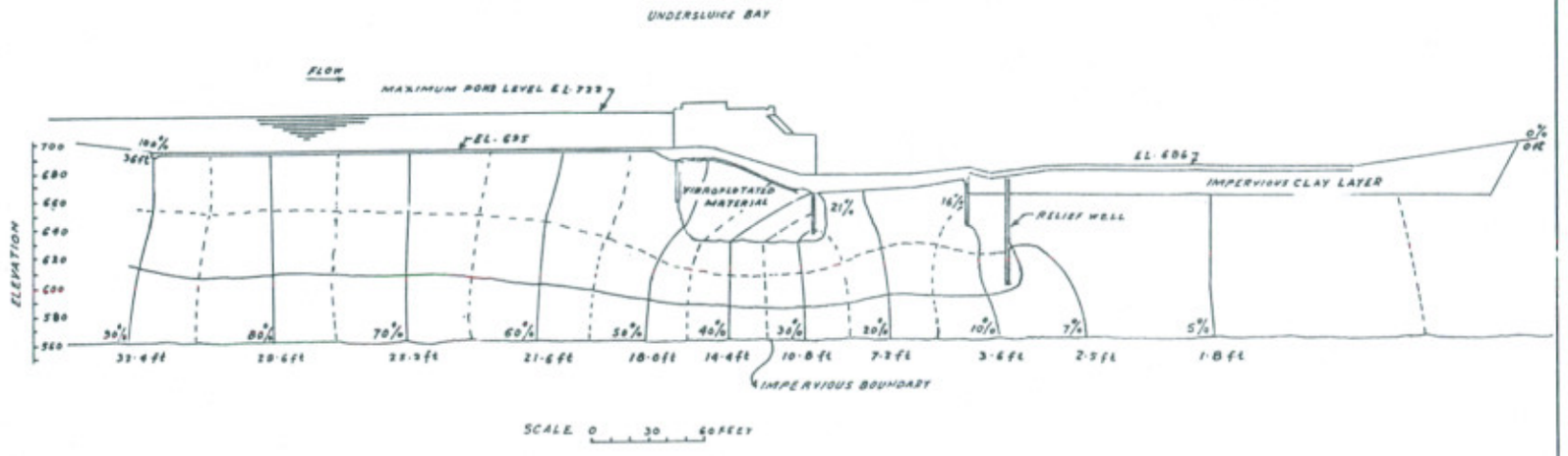
1. ℄ OF PRESSURE POINTS UNDER FLOOR COINCIDES WITH ℄ OF PIER IN PLAN. AT THESE POINTS PIERS ARE EITENDED BY 8'-6" DOWNSTREAM COMPARED WITH STANDARD PIER
2. THE PRESSURE POINTS PIER SKETCHED ABOVE IS A PIER FOR STANDARD BAY. THE ARRAGEMENT OF PRESSURE POINTS FOR UNDERSLUICES IS SIMILAR.

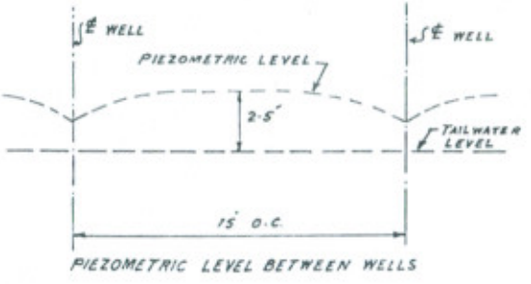
LOCATION OF PRESSURE POINTS



GEOLOGICAL SECTIONS
UNDERSLUICES

FIG:- 6·1





PIEZOMETRIC LEVEL BETWEEN WELLS

NOTES

1. AQUIFER YIELD 133,000 GPD PER ft, DETERMINED FROM LEFT BANK PUMPING TEST.
2. PERMEABILITY ASSUMED TO BE APPROXIMATELY THE SAME IN HORIZONTAL AND VERTICAL DIRECTIONS. AQUIFER ASSUMED 100 ft DEEP.
3. COMPUTED DISCHARGE OF RELIEF WELL 170 GPM

RASUL BARRAGE

ANALOG STUDY - 24 RELIEF WELLS IN OPERATION UNDER CROSSHEAD OF 36ft.

