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RASUL BARRAGE

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Of the five Barrages, which are included in the Indus Basin Project, the Rasul Barrage is the first key structure through which flow waters of the Jhelum river, and Mangla storage are transferred to meet the major portion of the replacement needs of the Eastern rivers Sutlej and Ravi. The Indus Water Treaty of 1960 envisaged Remodelling of the existing Rasul Weir to withstand the increased head of approximately 7 ft. necessitated for the withdrawals of Rasul-Qadirabad Link. This weir is a stone masonry structure constructed during the year 1901, when the theory of sub-soil flow under the hydraulic structures had not been sufficiently advanced and, consequently, its design features did not incorporate the modern concepts of a Barrage design. The existing weir based on unconventional design had a flat glacis, shallow well line cut-offs and practically no devices for energy dissipation. It is also known that the cut-offs have failed on previous occasions. On detailed examination, the scheme of Remodelling the existing weir proved to be unrealistic, non-dependable and economically unjustified.

1.1. Concept of New Barrage

In order to derive maximum use of existing training works, the second scheme considered was to build a new Barrage just downstream of the weir, but at an angle of about 26 degrees to it as it had been noticed that major floods approached the weir at that angle. Site investigation revealed most unsatisfactory ground at the east end of the alignment. This scheme had in addition a basic drawback, as with the site adjacent to the weir, it would be necessary to stop construction at the beginning of the flood seasons and to rebuild the cofferdams and re-excavate the site at the end of each flood season. Thus construction might have only been possible for six months of each year. This site was, therefore, completely abandoned.

1.2. New Barrage

The next alternative was to site the structure sufficiently downstream where the contractor should be able to work inside a cofferdam throughout the

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year. After long deliberations, economic appraisals and model studies in regard to alternative alignments of the off-taking channels, the final decision was to construct a new Barrage about 2½ miles downstream of the existing Rasul Weir (Refer Plate No. 1 for General layout).

The contract for the construction was awarded to M/s COGEFAR ASTALDI of ITALY on 31st March 1965 and the structure was ready for commissioning on 22nd December 1967, about 3 months ahead of schedule. M/s Coodes and Partners, Consulting Engineers of England, investigated the sites, designed the works and supervised the construction in the capacity of Engineers. M/s HARZA Engineering Company International of U.S.A., acted as general consultants to Water and Power Development Authority, West Pakistan.

1.3. Salient Features

The main structure 3209 ft. long has 48 bays of 60 ft. span each, fitted with electrically operated radial gates. The piers (47 No. each 7 ft. wide) carry two bridges, which comprise a number of precast, prestressed concrete beams. One bridge is narrow service bridge for operating the gates and the other is a 70 ton loading 24 ft. wide arterial road bridge. The barrage has one long divide wall towards the off-taking channels, on the left side and another small one on the right side to cater for the future right bank Jalalpur Canal. There is no fish ladder provided due to construction of a high Dam at Mangla about 50 miles upstream. The Head Regulator of Rasul Qadirabad Link has 6 spans of 40 ft. each and Lower Jhelum Feeder canal 2 spans of 40 ft. each with pier width in each case of 6 ft.

The floors upstream and downstream are of mass concrete with surface reinforcement to prevent cracking etc. The central portion is non-gravity reinforced concrete continuous beam over which the piers rest to support the bridges and other superstructures. The mass concrete of floors was done in lifts normally not exceeding 6 ft. in height and maximum area placed at one pour was restricted to 4000 sq. ft. The concreting was done in blocks of specified sizes by providing construction, expansion and control joints at designated locations. The details of vertical, horizonal and other joints with provisions for water stops is shown in Plate No. 2.

For scour protection, seepage and pressure control, there are four rows of Larssen No. 10 B 20 System steel sheet piles. Additional upstream and downstream protections against scour as usual are provided by way of concrete blocks and stone aprons. Arrangements to read the water pressure at points immediately beneath the Barrage floor has been made by installing 5 sets of water pressure recording tubes (one in the undersluice portion and four in the

normal bays) at piers No. 1, 8, 16, 27 and 38 with 10 pressure points in each row as shown in Plate No. 3. The Barrage is designed for a flood capacity of 850,000 cusecs (almost equivalent to the historic maximum discharge of 875,000 cusecs on 29th August 1929) and to cater for a differential head of 36 ft. corresponding to upstream maximum pond up to top of gates at RL 721.0 and dry downstream floor at R. L. 685.0. Designed normal and retrogressed river bed is, however, at R. L. 695 and R. L. 690.0 which give a cushion of 10 ft. and 5 ft. respectively above the top of downstream floor.

Ground densification by means of vibroflotation method of the substrata was undertaken to increase the bearing pressure in the zone of central reinforced raft floor, piers, abutments, divide wall and other heavily loaded regions. In certain areas (undersluice portion) lenses of cohesive or semi-cohesive soil too fine to be densified by vibration method were met. All such areas were either excavated, refilled duly compacted with suitable non-cohesive sandy material or aggregate back filling was done in conjunction with sand fill material for the compaction procedure to improve the subsoil conditions against differential settlements etc. The cohesive soil of the head regulators was, however, not removed as it exists sufficiently below the concrete floor (shown in Plate No. 3) and offered no structural problems.

Typical cross-sections of the structure are shown in Plate No. 1 and the important Barrage data has been abstracted at Appendix 'A'.

2. BARRAGE OPERATION

Soon after completion of works, WAPDA gradually started creation of the pond on upstream of the Barrage with effect from 8-1-1968. In the beginning head across the structure was relatively small because of the high tail water due to releases for the Irrigation Systems lower down. During February 1968, the releases lower down decreased and thereafter the structure was gradually subjected to increasing heads.

2.1. Uplift Pressures

The initial picture presented by the pressure readings was confused by the fact that some of the pressure points had been incorrectly indentified at read out stations and a few appeared to be blocked and not responding to head water and tail water fluctuations. In view of the confused pressure situation and the apparent high pressures being recorded at some points, it was decided to flush the pipes and remove blockages therefrom if any. Accordingly the pipes from top were subjected to high water pressures (approximately 5 feet) and compressed air (about 50 lbs/sq. inch) which effectively blew out all the obstructions except at point 7 pier 27 which seemed to have been choked.

Analysis of the readings, with most pressure points operating properly, permitted the pressure points to be correctly identified at read out stations and by early March 1968, a reliable date was usually obtainable. In this study, it was inferred that at pier 27 pipes 9 and 10 and at piers 1, 8, 27 and 38, the pipes from position 1 and 2 were interchanged and had been wrongly connected.

The pressure readings for the period up to December 1969, at the five instrumental piers (Nos. 1, 8, 16, 27 and 38) are plotted in Plate No. 3. The readings for each point have been converted to observed values ($\phi = \frac{P}{H} \times 100$) *i.e.* pressure at each point is expressed as a percentage of thc total head across the Barrage. The theoretical designed values determined by the method of Khosla are also shown for comparison with the observed values.

A brief resume on the behaviour of uplift pressures in different periods up to September 1968, is given hereunder:—

(*a*) **Period—Ending March 1968**

(*i*) The pressure at Pier No. 1 under the downstream floor at Points 8, 9 and at Point No. 10 under the concrete block apron are substantially higher than the theoretical.

(*ii*) These readings (*i.e.* Points 8, 9 and 10) at Pier No. 8 are also higher than theoretical, but to a lesser degree as compared to Pier No. 1.

(*iii*) The pressure at Pier No. 16 follows almost the theoretical, except for Point No. 10, indicating some local blockage of the exit.

(*iv*) The pressure readings at Pier No. 27 and 38 follow fairly close to the theoretical values except for some rise in the central floor of Pier No. 38. The central portion is R.C.C. raft having ample designed safety factor. The rise of pressure in this reach is therefore not of much concern.

(*b*) **Period—June to September 1968**

The readings at Point No. 1 have been steadily falling below theoretical, at all the 5 Piers. This is probably caused by the deposition of impervious silt layer on the bottom of Pond Upstream. As the underseepage moves downstream under the Barrage, the pressures become influenced slightly by the fall of pressure at entry *viż*. Point No. 1, but its effect is not very prominent lower down as the readings under the downstream floor are more influenced by the exit conditions. There has however, been slight improvement in all the pressure readings probably due to steady development of underseepage by removal

of initial obstructions, local blockages and air pockets. A brief resume of the pressures is however given hereunder:—

(*i*) The pressures at Pier No. 1, Points 8 and 9 under the downstream floor and at Point No. 10 have registered a slight fall, but they remain sufficiently higher than the theoretical.

(*ii*) These readings at Pier No. 8 have steadily fallen to almost the theoretical except for Point No. 10 indicating existence of still some local blockage or the influence of higher pressure from the under-sluice portion.

(*iii*) The pressures at Pier No. 16 have fallen below theoretical except for Point No. 10.

(*iv*) The pressure readings at Pier No. 27 and 38 continue to remain below theoretical. The readings of the central floor at Pier No. 38 have also fallen below theoretical.

2.2. Appearance of Bubbles

By early February 1968, the requirements of Irrigation System lower down stopped, and this created NIL flow conditions downstream of the Barrage. Under these conditions patches of silty circles were noticed in the zone of block apron opposite bays No. 11, 12, 13, 17 to 21, 23, 24, 47 and 48. The silt circles were 4 ft. to 9 ft. in diameter and the bubbling activity of gas or air escaped at intervals from some of the silt patches and elsewhere. With the start of escapage from 11 Feb. 1968, the still conditions, below the Barrage, got disturbed and this phenomenon could no longer remain visible.

Again during the period from March 25th to April 8th, there were several days when still water conditions prevailed downstream. Silt circles and bubbles of escaping air or gas to the surface were again observed in the area of downstream block aprons of standard bays. On March 28th and 29th skin divers were sent down to investigate the craters in the silt cover from where bubbles appeared to originate. Some of the craters were reported to have been an arm's length deep and to be warmer than the surrounding water.

Bubbles have continued to be seen thereafter from time to time. These sighting have invariably been at times, when tail water was low and still water conditions existed below the Barrage. It was noticed that bubbles have not been observed downstream of the undersluice bays, and that they have been observed, in general, downstream of almost all standard bays, although the number of observations is more frequent below some bays as compared to others. The appearance of bubbles varied considerably under varying differential heads, rate of fall in tail waters, period of static hydraulic conditions lower down,

degree of silt deposition in the affected area etc. etc. Usual location of the bubbles spots is shown in Plate No. 4.

Almost simultaneously some bubble spots were observed from the expansion and construction joints of the downstream impervious concrete floor, as well. The extent and location of these spots have also been changing under variable conditions. Normal location of these bubbles is also shown in the plate referred to above. It shall be of interest to mention here that shortly before the completion of works in October 1967 leakage of water in the joints of undersluice portion was noticed and such spots were satisfactorily treated by grouting with bentonite as under :

3 gallons bentonite, 1 gallon cement and 1 pint of sodium silicate were mixed with about 60 gallons of water (reduced steadily) and injected at a pressure of 25 to 30 lbs/sq. inch into the grouting holes drilled at expansion joints between the water stops. A time limit was allowed for this injected material to jellify and then second and if needed the third stage grouting was carried out up till the joint was sealed. No leaky spot in this portion of the Barrage was seen after this treatment.

2.3. Apprehensions

Rasul Barrage is the first key structure for the distribution of Mangla storage and transference of water to the Eastern rivers. The operation of Rasul-Qadirabad-Balloki-Sulemanki Link Canal System is almost totally dependent on the complete safety and full reliance of this vital structure.

The strange behaviour of excessive uplift pressures and bubbling phenomenon was of grave concern and therefore drastic operational restrictions had to be introduced to ensure the safety of this important structure. Excessive uplift pressures indicated that the thickness of the floor at certain places was inadequate to withstand the anticipated uplift pressures when subjected to the designed differential head. Similarly appearance of bubbles created strong doubts regarding the formation of runnels or piping action underneath the structure. As an immediate precaution, it was decided to restrict differential head across the structure to 16 feet till this limit could be modified on detailed analysis and checking hydraulic/structural design of the Barrage.

The third and last phase of Indus Basin Replacement works *viz.* the construction of Taunsa-Panjnad Link, Chashma Barrage, Chashma-Jhelum Link and Tarbela Dam have not yet been completed. In the absence of these media, the only source for meeting the requirements of Irrigation Systems lower down is from Mangla Dam through Rasul Barrage. Luckily this stage of replacement works necessitated ample controlled releases from Mangla almost throughout the period of investigations. This pattern of regulation

resulted into escapage of water down below and thus created favourable conditions of high tail water at this Barrage. In addition due to non-receipt of high floods during the year 1968 at this stage of initial operation of the Barrage, the leading cuts for diversion of the river downstream, did not develop fully and the extent of retrogression remained much lower than it could be expected. All these events averted the anticipated difficult situation, otherwise with the restricted differential head, the Barrage operation and feeding of the Link Canal System would have suffered immensely.

3. OFFICE STUDIES

Immediate studies were taken up to find out the causes of the unusual phenomena and a brief resume is given hereunder :—

3.1. Sub-soil strata

A total of 55 bore holes, 30 ft. to 100 ft. deep, were drilled in 1962 along a line about 1000 ft. downstream of the Barrage alignment. A total of 264 bore holes, 40 ft. to 110 ft. deep, were drilled as a first step towards the construction of the Barrage in 1965 along the centreline, upstream sheet pile row, downstream sheet pile rows and in the abutment areas of the Barrage and Head-regulators. Bore holes 14 in number on the right flank (150′ to 250′ deep) and 30 in number on the left flank (40′ to 150′ deep) were drilled in 1968 for further studies. Additional 8 bore holes each about 75' deep were drilled downstream of bays No. 28 to 38 in early 1969. All these drillings shown in Plate No. 4 indicate that in general, the foundation is made up of pervious materials deposited in layers by the river. The general trend of these deposits is parallel to the barrage with a slight dip and thickening of the layers in the downstream direction. The pervious river-lain materials are composed of layers of various mixtures of sands, gravels and silty sands. Small lenses or pockets of clay were encountered in some of the bore holes of which standard bays 29 to 35 deserve a particular mention. However, the bore holes at the left hand side encountered a massive layer of tough, plastic and impermeable clay. This clay layer increases in thickness in the downstream direction and is in places 25′ thick. Limits of clay layers liable to affect the uplift pressures have been marked on the plan referred to above.

3.2. Uplift Pressures

The design calculations of the Barrage for uplift pressures and exit gradients were reviewed and by the method of Khosla, they were found satisfactory for a maximum head across of 36 feet as originally planned. A review of the geological data below the foundations however, lead to the suspicion that some of the layers of "gravels mixed with sand" might be more porous than

sandy silt and silty sands normally found under the barrages in the Punjab on which Khosla's theory and formulae are based, and that the bubbles and some of the high pressures recorded in the downstream floor might be the result of 'Piping' through the gravel strata.

Of the 5 rows at instrumental piers, the uplift pressures observed along Pier No. 16, 27 and 38 were reasonably close to safe limits. The uplift pressures observed in the Piers No. 1 and 8, however, indicated higher percentages of uplift pressures on the downstream floor than envisaged in the design. It was initially felt that these higher pressures may be as a result of excessive artisian side pressure from canals or back of guide banks. Additional pressure points (piezometers) were therefore installed in the suspected zone, but their study did not prove any such apprehension.

The bore holes at the left hand side of the Barrage encountered a tough, plastic and impervious clay layer ranging between approximately elevation 665 to 675. Examination of the bore logs and sections show that this is a massive clay layer which extends for considerable distance left to the left abutment. The upstream edge of the clay moves from left to right as hown in *Plate No.* 4, so that it is clear of the downstream floor at approximately pier 6. This clay (shown shaded in the plan) was encountered in the excavation for the under-sluice bays and was removed from beneath the floor during construction. The removal of cohesive or semi-cohesive clayey soil was primarily done for the purpose of ground densification and to improve the bearing capcity of the strata for heavy loading considerations.

No clay was removed from downstream concrete blocks and stone apron and its existence there forms an impervious barrier blocking the escape of seepage under the undersluice bays. Seepage under these bays is thus forced to turn to the right and seek an exit through the aprons of bay 7 and the adjoining portion. The presence of clay blocking the natural seepage exit is the cause of the high pressures recorded under pier 1. As the seepage moves to the right to find a point of exit, it also raises the pressures under the adjoining bays which is evidenced by the higher pressures under pier 8.

Point 9 at Pier No. 1 tended to give critical results and thus it demanded a constant watch. Assuming submerged weight of concrete at 87.5 lbs/cu. ft. (as considered in design), floor thickness of 8.5 ft. at this spot can withstand a residual differential head equivalent to about 12 ft. of water. As a further caution, it was therefore decided that at no stage the water level recorded in this pipe should be higher than 10 ft. as compared to the normal water level on the downstream of the Barrage. Calculations also showed that if the observed levels in the pressure pipes were a true indication of the uplift pressures, the downstream floor in the undersluice bays will be virtually in a state of

floatation and not safe when subjected to a differential head exceeding 21 ft. as against 36 ft. head for which the downstream floor was designed. The initial restriction of 16 ft. differential head was, therefore, modified to 21 ft. pending implementation of remedial measures and their subsequent behaviour.

