PAPER No. 364

Marala Headworks—Site Investigation

By
H. J. ASAR*

Introduction

The Marala Headworks is situated on the river Chenab north of Sialkot as indicated on general plan showing site of Headworks (Plate I).

In Paper No. 360 of West Pakistan Engineering Congress, Session 1963 namely "Alternative Proposals for Remodelling Marala Headworks", the author described in detail ten alternatives which had been gone into for improving conditions at Marala Headworks. Out of these ten alternatives the last two were finally approved to be taken up by the concerned authorities. These two alternatives are as under :—

1. Combination scheme—new barrage immediately downstream of the existing weir.
2. Remodelling of existing Marala Weir with provision of radial gates etc.

Before the designs of these two schemes could be proceeded with it was essential to carry out site investigations including sub-surface exploration etc. The importance of these investigations cannot be over-emphasised. This paper aims at describing in brief the investigations done in this behalf.

The investigations done for the new barrage site are the standard ones and are useful to know the quality of the sub-strata. The sub-surface explorations made at the existing weir brought about results of special significance which formed basis for the decision to give up the said scheme without detailed designs. The real objective of this paper is to bring these conditions to the notice of the Irrigation Engineer with a note of caution that such conditions may be existing even under other weirs. May be that Marala Weir originally designed with scanty knowledge of river works and thereafter having undergone changes from time to time had been subjected to undesired sub-soil flow resulting in the state of affairs revealed by the sub-surface explorations. Still it is an exemplary case and investigation on the same lines on other weirs is indicated.

Investigations for New Barrage Site

The exploratory borings were to be carried out with the object of obtaining information about the sub-soil where the foundation of the proposed new

*Chief Engineer, Remodelling Organization, Lahore.

barrage is to be placed. This work was entrusted to Swiss Boring Overseas Corporation in collaboration with the Indus Basin Valley Construction Company, Limited, Lahore and was carried out under the general supervision of the Harza Engineering Company International, General Consultants to WAPDA.

As indicated on Plate II the new Marala Barrage was proposed 736 feet below the centre line of the existing weir. Borings proposed were carried out along 3 lines A, B and C running from the right to the left bank. The line B is the central line of the proposed barrage. Line A is 100 feet and line C 200 feet on the downstream of the line B. These borings were located in such a way that they cover the entire area to be occupied by the proposed weir. In all 26 bores were done against 33 originally proposed. The number of holes bored were as under :—

A Line	..	7
B Line	..	11
C Line	..	8

The borings were carried out by percussion method using $4\frac{1}{2}''$ and 8" diameter pipe casings.

Standard penetration tests were carried out in alternate holes at 5 feet intervals starting from the foundation level. Samples were extracted by Penetro-meter and were preserved in glass jars for laboratory tests.

Disturbed samples with a split spoon smapler of $1\frac{3}{8}''$ diameter driven into the soil with 140 lbs. drop hammer falling through 30" were collected at 5 feet intervals or at each change of the strata. These samples too were preserved in glass jars for complete grain size analysis in the laboratory.

Seven number permeability tests were undertaken in different holes on all the three lines through "Falling Head" method.

The Statement showing the bore holes actually carried out, their top and bottom level and length of holes etc., is given as Appendix I.

Undisturbed samples were also obtained wherever cohesive soil was met with. These samples were extracted by means of thin walled shelby tubes made air-tight by wax immediately after their extraction and sent to the laboratory for tests. The geological cross sections plotted as a result of these explorations are indicated on Plates III, IV and V.

Concluding it could be stated that the sub-grade at the proposed site of the barrage consists of three strata. The top one generally consists of medium to fine sand which is under-lain by a stratum of medium sand mixed with pebbles and cobbles of sizes varying from 1" to 4" diameter. It is significant that sand becomes coarser downwards. The second stratum consisting of

