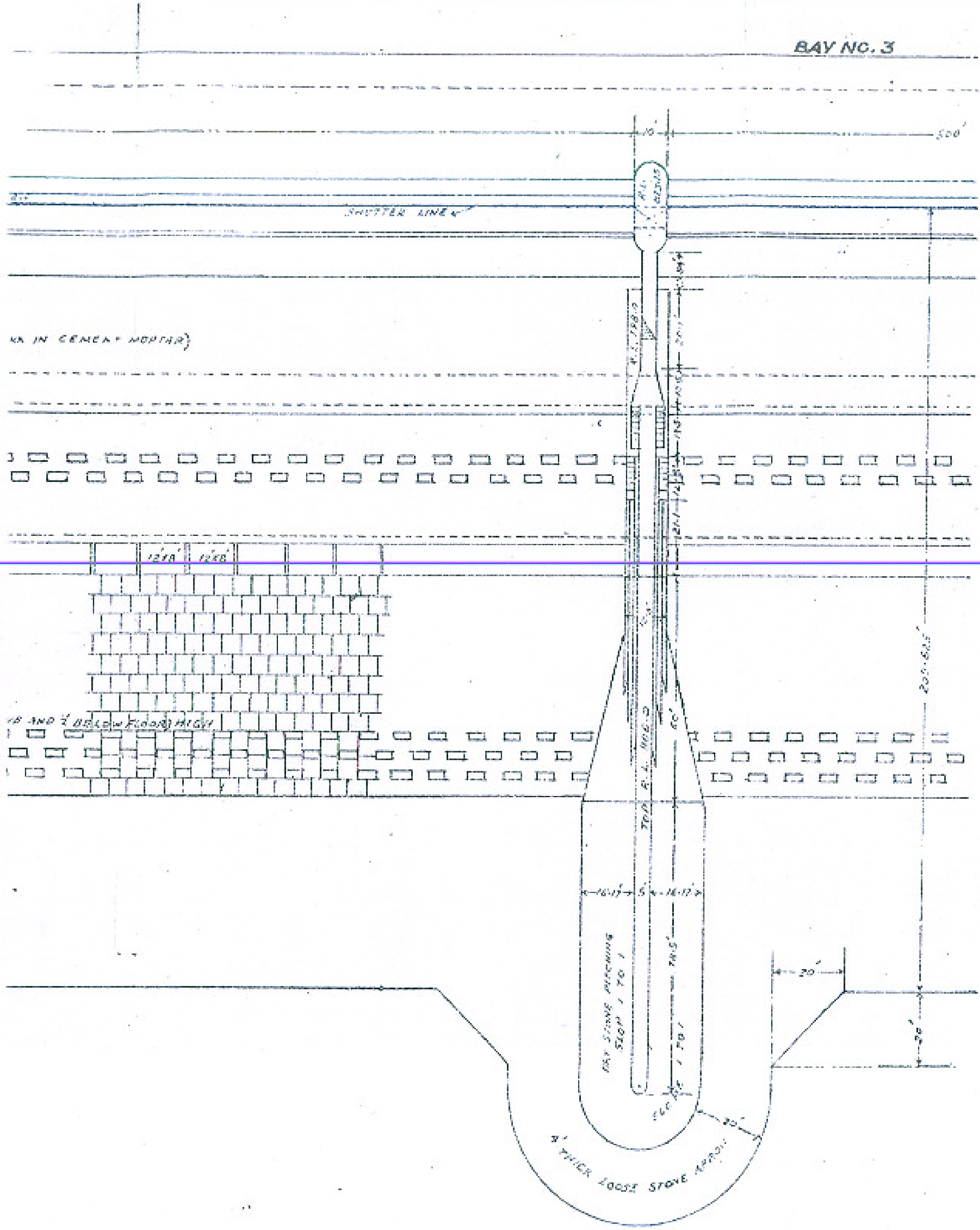


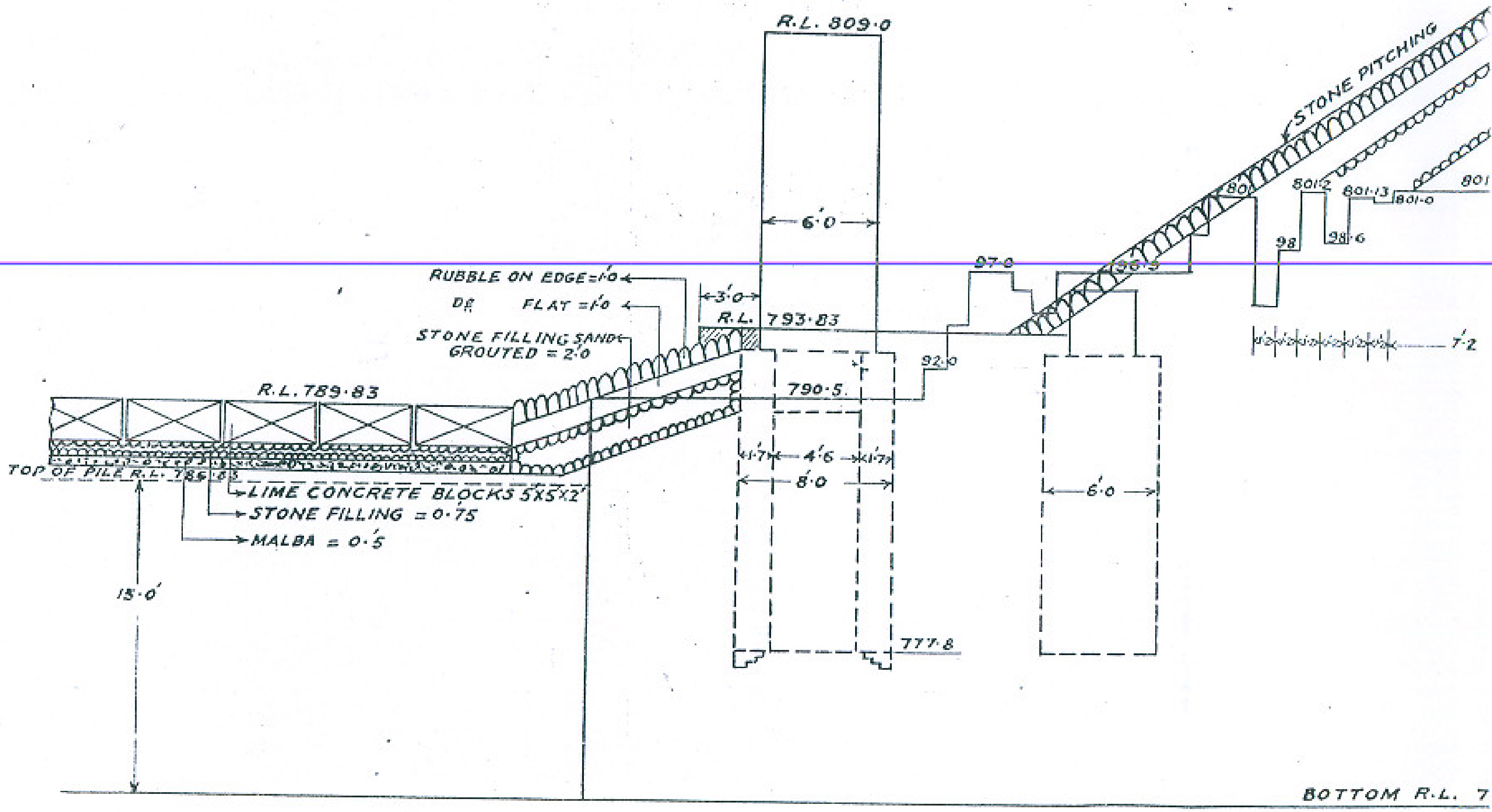
RECONDITIONING OF MARALA WEIR

PLAN OF WEIR (BAY No. 4)

SCALE 1/500

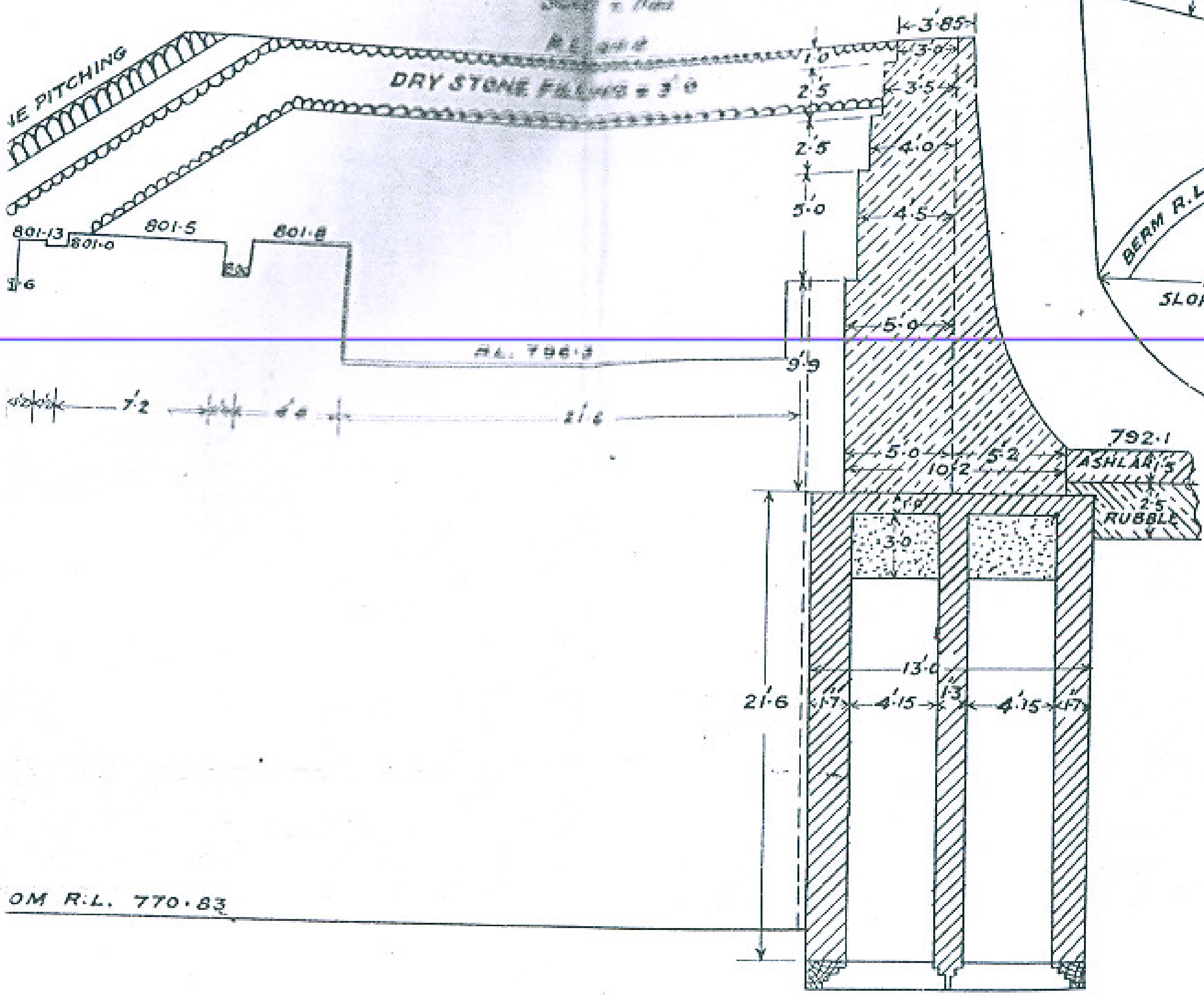
PUNJAB ENGINEERING CONGRESS
1938

RECONDITIONING OF MA



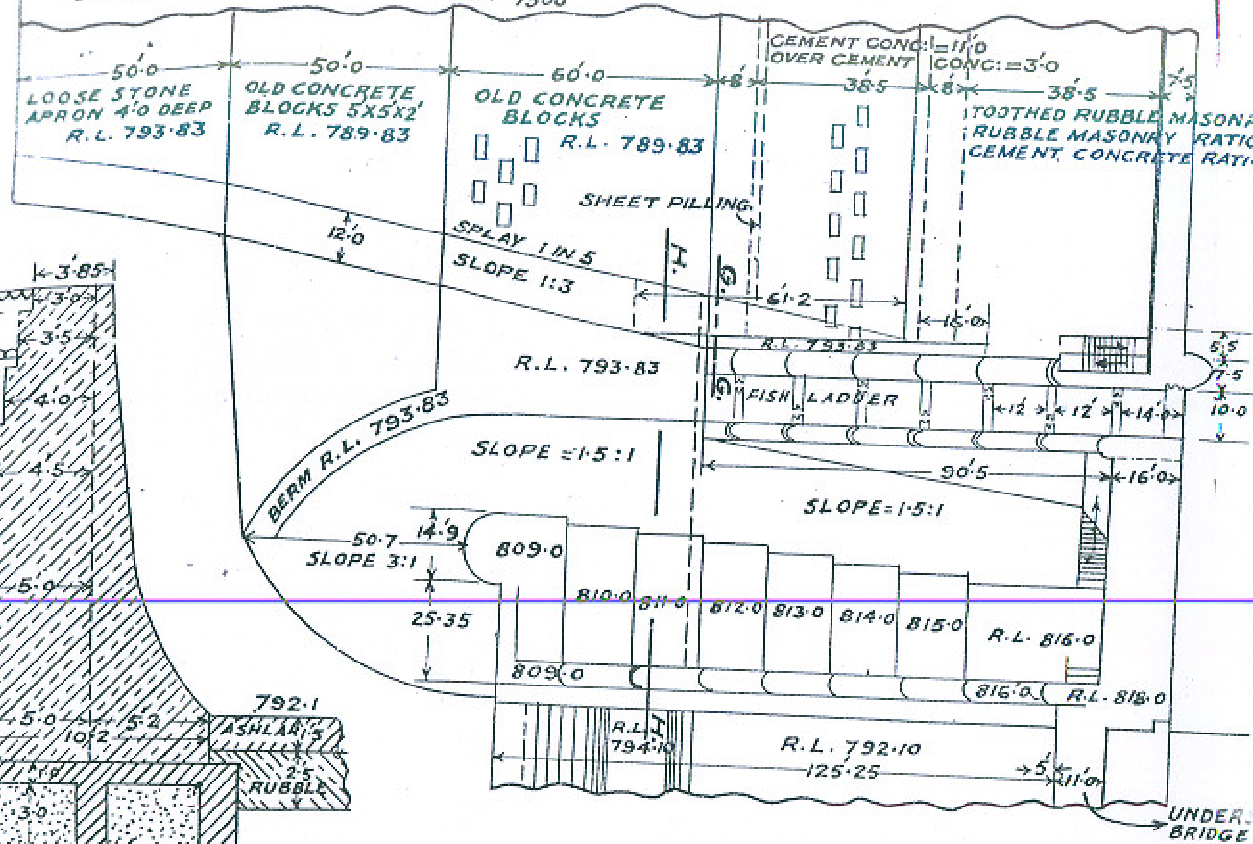
F MARALA WEIR

SECTION ON H.H.