FIG.7

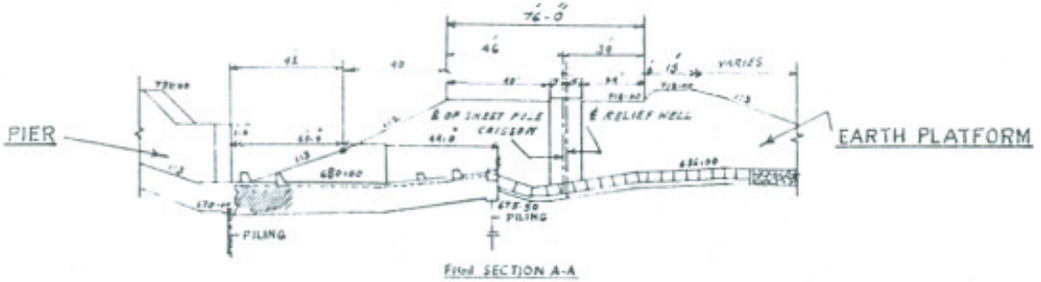


Fig.I SECTION A-A

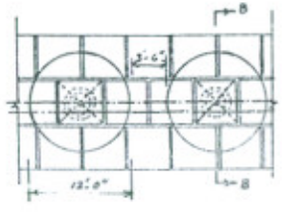


FIG.III PLAN SHOWING CELLS LOCATION & BLOCKS TO BE REMOVED & REPLACED

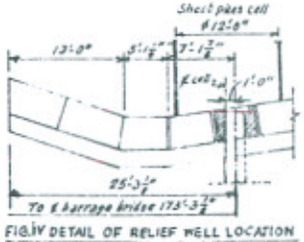


FIG.IV DETAIL OF RELIEF WELL LOCATION