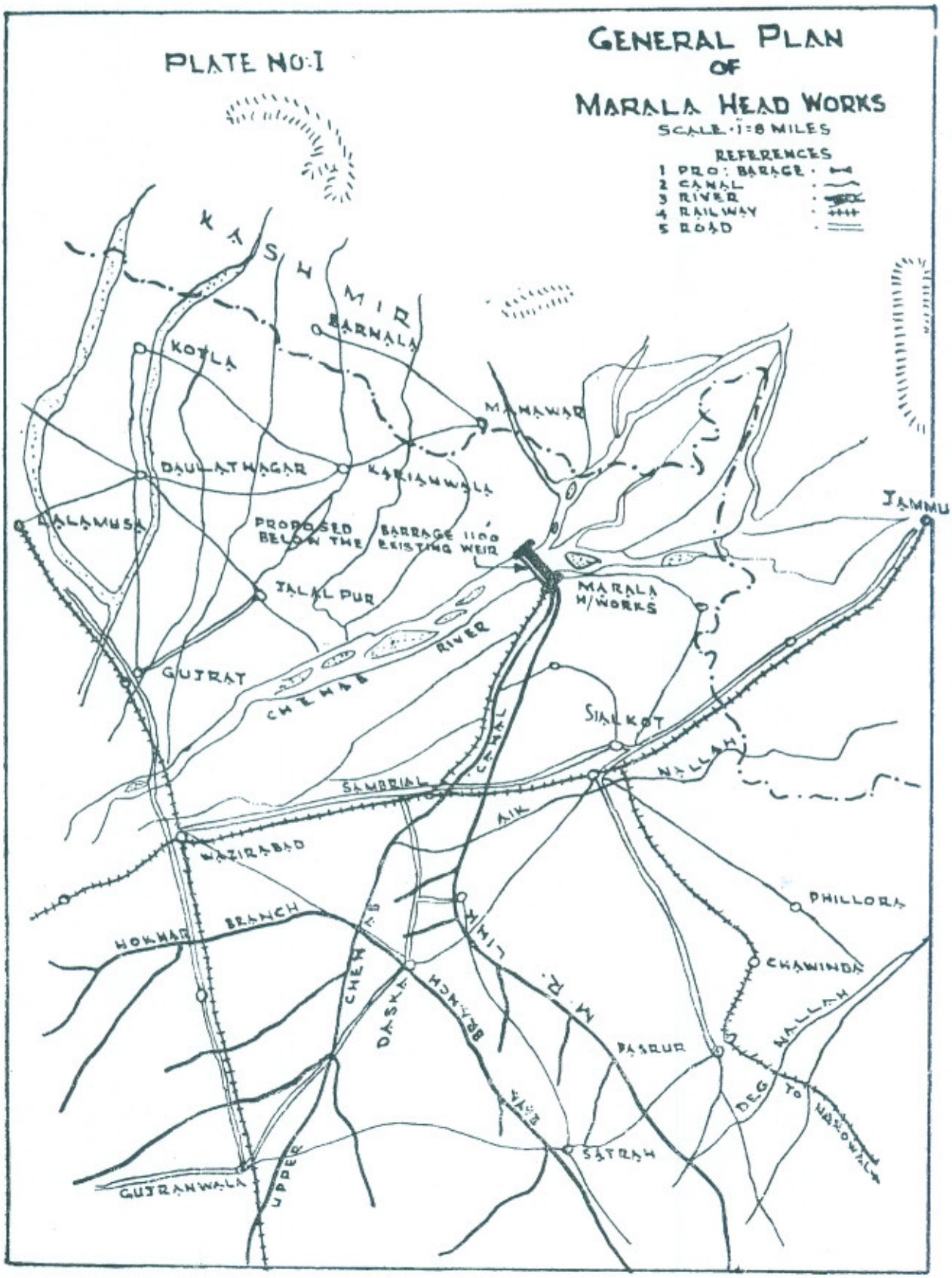


PLATE NO: I
GENERAL PLAN
OF
MARALA HEAD WORKS
SCALE 1:8 MILES
REFERENCES
1 PROJ: BARAGE
2 CANAL
3 RIVER
4 RAILWAY
5 ROAD
KASHMIR
KOTLA
BARNALA
MANAWAR
DAULATNAGAR
KARIANWALA
LALAMUSA
PROPOSED BARRAGE 1100' BELOW THE EXISTING WEIR
JALALPUR
MARALA H/WORKS
JAMMU
GUJRAT
CHENAB RIVER
SIALKOT
CANAL
NALLAH
SAMBRIAL
AIK
WAZIRABAD
PHILLORA
NOKHAR BRANCH
LINK
CHENAB
CHAWINDA
DASKA
BRANCH
M.R.
NALLAH
PASRUR
SATRAH
D.G. TO NAROWALA
GUJRANWALA
UPPER

medium to coarse sand mixed with some pebbles or cobbles varies in depth from 15 to 45 feet. The boring investigation for final site of the barrage about 1100 feet downstream are in progress and the results so far obtained indicate that the strata encountered is similar. It is also noticeable that the second stratum is situated at greater depth towards the right bank and smaller towards the left bank. The third stratum below the second one consists of stiff clay. This clay layer is also inclined from left to right. This is indicative of the fact that the river stream at one time had scoured down to the clay layers in that inclined plan and later filled with sand deposits of varying grades.

The ultimate bearing capacity of cohesive soils is generally estimated from bearing capacity theories and on the sheering strength of undisturbed soil samples. For soils without cohesion where undisturbed sampling is not possible penetration tests can only be used for calculating the bearing capacity of a soil. The standard penetration test is the one commonly used for the purpose. In this a 2 inches outside diameter split spoon samples of $1\frac{3}{8}''$ internal diameter is driven into the ground at bottom of a clean bore hole. A drop hammer weighing 140 lbs is made to fall on the top of the drill rods to make the spoon penetrate about 6" into the soil at each point where the penetration test starts. The number of blows required to make the spoon penetrate further 1 foot are noted. The results of standard penetration tests co-related with the relative density of sand and the corresponding bearing capacity are tabulated below:—

Standard Penetration Resistance N	State of Packing	Relative Density	Angle of Internal Friction	Allowable soil pressure q_a (tons/sq. ft.)
4	Very loose	0.2	28°	0.4
4—10	Loose	0.2—0.4	28°—30°	0.4—0.7
10—30	Medium	0.4—0.6	30°—36°	0.7—2.5
30—50	Dense	0.6—0.8	36°—41°	2.5—4.5
Over 50	Very dense	0.6—0.8	41°	4.5

The allowable soil pressures shown above are adjustable for reduction in the case of silty sands and increase in the case of clean sand gravel mixtures in accordance with the above shown valuation of 'n' and the grain size distribution.

The new barrage is now proposed about 1100 feet below the existing

weir to enable locating the new head regulators for Upper Chenab Canal and Marala-Ravi Link immediately above the new barrage. The investigations for the new site had to be carried out afresh under the advice of the Engineer, Messrs Coode and Partners, Consultants for Barrages, WAPDA. Conditions of sub-strata are not very different from those explored for the previous site.

2. Remodelling of Existing Weir

Before describing the sub-soil explorations carried out at the existing Marala Weir it might prove useful to describe briefly the history of the existing Marala Weir.