LINKING OF PILE LINE TO THE FOUNDATION WELLS UNDER THE D/S RIGHT FLANK OF THE UNDERSLUICES

SCALE 1/500



DETAIL OF REINFORCEMENT OF

SCALE

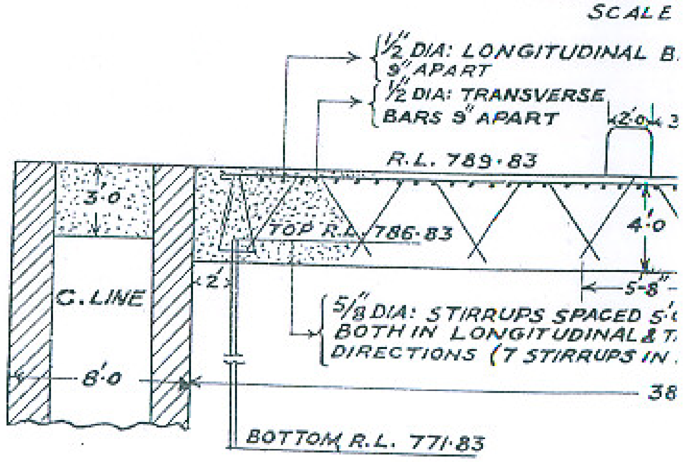
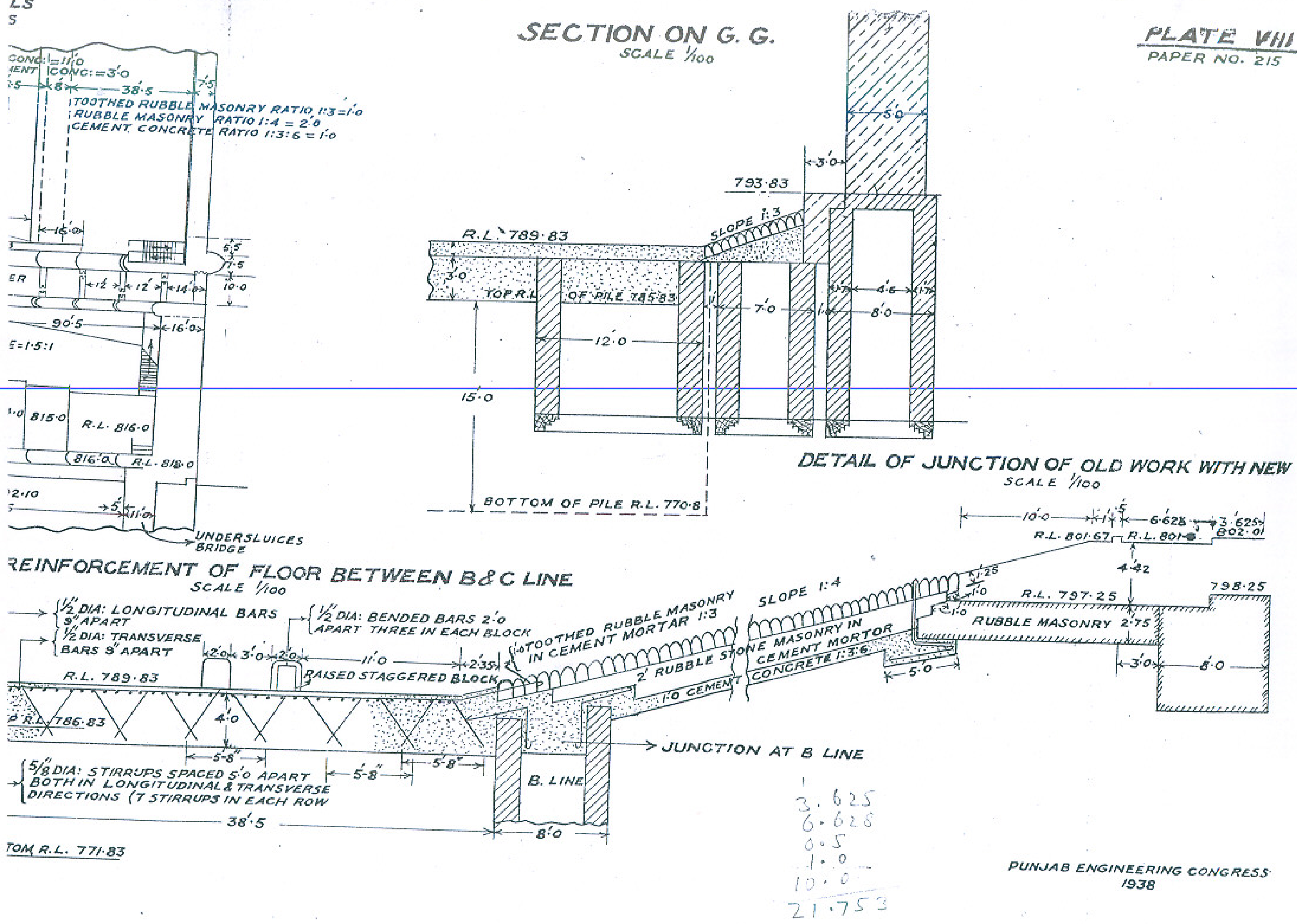
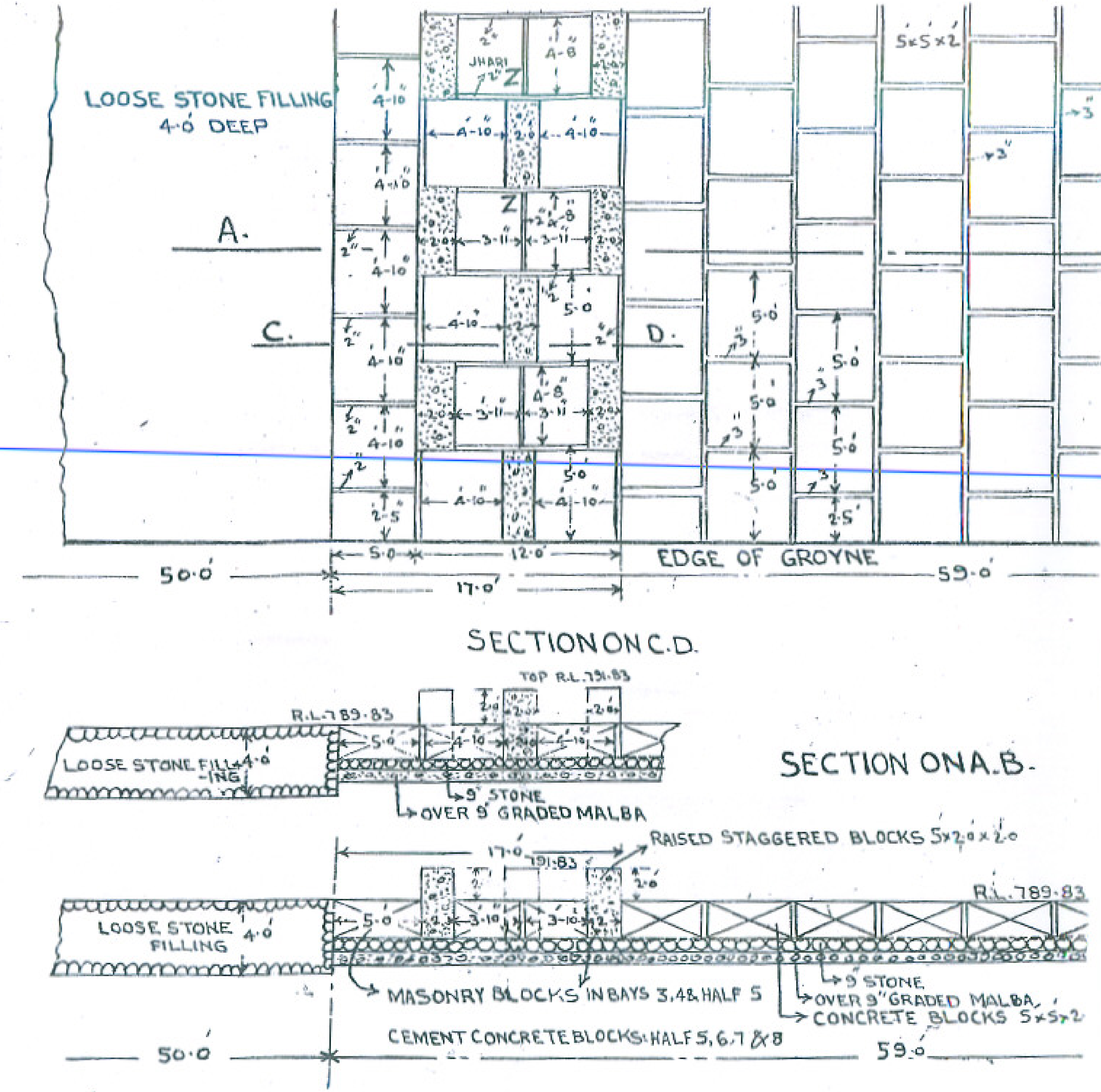


PLATE VIII
PAPER NO. 215





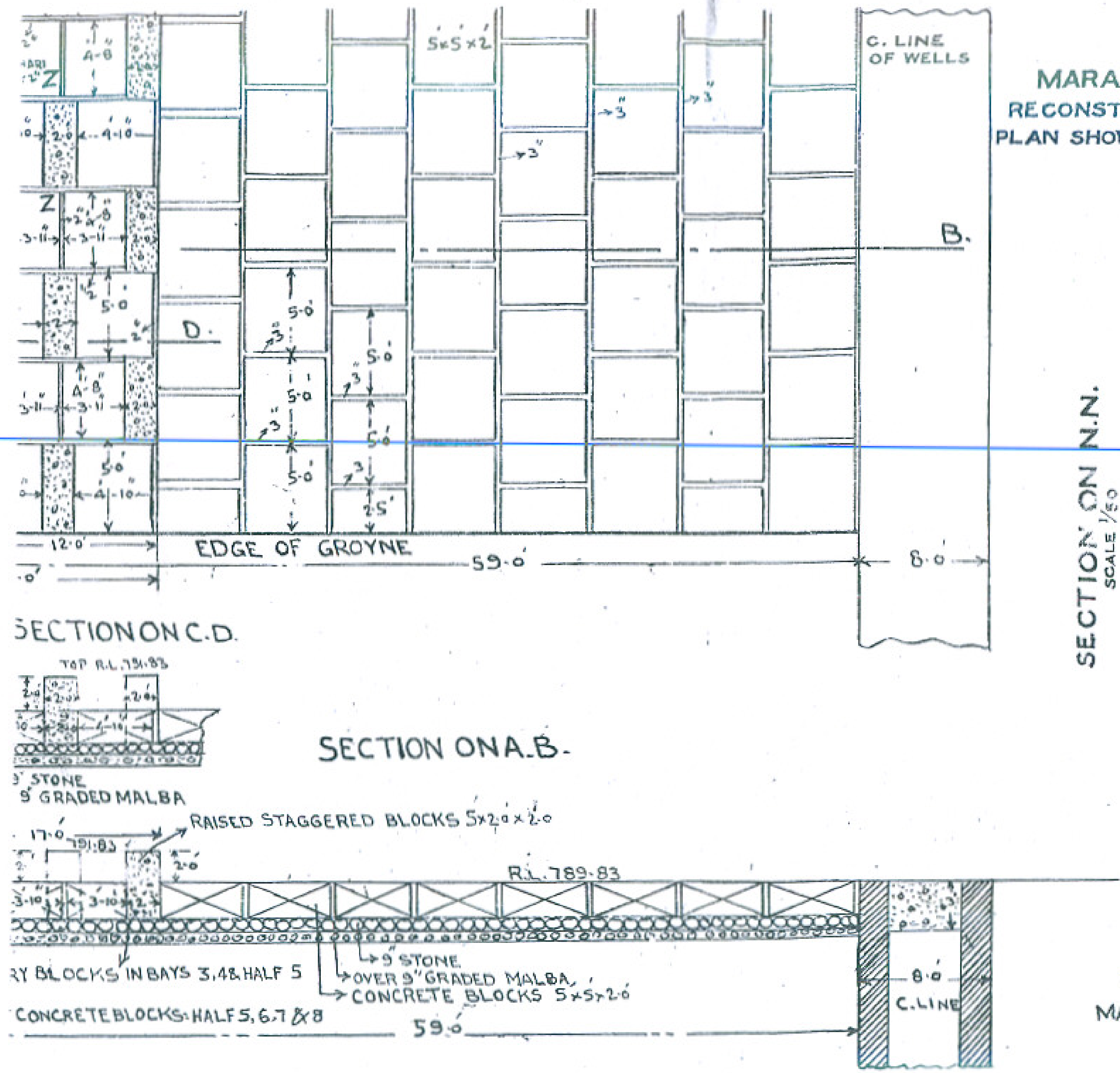
LOOSE STONE FILLING 4·0' DEEP
JHARI
A.
C.
D.
EDGE OF GROYNE
50·0'
17·0'
59·0'
5×5×2
SECTION ON C.D.
TOP R.L.791·83
R.L.789·83
LOOSE STONE FILL-ING
9" STONE
OVER 9" GRADED MALBA
SECTION ON A.B.
RAISED STAGGERED BLOCKS 5×2·0×2·0
R.L.789·83
LOOSE STONE FILLING
MASONRY BLOCKS IN BAYS 3,4 & HALF 5
9" STONE
OVER 9" GRADED MALBA
CONCRETE BLOCKS 5×5×2
CEMENT CONCRETE BLOCKS HALF 5,6,7 & 8

MARALA DIVISION U.C.C.