Bore logs for the year 1965 indicated possibility of a continuous clay layer in the sub-soil strata on the downstream opposite the barrage bays 29 to 36. Such condition was likely to create high uplift pressures somewhat similar to those as had been encountered in case of the undersluice portion. As no piezometers are installed in the affected zone to contradict this apprehension, 8 No. bore holes were drilled during the year 1969 to establish the extremities of clay band. These bore holes indicated localized clay laminations only (as shown in Plate No. 4) and this does not appear to generate any untoward conditions of high uplift pressures.

The pressure build-up if any shall be extremely negligible due to three dimensional drainage at either side of, and downstream beyond, the limits of this clay band. This inference is amply confirmed from the adjoining piezometer readings at Pier No. 27 and 38 which rather register pressure readings below theoretical.

3.3. Bubbles

Several explanations were advanced for the bubble phenomenon, but no conclusion could be initially agreed upon by all concerned. Possible reasons put forward were :—

(*i*) The bubbles resulted from the decomposition of organic material or alternatively were from some gaseous source other than air.

Such extengency might indicate the escapage of some subterranean gas, which may cause excessive uplift pressures. Likewise it was apprehended that decomposition of organic material might in due course cause excessive or unequal settlements and consequental damage of the Barrage structure. The intensive geological information, which include 5 bore holes for each bay, did not reveal any organic material in the foundation. In view of the broad distribution of the bubbles and the thoroughness of the drilling investigations, it is inconceivable that the bore holes would not have encountered organic material, if such had been present in the foundation. Samples of gas in the bubbles and water at these spots were collected and their chemical composition was analysed in London, Lahore and at site. A brief description of bubbles sampling and testing as at Appendix B is given hereunder :—

(*a*) Oxygen contents range from 1 to 36%. Oxygen is an active gas and its content in the bubble samples would be expected to vary widely.

(*b*) Inert gas (Nitrogen) ranges from 64 to 99%.

(*c*) Carbon dioxide is in small quantity. It being so as this gas is very soluble in water.

(*d*) Small amounts of combustible gases in a few samples are probably due to gases from decomposition of humus in the surface soils left by the river in the silt deposits.

General conclusion that can be drawn from all the bubble samples tested is the consistently high percentage of inert gas (Nitrogen). The only source of this nitrogen is atmospheric air. Therefore, it is concluded that the original source of the gas is air from the atmosphere.

The chemical tests were made of the sub-soil water as obtained from the left bank Test wells (discussed later), tail-water and the upstream pond. The result of the tests as tabulated in Appendix C indicated that these waters had basically a uniform composition, were essentially saturated with dissolved air and had an alkalinity equivalent to PH value of 8. The movement of water under the structure was therefore, direct from Head water to tail water and not through artisian underground sources.

Apart from above, the majority of pressure point readings under the standard bays are not higher than theoretical which lead to conclude that there is no subterranean gas seepage or artisian water pressure occurring under the Barrage. It could also be argued that the source of the gas is the decomposition of organic material in the pond reservoir. In such conditions, most of this gas would bubble to the surface in the upstream pond. Only a very small part dissolved by the water and seep under the structure. But the chemical analysis of the gas does not support this explanation as well.

(*ii*) We have established above that the origin of the gas in the bubbles is air from the atmosphere. In order to generate bubbles, it is therefore necessary to have a source of supply of air. It is not possible to replenish this supply from the atmosphere after the pond is filled with water. Therefore, the air supply for the bubbles must necessarily have been provided during or prior to filling of the pond. The source of air may be due to the following :—

(*a*) The bubbles resulted by the escape of air trapped in the foundation at the time of huge dewatering during construction.

(*b*) The bubbles were caused by the escape of air pumped under the structure to clear the pressure points.

(*c*) The bubbles resulted from the release of air in solution in the underseepage due to change in pressure and/or temperature.

The aforementioned alternatives present a possible explanation as it is consistent with the chemical analysis of the gas which is almost similar to atmospheric air. Dewatering at this Barrage was done by tubewells which effectively lowered the water table to about 5 to 10 ft. below the bottom of downstream concrete floor. Lowering of the water table in the granular foundation by deep unwatering wells permitted air to enter the voids between the sand grains in the foundations. An indication of the extensive cross section area which is made available for storage of air through this method of unwatering of foundations can be seen from Plate No. 5. On completion of the works and stoppage of dewatering, large air in this reservoir was trapped between the piles under the Barrage floor. Likewise air was also trapped in the unwatered foundations under the block and stone aprons and the guide banks.

The trapped air will be slowely forced downstream by the movement of seepage. The air has not place to go except to escape to the atmosphere, and in so doing creates the bubble phenomenon which can occasionally be seen under still water conditions downstream of the Barrage. This assertion is also supported by the fact that :—

(*a*) The bubble locations are intermittent and are not constantly emanating at any fixed place.

(*b*) The bubbling spots which rose to as high a figure as 537 in the months of May, June and July 1968 dropped to about 100 in the month of October same year and this number appears to be gradually on the decrease.

(*c*) The bubbles which were prominently visible from long distance and of which intensity was quite frightening in the beginning has greatly reduced in intensity now and at this stage the bubbling phenomenon cannot ordinarily be spotted unless the inspecting officials make a particular search for them.

(*d*) The silty circles or discoloration at the bubbling spot have now completely disappeared.

(*e*) Formation of craters in the silt cover wherefrom bubbles appeared to originate as reported in March 1968, was no longer visible in subsequent inspections of October 1968 and now the skin diver could hardly locate even the source at the surface of silt blanket.

It is anticipated that this emission of bubbles will continue until most of the trapped air is forced out of the foundation.

Air bubbling in the zone of blocks and stone apron downstream may rather be taken a sign of soundness of the concrete massive structure and its joints as it does not even permit the cozing out of air from the structure when

subjected to excessive differential heads. In the United States similar bubbles have been observed downstream of the dams founded on sand foundations (North Santee Dam of South Carolina, Dams on tributaries of Lower Mississippi River etc.). Based on such experience, the emission of bubbles can be expected to continue for quite a long period subsequent to construction.

The phenomenon, is, therefore, a normal process wherein some of the air trapped in the voids of the granular foundation is gradually being displaced by the downstream movement of the seepage water.

4. FIELD INVESTIGATIONS

It can be safely concluded from the aforementioned studies that the bubbles appearing downstream of the standard bays do not appear to create a situation of structural or functional concern and their presence does not depict any danger towards the safety or operation of the Barrage. There was yet another school of thought which apprehended that the bubbles may be an indication of concentration of underseepage or piping finding its exit into tail water possibly caused by temporary blockage of a substantial area of the openings in the concrete block aprons (2″ wide gap between the blocks provide 1.7 sq. ft. of opening per block or 265 sq. ft. of opening per bay). This possibility is extremely remote as the concrete block apron is underlain by a four layer inverted filter 24″ thick, comprising 6″ layer each of quarry sand, graded shingle, broken stone and spall. This filter is definitely expected to prevent any element of piping and thus loss of the foundation material. Furthermore the exit gradients through the block apron and pressures under the standard bays are so low that possibility of piping is highly unlikely. As even the remotest possibility of piping action or loss of foundation material or formation of runnels was an apprehension of grave consequences, it was decided to re-establish the substrata bore hole logs, study at length ground water flow rates and to determine the permeabilities.

4.1. Permeability of Soil Strata

Permeability of a soil strata can roughly be judged by study and computations from bore log charts, sieve analysis and grain size curves. Pumping test is, however, the most reliable method for scientific determination of the composite permeability of any strata. In this study, the theory of establishing flow into a well forms the basic principle of which a brief description is given as follows:—

According to Darcy's law the velocity of flow through any concentric cylindrical section of the water-bearing material of unit thickness is given by

$$V = P\ I$$

where

I is called the hydraulic gradient and is the ratio of $\frac{h}{r}$ or $\frac{dh}{dr}$ for very small water head considered at a small distance from a tubewell.

P is the permeability coefficient of the medium.

For a concentric cylindrical section, the flow of water is expressed as

$$Q = VA$$
$$= P\ I\ A \ldots\ldots\ldots (1)$$

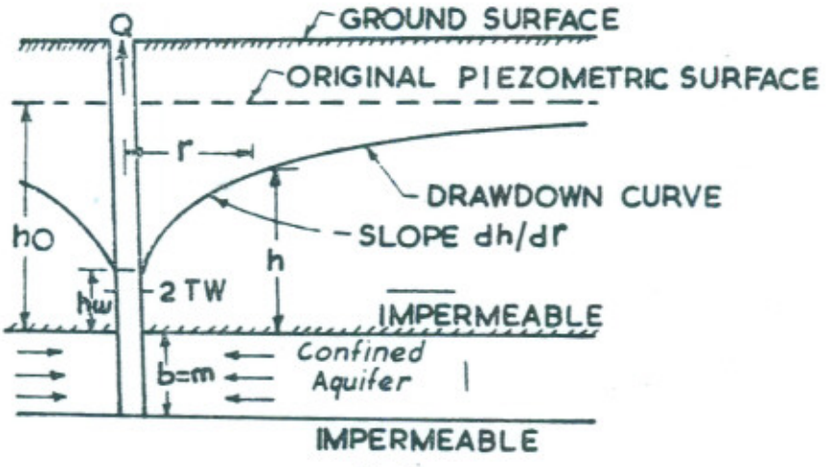


Fig. 1

where A is the area of a section and is equal to :

$$A = 2\pi\ rh$$

$$\text{or } Q = P.\ \frac{dh}{dr}\ .\ 2\pi\ rh$$

$$\text{or } \frac{dr}{r} = \frac{2\pi\ p}{Q}\ p.h.\ dh \ldots\ldots\ldots (2)$$

Integrating between the limits r1, r2, h1, and h2, we have

$$\int_{r1}^{r2} \frac{dr}{r} = \frac{2\pi\ P}{Q} \int_{h1}^{h2} hdh$$

$$\text{or } \log_e \frac{r2}{r1} = \frac{\pi P}{Q}\ (h_2^{\,2} - h_1^{\,2})$$

The computed coefficients of transmissibility for the various lines of observation wells (as determined by the Jacob Method) are as follows:—

Description of test	Line of observation wells		
	L-SE glns : dat/ft.	L-SW glns : day/ft.	L-NE and piers 1 & 8 glns/day/ ft.
Pump out Test No. I	144,000	125,000	121,800
Pump out Test No. II	146,700	128,400	126,900
Pump out Test No. III ..	148,700	118,500	108,000
Recovery Test No. I ..	147,000	144,600	125,500
Recovery Test No. II ..	140,000	136,400	137,500

Average coefficient of permeability (T) = 133,000 gallons per day per ft. of aquifer.

Computed coefficient of permeability for 100' thickness of aquifer (P) = $\frac{T \times \text{centimeters per foot}}{\text{gallons per cu. ft.} \times \text{seconds per day} \times \text{aquifer thickness in feet.}}$

$$= \frac{133{,}000 \times 30.5}{7.5 \times 86{,}400 \times 100} = 0.063 \text{ cm/sec.}$$

or 6×10^{-2} cm/sec : (Approximately)

$= 2 \times 10^{-3}$ ft./sec : (Approximately)

(*b*) **Right Flank:**

Three pumping tests and two recovery tests were conducted. Pump Test No. 1 was a constant discharge at 700 gallons per minute for 22 hours followed by a recovery test of 22 hours. Pump Test No. II was a step discharge for a period of 6 hours of 500 gallons and 900 gallons per minute.

Pump Test No. III was again a constant discharge at 900 gallons per minute for 22 hours followed by a recovery test lasting 7 hours.

The computed co-efficients of transmissibility for the various lines of observation wells (as determined by the Jacob Method) are as follows :

Description of Test	Line of observation wells		
	R-SW glns/ day/ft.	R-NW glns day/ft.	R-NE & piers 27 & 38 glns/ day/ft.
Pump out Test No. I ..	85,500	97,000	86,000
Pump out Test No. II ..	82,700	94,500	68,400
Pump out Test No. III ..	101,000	102,000	92,500
Recovery Test No. I ..	81,200	93,000	89,000
Recovery Test No. III ..	100,000	97,400	87,700

$\therefore$ Average coefficient of permeability (T) = 91,000 gallons/day/ft. of aquifer.

$\therefore$ Computed coefficient of permeability for 100′ thickness of aquifer (p)

$$= \frac{91{,}000 \times 30.5}{7.5 \times 86{,}400 \times 100} = 0.043 \text{ cm/sec.}$$

or $= 4 \times 10^{-2}$ cm/sec. (Approximately)

$= 13 \times 10^{-4}$ ft./sec. (Approximately)

4.3. Dye Test

In order to physically establish the sub-soil seepage flow velocities, Fluorescin Sodium dye was placed in small quantities on the upstream side in the Pond on 9-10-68 as under :—

(*i*) Barrage portion— In the zone of upstream stone apron opposite piers No. 16, 27 and bay 48.

(*ii*) Right closure bund— Near the upstream toe of the bund opposite RD 1000′ & 2125′.

This chemical on dilution makes the water fluorescin in sun and also when exposed in darkness to Black Light (Invisible Ray Black light product with rechargeable battery unit. B. L. 6 B manufactured at 1057 Lawrence, Chicago, U.S.A.).

The idea was that pond water while seeping under pressure through the sub-soil towards the downstream of the Barrage shall take along the florescine dye and its presence at various locations, when corelated with time of travel shall indicate in different zones the seepage velocities.

Water from the sub-soil was thus pumped out from the pressure pipes at the read-out stations, to trace the dye. While taking the observations, it was found that some oily material of the pressure pipe system was already causing florescine. The florescine effect also failed to appear in the tail waters showing either the dye had diluted too much as not to show the florescine effect or probably siltation on upstream had made the experiment a failure. The dye also did not appear on the downstream of closure bund probably due to siltation on the upstream. This test therefore gave no conclusive results.

This experiment was again repeated on 13 and 14 March 1969 by using sufficient quantity of another type of dye (KEYSTONE STANDARD COLOURS) in three different colours as under :—

(*i*) Keystone acid Turquoise 7BC 110% (Blue colour).	In the zone of upstream stone apron opposite bay 38 & 46.
(*ii*) Bazo Pink RB (Red colour).	In the zone of upstream stone apron opposite piers, 8, 39 and 38.
(*iii*) Keystone Uranine (Yellow colour).	In the zone of upstream stone apron opposite piers 44, and left divide wall nose.

The florescine effect failed to appear in the pressure pipes and also in the tail waters although observations were continued over a long period. The method of physical establishment of the sub-soil flow velocities, therefore, proved to be a total failure.

4.4. Under Seepage Velocities

The permeabilities as determined from the pumping tests are normal for the fine sand and sand gravel mixtures in the Barrage foundation. Water seeping beneath the standard bays from the upstream pond to the tail pond travels a distance of about 350 ft. for the foundation permeability of 6×10^{-2} cm/sec, and a cross head of 36 ft. a period of about 20 days will be required for a particle of water to move a distance of 350 ft. Likewise for a permeability of

4×10^{-2} cm/sec. this period will be 32 days. Corresponding average velocities will be in the order of 0.7 ft. per hour and 0.5 ft. per hour. In order for piping to occur, the velocities would have to be at least 5 ft. per hour. The factor of safety against piping is thus about 8 and the under seepage velocities are well below the piping velocities for the foundation materials under the Barrage. The possibility of piping, runnel formation or movement of foundation materials is thus completely ruled out.

5. REMEDIAL MEASURES

Remedial measures were only required to alleviate the high pressures under the downstream floor of the undersluice. It had become evident from analysis of the geology and pressure point readings, that the clay layer under the downstream block and stone apron of undersluices was the cause of these high pressures.

5.1. Ordinary Solutions

A customary solution of the problem as per past practices of the Irrigation Department could be as under :—

(*i*) Weighting of the floor

This comprised of resisting the high uplift pressures by weighting the floor. Such a scheme would have consisted of precast concrete blocks of sufficient size to be stable against the hydraulic action in the stilling basin, placed in standing water to counterbalance the excessive pressure. The pressure point readings at pier 1 indicate that additional thickness of the floor concrete would have been of the order of about 3.5 ft. The objection to this scheme was :—

(*a*) A possible creation of surface flow condition unfavourable for proper energy dissipation on the downstream and this may cause recurring future damages to the stone apron.

(*b*) The scheme fails to go to the root cause of the problem which is due to the blockage caused by the clay layer under the downstream blocks and apron.

For these reasons, WAPDA's general Consultants, M/s HARZA did not approve of this scheme and was, therefore, rejected as a solution.

Subsequent hydraulic model studies by the Irrigation Research Institute at Nandipur during October and November 1968,

however, did not approve of the apprehensions at (*a*) above. The tests indicated that by adding a system of blocks on the downstream floor, the upper/lower limits of jump formation shrinked, the overall velocity distribution got improved and the scour depths also reduced slightly. The scheme of adding blocks and the results of model studies are compared at Plate No. 7.