B-B

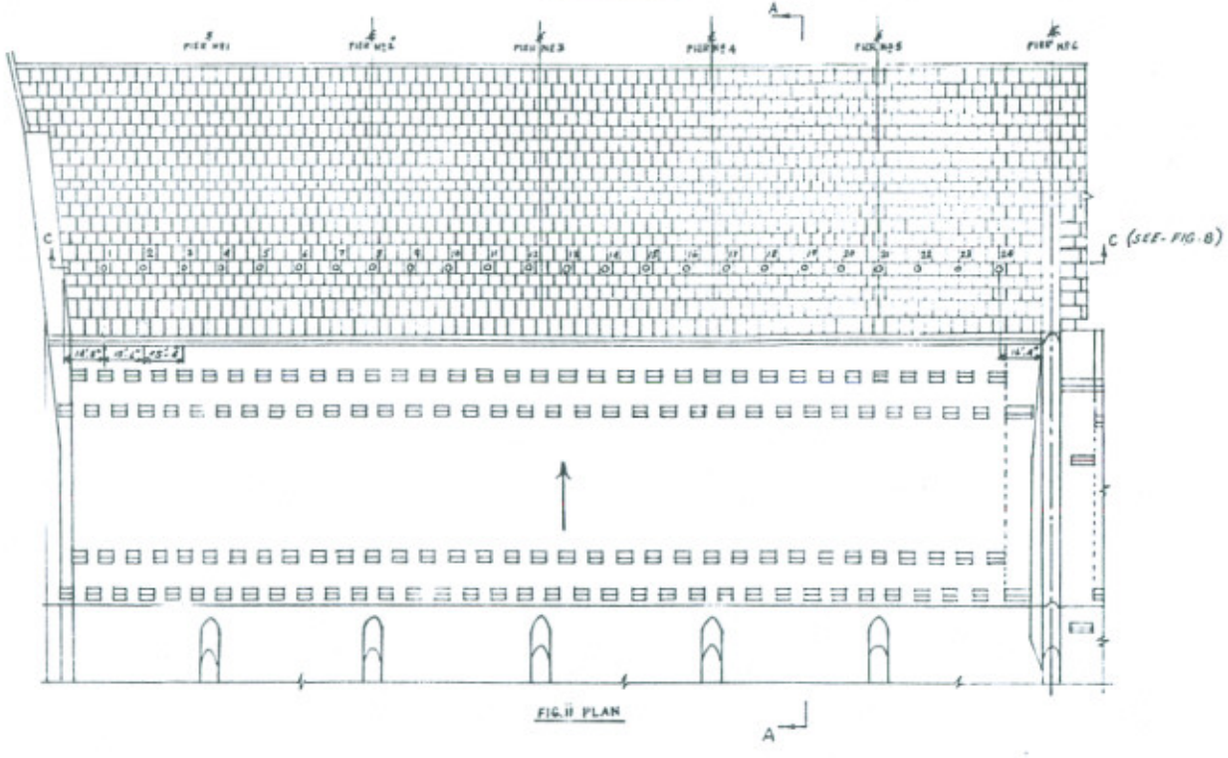


FIG.II PLAN

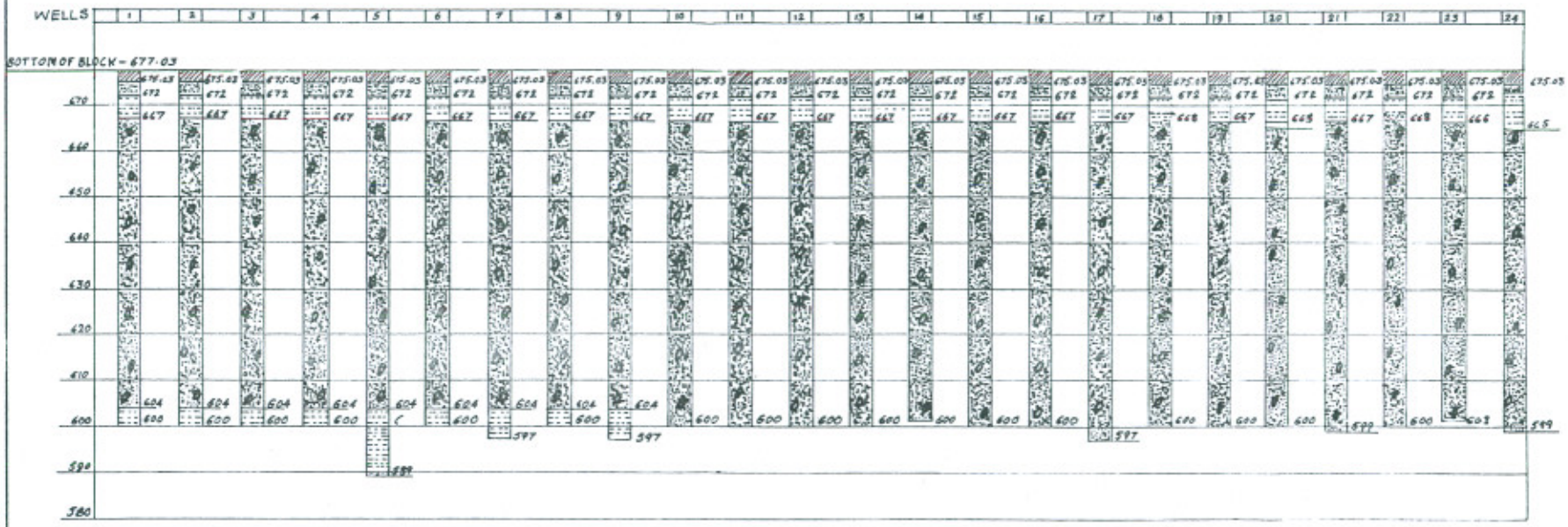
FIG. I, II 0 20 40 feet

FIG. III, IV 0 6 12 feet

SCALE

RASUL BARRAGE
RELIEF WELLS
LOCATION PLAN

Fig. 8



SECTION C-C
(FOR PLAN SEE FIG. 7)