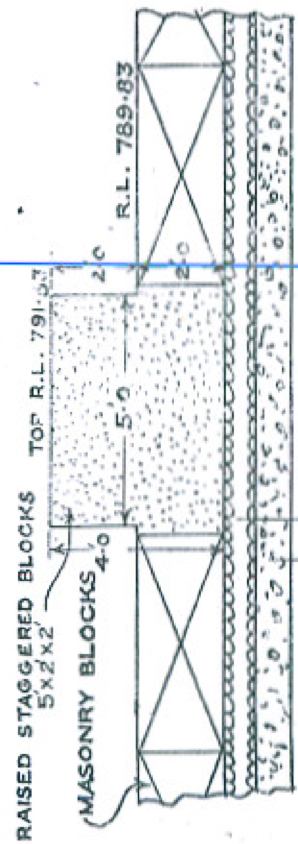
RECONSTRUCTION OF MARALA WEIR

PLAN SHOWING ARRANGEMENT OF BLOCKS

SCALE = 1/100



SECTION ON N.N.
SCALE 1/50



EXECUTIVE ENGINEER
MARALA DIVISION U.C.C.

PUNJAB ENGINEERING CONGRESS
1938

RE-CONDITIONING OF

PUMPING DI

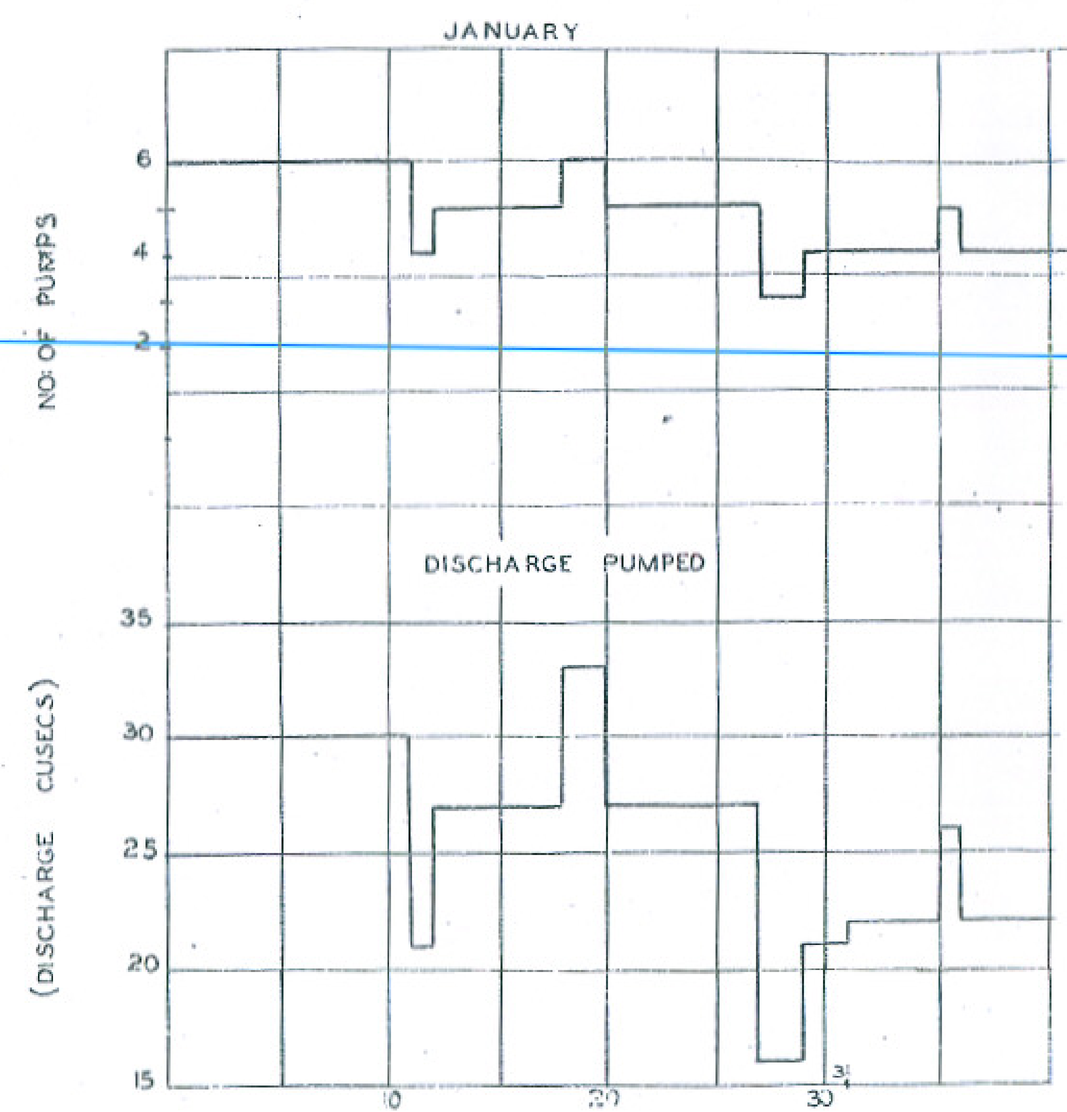
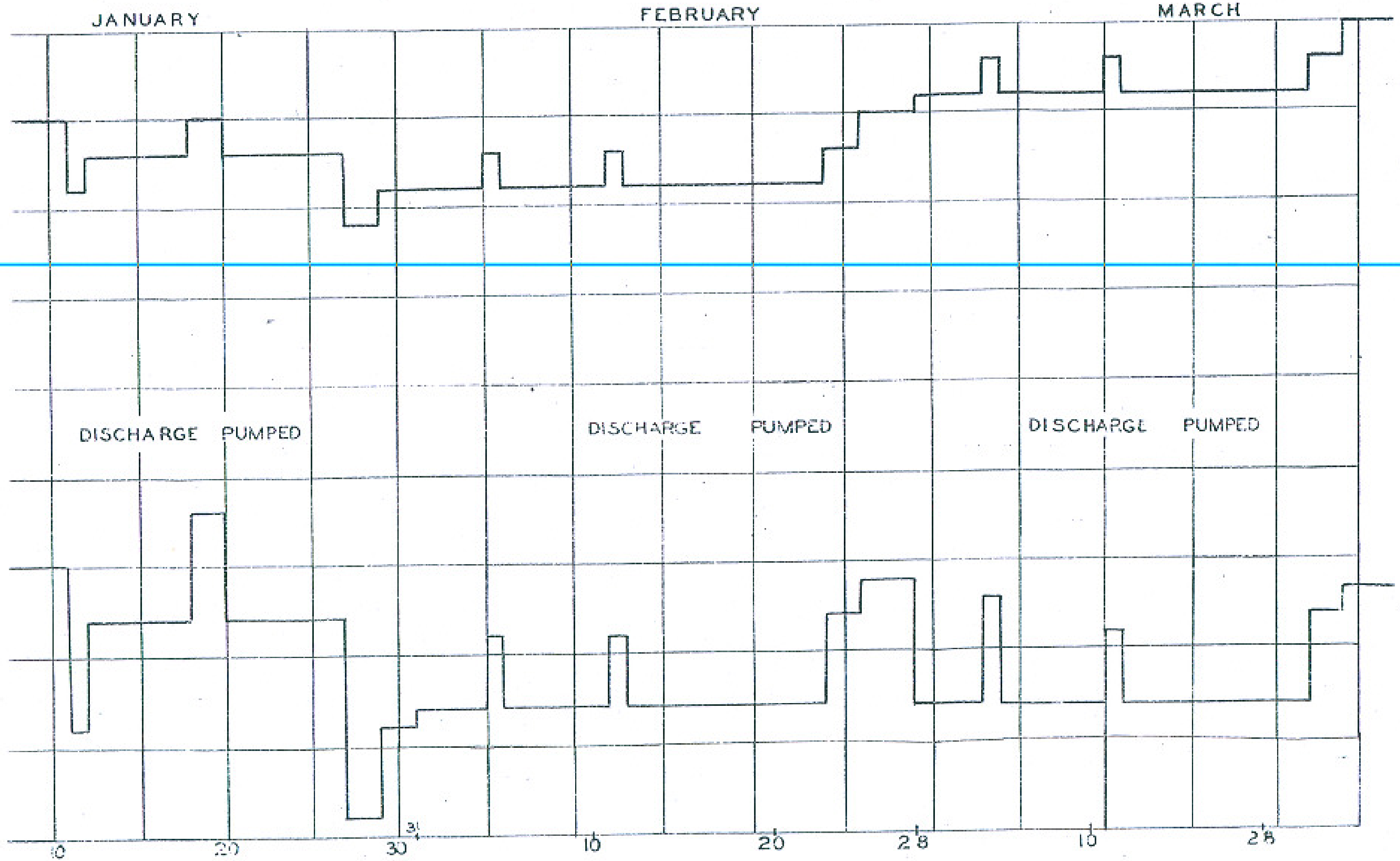
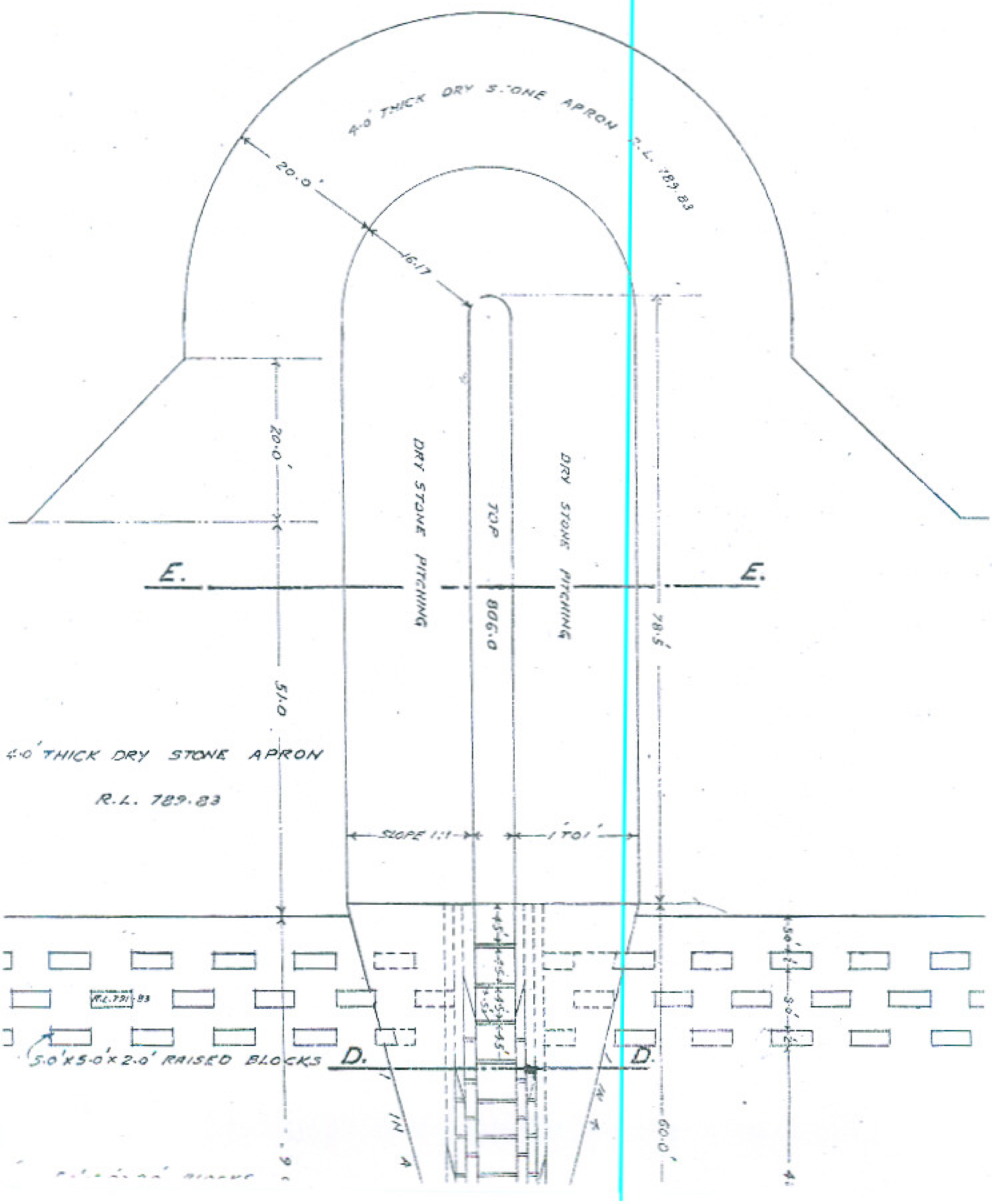