(*ii*) **Subsidiary Weir**

As concluded previously, the observed levels in the pressure pipes indicated that without remedial works, the downstream floor in the undersluice bays shall be in a state of floatation when subjected to a differential head exceeding 21 ft. approximately. A subsidiary weir to withstand the residual differential head of about 15′ having its crest at RL 700.00 was, therefore, another possible solution to cater for the designed head across limit of 36 ft. Such a scheme tantamounted to re-construction of a full-fledged weir at an exorbitant cost and was, therefore, rejected as a possible solution.

5.2. Scientific Solutions

In order to find a scientific answer to the problem, elaborate ELECTRIC ANALOG studies were made by WAPDA's general Consultants, M/s HARZA at Chicago, U.S.A. This method is a convenient way of determining equipotential lines in a porous media by using an electric analog of conductive paper type. It is based on the principle that the flow characteristics of electric current through a conductive surface are similar to flow of water in a permeable media. The device consists of conductive paper and an instrument, which is basically a wheatstone bridge, for measuring the potential between various points on the conductive paper. The full potential is set at the upstream end of the structure as layed out on the conductive paper and the zero or residual potential on the downstream end. By presenting various percentages of this potential on the wheatstone bridge, it is possible to trace the configuration of each of these potential lines on the conductive paper. These lines in turn shall represent the trace of the equipotential lines under the structure in terms of percentage of head across the structure.

In this type of analog, the conductive paper is analogous to the permeable foundation. The geometry of the structure and other impermeable boundaries are reproduced on the model by cutting and removing portions of the conductive paper. Differential head is represented by applying a differential voltage to the paper, and pressure gradients are measured by determining the voltage

drop at various points on the paper. Variations of permeability in the foundation strata can be reproduced in the analog by increasing the conductivity of portions of the paper by painting on a more conductive material. By distorting the horizontal and verticle scales, it is also possible to account for differences in vertical and horizontal permeabilities. The chief advantage of the analog is the speed with which results can be obtained, and that many complicated conditions can be studied for which mathematical solutions are not readily available. Sketches showing various schemes and the results of analog studies for undersluice and standard bays are shown in *Plate No.* 8. A brief resume of these solutions is given hereunder :—

(*i*) Deep Upstream cut-off

The results of the analog studies indicated that even a deep upstream cut-off, approximately 100 ft. deeper than the upstream sheetpile row, would reduce the pressures under the upstream and middle floors but would have little effect on the pressures under the downstream floor. It is not practicable to drive sheet piling to this depth and a slurry trench or grouted type cut-off would have been required. The cost per square foot for either of these types is higher than for an ordinary sheet pile cut-off. Any type of such construction under operating conditions of the Barrage would have required special working platforms. Therefore, this scheme was rejected as a solution because of its ineffectiveness in reducing pressures under the downstream floor, difficult construction and high cost.

(*ii*) Gravel filled trench in the block apron

The analog study of this scheme gave satisfactory but not the best results. This scheme envisaged complete enclosure and dewatering of the working area. In order to ensure safety of the structure, substantial lowering of the upstream pond throughout the construction period (or even diversion of the river) was necessary and the same would not have been practicable for proper utilization of Mangla storage and other requirements of Irrigation systems. The safety of the cofferdams under this scheme resting over pervious foundation of blocks and stone aprons would have always posed a threat of failure and in addition huge pumping arrangements were necessary. The difficult and unreliable construction under high tail water condition's (about 10 to 15 ft. above floor level corresponding to nil flow downstream) and the expense of removing and replacing a large number of concrete blocks, made the scheme extremely difficult and costly.

Since the scheme was also less effective as compared to another alternative, it was rejected as a solution.

(*iii*) **Excavation of clay layer under stone apron**

Excavation of clay layer under the stone apron and replacing the same with gravel was considered as a possible solution to provide a relief drain. The analog studies, however, indicated that this did not provide sufficient relief under the undersluice floor due to its distance downwards and was, therefore, rejected as a solution.

(*iv*) **Gravity Relief Wells**

Analog studies indicated that a change of status towards upstream reflected its influence predominantly in the upstream and middle portions of the floors and gave little relief on the downstream pressures. Conversely changes in the exit conditions primarily affected the underseepage in the zone of downstream floor, but had little effect upstream. Also when pervious strata beneath dams or such structures are too deep to excavate where excavation of shallow drains do not appear practicable, relief wells offer a good means of relieving uplift pressures because they can penetrate the most pervious water-bearing strata in the foundations. Working on these principles, a scheme of providing drainage relief wells through the clay layer was considered. Analog studies revealed that shallow wells reduced the pressures to acceptable values, but deep wells were found to have a much greater effect.

This treatment appeared most effective, less costly and easily workable. The scheme could be implemented without any fear of damage to the structure or disturbance of flow into the off-taking channels and also across the Barrage. The selection was therefore in favour of gravity relief wells.

6. DESIGN OF RELIEF WELLS

The readings were reasonably within the designed values in the standard bays and hence no remedial measures were considered appropriate in this region. The scheme of installing the relief wells was restricted for the treatment of undersluice bays only where excessive pressures threatened the safety of the structure.

Designing a relief well system requires determination of the most economical spacing and depth of penetration which will lower the uplift pressures to a safe level. The location of well nearest to the floor could be at the 1st block downstream of the horizontal section of concrete block apron that could practically be lifted out without any detriment to the stability, tightness

and integrity of the works. The relief wells are designed in such a manner that their discharge resistance is small, that they are spaced sufficiently close together to lower the adjacent water pressures to a safe level. They must have screens or filters that prevent the loss of soil out through the wells and are resistant to corrosion and deterioration caused by bacteria.

6.1. Wells' location

Alternative studies indicated 8t ft. deep wells located in the 5th row of concrete blocks downstream of undersluice bays to give the best results. The selection was also in favour of 8 inch diameter slotted fibre glass well screen surrounded with 7 inch thick filter pack thereby making effective diameter of well to be 22 inches. Since the block is nominally five feet square with 2" all-round gravel filled cavity, the well spacings will, therefore, be a multiple of 5'-2" approximately.

The 5th block has its top elevation at RL 681.0. The bottom of 80' deep well after allowing some recess for fixing cap etc. will, therefore, be at approximate RL 600.0. In relation to these elevations, the bottom of clay layer under downstream block apron is located at approximate R. L. 665.0.

6.2. Seepage shape factor

Seepage will feed to the well system from upstream pond through pervious aquifer under the Barrage. Average distance from upstream cut-off to middle of relief well system along shortest path is 400 ft. Similarly average distance from reservoir around abutment to relief wells is approx: 1000 ft. For the seepage flow under the Barrge, the flow net indicates about 2.5 flow channels for 10 equipotential drops. This gives a seepage shape factor of 0.25 which is also in close agreement with the approximate ratio $D/(L \times 0.4.D)$ suggested by H. R. Cedergren on 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 and (D) is the composite aquifer thickness as established in the pump out tests.

$\therefore$ Seepage shape factor under the Barrage $= \dfrac{D}{L \times 0.4.D} = \dfrac{100}{400 + 0.4 \times 100} = 0.23$

Similarly seepage shape factor around the abutment $= \dfrac{D}{L + 0.4D} = \dfrac{100}{1000 + 0.4 \times 100} = 0.096$ Say 0.10

6.3. Seepage Flow

The quantity of seepage per linear foot of aquifer to the relief well system is governed by the following:

$q = K \times$ seepage shape factor $\times h$

where K= coefficient of permeability
= 6×10^{-2} cum/sec.
= 2..10^{-3} ft./sec.
and h = designed head = 36 ft.

∴ Seepage under the Barrage=0.002..0.25×36=0.018 cus/Linear ft.
Similarly seepage around abutment=0.002×0.10×36=0.007 cus/Linear foot.

∴ Total seepage (q)=0.018+0.007=0.025 cus/Linear ft.

6.4. Well Spacing

The well spacing is usually designed in consideration with the average pressure above the tail water which will be developed by such installations. This is derived in the book referred to at 6.2 above by the *application of a well formula for flow to an artesian well* modified for 180 degrees, on the assumption that inflow occurs only in the half circle upstream of the well.

$$H_m = \frac{Q \log 10 \left(R/rW\right)}{1.37\ KD}$$

where H_m=the Head difference between the edge of relief well and midpoint between the wells.

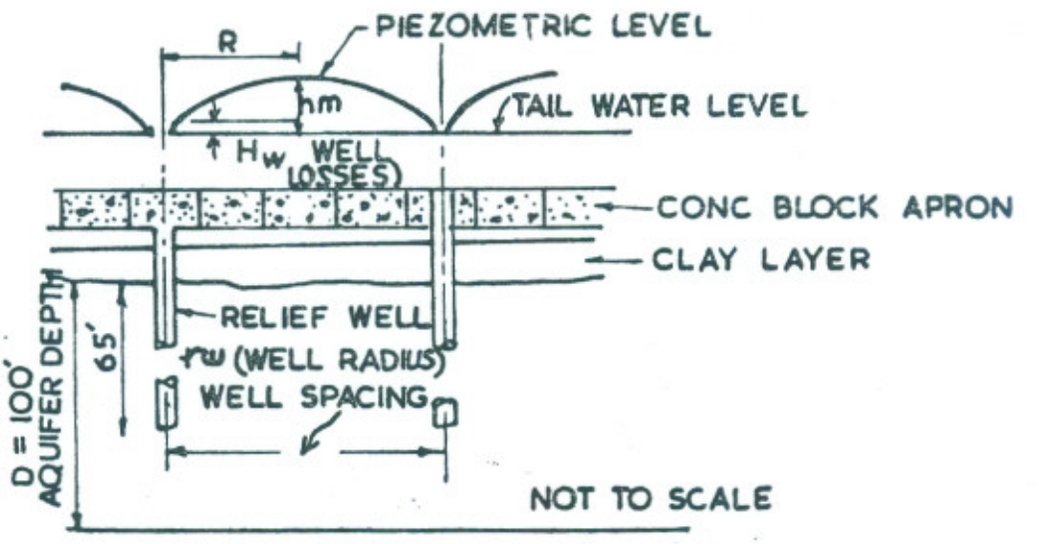


Fig. 3

Q=Disch : of well in cus.
R=Radius of influence
i.e. ½ of well spacing
in feet.
K=coefficient of permeability in ft. per sec.
D=Depth of aquifer in ft.

In the sketch H_w indicates head losses at the wells. This figure depends on numerous factors like unknown screen resistances, smear effect caused by the well drilling processes etc. and cannot be derived by arithmetical calculations. It is usually advisable to make a reasonable allowance for such losses which our consultants recommended as 1.5′ for well spacing of 20′ approximately.

(*i*) *Trial for* 20.7′ *spacing*

$$R = \frac{20.7}{2} = 10.35 \text{ ft.}$$

$Q =$ —— well spacing × seepage per linner foot

$= 20.7 \times 0.018 = 0.375$ cusecs.

$$H_m = \frac{Q \log 10 (R\ rw)}{1.37\ KD} = \frac{0.375 \times 1.05}{1.37 \times 0.002} \times 100 = 1.44 \text{ ft.}$$

∴ Total head at midpoint between wells $= H_m + H_w = 1.44 + 1.50 = 2.94$ ft.

∴ %age residual pressure $= \phi = \frac{2.94}{36} \times 100 = 8.2\ \%$ against designed figure of 6.2% at the exit end *i.e.* at pressure pipe No. 10.

Although residual pressure of 8.2% can safely be tolerated still it would be prudent to reduce the spacing.

(*ii*) *Trial for* 15.5 ft.

$$R = \frac{15.5}{2} 17.75 \text{ ft.}$$

$Q = 15.5 \times 0.018 = 0.28$ cusecs.

$$\therefore H_m = \frac{0.28..0.92}{1.37..0.002} \times 100 = 0.95 \text{ ft.}$$

$$H_w = 1.5 \times \frac{0.28 \text{ cus}}{0.375 \text{ cus}} = 1.12 \text{ ft.}$$

p∴ total head at midpoint between wells $= 0.95 + 1.12 = 2.07$ ft.

∴ %age residual pressure $= \phi = \frac{2.07}{36} = 6\%$ against designed figure of 6.2% at the exit end *i.e.* at pressure pipe No. 10.

Hence safe.

middle of flood season. Undersluice bays were completely closed to facilitate construction of a temporary access road and the working platform (fig : 1 and 2 Plate No. 11). The earthwork was started in early July and by the end of August 1968 working platform/cofferdam was ready. This platform was kept sufficiently high to avoid its over-topping by a normal high flood of about 5 lac. cusecs which could be handled in the remaining standard bays.

7.3. Sheetpile Caissons

A guide frame of angle iron (fig. 3, Plate No. 11) was made to drive the sheetpile caisson. After adjusting the frame at its correct position and checking its verticality, the sheetpiles each 16'' wide and about 33' long were pitched inside this frame in the shape of about 12 ft. ring. A set of 3 such caissons were worked in stages. The difficulty encountered in driving the sheetpile rings was that after repeated driving and pulling out, the piles got damaged badly and these had to be repaired on site by cutting and rewelding. To deal with this problem, it was decided that the piles should be driven through less length. For this purpose a trench about 12 ft. deep was dug in the dry platform with the help of an excavator. The sheetpiles were then driven from the bottom of this pit, thereby saving in time as well as heavy damages.

7.4. Cleaning inside of Caissons

After driving the sheetpile ring, the soil from inside was removed with Bucyrus 22 B Excavator firstly in dry and then lower down in water as far as practicable (fig. 4, Plate No. 11). Final clearance of the caisson was done by churning the bed by compressed air and simultaneously pulling out the muddy water with the help of air water pump (fig. 5, Plate No. 11). In this process cleaner water was supplied in the caisson by constant pumping from river.

After cleaning, the caisson was all the time kept full of water so as to :

(*a*) Guard against refilling from sides,

(*b*) Avoid disturbance or collapse of the hole to be drilled, and

(*c*) Avoid movement or blow out of substrata on replacement of the concrete block and during the process of drilling, boring and other operations.

.5. Removal of old block

After cleaning the caisson and exposing the block to be removed, two No. holes 1 ft. deep and 2 ft. apart were drilled in the existing block in standing water (fig. 6 Plate No. 11). The mobile drill was positioned over iron beams placed on the sheetpile cell which served as a bridge. The displacement of the mobile diamond drill in a length of about 35 ft. (in air and under water)

was precisely controlled by fixing guide frames at interval in the caisson. After drilling the holes, two No. special expansion bolts were inserted there. The bajri from the adjoining 2 inch openings around the concrete block was also cleared as far as possible to loose the grip for easy displacements. The old block was then pulled out by applying 2 cranes on each hook of the expansion bolts. Difficulties were encountered in pulling out of the 1st block where trial provision of only one expansion bolt did not work even by application of 2 cranes.

7.6. Placing new block

After removal of the old blocks, the new precast blocks were placed in position (fig. 7, Plate No. 11). The new blocks were cast 4′—10″ × 4′—10″ × 4′ to admit 4″ all-round gap with the adjoining blocks for easy placement. This block had 2′—4″ diameter hole at the centre, 3′—6″ diameter chamber at the top to accommodate the perforated steel cover plate and 8 No. bolts for fixing the cover plate to the block top. For placing the new block in position, a lifting cover consisting of Iron Plate was fixed to the block with the bolts. Then the block was lifted from the handles provided on this plate with the help of a crane and placed in position. The bolts were then unfastened by the divers and the lifting cover removed from the block. The central hole in the block was so located that when the block was placed in position on the slope, the hole remained vertical. The 4″ gap around the block was then filled with gravel as existing previously.

7.7. Drilling of borehole

Soon after placing the new block in position, the drilling of 22″ diameter hole (fig. 8, Plate No. 11) was done with the help of Idrosonda machine (wash boring). Average depth of the drill was 80 ft. and the boring was stopped in the sand gravel water BEARING STRATA for its effectiveness. For drilling of the borehole, 15 ft. long casing at the top was used and that also to avoid caving in of the four stage filter material into the hole. Lower down the borehole was drilled without casing and no difficulty was encountered in this process. All the holes remained standing except one at Serial No. 4 from left which collapsed and had to be redrilled.

7.8. Installation of slotted pipe

After the borehole had been completed, the slotted fabricated fibre glass 8″ dia pipe and about 80 ft. long was installed in the hole (fig. 9, Plate No. 11). The length of the pipe which had to come in contact with clay layer was blinded by wrapping plastic tape around that portion. The opening of the pipe at bottom was also plugged with concrete for a length of about 2 ft. The

of silt in the rubble around the pipe, it will be removed by flushing with an airjet or by pumping water into the well under pressure.