History of Marala Weir.—Marala was one of the few weirs built in the actual bed of the river and the work was accomplished under great difficulties; the construction having started in the winter of 1908 and completed in May 1910. Due to record winter freshet of 1,50,000 cusecs in February 1909 the work was stopped and resumed in the month of November. In the glacis above the A line of the wells, the bottom layer of masonry was laid dry and a weak grout of lime surki and sand poured over it which did not completely fill the joints between the stone. The study of these open joints on examination in the year 1937 showed that water had been flowing freely through them over the sand below. The weir was first commissioned in 1912 and after a few moderate floods during monsoon of the same year the glacis was found full of cracks and springs in October the same year. It was thought that the springs were due to the inter-stices in the rubble masonry not having been filled up with mortar during construction. In the following year fresh cracks appeared and in 1917 after a flood of 5,50,000 cusecs the stone on edge coarse in Bay No. 6 was found uplifted or bulged in an area of 19-50 feet above the B line of the wells. In subsequent years this bulging or "blistering" as it was called, occurred almost all over the glacis between the crest and C line of wells. In the year 1922 the Executive Engineer drew attention to a serious depression that had existed in Bay No. 8 between B and C lines of wells where springs were active. Next year he pointed out that most of the joints in the weir were open and that there were numerous springs in Bay No. 1 from which he concluded that there was much cavity in this bay. In the winters of 1924-25, 1925-26 the crest of the weir was raised by 2 feet to R. L. of 802.00 and this was an opportune time to repair upstream floor which was found full of hollows and open joints. During floods of 6,86,000 cusecs in September, 1928 and 6,60,000 cusecs in August, 1929 a good deal of damage was done below the weir. Bay No. 4 was worst affected. In addition to the removal of the loose protection in this bay the glacis between B and C lines was carried away in a length of 150 feet. After drawing various methods of repair between the year 1932-36 almost half the stone on edge coarse between the crest and B line of the wells was removed

and placed in cement mortar. A detailed examination of the weir in October 1934 showed its condition was far from satisfactory. Between the crest and the B line parts of Bays 1, 3, 4, and 5 and the whole of Bay No. 2 was blistered. Pits were dug in every bay through the stone on edge coarse and it was discovered that fine sand was found deposited between this course and the layer of masonry below and springs were working in every pit. In Bay No. 2 there were two longitudinal cracks and innumerable springs above the B line. In view of the state of the weir in 1934 experiments were carried out on a model in the hydrologic laboratory at Lahore. It was felt vitally important to the safety of these weirs to devise an instrument which can conveniently and speedily detect cavity under them, but no such device could be developed even uptil today. Various proposals for reconditioning of the weir were framed but fell through for one reason or the other. Finally it was decided on dismantling the existing glacis from a line a few feet below the crest where 1 in 4 meets the 1 in 15 slope, are built at a slope of 1 in 4 ending in a floor 4 feet thick with its top 4 feet below the existing floor so as to give a proper depth of water downstream. This necessitated the whole of the loose blocks and stone being removed and lowered 4 feet. It was contended thereby that with the extra width after the extra depth of water available as a result of reconditioning the waves of high flood in future will not be dangerously high. A continuous line of sheet piles fifteen feet deep upstream of C-line of wells was driven. A set of friction blocks was also provided on the downstream floor. This reconditioning was carried out in the year 1937. The sections of the weir before and after reconditioning are depicted in Plate VI. During the process of reconditioning open pumping was resorted to.

In 1954-56 a new 17 bay regulator was provided for feeding Marala-Ravi Link Canal. The weir was provided with a baffle wall at the lower end of the downstream floor to create water load against uplifts.

Condition of the Existing Structure

The weir is designed for a head across of 10 feet with full pond level 808.00. The pressure observations prior to reconditioning done in 1936-37 had shown that well lines were ineffective as cut-offs. As earlier stated examination of the weir floor from time to time had shown open gaps between the wells. In a few cases wells were found all together missing and the construction was found defective in many locations. In 1957 and 1959 the previous block apron on downstream was found damaged in bays 1 and 2. The damage was repaired by providing new blocks in place of old ones which had probably lifted and washed off.

During construction of baffle wall in 1957-58 the weir floor and glacis

were examined and found to have cracks in a few places. The floor and underlying sand were grouted with cement under pressure subsequently.

The repairs to block areas in bay 2 were carried out in 1961 winter. While dewatering numerous springs blowing sand appeared in the block area when water level was down to elevation 794.00 even though head across was only ten feet. The pumping had to be stopped and blocks precast and placed under water. The appearance of springs under the stated conditions was surprising as with sheet pile effective the exit gradient would be as low as 1 in 10. The efficacy of 1937 pile line was therefore doubtful in at least some locations. The piles driven at site of old constructions are likely to have met obstruction from old construction and may have separated while being driven. The bed sand at Marala is generally fine or medium but is found to contain odd cobbles which also could cause an obstruction. In the rush of work it is possible that piles separated at the clutch were not detected or taken care of. It may also not have been possible to drive the piles to full depth in some locations.

Examination of glacis this winter showed a horizontal crack more or less continuous located 15 feet from downstream edge of crest in bays 4, 5 and 6. The crack was sealed with plain cement concrete. The underlying sand was grouted with cement under pressure through holes drilled at 30 feet intervals on the crack line. Large quantities of grout were taken by foundation sand with a limiting pressure of 10 lbs. per square inch by means of grouting machine. Appendix I shows data collected on drill holes in Bay No. 6.

Operation

Prior to 1956 it was seldom necessary to impose high heads across the weir and it was always well below 10 feet the design head. After 1956 the operation of M. R. Link Canal necessitated cross heads as high as 12 feet on weir for maximum utilization of supplies during critical periods of early and late summer when river flow is low and demand keen. These requirements had been anticipated and provided for by additions of a baffle wall in 1957. The weir section was checked and found to be safe for a head of 12.6 feet against uplift and about twenty feet against exit gradient, but head across of 10 feet was never exceeded under any circumstances.

Pressure pipes are installed in all bays of the weir and record of observations has been maintained at the headworks. A good number of these pipes developed defects and many were dropped from observation. The need for increasing permissible head across the weir after 1956 attracted attention to the pressure pipe data. The pipes were tested and efforts made to clean and restore to function the defective ones but these met with only a limited success.

The pressure pipe data for 1962 was examined in detail. The pipes which were found to be reliable on testing and on basis of past records were found to be very few and scattered. Only in weir bay 6 was a fairly good number of functioning pipes available.

Plate VII shows location of pressure pipes on weir profile and anticipated and observed (1962) average pressure lines for weir bay 6 line 450/L. The observed pressure at key points are very close to computed figures in this bay. In other bays the pressure points observation are very few and in some of these, observed pressure are excessive.