PLATE X
PAPER No. 215

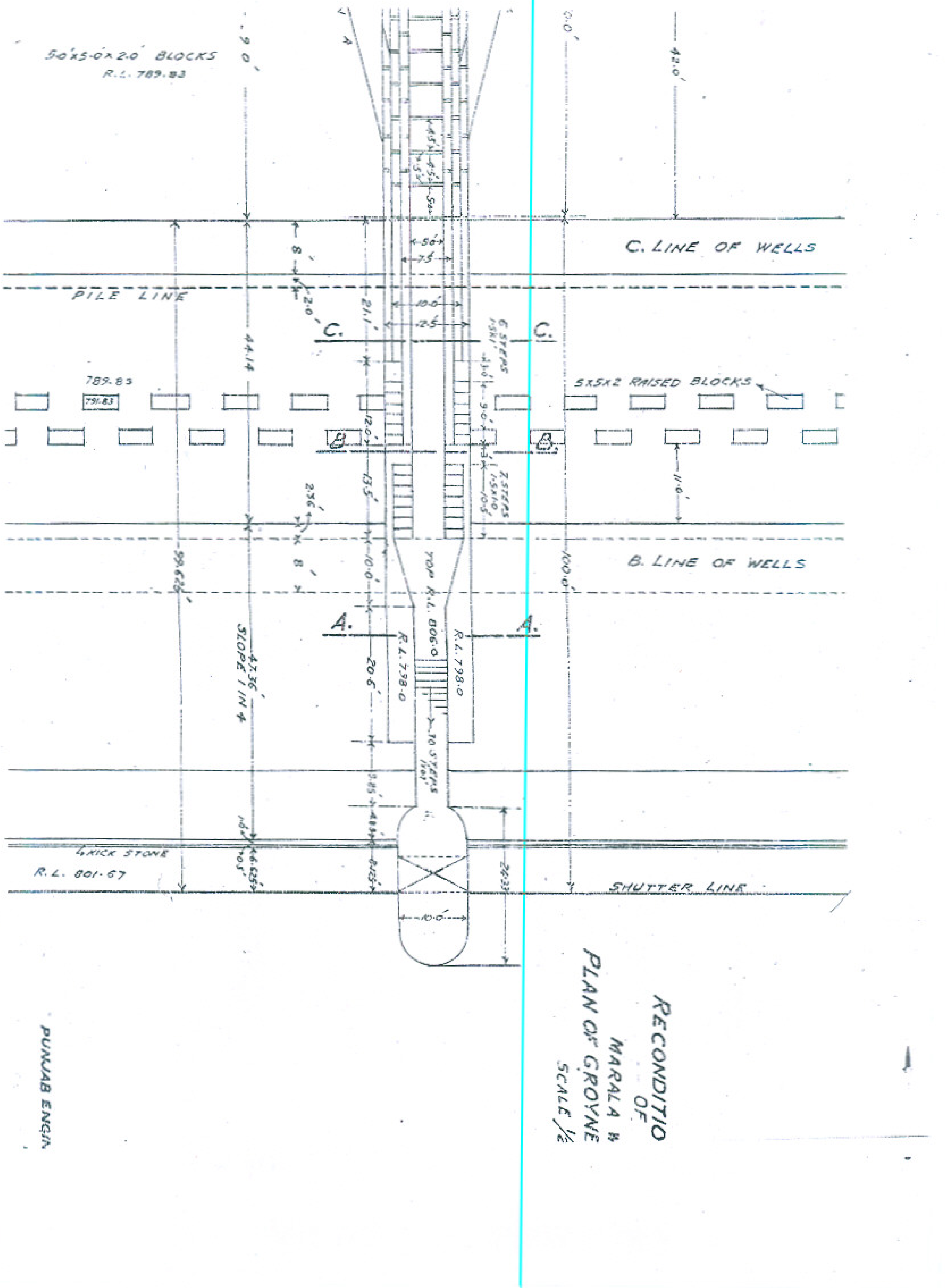
RE-CONDITIONING OF MARALA WEIR

PUMPING DIAGRAM





4·0' THICK DRY STONE APRON R.L. 789·83
20·0'
16.17
20·0'
DRY STONE PITCHING
TOP 806·0
DRY STONE PITCHING
E.
E.
78·5'
51.0
4·0' THICK DRY STONE APRON
R.L. 789·83
SLOPE 1:1
1 TO 1
R.L. 791·83
5·0'x5·0'x2·0' RAISED BLOCKS
D.
D.
60·0'



5.0'x5.0'x2.0' BLOCKS
R.L. 789.83
C. LINE OF WELLS
PILE LINE
C.
C.
789.83
791.83
5X5X2 RAISED BLOCKS
6 STEPS
7 STEPS
B.
B.
B. LINE OF WELLS
TOP R.L. 806.0
R.L. 798.0
R.L. 798.0
A.
A.
SLOPE 1 IN 4
10 STEPS
4 KICK STONE
R.L. 801.67
SHUTTER LINE
RECONDITIO
OF
MARALA H
PLAN OF GROYNE
SCALE 1/2
PUNJAB ENGIN

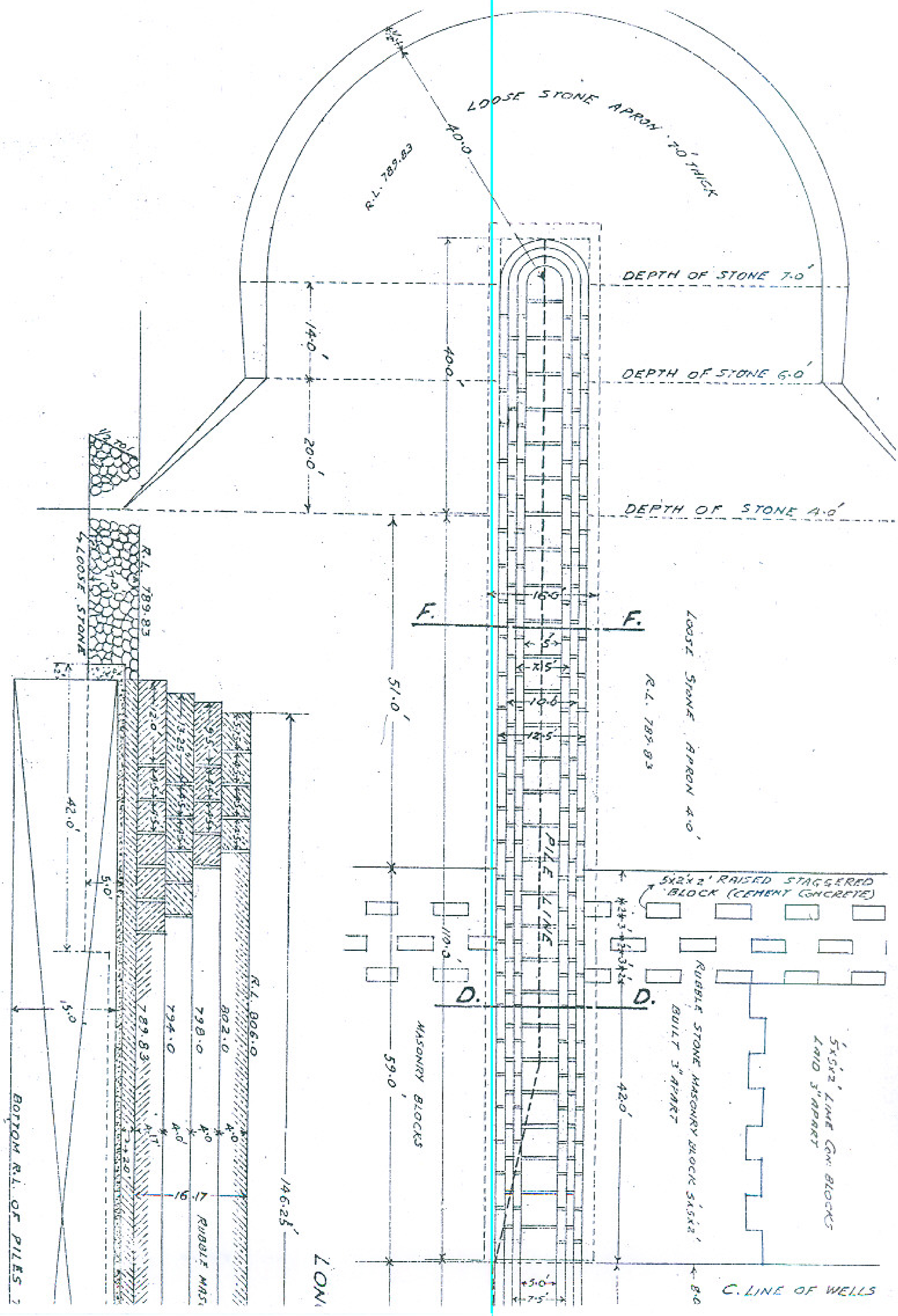

LOOSE STONE APRON 7.0' THICK
R.L. 789.83
40.0
DEPTH OF STONE 7.0'
DEPTH OF STONE 6.0'
DEPTH OF STONE 4.0'
14.0'
20.0'
40.0'
51.0'
110.0'
59.0'
42.0'
146.25'
16.0'
F. F.
D. D.
LOOSE STONE APRON 4.0'
R.L. 789.83
PILE LINE
5'x2'x2' RAISED STAGGERED BLOCK (CEMENT CONCRETE)
RUBBLE STONE MASONRY BLOCK 5'x5'x2' BUILT 3" APART
5'x5'x2' LIME CON: BLOCKS LAID 3" APART
MASONRY BLOCKS
LOOSE STONE
R.L. 789.83
R.L. 806.0
802.0
798.0
794.0
789.83
16.17
RUBBLE MAS
BOTTOM R.L. OF PILES
C. LINE OF WELLS
LON