Efferts should be made to obtain physical or chemical samples of incrustation (if these exist) or of the water, whereby the nature of the trouble can be determined. The water samples should be analysed and also examined under the microscope to determine the iron bacteria. In case the screens have become clogged by carbonate deposits or by the action of iron bacteria, conventional cleaning procedure can be used. The sulphamic acid or other cleaning mixture will have to be poured through a plastic or resistant pipe forcing out through the screen slotts by creating a reverse static hydraulic pressure. In spite of the measures taken above, if the ϕ values still register higher results, and the safety against flotation approaches 1.1, the only course left open would then be to instal more Relief Wells on the lines detailed previously.

9. RELIEF WELLS PERFORMANCE

Soon after the installations of pressure relief wells in November 1968, higher uplift pressures in the zone of undersluice bays started falling gradually and ultimately in December 1968 they dropped sufficiently below the designed figures. (Refer plotting for the month of 12/1968 in Plate No. 3). The pressures in the standard bays as recorded at Piers No. 8, 16, 27 and 38 were already within permissible range. This situation continued up till the commencement of annual closure from 26-12-1968.

On re-commissioning of the Barrage after closure on 6-1-1969, nil flow conditions on the downstream side were again created. It was then noticed that heavy silt deposits had taken place on the downstream floor, concrete blocks and stone apron from left abutment up to approximately Bay No. 29. This deposition occurred because the leading cut on the left had not fully developed due to non-receipt of floods during the year 1968 and also due to obstruction caused by the remnants of cofferdam opposite Bays No. 8 to 26 approx. Such silt deposition at this Barrage may for some period be a common feature till the retrogression of the river bed on the downstream sufficiently takes place after passage of floods in the coming years.

Survey of the downstream area after the closure indicated that the depth of the silt ranged from 9 ft. to 11 ft. in front of Bays No. 1 to 29. This heavy siltation at once affected the seepage flow underneath the structure. The plezometer readings in such condition again gave results sufficiently higher than the designed figures particularly in the portion of downstream floor and exit point. (Refer plotting for the month of 2/69 in Plate No. 3). This deposition affected the working of pressure relief wells also as the pressures had shot up in the undersluice portion as well.

The percentage rise of pressure apparently showed a cause of concern,

but in reality, the situation was not so bad. The designed optimum pressures generally hold good in case the cistern on the downstream is completely empty. In the condition then existing, the floor was heavily laden with silt deposit and this tended to give an added advantage of counteracting the excessive uplift pressure. The phenomena of siltation *vi,-a-vis* the higher uplift pressures therefore did not impose any restrictions on the normal working of the Barrage.

It was anticipated that with the gushing of water lower down, the silt whould get washed off and the pressures would then again fall down. In that event, the pressure relief wells were also expected to function properly. This assumption was fully confirmed as with the escapage of even low discharge towards the end of 3/1969 showed remarkable results. The silt deposits were successfully washed off and the uplift pressures thereupon again dropped to permissible limits as would be clear from the representative plotting for the month of April, July & December 1969 in Plate No. 3.

10. CONCLUSIONS

The initial operation of the Barrage is always an intricate job and more so far Rasul on account of the peculiar phenomena. Certain quarters had the feeling that this vital structure may not be able to give its due performance in accordance with the designed criteria as envisaged in the replacement plan. Critical appraisal of the complicated situation, coupled with additional investigations and the remedial works, have proved that the structure is reasonably safe and for the normal operation and maintenance no untoward limitations are now required to be imposed. A constant watch on the deposition of silt at the downstream side in relation to the uplift pressures shall, however, be required and this may occassionally warrant adjustments in the regulation pattern at site. This caution is not peculiar to Rasul Barrage only as such vigilance is necessarily required at all the Headworks, particularly during the initial stage of operating these structures.

11. ACKNOWLEDGEMENTS

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Appendix—A

IMPORTANT DATA OF BARRAGE AND REGULATOR

(*a*) RASUL BARRAGE

1.	Undersluice Bays (1 to 6)	.. 6 No of 60′ each
2.	Standard Bays (7 to 48)	.. 42 No. of 60′ each
3.	Thickness of piers and divide walls	.. 7.0
4.	Crest level of undersluice Bays (1-6)	.. 698.50
5.	Crest level of standard Bays (7 to 48)	.. 703.00
6.	U/S floor level in standard & undersluice bays	.. 695.00
7.	D/S floor level in stilling pool of undersluice bays	.. 680.00
8.	D/S floor level of standard & undersluice at end of impervious floor.	.. 695.00
9.	Top RL of Bridge	.. 729.00′
10.	Top RL of Gates	.. 721.00′
11.	Height of gates in standard bays in undersluice bays	.. 22.50
12.	Normal pond level upstream	.. 719.00
13.	Height of flood level upstream	.. 722.75
14.	Highest flood level D/S	
	(accerted)	.. 719.00
	(Retrogressed)	.. 712.00
	(Normal)	.. 718.00
15.	Maximum head across	.. 36.00′
16.	Width of Road Bridge carriage way	.. 24.0′
	Footway	.. 4.0. on either side.
17.	Loading of Bridge=as for Class AA (70 ton Tank).	
18.	Width of service bridge	.. 5.0′
19.	Length of Barrage between abutments	.. 3209′

20.	Designed capacity of Barrage	.. 850,000 cs.
21.	Length of Left Guide Bank U/S	.. 3360′
22.	„ „ „ „ D/S	.. 617′
23.	Length of Right Guide Bund U/S	.. 2021′
	D/S	.. 617′
24.	Designed discharge intensity of undersluice bays	
	At the crest ——410 cs. per ft. run.	
	At the U/S floor —— 348 cs. per ft. run.	
25.	Designed discharge intensity of standardy Bays	
	At the crest —— 299 cs. per ft. run.	
	At the U/S floor —— 262 cs. per ft. run.	
26.	Right Bank Spur length	.. 2762 ft.
27.	Right Tee Spur Length	.. 3260 ft.
28.	Left Tee Spur length	.. 1950 ft.
29.	Pressure Relief Wells	.. 24 Nos.

(*b*) HEAD REGULATOR, RASUL-QADIRABAD LINK

1.	Discharge (full supply) at head	.. 19000 cs.
2.	No of Spans	.. 6 Nos. of 40′ each
3.	Length between abutments	.. 270′
4.	Thickness of Pier	.. 6′
5.	Crest level of head regulator	.. 708.50
6.	Floor level upstream	.. 704.50
7.	Floor level downstream	.. 698.00
8.	Top of lip wall at end of D/S floor	.. 699.50
9.	Designed Head across	.. 25.25
10.	Full supply level of canal	.. 717.00
11.	Top RL of Gate	.. 720.50
12.	Height of Gates	.. 12.00
13.	Bottom RL of diaphragm	.. 719.50
14.	Bed width of canal	.. 330′ ..
15.	Full supply depth	.. 14.0′
16.	Normal bed level of Canal at head	.. 703.00

(c) HEAD REGULATOR LOWER JHELUM CANAL FEEDER

1.	Full supply discharge	(*i*) Future	.. 6600 cs.
		(*ii*) Present	.. 3000 cs.
2.	No of spans		.. 2 No. 60′ each
3.	Length between abutments		.. 126.0′
4.	Thickness of pier		.. 6′
5.	Crest level of head regulator		.. 708.50
6.	Floor level upstream		.. 704.50
7.	Floor level downstream		.. 698.00
8.	Top of lip wall at end of D/S floor		.. 699.50
9.	Designed Head a cross		.. 24.75
10.	Full supply level of canal at head :		
		(*i*) Present Discharge 3000 cs.	.. 712.11
		(*ii*) Future discharge 6600 cs.	.. 715.60
11.	Normal bed level of canal at head		.. 704.50

Appendix "B"

BUBBLE SAMPLING AND CHEMICAL TESTING

Gas Composition Test					Sampling	
Bubble sample No.	Oxygen %vol.	Carbon Dioxide %vol.	Combus-tible gas-ses %vol	Nitro- %vol.	Date	Location and Remarks
1	2	3	4	5	6	7
1.	30.4	NIL	..	69.6	Oct. 6 1968	Bay No. 19 near downstream end of Pier.
2.	24.2	NIL	..	75.8	,,	Centre of Bay 41 at D/S end of Piers.
3.	32.2	NIL	NIL	67.8	,,	Near D/S end of Pier 36-37.
4.	36.4	NIL	..	63.6	,,	D/S of Bay 22, beginning of block apron.
5.	21.8	0.4	NIL	77.8	Oct. 7 1968	Near right G/Bank D/S of Barrage.
6.	22.9	0.2	NIL	76.9	,,	Downstream of Pier 10-11 on block apron.
7.	22.5	0.2	NIL	77.3	,,	Downstream of Bay 13 on block apron.
8.	23.2	0.1	NIL	76.7	,,	Downstream of Pier 14-15 on block apron, half of sample collected after disturbing mud on the apron.
9.	22.8	0.2	NIL	77.0	,,	Downstream of Bay 8 at end of block apr.
10.	9.4	NIL	NIL	80.6	,,	Downstream of Bay 21 near centre of block apron.
11.	13.4	NIL	NIL	86.6	,,	Downstream of Bay 23 on block apron.
12.	2.7	0.2	..	97.1	Oct. 8 1968	Downstream of Bay 23 on block apron.
13.	20.4	NIL	..	79.6	,,	Check on sampling method. Sample contained air injected in the water by a hand pump.
14.	3.9	0.1	0	96.0	,,	Downstream of Bay 23 on block apron.
15.	3.1	NIL	0	96.0	,,	Downstream of Pier 24-25 at edge of concrete apron.
16.	20.2	NIL	..	79.8	,,	Check sampling method, Sample collected for 2 hrs. in a bottle containing 50% of air.

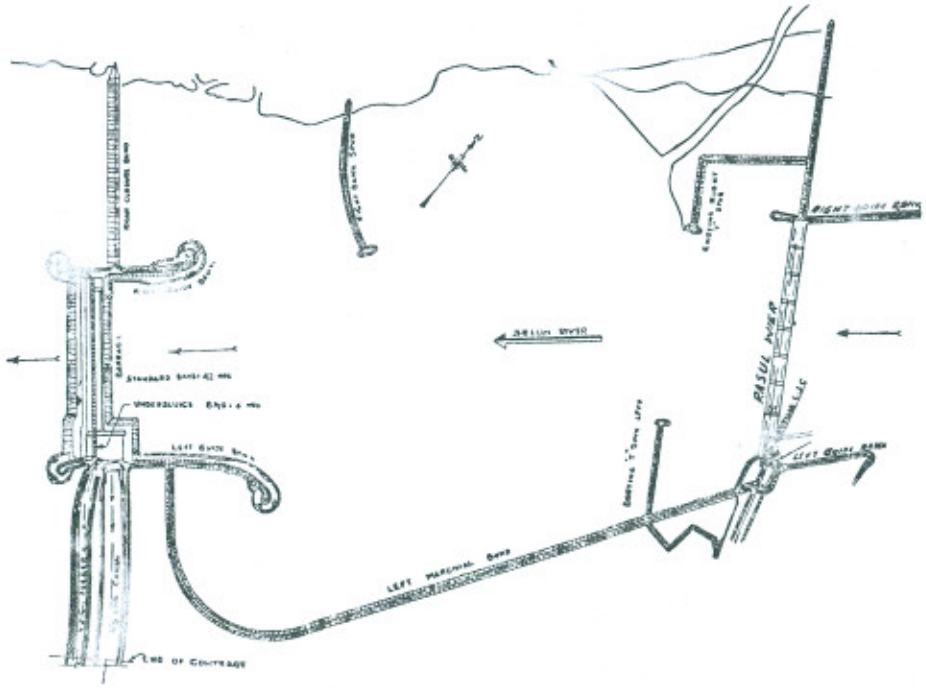
1	2	3	4	5	6	7
17.	20.8	NIL	..	69.2	Oct. 8 1968	D/S of Bay 44 on block apron. Sample bottle filled only after mud on the floor was disturbed.
18.	1.9	NIL	0	98.1	,,	D/S of right abutment sample made without funnel in shallow water.
19.	2.8	NIL	..	97.2	,,	D/S of Pier 25-26 at end of block apron. Sample taken after bottom was disturbed.
20.	4.4	NIL	0	95.6	,,	About one mile U/S of Barrage. Sample taken in 3 ft. of water after bottom was disturbed.
21.	2.4	NIL	1.0	96.6	,,	Same as sample 20.
22.	4.1	NIL	0	95.9	,,	D/S of Pier 22-23 at end of concrete apron.
23.	2.0	NIL	NIL	98.0	Oct. 9 1968	About 1.5 miles upstream of Barrage below left tee Sample taken in 3 ft. of water after disturbing bottom.
24.	21.3	NIL	NIL	68.7	,,	Same as sample 23.
25.	3.4	0.9	NIL	95.7	Oct. 10 1968	About 2 miles U/S of Barrage and 200 ft. U/S of right spur.
26.	2.4	NIL	0	97.6	,,	Same as sample 25.
27.	2.5	0.4	NIL	97.1	,,	In pond near left bank.
28.	2.5	0.4	..	97.1	,,	Same as sample 27.
29.	1.1	NIL	0	98.9	,,	Downstream of Bay 10 on block apron.
30.	3.1	0.1	NIL	96.8	,,	Downstream of Bay 47 below rock apron.
31.	2.6	NIL	0	97.4	,,	Along right bank downstream of rock 2.
32.	1.4	0.2	NIL	98.4	Oct. 11 1968	Downstream of Bay 10 on block apron.
33.	2.0	NIL	1.4	97.6	,,	Downstream of Bay 20 on block apron.
34.	3.9	0.1	2.0	94.0	,,	Downstream of Bay 12 on block apron.
35.	3.6	NIL	1.8	94.6	,,	Downstream of Pier 23-24 on block apron.

Appendix "C"

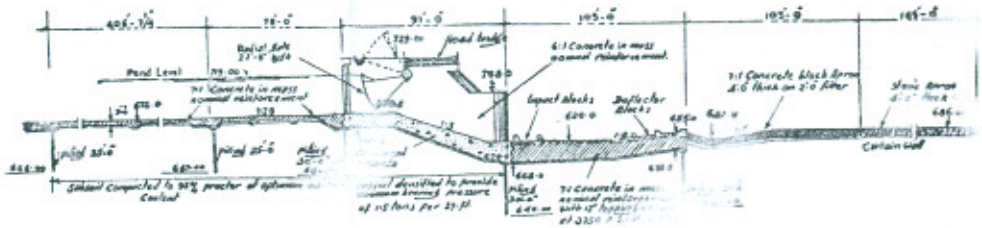
RASUL BARRAGE—RIVER JHELUM SUMMARY OF WATER ANALYSIS

Source		Milliequivalents per litre								D.S. (by Evap.)	EC_x 106 at 25°C	PH	Remarks
		C_a	M_g	N_a	CO_3	HCO_8	C_1	SO_4	Total Cations Anions				
(a) Subsoil water from Left Bank Test Well										ppm			
Left Bank	Test Well	2.10	.60	.40	0	2.50	.10	.46	3.10/3.06	162	290	7.48	Iron content was found to be 0.1 to 0.01 parts per million.
Left Bank	Test Well	0.95	1.65	0.42	0	2.70	0.10	0.35	3.02/3.15	156	300	7.50	
Left Bank	Test Well	2.00	0.70	0.22	0	2.66	0.10	0.40	2.92/3.16	150	290	7.48	
(b) River water on the downstream													
Bay 21	Surface	1.58	0.52	0.30	0	1.73	0.15	0.45	2.40/2.33	120	220	7.79	
Bay 21	Bottom	1.79	0.51	0.30	0	1.95	0.15	0.50	2.60/2.65	130	240	7.87	
Bay 35	Surface	1.58	0.52	0.30	0	1.74	0.15	0.50	2.40/2.39	124	230	7.71	
Bay 35	Bottom	1.89	0.51	0.30	0	1.95	0.20	0.50	2/70.2.65	140	250	7.92	
Bay 40	Surface	1.58	0.42	0.30	0	1.69	0.15	0.45	2.30/2.29	116	210	7.57	
Bay 40	Bottom	1.79	0.51	0.30	0	1.91	0.20	0.50	2.60/2.61	130	240	7.80	
Bay 46	Surface	1.47	0.43	0.25	0	1.64	0.15	0.30	2.15/2.09	114	200	7.75	
Bay 46	Bottom	1.58	0.42	0.30	0	1.86	0.15	0.20	2.30/2.21	134	230	7.74	
(c) River water in Pond Upstream													
Bay 8	Surface	1.47	0.43	0.25	0	1.64	0.15	0.30	2.15/2.09	110	200	7.69	
Bay 8	Bottom	1.47	0.43	0.25	0	1.60	0.15	0.30	2.15/2.05	110	200	7.70	
Bay 20	Surface	1.47	0.43	0.25	0	1.64	0.15	0.30	2.15/2.09	110	200	7.68	
Bay 20	Bottom	1.47	0.43	0.25	0	1.64	0.10	0.30	2.15/2.04	108	200	7.75	
Bay 32	Surface	1.47	0.43	0.25	0	1.64	0.15	0.35	2.15/2.14	110	200	7.57	
Bay 32	Bottom	1.47	0.43	0.25	0	1.64	0.10	0.35	2.15/2.09	110	220	7.72	
Bay 44	Surface	1.47	0.43	0.25	0	1.64	0.15	0.30	2.15/2.09	108	200	7.61	
Bay 44	Bottom	1.47	0.43	0.25	0	1.64	0.15	0.30	2.15/2.09	108	200	7.80	