SUBSURFACE INVESTIGATION

Purpose

Site investigations involving drilling through existing masonry and concrete of Marala Weir in suitable locations and wash boring through underlying soil were carried out in 1963-64. The object of these investigations was to ascertain soundness of existing work and nature of foundation sand and clay to determine feasibility of Remodelling Weir to make it fit for improved operational conditions. The main features of these requirements were:—

1. Increase in pool levels from 808.0 to 812.0.
2. Increase in head across from 10 to 22 feet for no escapage.
3. Increase in width, length and depth of undersluices.
4. Lowering crest and floors on weir for passage of design flood of 900,000 Cs.
5. Providing gated control and bridge over the undersluices and weir.

The locations of boreholes shown in Plate VIII are briefly stated below:—

1. A series numbered MR-A_1 to MR-A_9 on crest in each weir bay the last of these being through masonry block under crest.
2. B series bearing numbers MR-B_2, MR-B_4, MR-B_6, and MR-B_8 over glacis in weir bays 2, 4, 6 and 8.
3. C series of holes in downstream floor were not allowed on considerations of safety of weir.
4. Hole MR-D_3 in upstream floor.
5. Hole MR-E_3 on groyn 3 downstream.

Method

The borings through concrete and masonry were made by rotary method with core recovery using double tube core barrels with diamond or tungston carbide bits. In the sand underneath wash borings were made using casing

pipe. The clay strata was cored by means of double tube core barrels. Bentonite drilling fluid was used in these borings.

In all cases a stand pipe was fitted before commencement of work and borings were made through these pipes. The stand pipe was kept filled with water to provide adequate head against piping. For the same reason borings in glacis were done under low head and low pond levels.

In the sandy layers below the weir disturbed samples by standard split spoon were taken. The spoon was driven 24 inches deep into soil from bottom of a cleaned hole by blows from 140 lb hammer with 30 inch drop. Number of blows was recorded for every three inches of penetration. The 'N-count' representing number of blows for one foot penetration from 6 inch to 18 inch depth was worked out and are shown in the bore logs (Plate IX).

The blow count for sand under crest upto elevation 780.00 and even lower still in some locations is below five indicating very loose sand. As the bed sand in the vicinity of headworks is of medium compaction with average blow count of 15 to 30 it appears that during construction operations or thereafter through seepage flow underneath fines have been washed off from sand resulting in loss of density and resistance to penetration.

It is of particular significance that in some holes such as A_3, A_4, and B_8 the blow count was zero indicating cavities and hollows underneath.

CONCLUSION

The results of the subsurface investigations, history of damages and site observation and records combined with need for imposing full 10 feet head for long periods have had a marked effect on thoughts on safety of Marala Weir. As a result of these investigations a second thought was given to the proposal of remodelling the existing weir into a gated barrage as an alternative to the construction of a new barrage. Since hollows were found underneath and the condition of the existing structure unsatisfactory to form basis for the remodelled structure it was decided to give up the idea of this alternative at all in favour of new barrage downstream in clear terms.

The new barrage 1115 feet downstream is scheduled for completion by June 1968 and with its coming into operation the existing weir will be redundant. In fact it would be desirable to remove the existing structure altogether except the flanks but on consideration of cost only depression of crest by two feet is provided in the contract for new barrage. Under these circumstances for safety

of weir for four more years of its life till June 1968 it is considered sufficient to:

1. Cement grout foundation under pressure through holes spaced 30 ft. on along crack line about 20 feet from the shutter line.
2. Limit head across to 10 feet for minimum period.
3. Examine weir glacis for cracks every year and take care of these.

Fortunately at Marala there is always some head across the weir even in closure and the resulting uplift pressure supports the glacis. When weir is open the submergence reduces weight of masonry and uplift due to static head and standing wave suction also reduce ground pressures.

It is likely that on other weirs also the sand underneath may be in a loose state due to open pumping during construction or flow through the soil subsequently during operation. The life of these weirs is important and it appears necessary that extensive investigations are carried out and necessary measures to safeguard these works against failure taken.

BIBLIOGRAPHY

1. Report from Swiss Boring Overseas Corporation Limited.
2. Report issued by M/s. Harza Engineering Company International, General Consultants to WAPDA.
3. West Pakistan Engineering Congress Proceedings for the year 1938.
4. West Pakistan Engineering Congress Proceedings for the year 1963—Paper 360 by Mr. H. J. Asar.

Appendix I

Grout Hole Data Weir Bay 6

Hole No.	Location		Water Levels			Head Across	Uplift	ϕ	Cement consumed in CWT
	Distance from shutter line	Distance from Groyne No. 6	Upstream	Down-stream	Bore Hole				
1	20′—10″	262	801.4	797.33	799.33	4.07	2.0	49.14	22.0
2	21′—0″	278	801.4	797.33	799.15	4.07	1.82	44.71	—
3	20′—5″	298	801.4	797.33	799.07	4.07	1.74	42.75	23.0
4	20′—4″	232	801.91	796.94	799.12	4.97	2.18	43.86	4
5	20′—9″	202	801.64	796.90	799.19	4.74	2.29	48.31	4
6	21′—0″	172	801.91	796.94	799.27	4.97	2.33	46.88	23
7	21′—1″	142	801.78	797.08	799.33	4.70	2.25	47.87	17
8	21′—9″	112	801.78	797.08	799.4	4.70	2.32	49.36	7
9	21′—7″	82	801.90	796.99	799.44	4.91	2.45	50.00	14
10	20′—5″	52	801.86	796.79	799.35	5.07	2.56	50.49	18
11	21′—5″	22	801.86	796.79	799.31	5.07	2.52	49.70	7
12	21′—3″	382	801.90	796.82	799.25	5.08	2.43	47.83	21