LEGEND

SAND

GRAVEL

CLAY

FILTER UNDER THE BLOCK

BORE HOLE LOGS FOR 24 WELLS

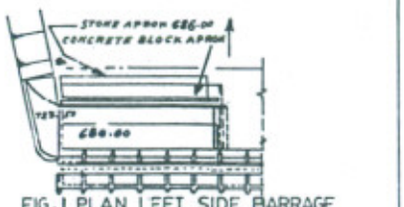


FIG.1 PLAN LEFT SIDE BARRAGE

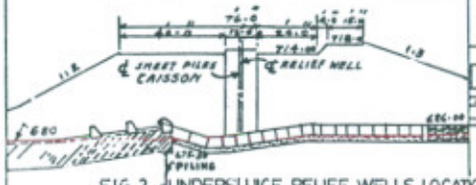


FIG.2 UNDERSLUICE RELIEF WELLS LOCATION

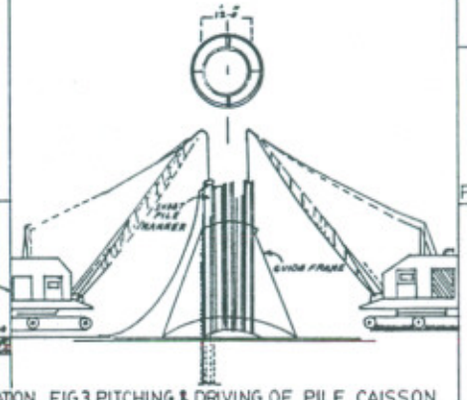


FIG.3 PITCHING & DRIVING OF PILE CAISSON

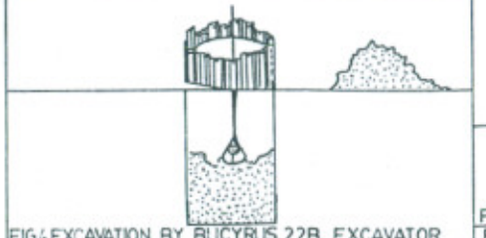


FIG.4 EXCAVATION BY BUCYRUS 22B EXCAVATOR



FIG.5 CLEANING WITH AIR WATER PUMP

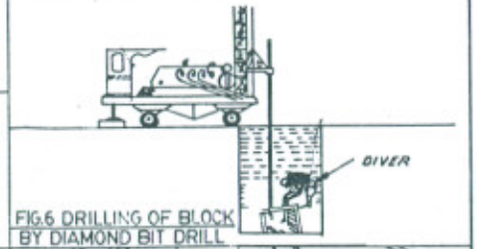


FIG.6 DRILLING OF BLOCK BY DIAMOND BIT DRILL

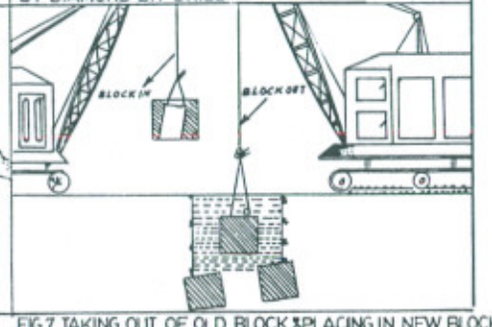


FIG.7 TAKING OUT OF OLD BLOCK & PLACING IN NEW BLOCK

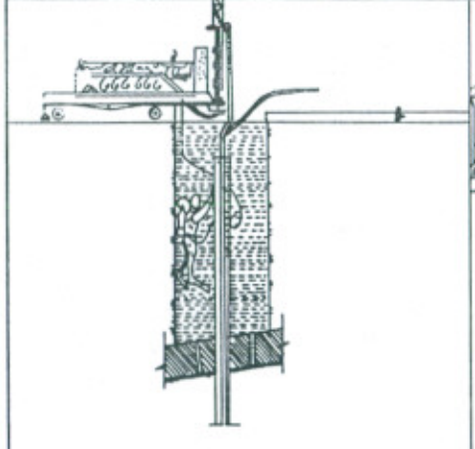


FIG.8 DRILLING WITH IDROSONDA MACHINE

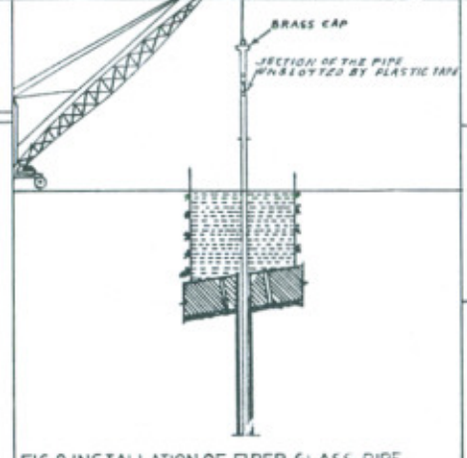


FIG.9 INSTALLATION OF FIBER GLASS PIPE