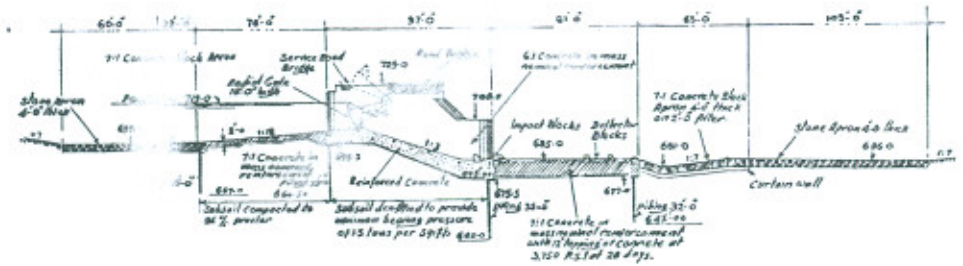
LAYOUT PLAN OF RASUL BARRAGE



TYPICAL CROSS SECTION – UNDERSLUICE BAY



TYPICAL CROSS SECTION – STANDARD



TYPICAL CROSS SECTION – LINK CANAL REGULATOR

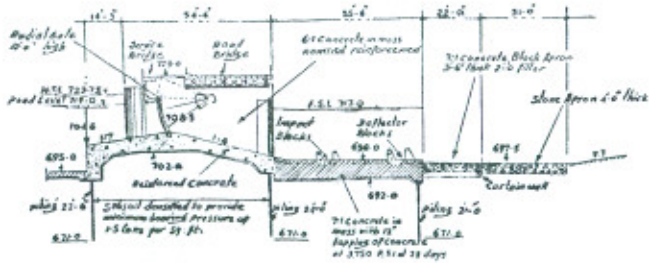
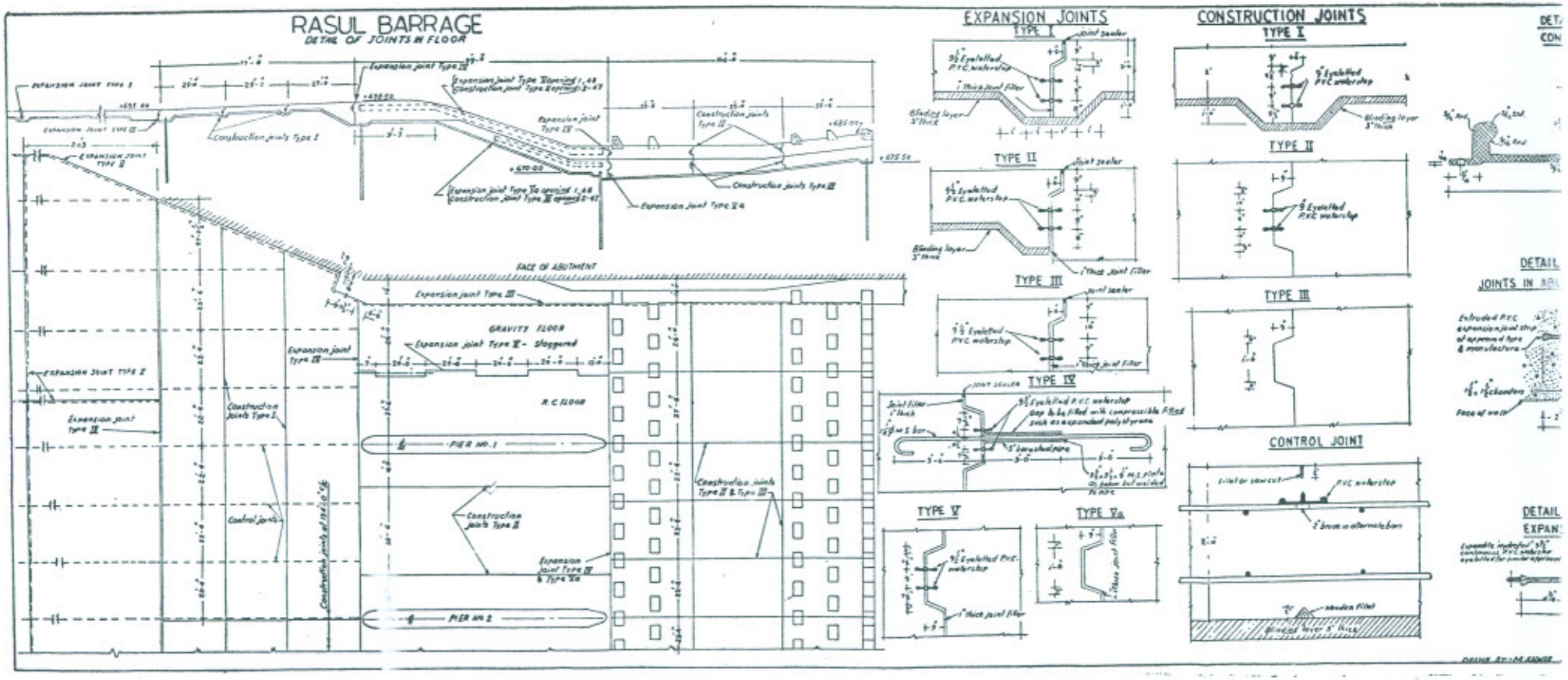
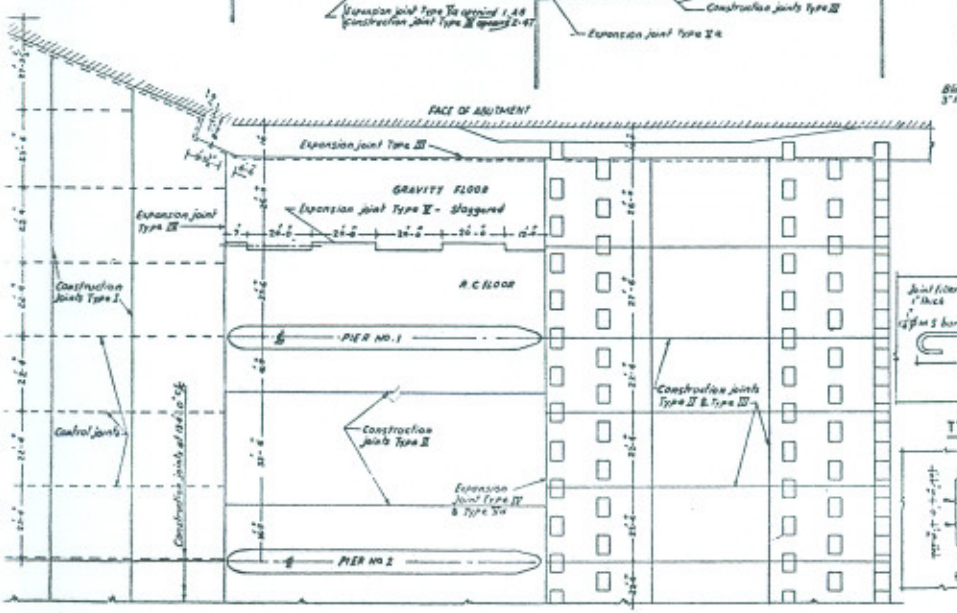


PLATE No. 1


RASUL BARRAGE
DETAIL OF JOINTS IN FLOOR
EXPANSION JOINTS
TYPE I
TYPE II
TYPE III
TYPE IV
TYPE V
TYPE Va
CONSTRUCTION JOINTS
TYPE I
TYPE II
TYPE III
CONTROL JOINT
GRAVITY FLOOR
R.C FLOOR
FACE OF ABUTMENT
PIER NO. 1
PIER NO. 2
Expansion joint Type I
Expansion joint Type II
Expansion joint Type III
Expansion joint Type IV - Staggered
Expansion joint Type Va
Construction joints Type I
Construction joints Type II
Construction joints Type III
Control joints
Joint sealer
Joint filler
Blinding layer
PVC waterstop
DETAIL
JOINTS IN ABU
DETAIL
EXPAN

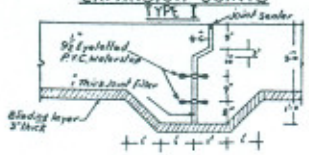
RASUL BARRAGE

DETAIL OF JOINTS IN FLOOR

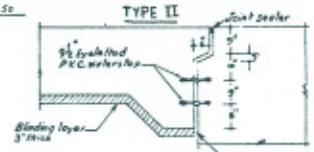


EXPANSION JOINTS

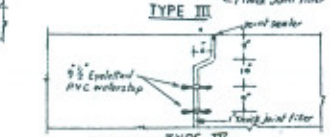
TYPE I



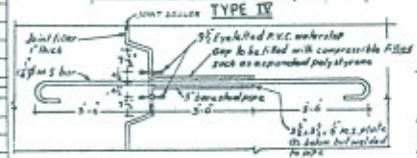
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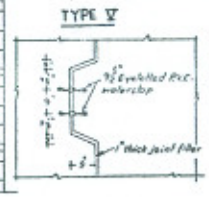
TYPE III



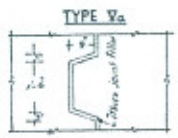
TYPE IV



TYPE V

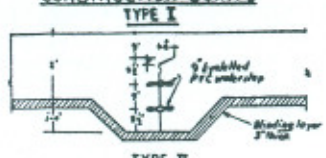


TYPE Va

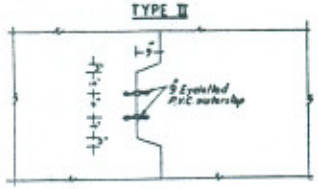


CONSTRUCTION JOINTS

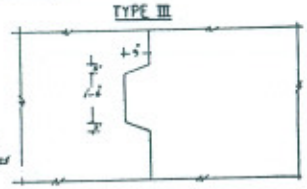
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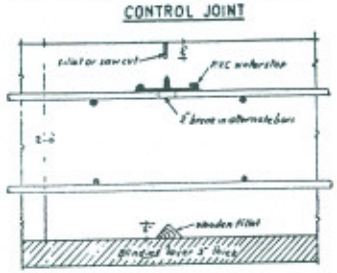
TYPE II



TYPE III

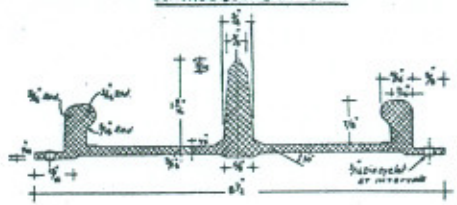


CONTROL JOINT

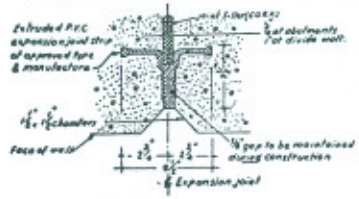


DETAIL OF P.V.C WATERSTOP FOR CONTROL JOINTS IN FLOOR

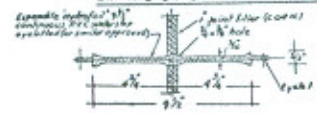
PLATE No. 2



DETAIL OF VERTICAL EXPANSION JOINTS IN ABUTMENT WALLS AND DIVIDE WALL



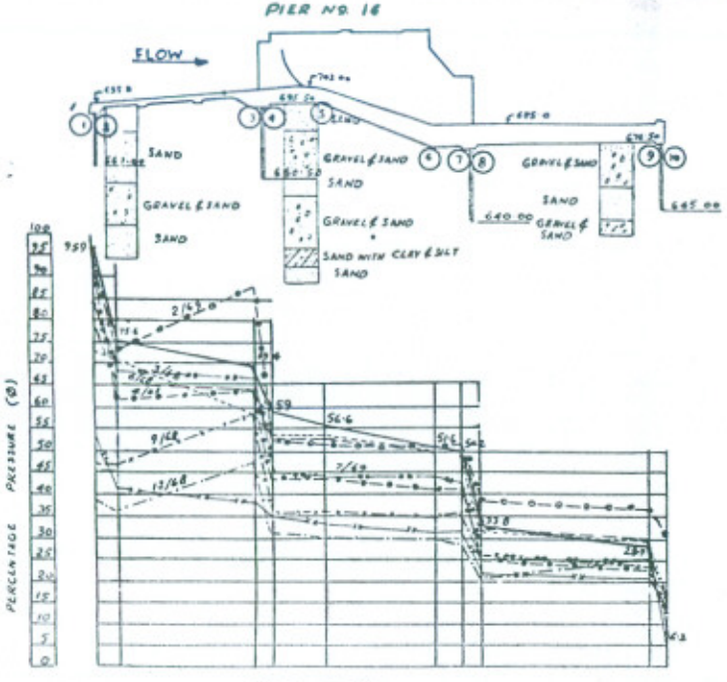
DETAIL OF P.V.C WATERSTOP FOR EXPANSION JOINTS IN FLOOR



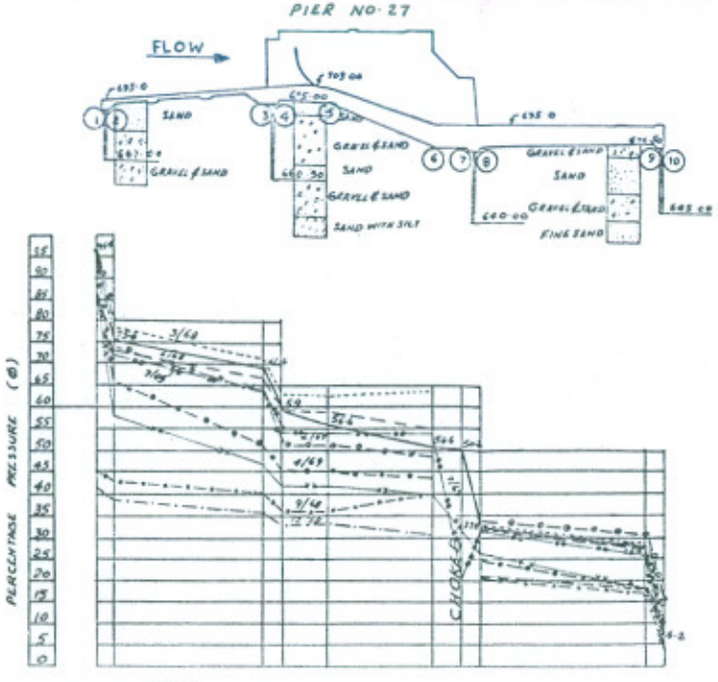
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TRACED BY: M. Ayub

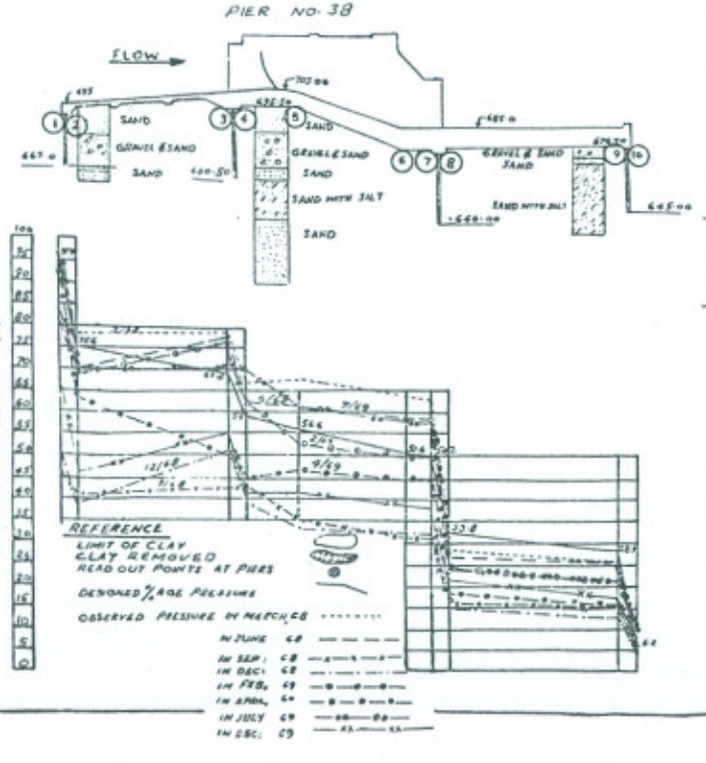
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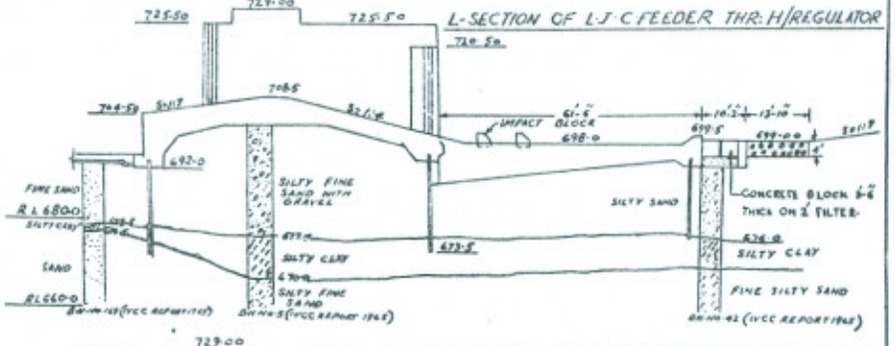
PIER NO. 27