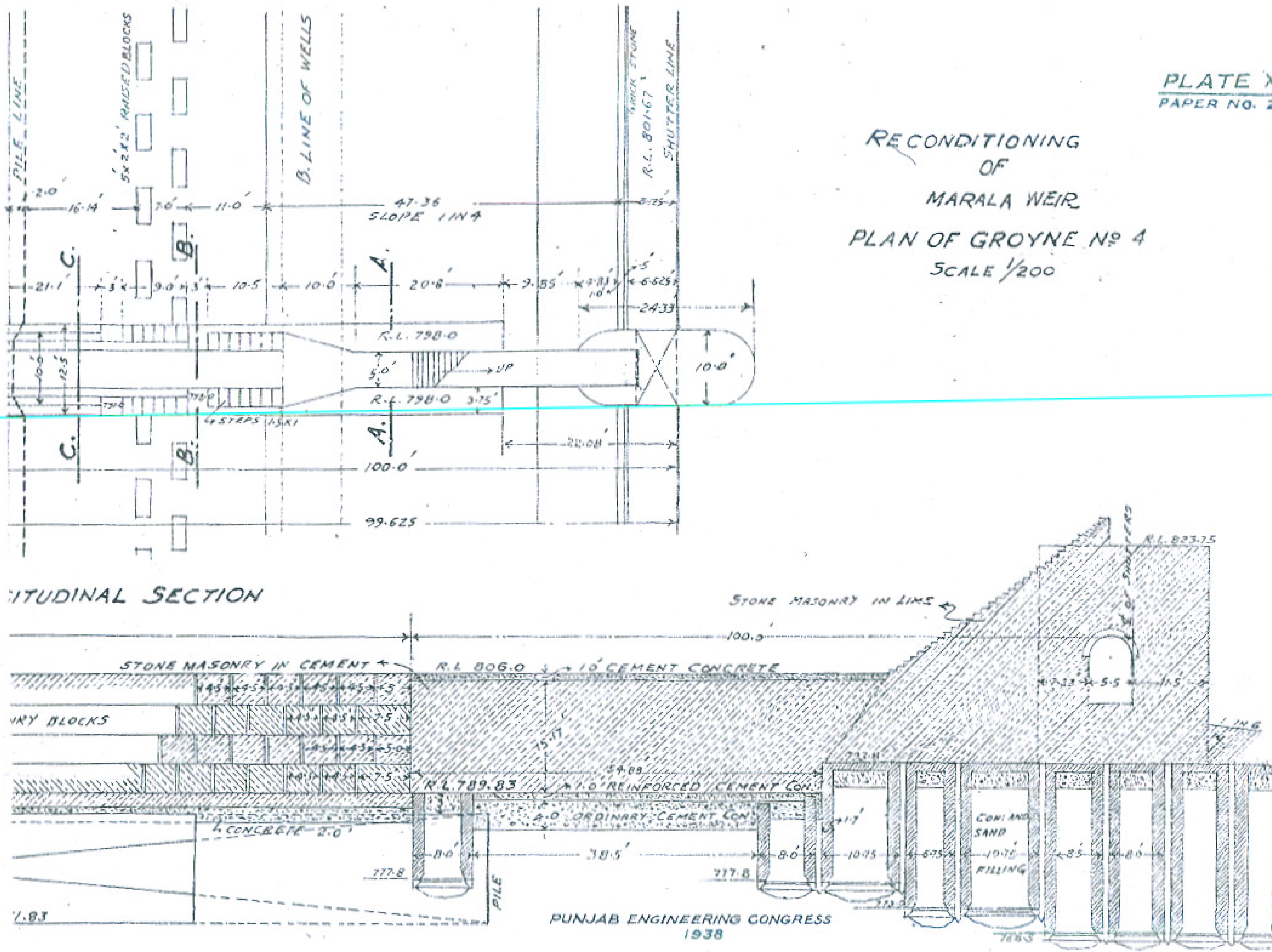
PLATE XII

PAPER No. 215

RECONDITIONING OF MARALA WEIR

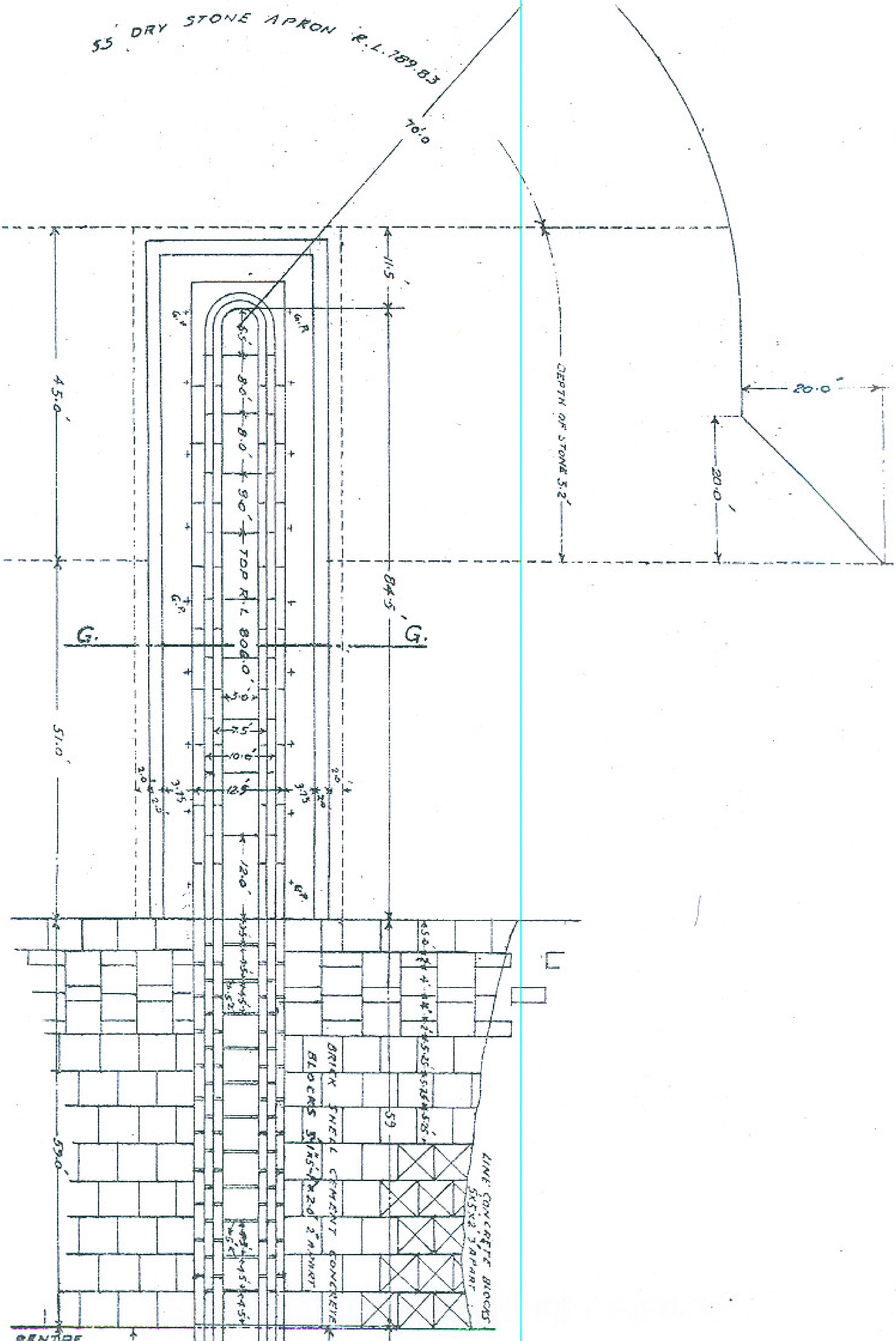
PLAN OF GROYNE No 4

SCALE 1/200



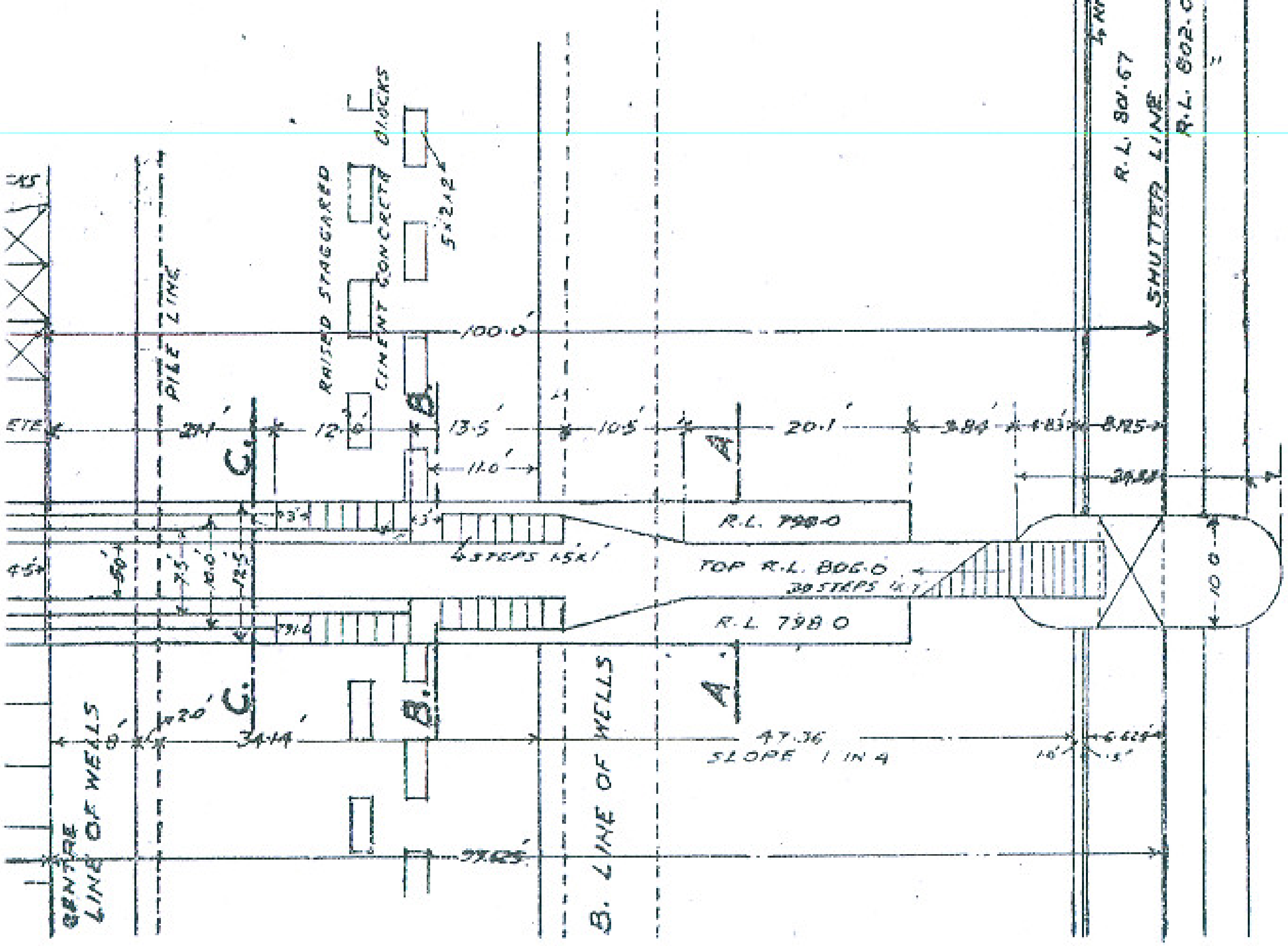
PUNJAB ENGINEERING CONGRESS

1938



5.5' DRY STONE APRON R.L.789.83
70'0
11.5'
45.0'
51.0'
59.0'
84.5'
DEPTH OF STONE 3.2'
20.0'
20.0'
G.
G.
TOP R.L 808.0'
8.0'
8.0'
9.0'
12.0'
7.5'
10.0'
12.9
BRICK SHELL CEMENT CONCRETE BLOCKS
LINE CONCRETE BLOCKS
CENTRE

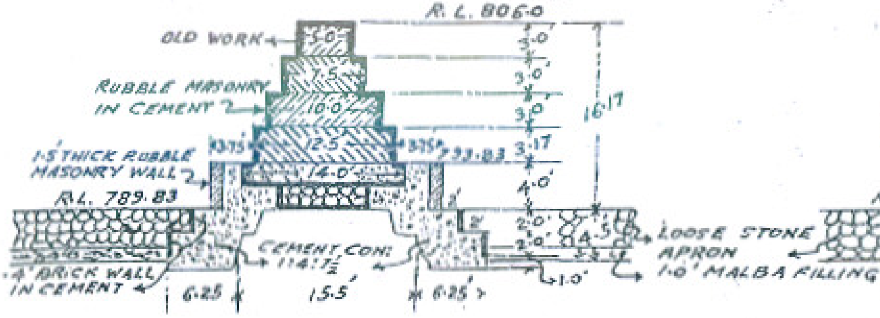
PLATE No XIII
PAPER No 215



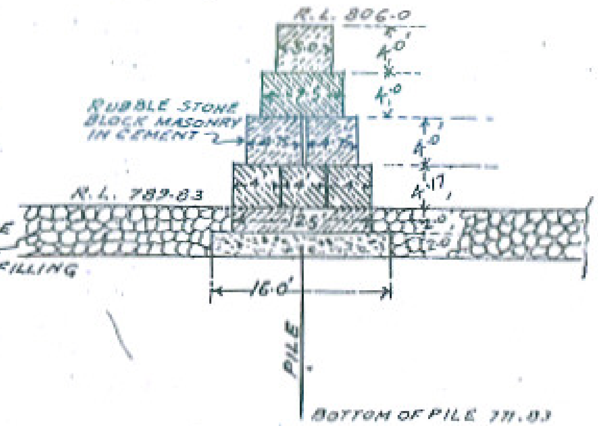
RECONDITIONING
OF
MARALA WEIR
PLAN OF GROYNE No 7.
SCALE 1/200

PUNJAB ENGINEERING CONGRESS
1938

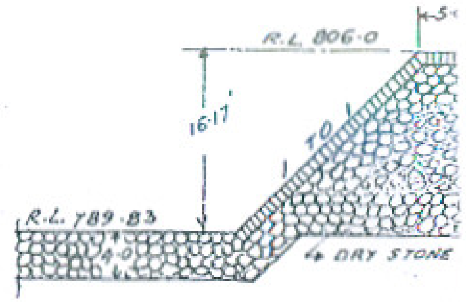
SECTION ON G. G.