PLATE I

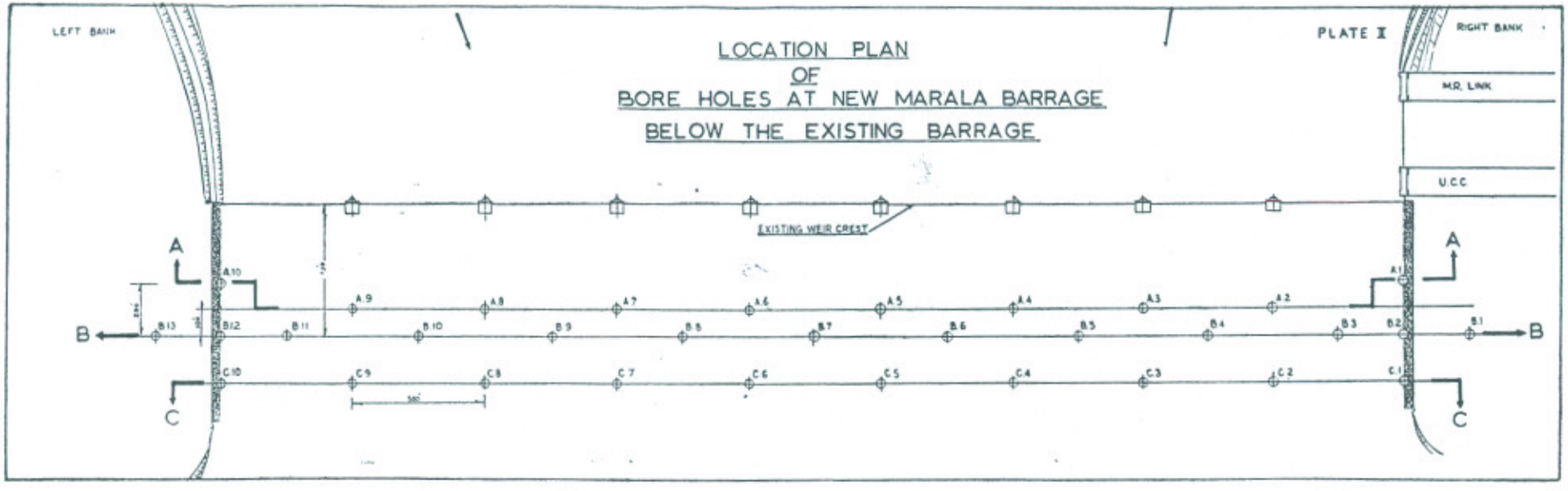
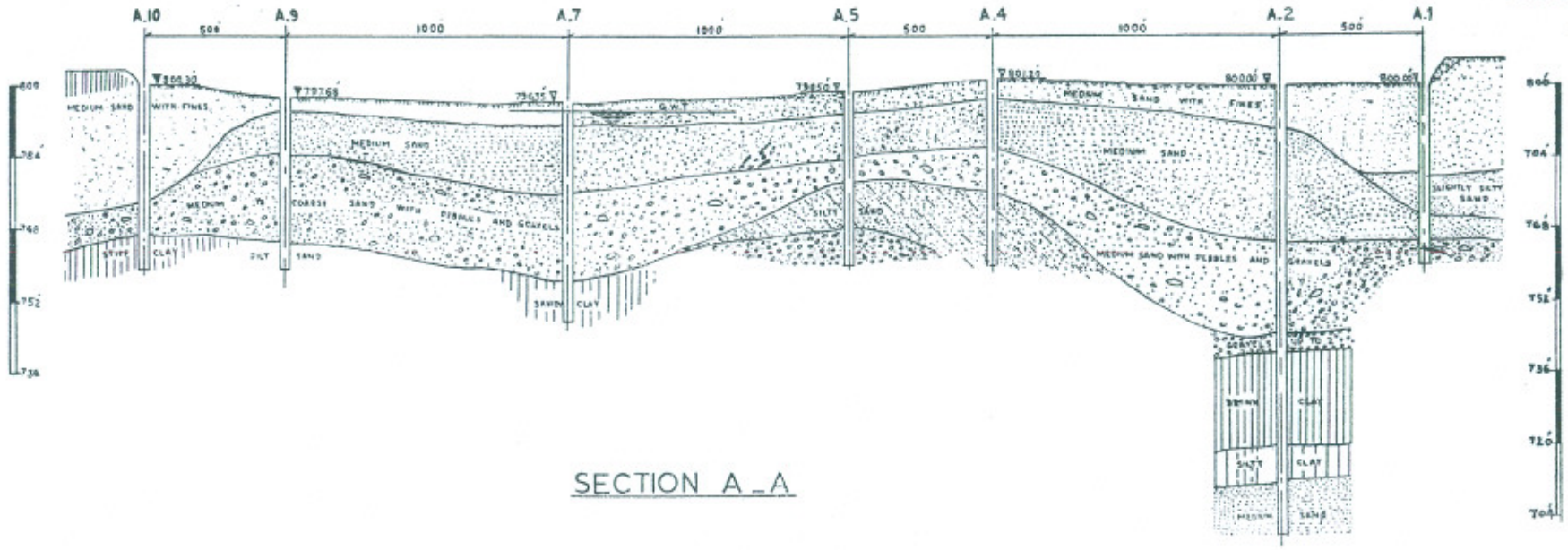
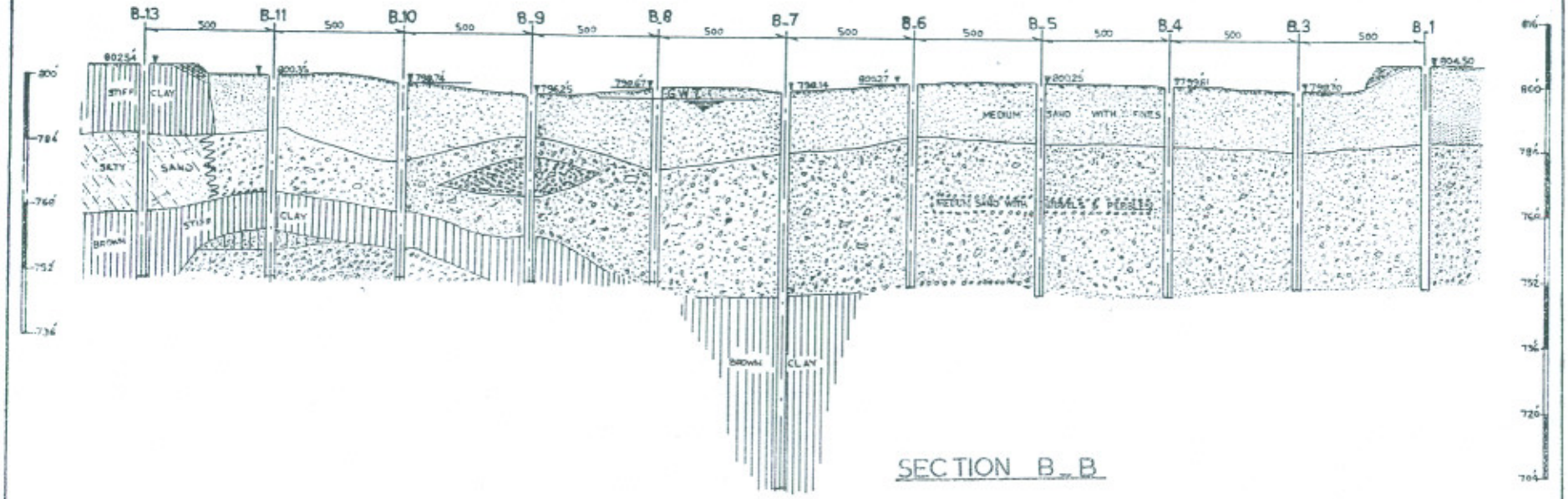