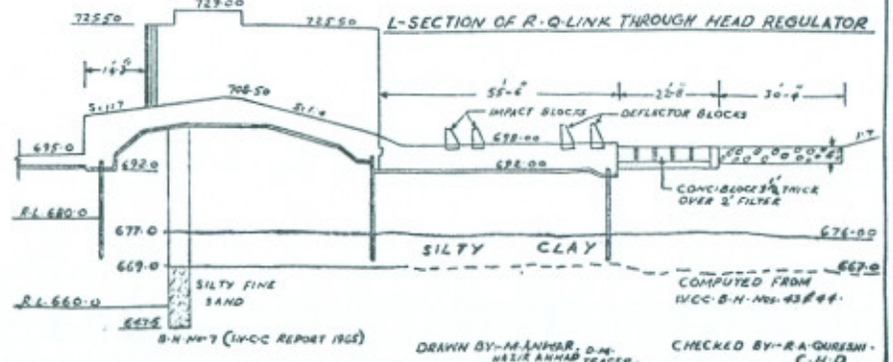
PIER NO. 38



L-SECTION OF L.J.C FEEDER THR: H/REGULATOR



L-SECTION OF R.Q.LINK THROUGH HEAD REGULATOR

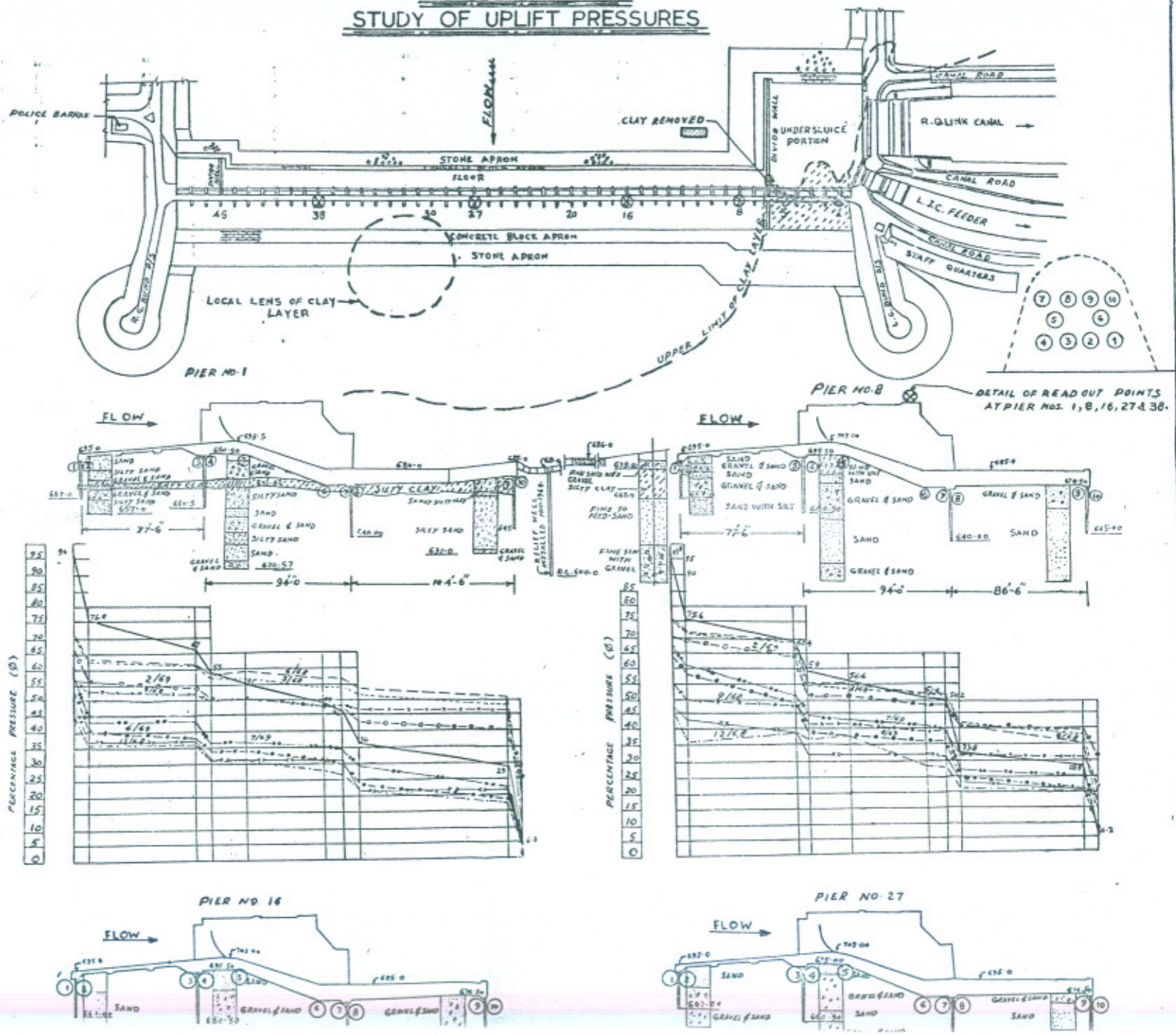


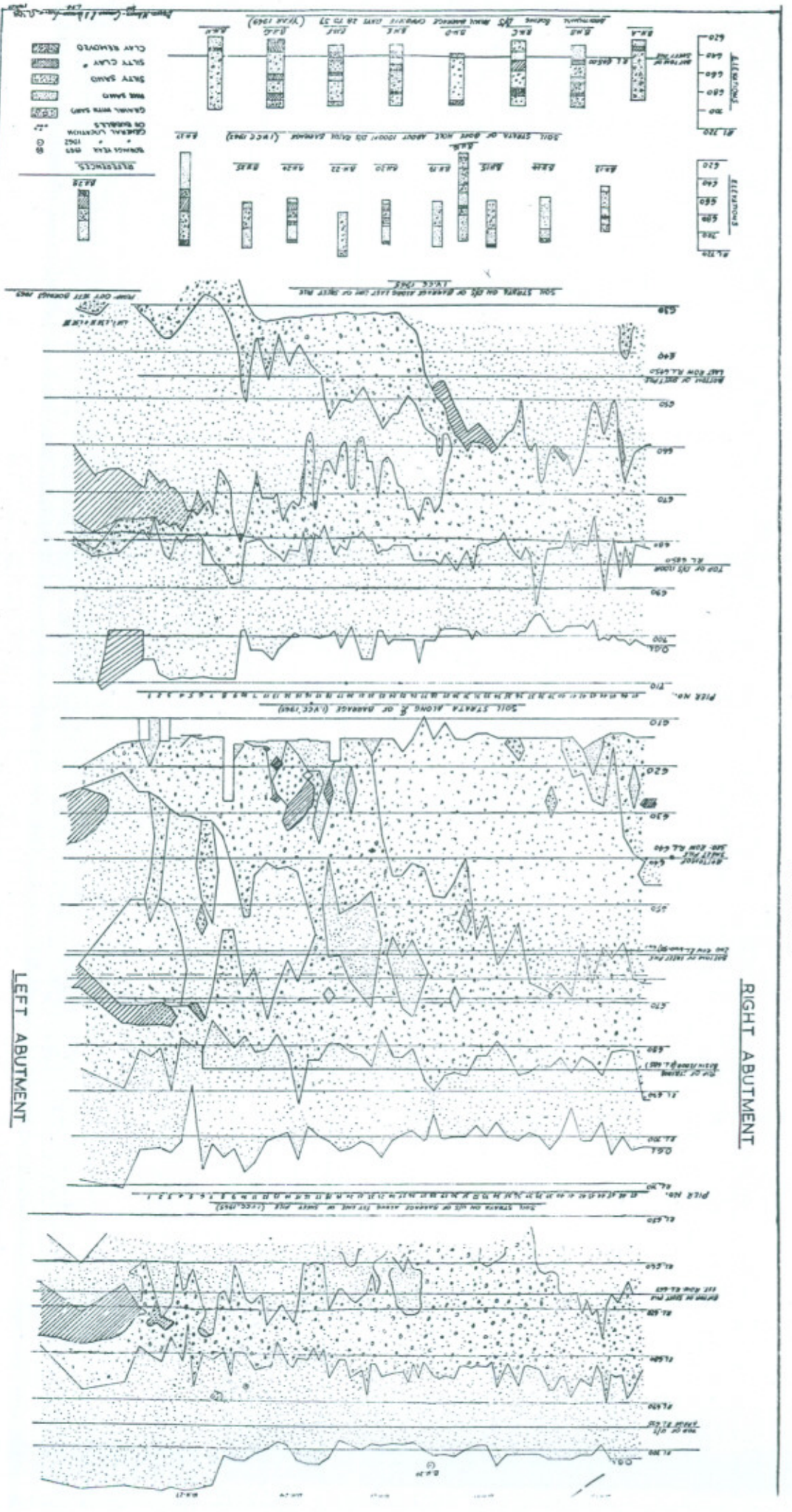
DRAWN BY:- M. ANWAR, D.M. NASIR AHMAD TRACER.

CHECKED BY:- R.A. QURESHI. C.H.D

PLATE Nº 3

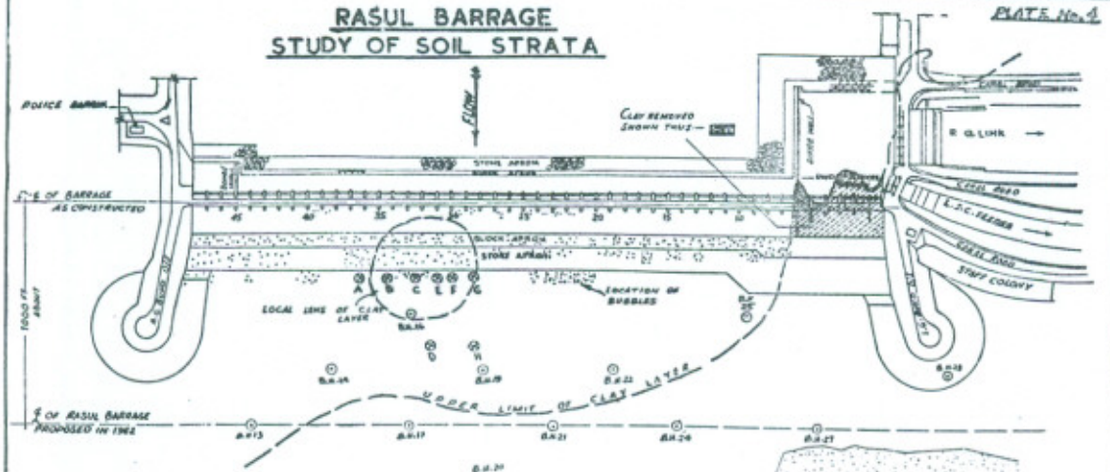
RASUL BARRAGE
STUDY OF UPLIFT PRESSURES

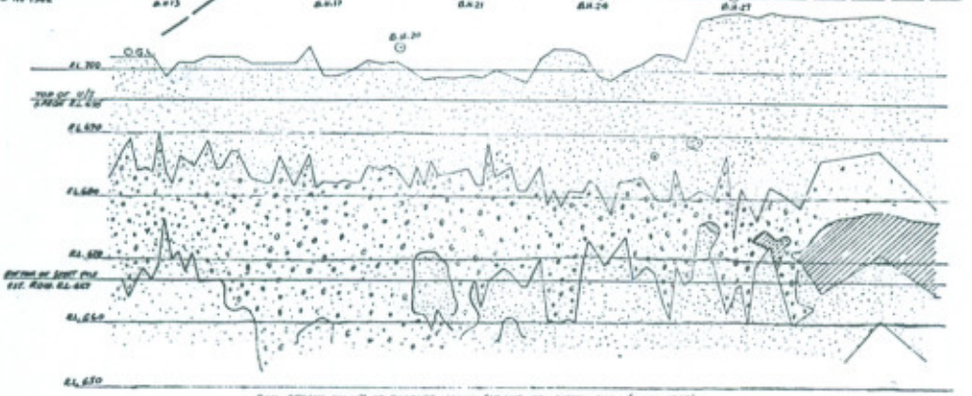



LEFT ABUTMENT
RIGHT ABUTMENT

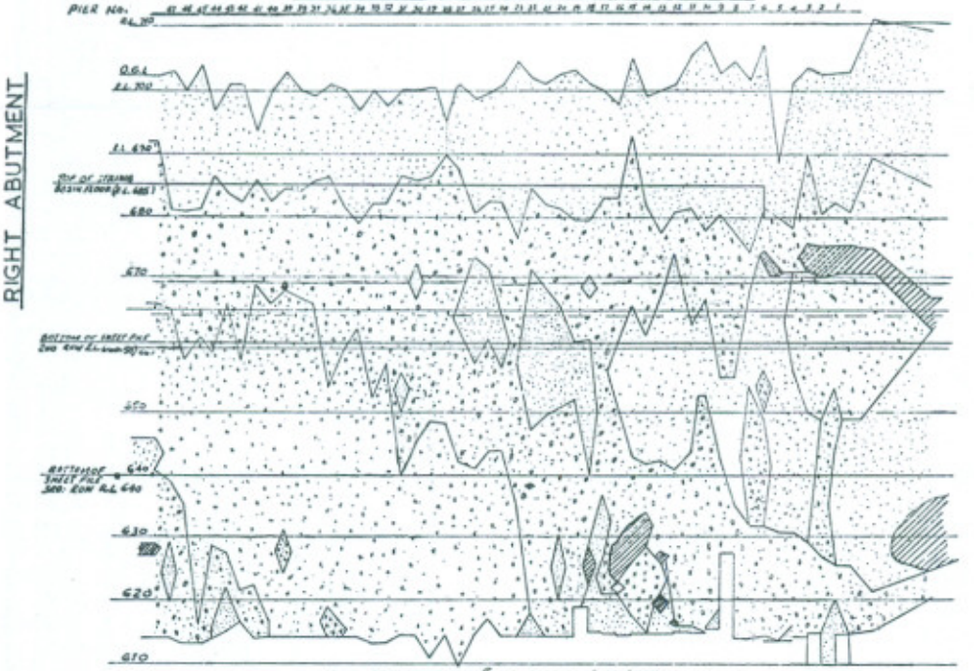
RASUL BARRAGE
STUDY OF SOIL STRATA

PLATE No. 1





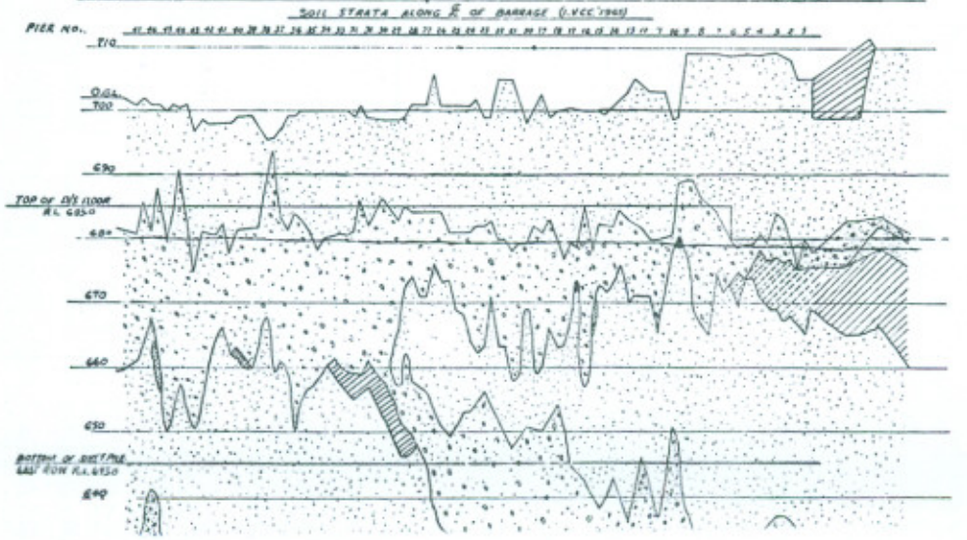
SOIL STRATA ON U/S OF BARRAGE ALONG 1ST LINE OF SHEET PILE (I.V.C.C. 1947)