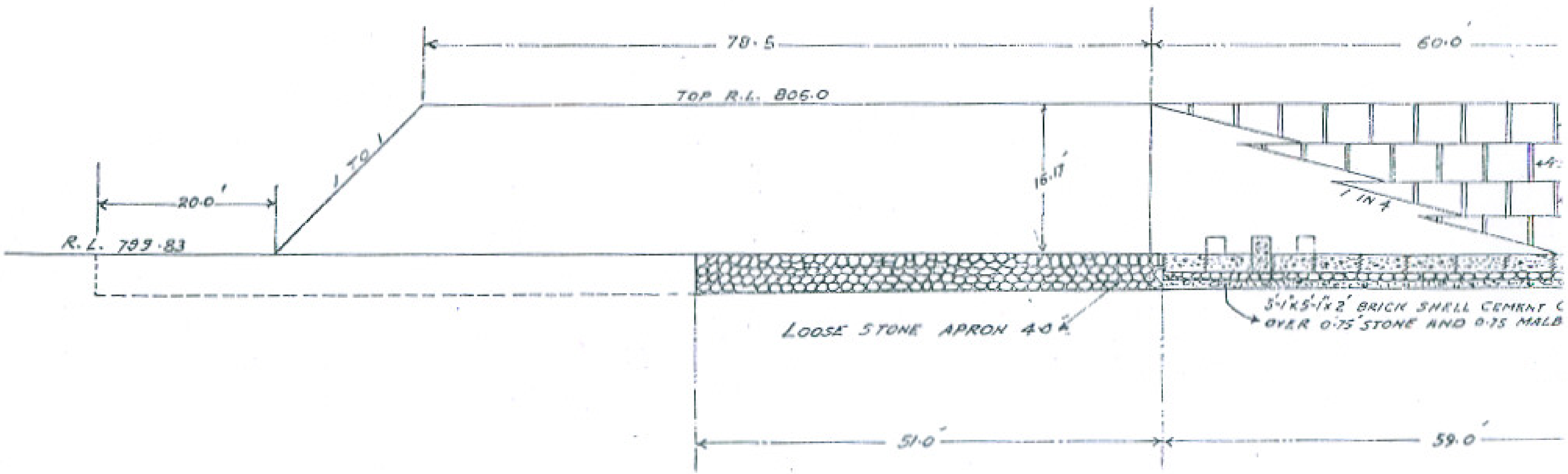
SECTION ON F. F.



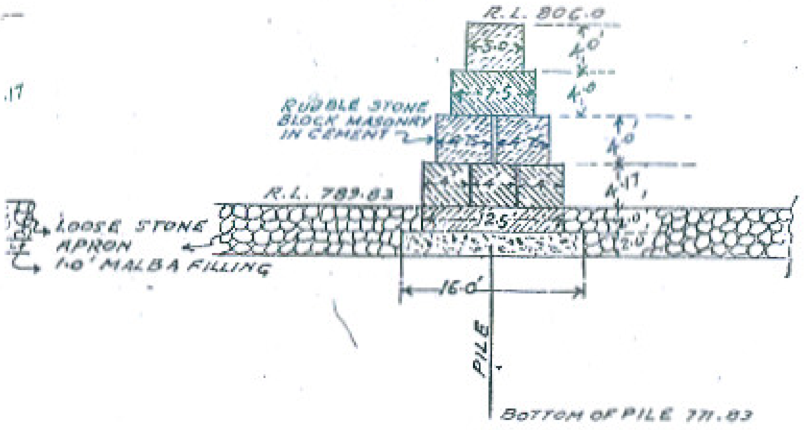
SECTION



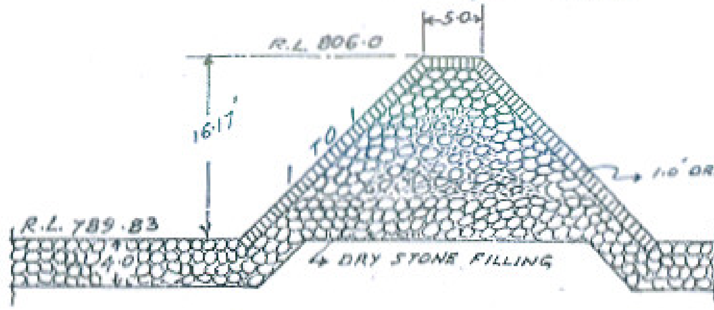
SECTION OF D/S GLACIS



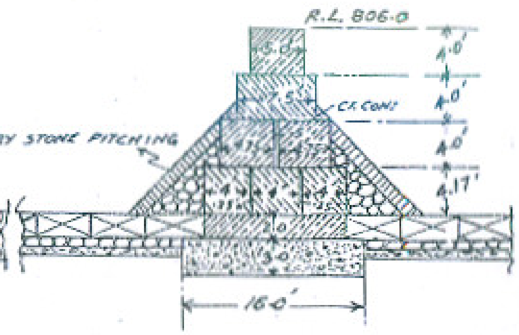
SECTION ON F.F.



SECTION ON E.E.



SECTION ON D.D.



SECTION OF D/S GLACIS & SIDE ELEVATION OF GROYNE

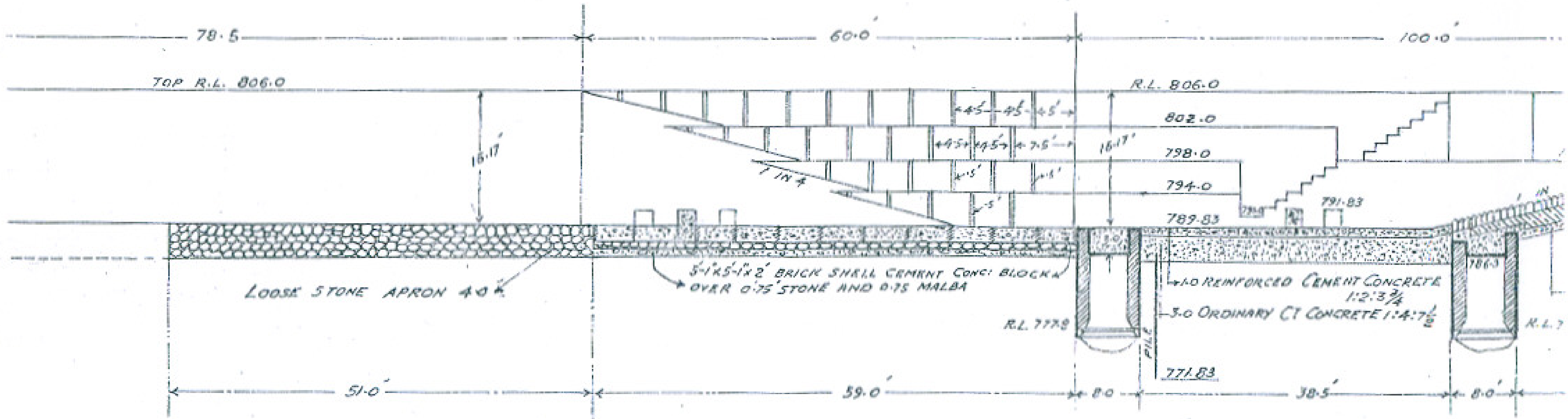
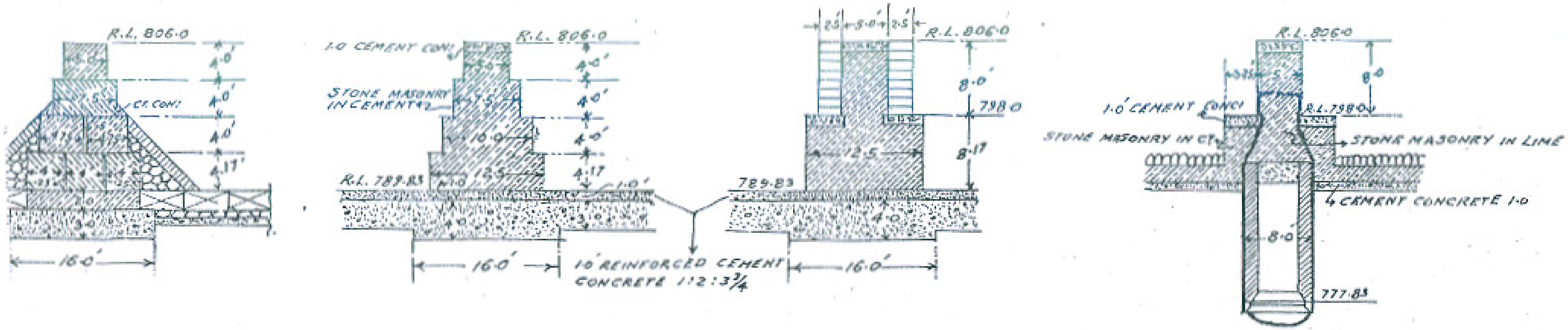
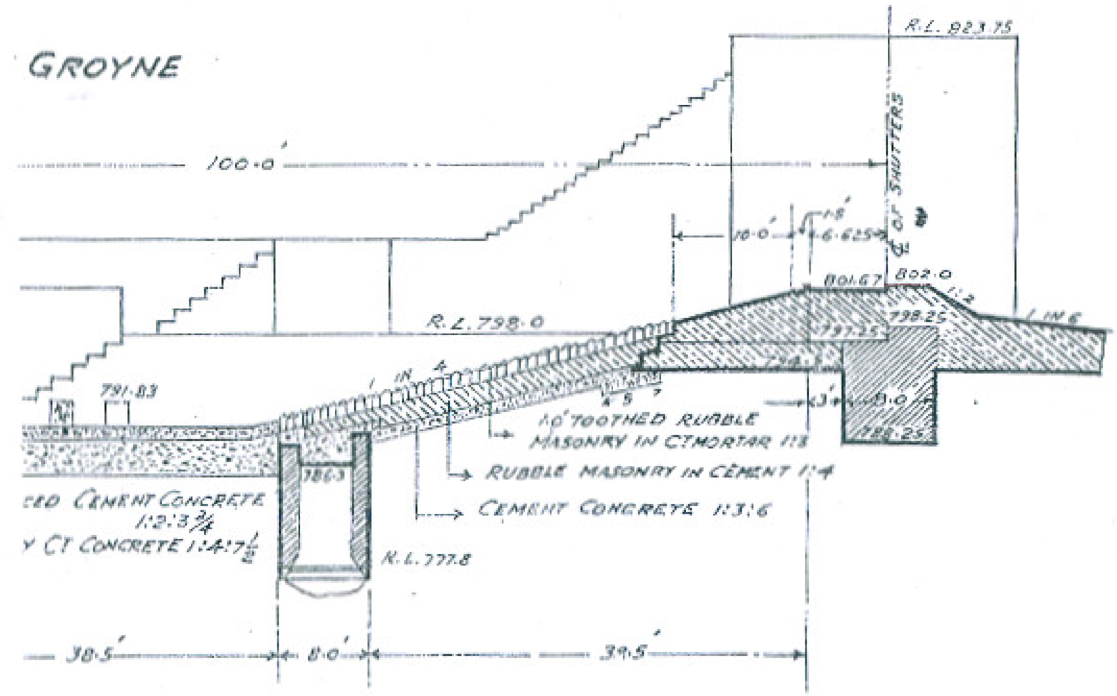


PLATE XIV
PAPER NO. 215

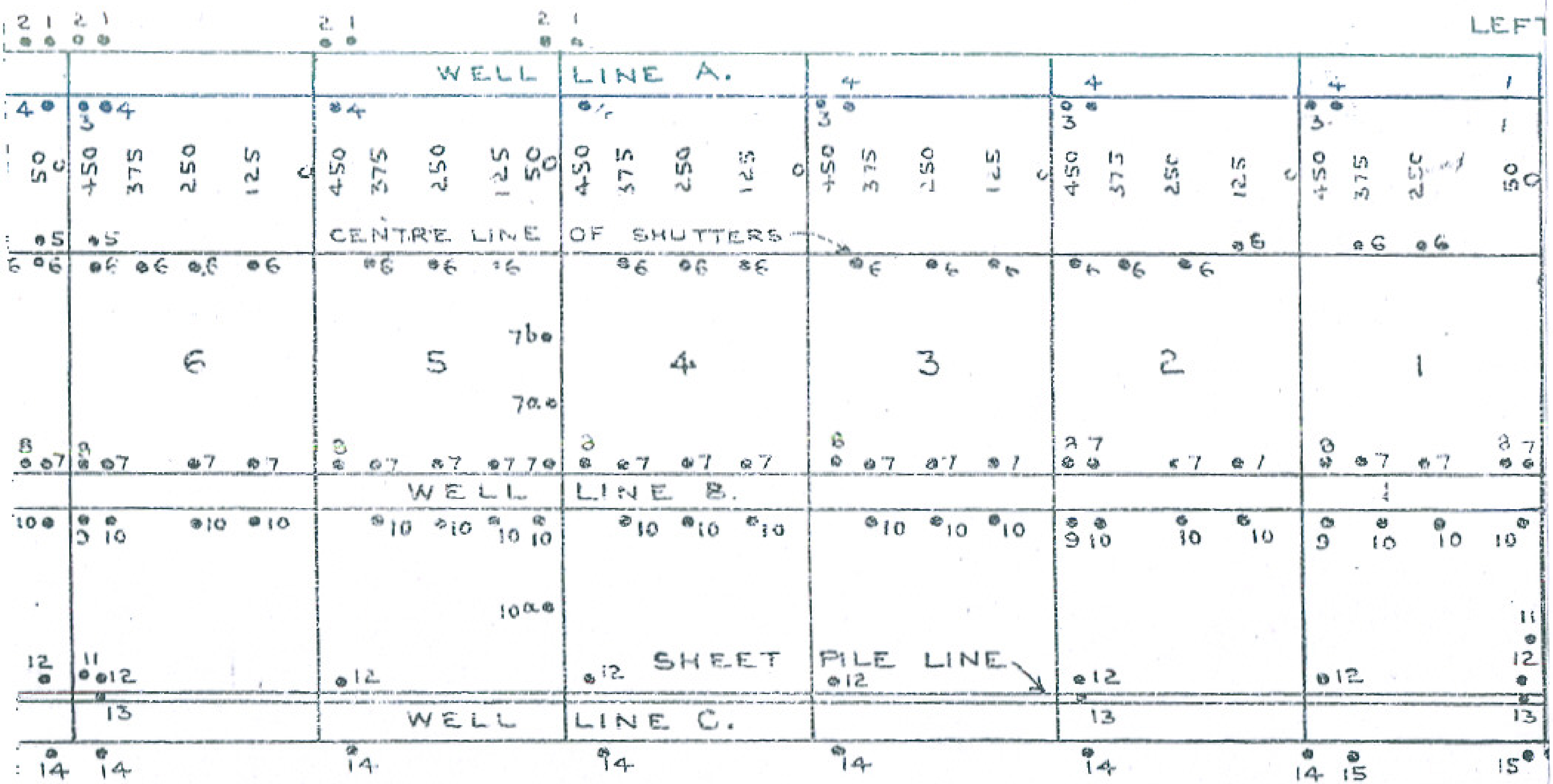




REECONDITIONING
OF
MARALA WEIR
SECTIONS
SCALE 1/200

PUNJAB ENGINEERING CONGRESS
1938

SCALE =1/5000



PUNJAB ENGINEE
193

DISCUSSION.