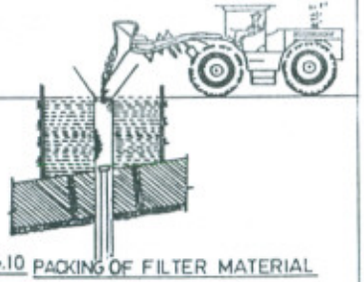


FIG.10 PACKING OF FILTER MATERIAL

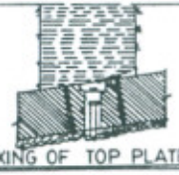
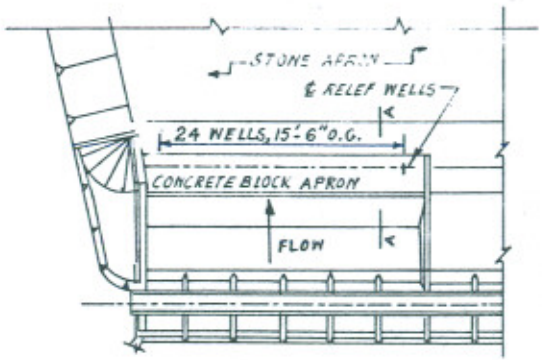
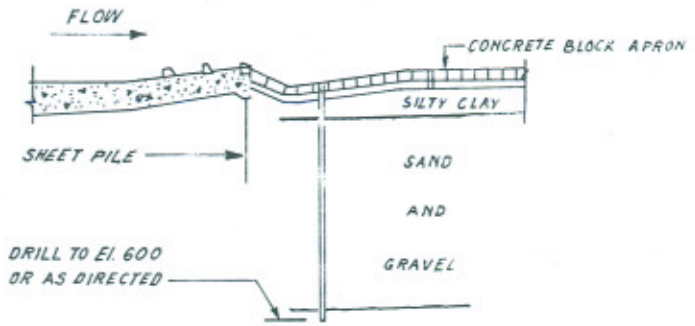


FIG.11 FIXING OF TOP PLATE COMPLETE

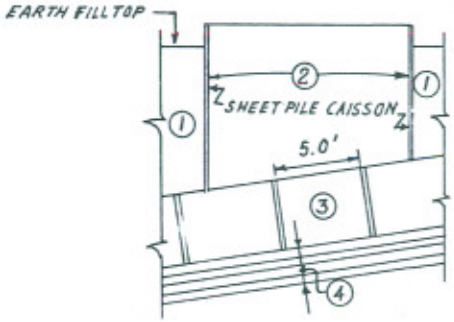
INSTALLATION DETAILS OF RELIEF WELLS



PLAN
LEFT SIDE BARRAGE



SECTION A - A

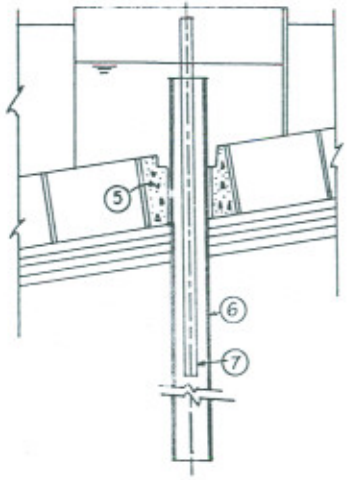


PHASE 1

1. PLACE WORKING FILL AT REQUIRED LEVEL
2. EXCAVATE AND UNWATER HOLE AS REQUIRED TO EXPOSE BLOCK
3. REMOVE BLOCK
4. CHECK, CLEAN AND REPLACE FILTER BENEATH BLOCK AS REQUIRED.

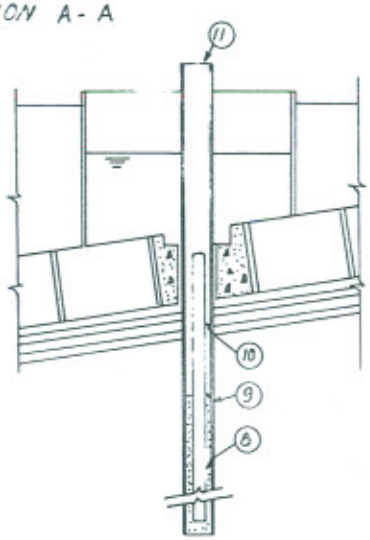
PHASE 2

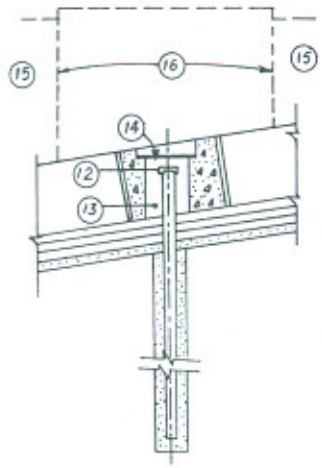
5. PLACE NEW PRECAST BLOCK
6. DRILL 22" HOLE WITH CASING TO DEPTH AS DIRECTED
7. PLACE SLOTTED FIBRE GLASS PIPE 8" DIA.