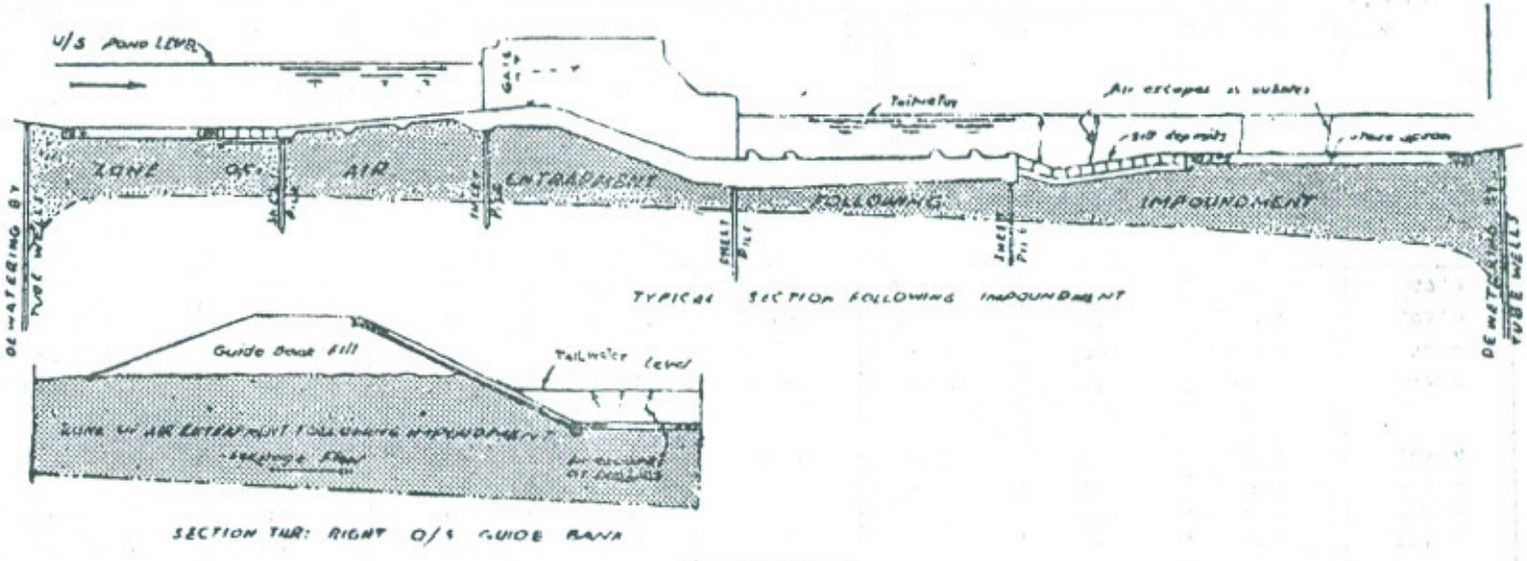
RIGHT ABUTMENT

LEFT ABUTMENT

SOIL STRATA ALONG ℄ OF BARRAGE (I.V.C.C. 1965)



RASUL BARRAGE
AIR ENTRAPMENT FROM CONSTRUCTION SEQUENCE



TYPICAL SECTION FOLLOWING IMPOUNDMENT

SECTION THRU: RIGHT O/S GUIDE BUND

PLATE No. 5

RASUL BARRAGE - UNDERSLUICES - SECTIONAL MODEL TESTS

CONDITION OF FLOOR	DISCHARGE INTENSITY IN CUS/FT RUN.	UPSTREAM POND LEVEL	VELOCITY AT END OF CONCRETE BLOCK APRON		MAX: SCOUR BELOW D/S BED LEVEL IN FT.	DISCHARGE OF SCOUR FROM END OF LOOSE APRON IN FT.	UPPER LIMIT OF JUMP FORMATION R.L.	LOWER LIMIT OF JUMP FORMATION R.L.
			SURFACE FT:/SEC:	BED FT:/SEC:				
ORIGINAL DESIGN	100	719	11.37	7.35	6.1	25	707.3	695.7
SKETCH - I	200	719	16.10	10.60	11.7	55	711.6	700.8
	250	719	17.87	11.00	13.0	65	712.7	703.6
	354	719	19.10	14.00	16.9	95	712.2	705.8
DOWNSTREAM	100	719	8.85	6.125	6.25	13	706.6	697.0
Floor with network of	200	719	14.20	8.500	12.05	35	711.1	702.0
concrete block	250	719	17.625	10.500	12.20	45	712.2	703.8
Sketch - II.	354	719	20.85	11.600	16.35	115	715.1	707.1

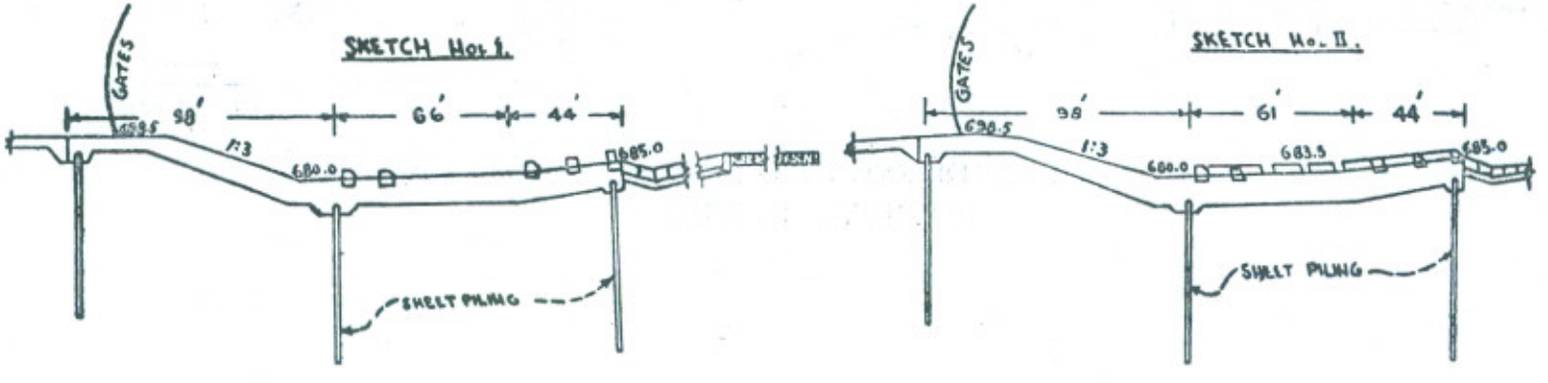


PLATE No. 7

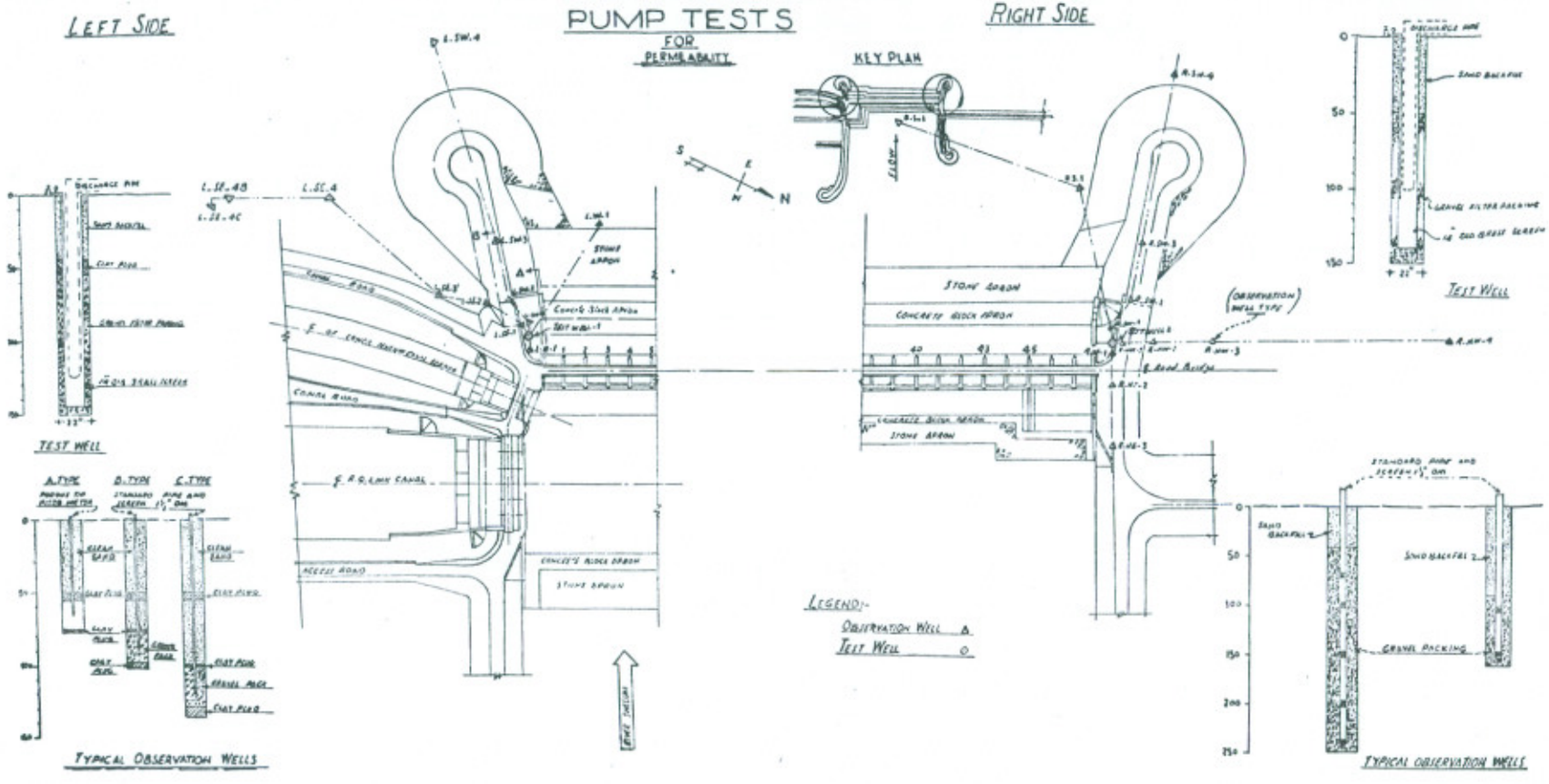
PLATE No. 6

RASUL BARRAGE

PUMP TESTS

FOR

PERMEABILITY



RASUL BARRAGE
ANALOG STUDIES FOR INVESTIGATING REMEDIAL MEASURES AGAINST EXCESSIVE UPLIFT PRESSURES

PLATE No. 3.

PROPOSED ADDITIONAL U/S CUTOFF FOR UNDERSLUICES (130' DEEP)

PROPOSED ADDITIONAL U/S CUTOFF FOR STANDARD BAYS (80' DEEP)

GRAVEL FILLED TRENCH IN THE BLOCK APRON OR SHALLOW DRAINAGE WELLS IN UNDER-SLUICES.

10' WIDE TRENCH

SHALLOW DRAINAGE WELLS.

EXCAVATION OF CLAY LAYER UNDER THE STONE APRON GRAVEL FILLED DRAINAGE TRENCH IN STONE APRON IN UNDERSLUICES

10' WIDE GRAVEL FILLED TRENCH

GRAVITY RELIEF WELLS (80' DEEP DRAINAGE WELLS) FOR UNDERSLUICES

GRAVITY RELIEF WELL 80' DEEP

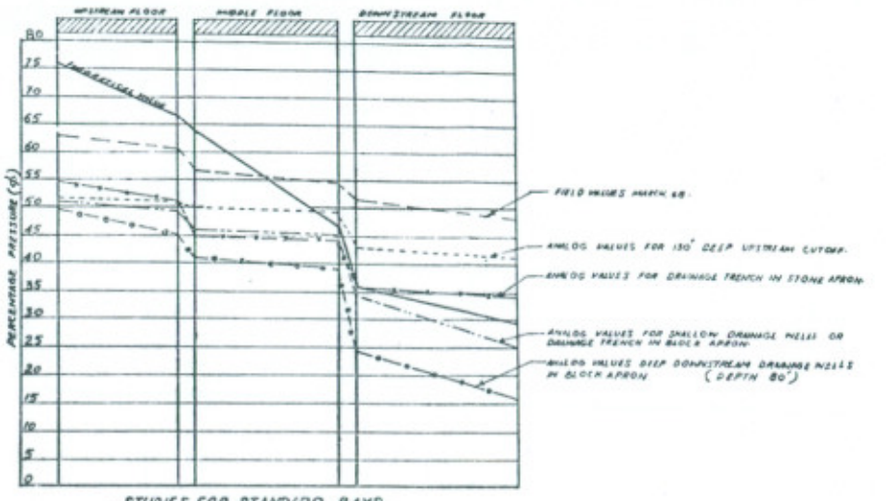
GRAVITY RELIEF WELLS (60' DEEP DRAINAGE WELLS) FOR STANDARD BAYS

GRAVITY RELIEF WELL 60' DEEP

REFERENCE:-

PROPOSALS SHOWN THUS :-

STUDIES FOR UNDERSLUICE BAYS



STUDIES FOR STANDARD BAYS

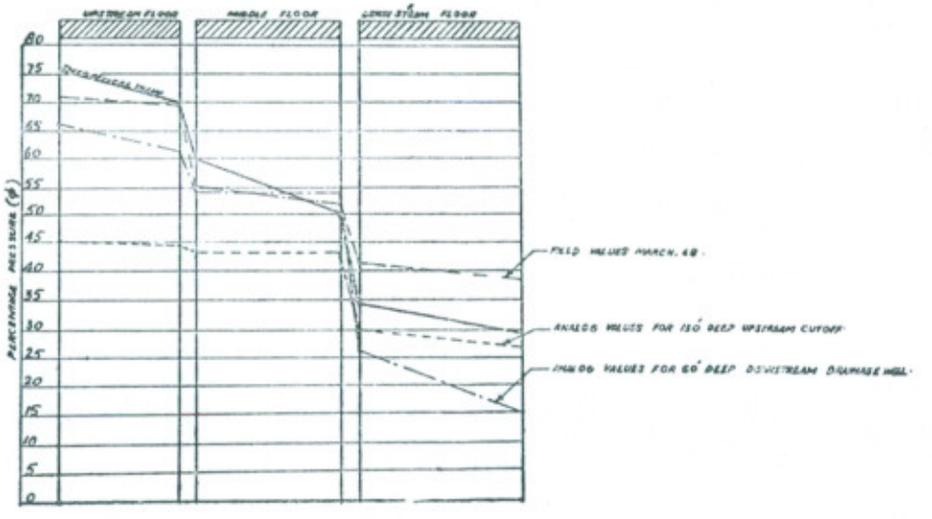
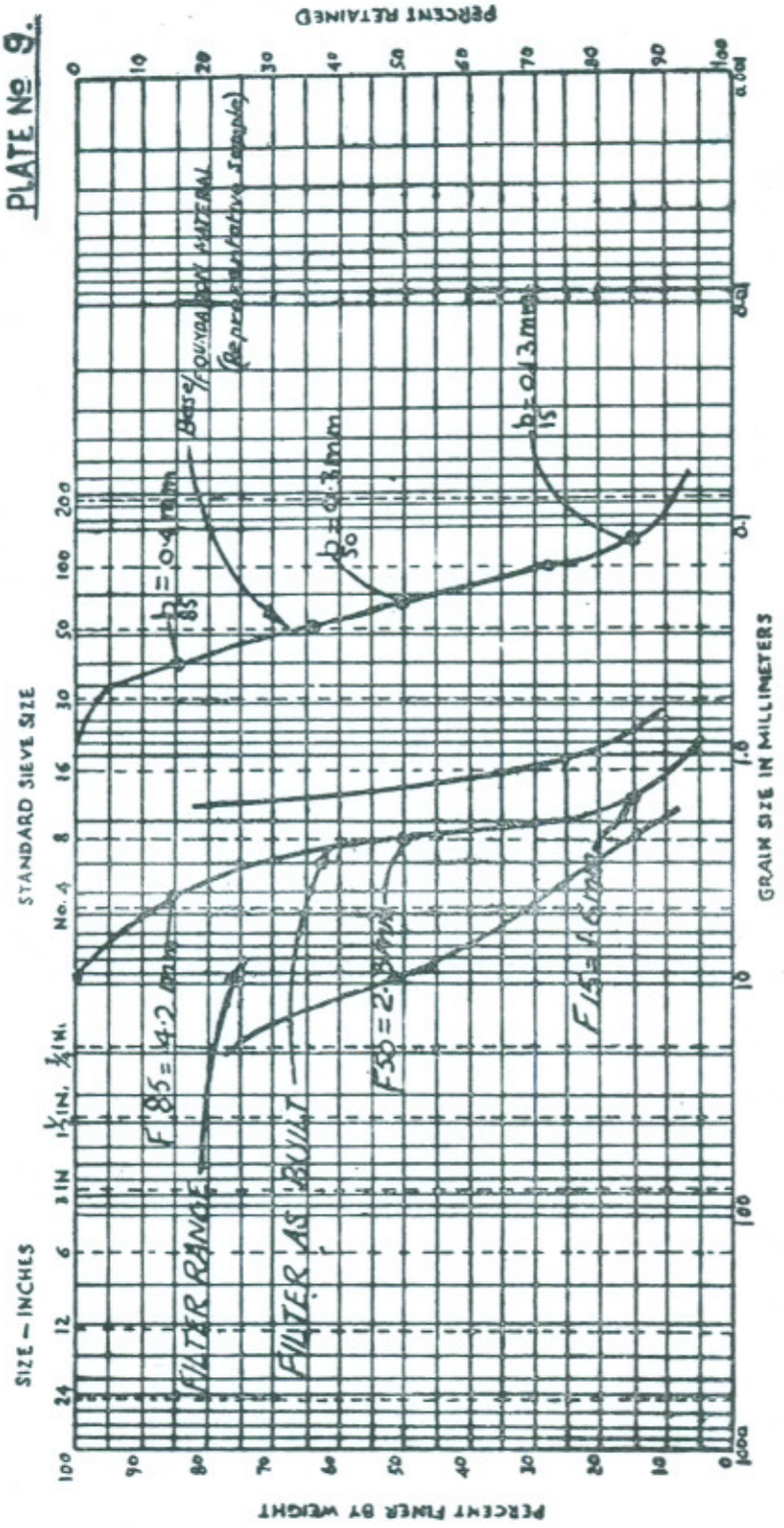


PLATE No 9.