Mr. **Ganpat Rai** in introducing his Paper No. 215 said that he had not much to add to it.

It had been stated in the last para. on page 172, that although there was a 24-day canal closure during the months of November and December, 1936, the undersluices could not be opened during that period because this would have interfered with the work of remodelling of the head regulator of the Upper Chenab Canal. The result was that high ring bunds had to be constructed for work in bays 1 an 2 of the weir as any water that had then to be passed below the weir had to be escaped over bays 6 to 8. On the other hand, as mentioned on page 45 of Paper No. 210, an unusually high pond level had to be maintained above the weir in October 1936 so as to facilitate the work of ring bund construction in bays 1 and 2 of the weir and this high pond level produced a number of complications in the work of remodelling the head regulator. Thus the execution of the two works in the same working season added to a certain extent to the difficulties of both.

In regard to the pumping arrangements, described on pages 176 to 178, care was taken to see that no sand found its way into the sumps, whether natural or artificial, over which the big pumps were installed. This was secured by proper maintenance of the unwatering drains which led into the sumps. This not only did away with the necessity for carrying out any dredging in the sumps but also eliminated the possibility of the impellers of the pumps being damaged by gritty water.

On page 177, para. 2, mention had been made of the use of a number of siphons to lower the water level upstream of the weir. Mr. Ganpat Rai said that the method used in priming these siphons might be of interest to members of the Congress. Each siphon had a foot-valve fitted at its upstream end and a sluice-valve near its downstream end. At the highest point, *i.e.*, over the crest of the weir, a stand pipe with a stop-cock attachment was fitted to the siphon pipe. After the pipe had been filled with water and the stop-cock closed, all that was necessary was to open the sluice valve when the siphon started working. The sluice-valve also rendered it possible to regulate the discharge through the siphon.

The interesting nature of the work had been brought to the notice of officers of the Irrigation Branch by the Chief Engineer last year. A good many members of the Congress had visited the work as a consequence and the engineers in charge of it had the privilege of showing them round. Mr. Ganpat Rai hoped that these visits and the description of the work given in the Paper would lead to an interesting and instructive discussion.

Mr. **C. L. Handa** remarked that the Joint Authors had given a very lucid description of the work and the reasons why it was imperative that the reconstruction of this important weir should not have been delayed. As Sub-Divisional Officer in charge of the reconstruction work at this weir he wished to speak on a couple of points of practical interest. The first related to the construction of bunds which were certainly the most vital part of the work at Marala. On account of the non-availability of any shoal on the upstream side, which could be linked to the weir by means of donkeys, the entire earthwork of upstream bunds had to be done by boats supplemented by donkeys bringing earth from the downstream side of the weir with the lead ranging from 12 to 30 chains.

In order to gain time it was decided to make a sub-compartment round bays 1 and 2 by linking a cross bund upstream of groyne No. 2. Looking back in retrospect he felt inclined to believe that this cross bund could well have been omitted and one compartment made of bays 1 to 4. This would necessarily have delayed the starting of concreting in bays 1 and 2 but the availability of a larger working area would have more than made up for the slight delay in the start of the concreting. It would also have avoided the struggle that had to be faced in linking the cross bund opposite groyne No. 2 when working in 12 feet of water and against a velocity of over 8 feet per second.

The protection of the bunds against erosion during freshets was a serious problem and presented peculiar difficulty on account of the non-availability of long *pilchhi*. It was therefore decided that the horizontal layer of *pilchhi* which formed part of the revetment might also be laid as a mattress, by the tying together of the individual rolls. By this means the revetment done was quite strong and would not be pulled out by a strong current. Another important feature of the bund was the sand grouting that was done in the stone apron wherever the bunds overlaid freshly built aprons.

In view of the fact that during one construction season at Suleimanke the bunds had given way on account of the heavy percolation through the loose stone, this precaution of sand grouting stone was given the attention it deserved. After each one foot of stone had been laid, a foot of sand was laid over the stone and then washed into the interstices by means of water jets ; and then the next layer of stone was laid. This involved a lot of labour, expense and time, but was worth while as any ungrouted stone in the apron would not only have admitted too much seepage but would also have risked the safety of the bunds and thus jeopardized the entire work.

The other item about which he wished to say a few words was the progress in concreting. On a work of this nature 31,500 cubic feet of concrete in one day constituted a record. This was obtained with 5 cubic yard mixers and one ¼ cubic yard mixer. Such heavy progress (sustained for well nigh a month) was solely due to the organization with which dismantling, sheet piling, building of blocks and laying of reinforcement were regulated. On the reconstruction of an old weir, the progress generally suffered from lack of room. At Marala, it was arranged that the maximum ouput of mixers should be absorbed and thus make possible the quick completion of the season's programme.

The mixers kept on turning out their peak progress within a week of the final completion. This was a feature which was not generally met with. A labour strength of 4000 in March dwindled to a few hundred early in April.

It had to be stated that the pumping operations done with the old steam engines were most satisfactory. In particular the pumps in the right group were so installed that the water level was depressed 3 or 4 feet below the foundation level and concreting was done almost under dry conditions. The Author had mentioned the programme of work drawn out at the beginning of the construction (Plate V). This programme was jointly drawn out by S. B. Raghbir Singh and the Speaker and supported by the Executive Engineer and accepted by the Superintending Engineer. It was remarkable that the final dates of completion worked very nearly to programme, except for a lag of about a fortnight, as far more concreting had to be done in bays 6, 7 and 8 due to the block protection having been found in a far worse condition than presumed.

It had, however, to be stated that the labour that had to be arranged had to be 50 per cent more than estimated, for the following reasons :—

1. The progress on earthwork was adversely affected on account of long lead.

2. The dismantling of cement concrete proved very difficult.

3. The quantity of work carried out under "dismantling" and "concrete" went up from 16 lacs to 36 lacs and from 8 lacs to 11·75 lacs respectively.

Mr. **Montagu** offered his congratulations to Mr. Cox. on the successful completion of a piece of work which reflected the greatest credit on every one concerned, from the Superintending Engineer himself, downwards. Further, he took leave to congratulate Mr. Cox and Mr. Ganpat Rai personally, on the presentation of a Paper which would be of lasting value as a guide to officers who might be employed in future on work of a similar nature.

The Speaker had read the Paper with the greatest interest, and in one respect only had failed to find complete information. On page 167, Mr. Cox recorded the decision to lower the floor downstream by 4 feet, the figure being fixed by the similarity of conditions at Khanki.

The Speaker enquired what method had been adopted for checking the position of the standing wave. He had himself tried to do so, as follows.

Maximum flood discharge from page 138, 170 cusecs per ft. run

H (in formula $q=3{\cdot}1\ H^{3/2}$). 14·4 feet

R. L. of "still water" upstream 816·4

R. L. of surface downstream from page 158 and Plate II 810·43

$H_L=5{\cdot}97$

$q=170$

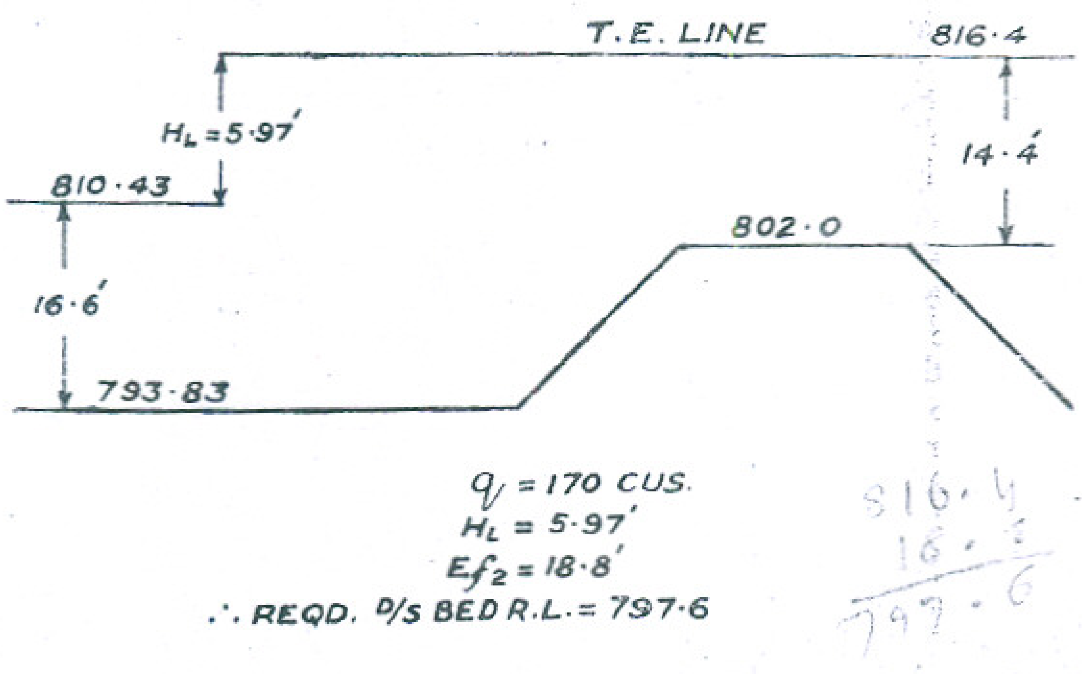
From Mr. Blench's diagrams for "clear" trough, the value of $E_{f_2}=18{\cdot}8$ and R.L. of bottom of standing wave R. L. 797·6

It would appear to the Speaker that in the absence of other information, it was unnecessary to lower the downstream floor at all.

It would be noted that friction had been neglected. The standing wave would form higher up the glacis. Also, a clear trough had been postulated instead of a full trough as advocated by Mr. Blench. This made not more than 2 feet of difference. Still, the old floor was low enough, the Speaker thought.

The point not examined in the above analysis was, whether the standing wave at maximum flood was the worst. There were not sufficient data in the Paper to determine this, but the Speaker suggested it should have been done. The calculations only took a few seconds each and by neglecting friction, were *always* safe.

The Speaker noted that the friction blocks were all in the sub-critical stream at all states of discharge but the lowest, and consequently would fulfill their proper function.



Mr. **Blench** commented that the Author's frankness on our general ignorance of physical conditions and priciples behind weir design indicated not only that model experiments were necessary but that they should be carried out and interpreted with great caution, by men acquainted with practical conditions and the dynamics of the subject. So great was the danger of inexpert use of models that Mr. C. C. Inglis, Director of the Central Research Station at Poona, had written a Paper entitled "The Use of Models for Elucidating Flow Problems" and contributed it to the symposium to be held by the Indian Science Congress that same year. He brought out very clearly that the different causes underlying hydraulic behaviour required different scale distortions to get them in their right perspective; and, even in simple problems, pure geometric similarity was not a criterion of dynamical similarity.

To illustrate the need for practical knowledge, fig. 1 (1) gave diagramatically, a weir in low river conditions, with no upstream or downstream cut-offs—fig. 1 (2) showed the same weir with flood conditions, when the bed of the river had scoured (apparently the designers of such works as Islam were unaware that this scour would occur). The result of the scour was obvious, but an experimenter who was unaware that it could occur would probably make a model like fig. 1 (3) and consider the weir safe if piping did not occur for unscoured conditions. Fig. 1 (4) showed a weir in flood with scoured bed, but with upstream and downstream pile lines. Obviously mere static experiments based on fig. 1 (3) were inadequate and might mislead. Thus fig. 1 (5) showed to a larger scale the downstream pile line in flood. If *ac* were a portion of floor, laden with water, and *ad* the slope to which the downstream bed adjusted itself (assuming the loose pitching did not settle vertically as it often did in practice, making matters even worse) then as the sand to the right of pile line *ab* was loaded from above, by water *plus* a heavy weir floor, and had a density greater than that of water (combined with a fluidity that was far from negligible), it followed that the pressure at the level of *b* to the right of *b* was very much greater than that at the level of *b* to the left where the imposed load was mainly due to water only. Therefore there was every likelihood of the sand to the right of *b* flowing along the arrow, round the bottom of the pile line. This meant that cavities would form under the floor, and be attributed wrongly to "piping". If the pile line was not perfect, *e.g.* if occasional piles had been cut off short because they fouled a boulder and refused to go further, the danger was accentuated.