LEFT BANK
RIGHT BANK
M.R. LINK
U.C.C.
LOCATION PLAN
OF
BORE HOLES AT NEW MARALA BARRAGE
BELOW THE EXISTING BARRAGE
EXISTING WEIR CREST
A
B
C
A10
A9
A8
A7
A6
A5
A4
A3
A2
A1
B13
B12
B11
B10
B9
B8
B7
B6
B5
B4
B3
B2
B1
C10
C9
C8
C7
C6
C5
C4
C3
C2
C1
300'

PLATE. III



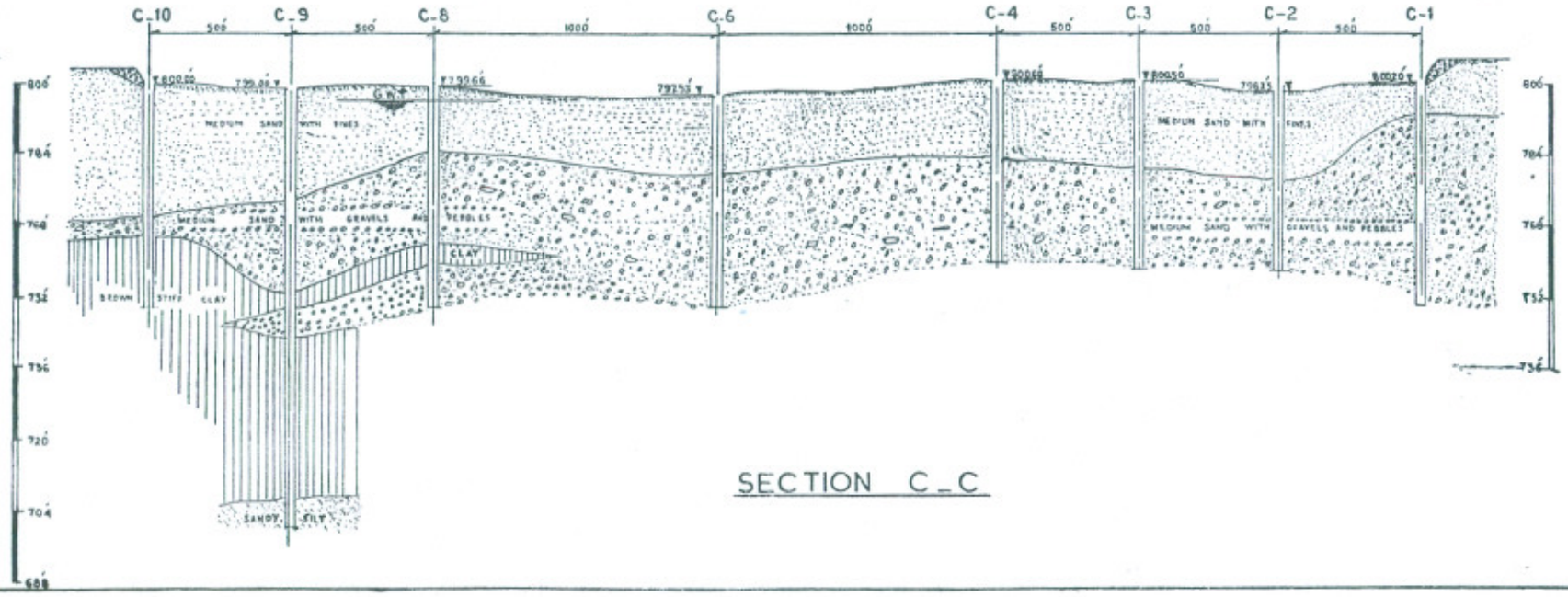
SECTION A_A

PLATE IV



SECTION B - B

PLATE V



SECTION C-C

PLATE VI

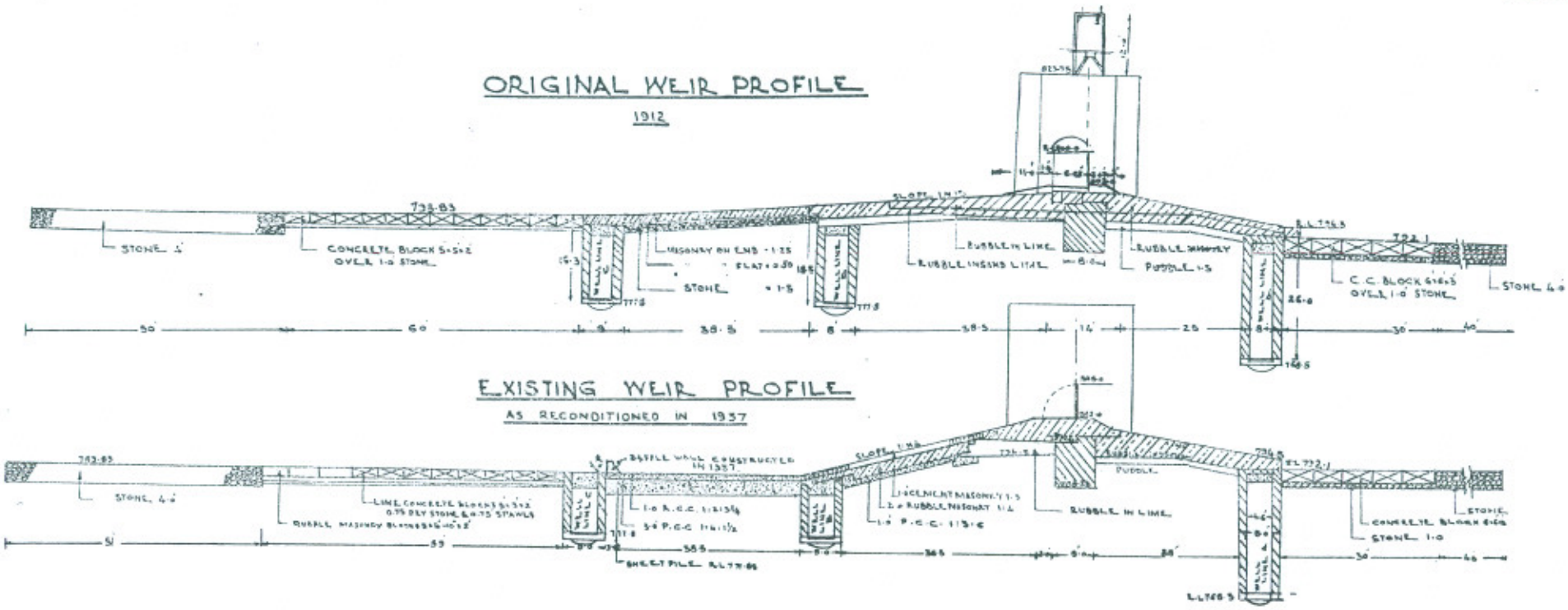

ORIGINAL WEIR PROFILE
1912
STONE 4'
CONCRETE BLOCK 5'×5'×2' OVER 1-0 STONE
MASONRY IN ENDS 1:3
FLAT 0.50
STONE 1:5
RUBBLE IN LIME
RUBBLE MASONRY
PUDDLE 1:5
C.C. BLOCK 6'×6'×5' OVER 1-0 STONE
STONE 4-0
50
60
38·5
8'
14'
25
30
40
EXISTING WEIR PROFILE
AS RECONDITIONED IN 1937
STONE 4-0
LINE CONCRETE BLOCKS 5'×5'×2' 0·75 DRY STONE & 0·75 STAKES
RUBBLE MASONRY BATTERED 1:0·2
BAFFLE WALL CONSTRUCTED IN 1937
1-0 R.C.C. 1:2:3½
3-0 P.C.C. 1:4:7½
SHEET PILE
SLOPE 1:16
FACING MASONRY 1:3
2-0 RUBBLE MASONRY 1:4
1-0 P.C.C. 1:3:6
RUBBLE IN LIME
PUDDLE
CONCRETE BLOCK 6'×6'
STONE 1-0
51
53
58·5
36·5
50
30
40