PHASE 3

6. PLACE FILTER PACK
9. PULL 22" CASING IN INCREMENTS AS FILTER IS PLACED
10. PLACE INVERTED FILTER MATERIALS INSIDE CASING AS IN EXISTING FILTER
11. REMOVE CASING





PHASE 4

12. PLACE CAP ON FIBRE GLASS PIPE
13. FILL SPACE WITH FILTER UP TO 1/4" SIZE
14. PLACE COVER
15. REMOVE ALL WORKING FILL MATERIAL
16. REMOVE SHEET PILES

RELIEF WELLS
CONSTRUCTION STEPS

Fig. 11

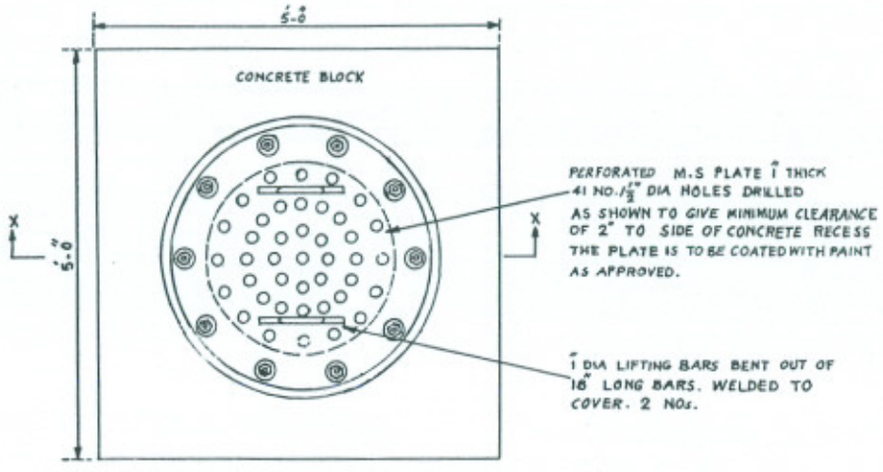


FIG. 1 PLAN

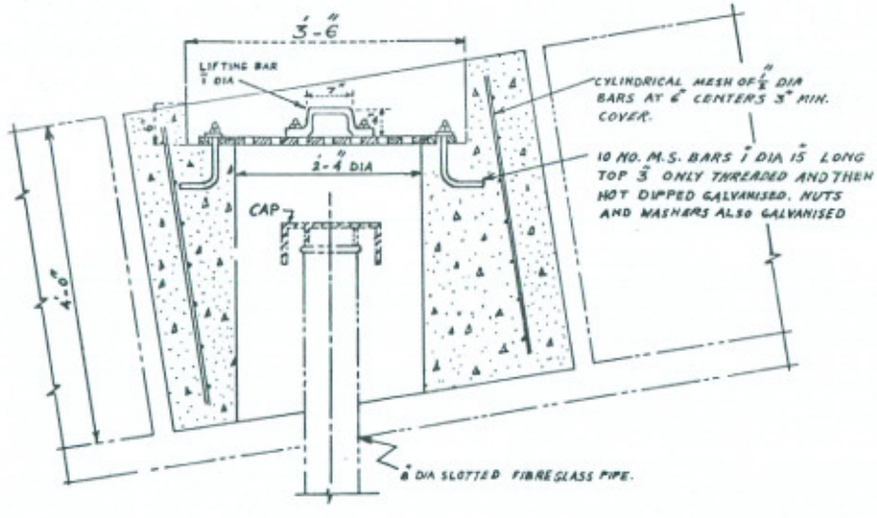
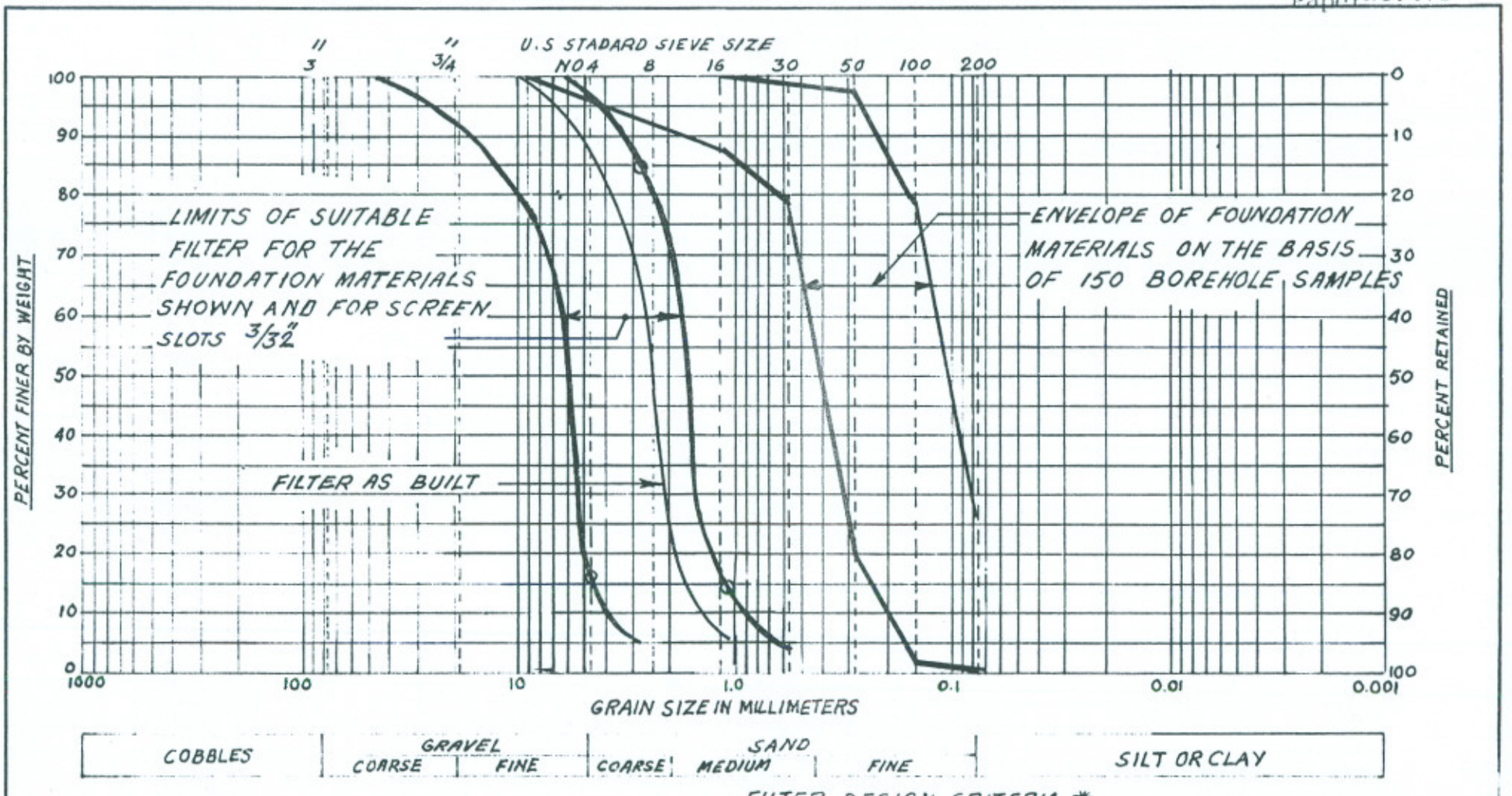