RASUL BARRAGE

PRESSURE RELIEF WELLS

DESIGN OF FILTER MATERIAL ARROUND GLASS FIBER STRAINER IN RELATION TO THE FOUNDATION MATERIAL

RASUL BARRAGE.

UNDERSLUICE PRESSURE RELIEF WELLS ——— INSTALLAT

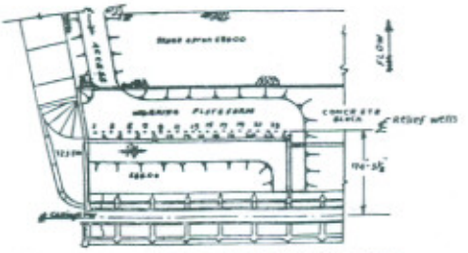


FIG.1 PLAN LEFT SIDE BARRAGE

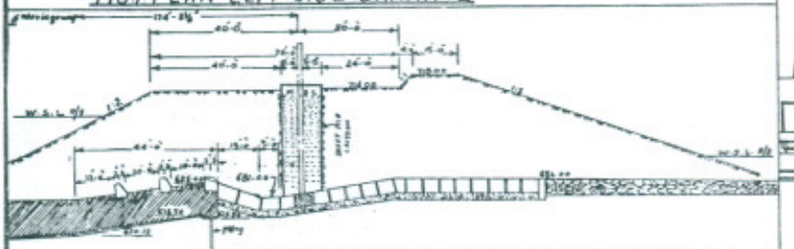


FIG. 2. UNDERSLUICE RELIEF WELLS LOCATION

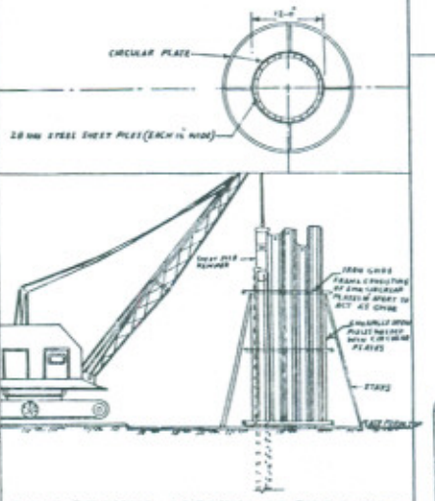


FIG. 3. PITCHING & DRIVING OF PILE CAISSON

FIG. 4 EXCAVATI

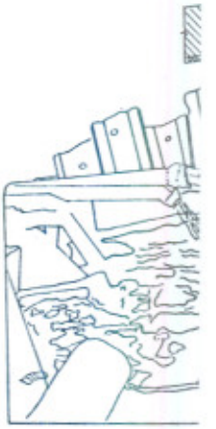


FIG. 5. CLEANING

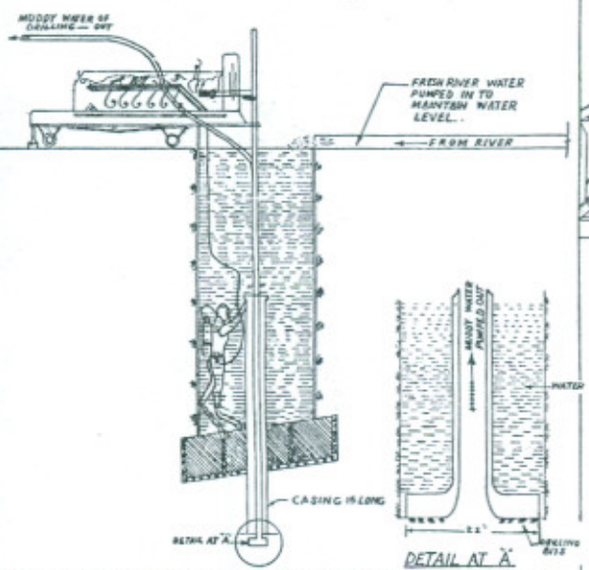


FIG. 8 DRILLING WITH IDROSONDA MACHINE (WASH BORING)

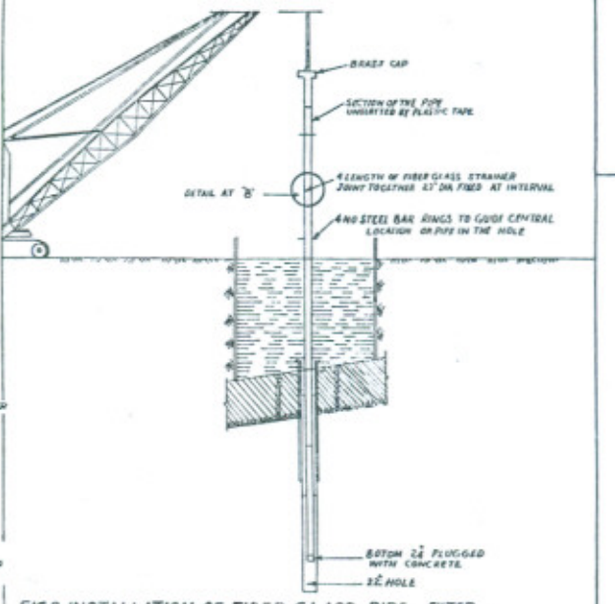


FIG 9. INSTALLATION OF FIBER GLASS PIPE FILTER



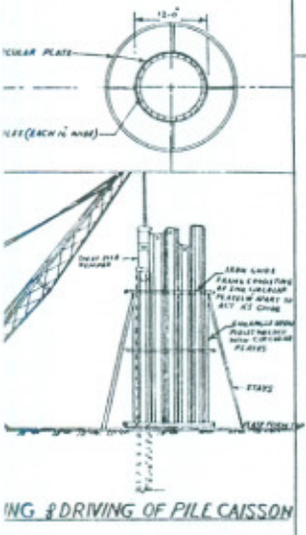
FIG. 10 PACKIN

FIG. 11. FIXING

RASUL BARRAGE.

PLATE NO. II

F WELLS ——— INSTALLATION DETAILS



ING. 8 DRIVING OF PILE CAISSON

FIG. 4 EXCAVATION BY BUCYRUS 22 B EXCAVATOR

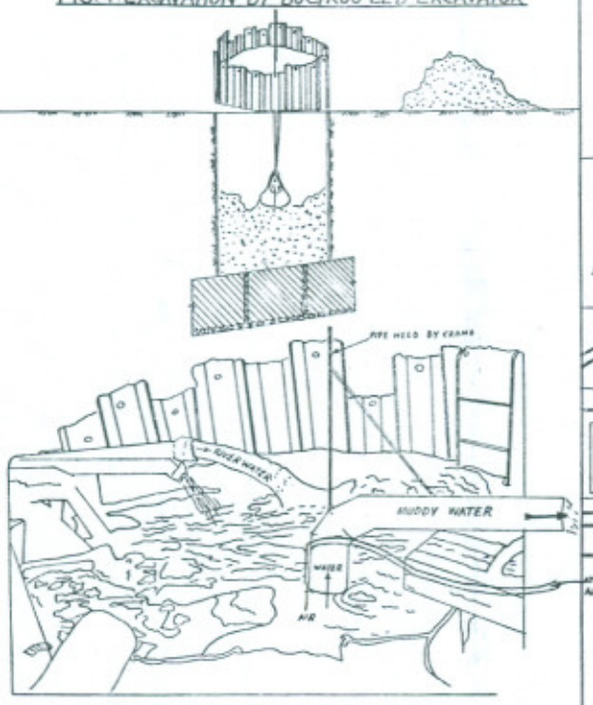


FIG. 5 CLEANING WITH AIR WATER PUMP

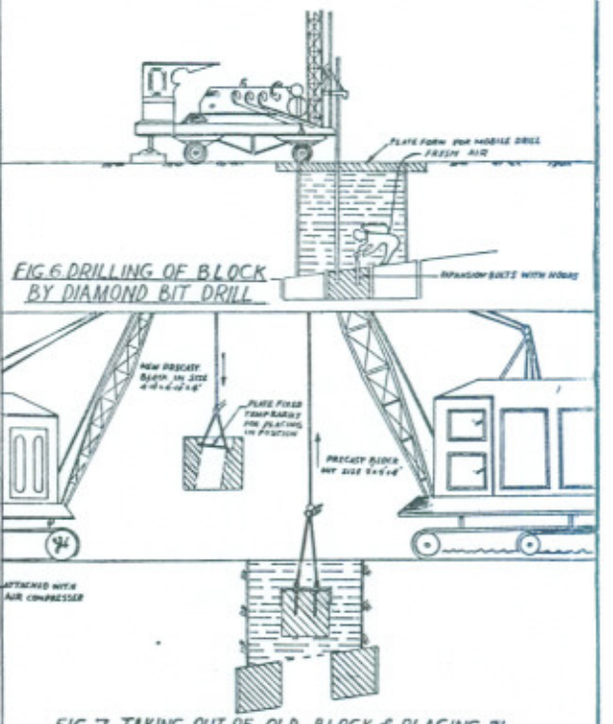
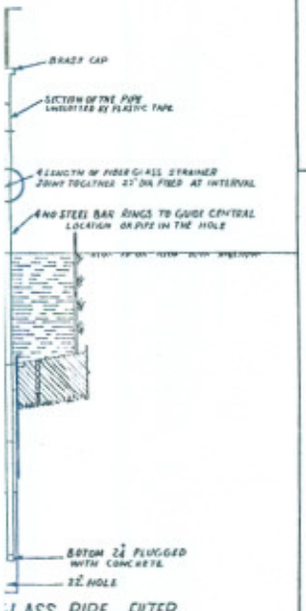


FIG. 6 DRILLING OF BLOCK BY DIAMOND BIT DRILL

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



LASS PIPE FILTER

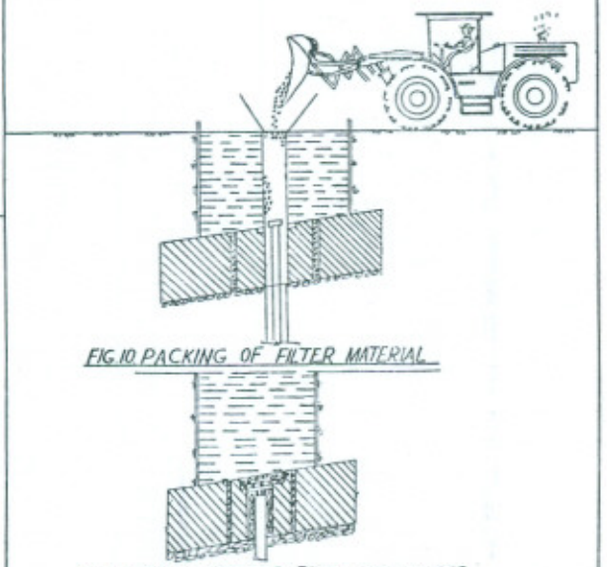
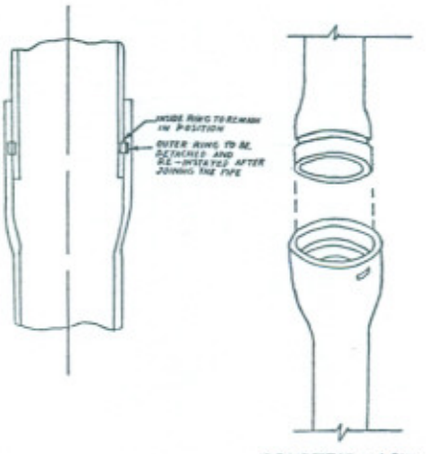


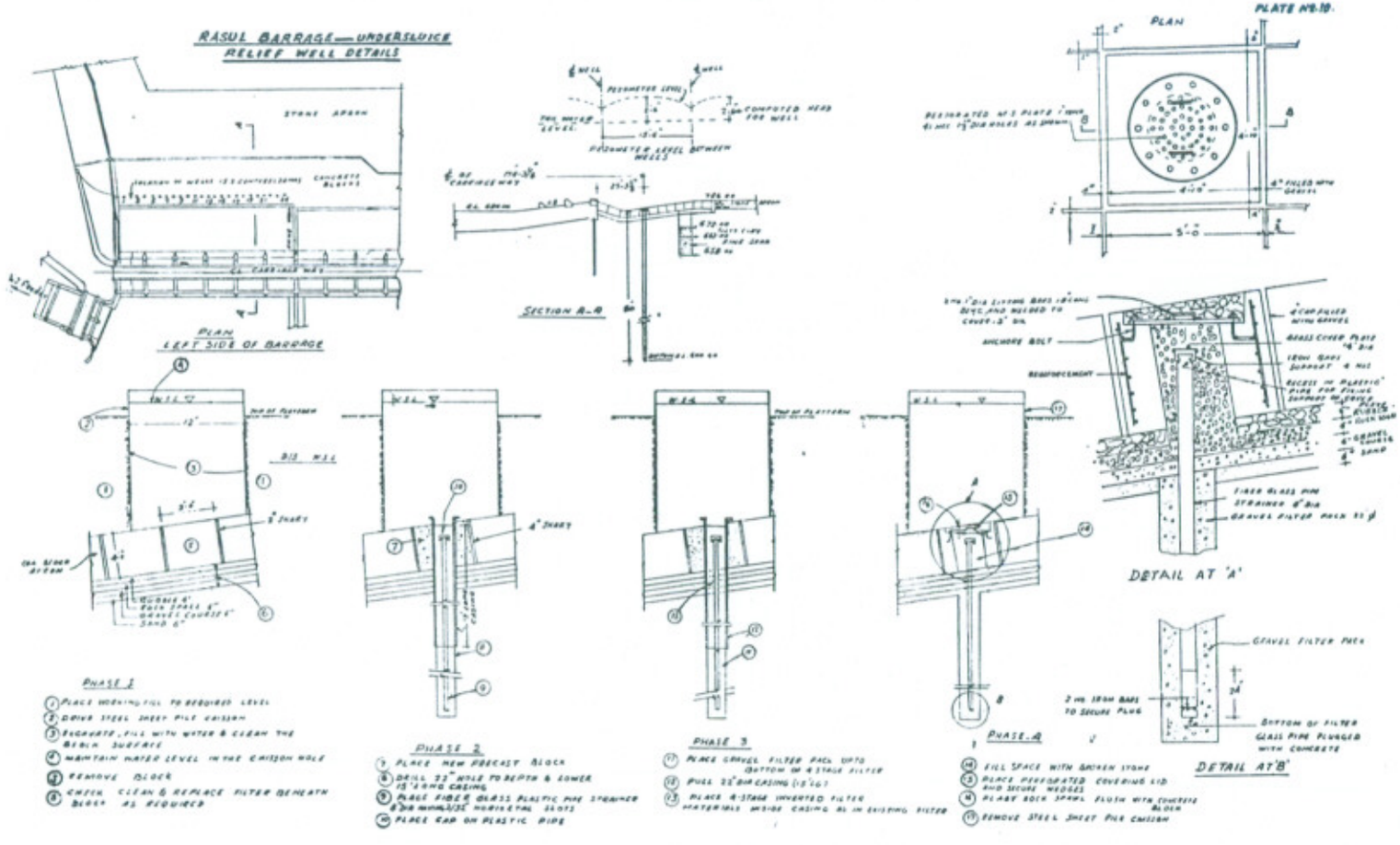
FIG. 10 PACKING OF FILTER MATERIAL

FIG. 11. FIXING OF TOP PLATE COMPLETE



ISOMETRIC VIEW

DETAIL AT 'B'


RASUL BARRAGE — UNDERSLUICE
RELIEF WELL DETAILS
PLATE No. 10.
PLAN
LEFT SIDE OF BARRAGE
SECTION A-B
PLAN
DETAIL AT 'A'
DETAIL AT 'B'
PHASE 1
PHASE 2
PHASE 3
PHASE A