PLATE VII

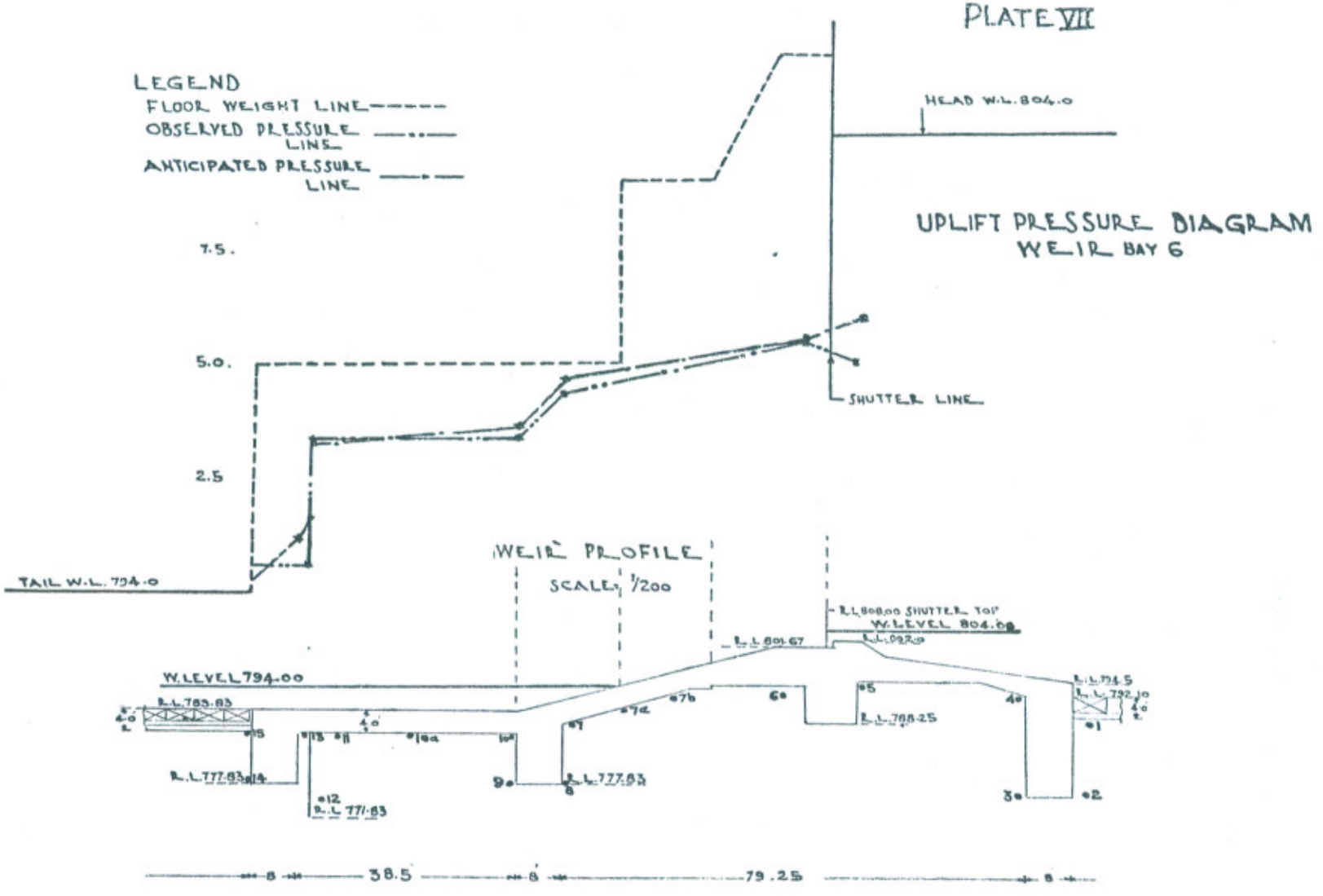

LEGEND
FLOOR WEIGHT LINE
OBSERVED PRESSURE LINE
ANTICIPATED PRESSURE LINE
HEAD W.L. 804.0
UPLIFT PRESSURE DIAGRAM
WEIR BAY 6
7.5.
5.0.
2.5
SHUTTER LINE
TAIL W.L. 794.0
WEIR PROFILE
SCALE 1/200
R.L 808.00 SHUTTER TOP
W.LEVEL 804.00
R.L 801.67
R.L 792.0
W.LEVEL 794.00
R.L 794.5
R.L 732.10
R.L 783.83
R.L 788.25
R.L 777.83
R.L 771.83