FIG. 2 SECTION X X

STEEL COVER PLATE DETAILS



FILTER DETAILS FOR RELIEF WELLS

FILTER DESIGN CRITERIA *

1. MAX. LINE IS DETERMINED BY $\frac{D15\ (OF\ FILTER)}{D15\ (OF\ FOUNDATION)} \leq 5$
2. $D85\ (OF\ FOUNDATION)$
3. MIN. LINE IS DETERMINED BY

$\frac{D15\ (OF\ FILTER)}{D15\ (OF\ FOUNDATION)} \geq 5$

$D85\ (OF\ FILTER) \geq 1.2$ MAX. SIZE OF SLOT

* CEDERGREN, H.R. "SEEPAGE, DRAINAGE AND FLOW NETS"

FIG. 17

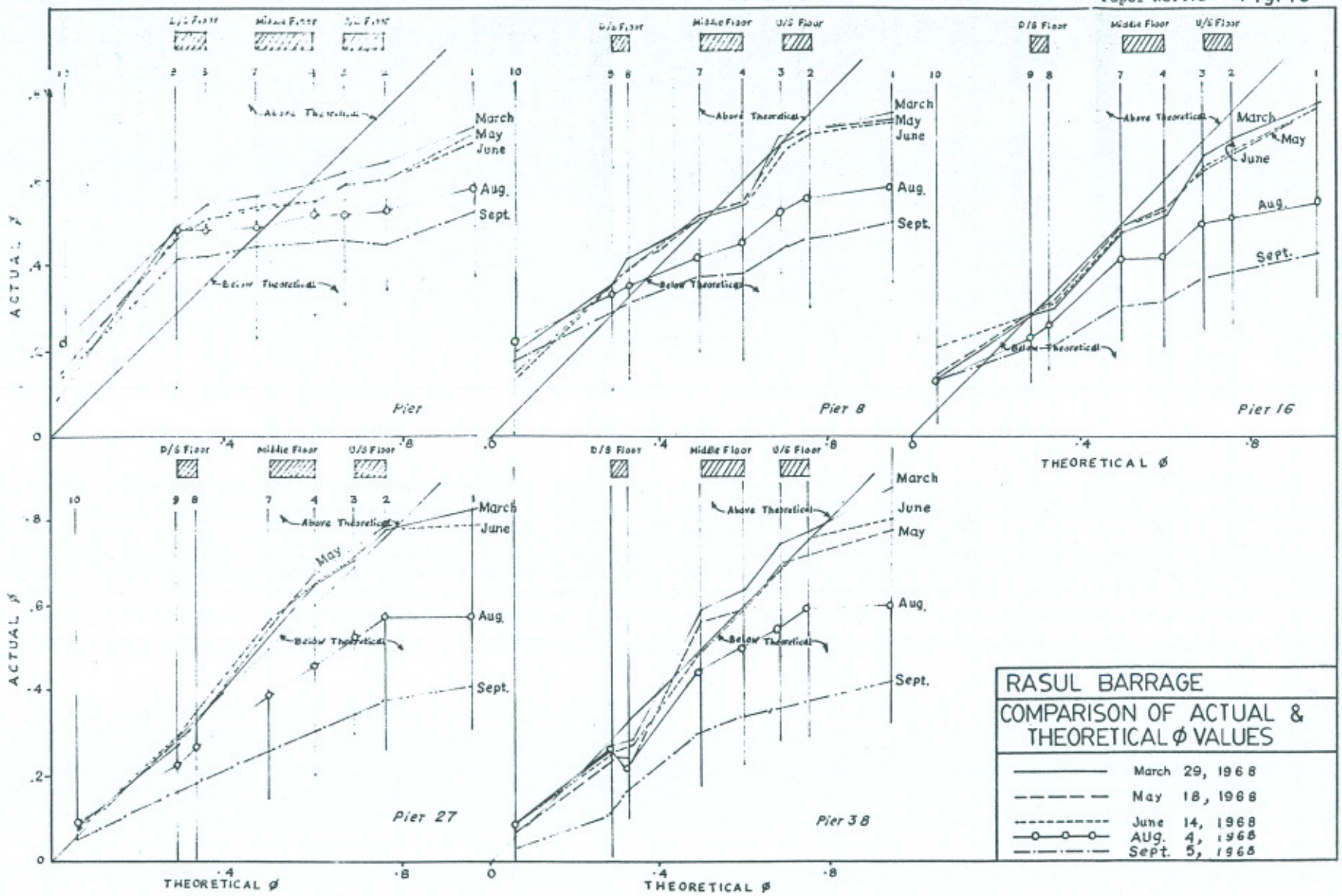

D/S Floor
Middle Floor
U/S Floor
March
May
June
Aug.
Sept.
Above Theoretical
Below Theoretical
Pier
Pier 8
Pier 16
Pier 27
Pier 38
ACTUAL ϕ
THEORETICAL ϕ
RASUL BARRAGE
COMPARISON OF ACTUAL & THEORETICAL ϕ VALUES
March 29, 1968
May 18, 1968
June 14, 1968
AUG. 4, 1968
Sept. 5, 1968

FIG. 14

1	8		16		27		38		BARRAGE PIER NO.
9	8	9	8	9	8	9	8	9	PRESSURE POINT NO.
29	33.8	28.9	33.8	28.9	33.8	28.9	33.8	28.9	DESIGN VALUE.
47	41	35	32	30	32	28	26	24	MID-MAY 1968, BEFORE REMEDIAL WORK.
42	32	30	20	18	19	17	15	12	EARLY NOVEMBER 1968, DURING CONSTRUCTION OF RELIEF WELLS.
18	23	20	24	22	23	21	17	15	EARLY DECEMBER 1968, AFTER COMPLETION OF WELLS, COFFERDAM REMOVED.
39	42	40	41	39	36	32	27	23	EARLY FEBRUARY 1969, WELLS COVERED WITH SEDIMENT.
19	26	24	25	23	26	24	20	17	MID-APRIL 1969, SEDIMENT DEPOSITS FLUSHED.
22	29	25	25	24	32	28	26	24	MID-JULY 1969, NORMAL CONDITION.
35	38	43	32	43	32	46	28	34	17TH SEPTEMBER 1984, COVERED WITH SEDIMENT DEPOSITS.
16	21	21	19	19	32	23	23	19	1ST DECEMBER 1984 AFTER CLEANING OF RELIEF WELLS.

TYPICAL PHI-VALUES FOR THE DOWNSTREAM FLOOR AT PRESSURE POINTS 8 AND 9