R.L 777.83
38.5
79.25

PLATE VIII

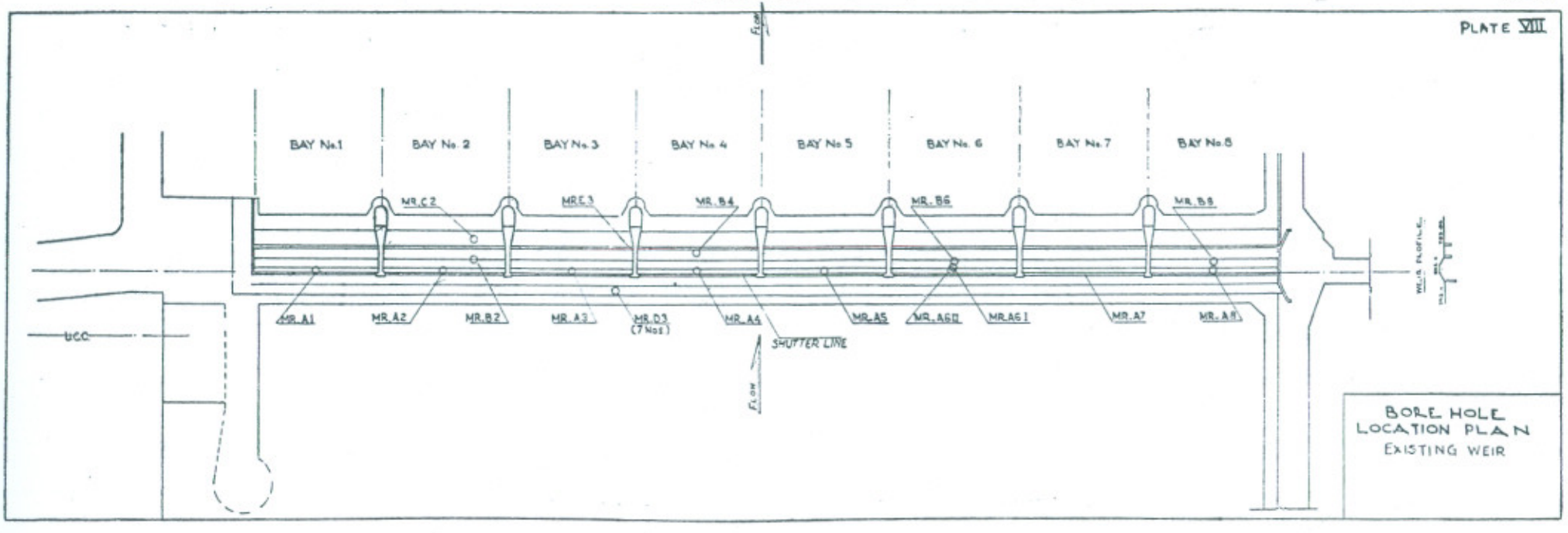

FLOW
BAY No.1
BAY No. 2
BAY No. 3
BAY No. 4
BAY No. 5
BAY No. 6
BAY No. 7
BAY No. 8
MR. C 2
MR. E 3
MR. B 4
MR. B 6
MR. B 8
MR. A1
MR. A2
MR. B 2
MR. A 3
MR. D 3
(7 Nos)
MR. A4
SHUTTER LINE
MR. A5
MR. A6 II
MR. A6 I
MR. A7
MR. A 8
U.C.C.
FLOW
BORE HOLE
LOCATION PLAN
EXISTING WEIR

PLATE IX

BORE HOLE LOGS
EXISTNG WEIR