A further point was that the sand (or earth-sand mix) under the weir was subject to alternations of pressure and wetness during the course of years, and, under the standing wave, to vibration. Little was known of the physics of wet earths; but we did know that patting apparently slightly moist and coherent sand would make it sloppy, and vibrating it might make it settle. It seemed, therefore, that satisfactory weir model tests should not confine themsevles to unscoured static conditions, but should find what happened in scoured bed conditions, (*a*) for fixed difference of water-level, (*b*) for repeated differences and (*c*) for slight vibration of the downstream floor. The scoured beds would have to be modelled from experience, for *self-formed regime channel conditions were not geometrically similar.*

It appeared that pile lines should not be designed merely to prevent piping, but to prevent sand from slipping out underneath into the scoured flood-bed conditions. Judged by the Author's experiment on page 164 there was never any real danger of "piping" unless the sand under the weir was removed by the causes stated above. The Speaker was informed that under Islam Headworks was a solid core of sand and that the hollows were local in its vicinity.

The need for understanding not only physical conditions but also physical *principles* was illustrated by the standing wave model for Marala which gave the wave much too small and too far up the glacis. (Another model, for Islam, was said to have reversed the error). The Central Board of Irigation Publication on the Standing Wave should show the errors likely to arise in model work, and how to correct for them, although some supplementary knowledge was also requisite. In fact there seemed no need for a wave model when it was easy to calculate accurately all that was required.

Thus, suppose Fig. 1 (6) represented conditions over a model, using the conventional symbolism of the Central Board of Irrigation Publication, the upstream total energy (T.E.) line would drop rapidly beyond the crest, due to friction (hf was the friction loss, which could be assessed almost entirely from the loss on the glacis while h_1 was the loss of head in the wave, the critical depth corresponding to the discharge q per foot breadth as given by $D^3{}_c=q^2/g$.

The equations determining the jump were then, for a clear trough,

$$D_1\,D_2\,(D_1+D_2)=2D^3{}_c \qquad (1)$$

$$h_1=E_1-E_2=\tfrac{1}{2}\,D^3c\,(1/D^2{}_1-1/D^2{}_2)-(D_2-D_1) \qquad (2)$$

Let us suppose, the Speaker said, that the prototype was n times as big. Could we arrange for exact geometeric similarilty of flow? To test, put nD_1 and nD_2 for D_1 and D_2 in (1) and it would be satisfied if nD_c replaced nD_C. This meant that the discharge per foot of the prototype would have to be $n^{1\cdot 5}$ times as great for the model. Neglecting friction between the upstream gauge and the downstream edge of crest, and remembering that the discharge over a long-crested weir varied as $E_t{}^{1\cdot 5}$ (not as $H_t{}^{1\cdot 5}$), we saw that the upstream still-pond level must be n times higher from the crest than in the prototype. The waterlevel at the gauge site would not be n times higher, but somewhat more, as velocity head varied as the square of velocity, which, in turn, meant direct variation almost exactly as H_t. To be specific, if model H_t be 1·05 and prototype H_t be 10·5 and the upstream floor level with crest velocity head.

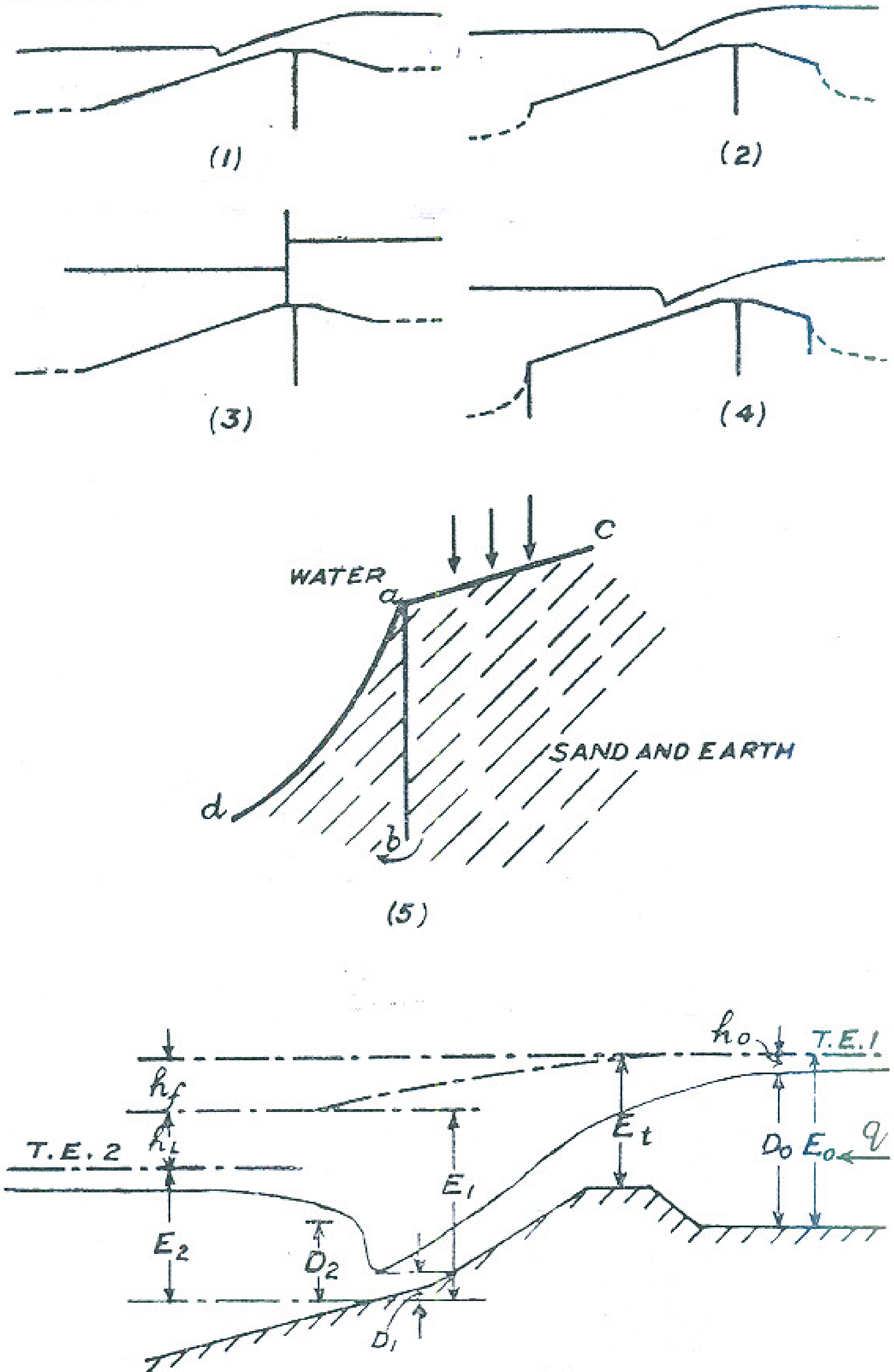

(1)
(2)
(3)
(4)
WATER
a
c
SAND AND EARTH
d
b
(5)
h_o
T.E.1
h_f
h_L
T.E.2
E_t
D_o
E_o
q
E_1
E_2
D_2
D_1

FIG. 1 (6)

would be 0·17 foot and 1·56 feet respectivlly. So generally we could not expect much deviation from truth by dealing with upstream gauge levels (relative to crest) instead of still-pond levels, although theoretically there was some deviation.

Friction, however, was more serious. If D denoted the mean depth of jet on the glacis, we calculated total friction nearly enough from this when there was a long glacis jet. The formula $V=CR^{\frac{3}{4}}\ S^{\frac{1}{2}}$ from Punjab Engineering Congress Paper No. 212 could be used ; S measured the rate of loss of head by friction. Using suffix p for prototype, and m for model, we would get,

$$S_p/S_m=(V_p{}^2/C_p{}^2D_p{}^{\frac{3}{2}})/(V_m{}^2/C_m{}^2D_m{}^{\frac{3}{2}})\ldots\ldots\ldots(3)$$

Using the fact that $V=q/D$ and q increased with $n^{1.5}$, (5) gave that

$$S_p/S_m=(1/\sqrt{n})\ (C_m/C_p)^2\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

and total friction losses, being proportional to glacis length, would be

$$_fh_p/_fh_m=\sqrt{n}\ (C_m/C_p)^2\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

The Appendix to Paper No. 212, combined with the Fig. 1 suggested that $C_m=160$ and $C_p=120$ are fair, So if we took $n=10$ (*i.e.*, modelling Marala flood by 1 foot of water over the model crest) we would get.

$$_fh_p/_fh_m=\sqrt{10}\times 16/9=5{\cdot}65,$$

against a geometric similarity ratio of 10. In other words, friction was twice as disturbing in the model as in the prototype, in spite of the greater absolute smoothness of the model. This would keep the model wave too far up the glacis.

If one was to believe dimensions scaled from Plate IV (Marala Weir) the loss in the wave was less than that in friction (taking friction as the difference between total loss and wave loss), whereas if friction were calculated directly the reverse held. If we took the friction in the Marala prototype as being about ⅓ total loss, then, in a model the friction would be ²/₃ total. Plate V of the Central Board of Irrigation's Publication "The Standing Wave" showed at a glance that such a model of Marala would predict a wave about 15 feet further up the glacis (longitudinally) than it ought.

The reverse effect could occur with suitable scale ratio and *very* rough prototype, compared with a smooth model.

The final source of error, and probably the biggest, lay in the fact that, in high flood, Marala weir crest was obviously "short"; and Mr. Inglis, in his Paper mentioned above, had shown that the co-efficient

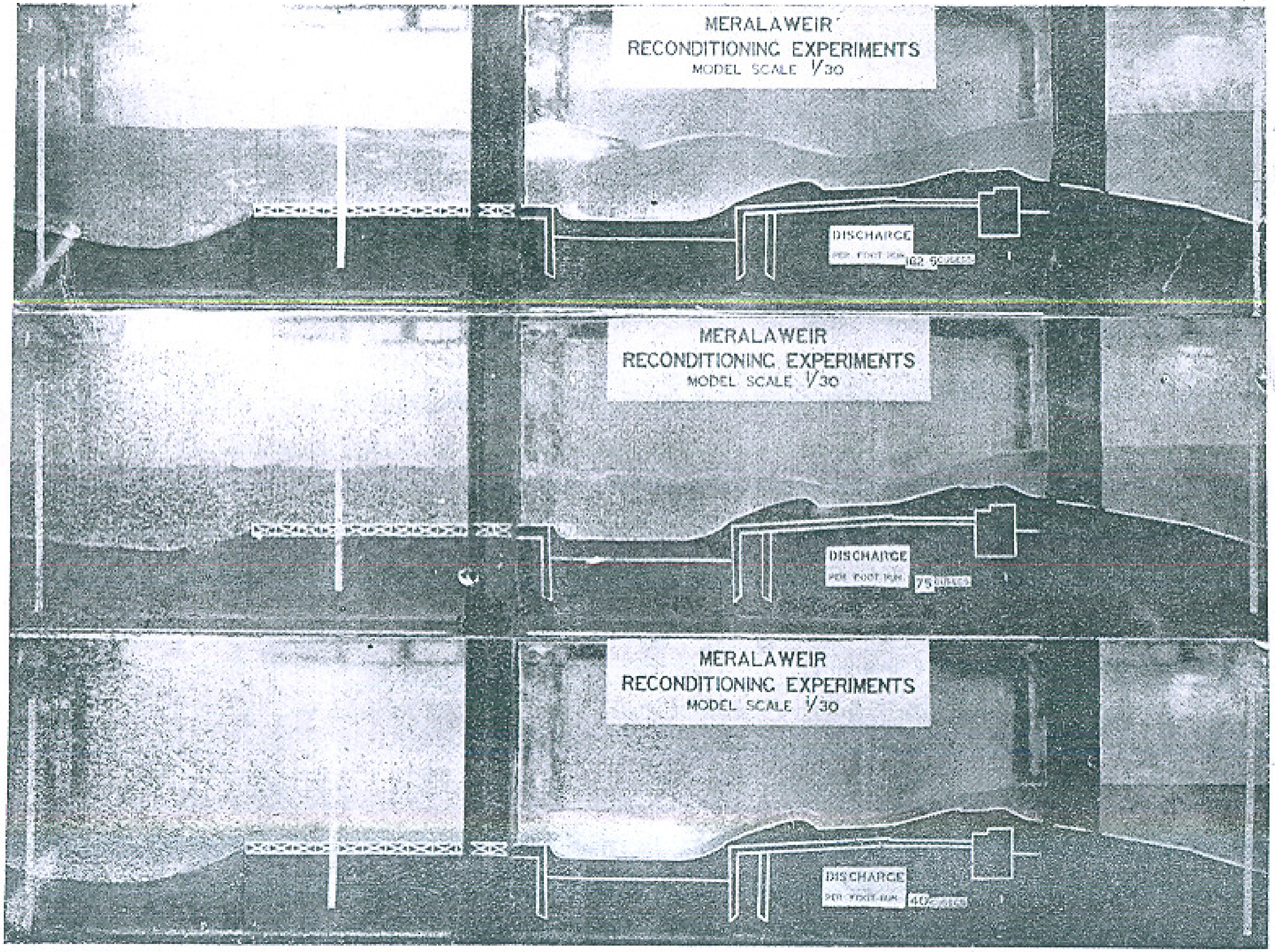

MERALAWEIR
RECONDITIONING EXPERIMENTS
MODEL SCALE 1/30
DISCHARGE
MERALAWEIR
RECONDITIONING EXPERIMENTS
MODEL SCALE 1/30
DISCHARGE
MERALAWEIR
RECONDITIONING EXPERIMENTS
MODEL SCALE 1/30
DISCHARGE

Photo No. 16.

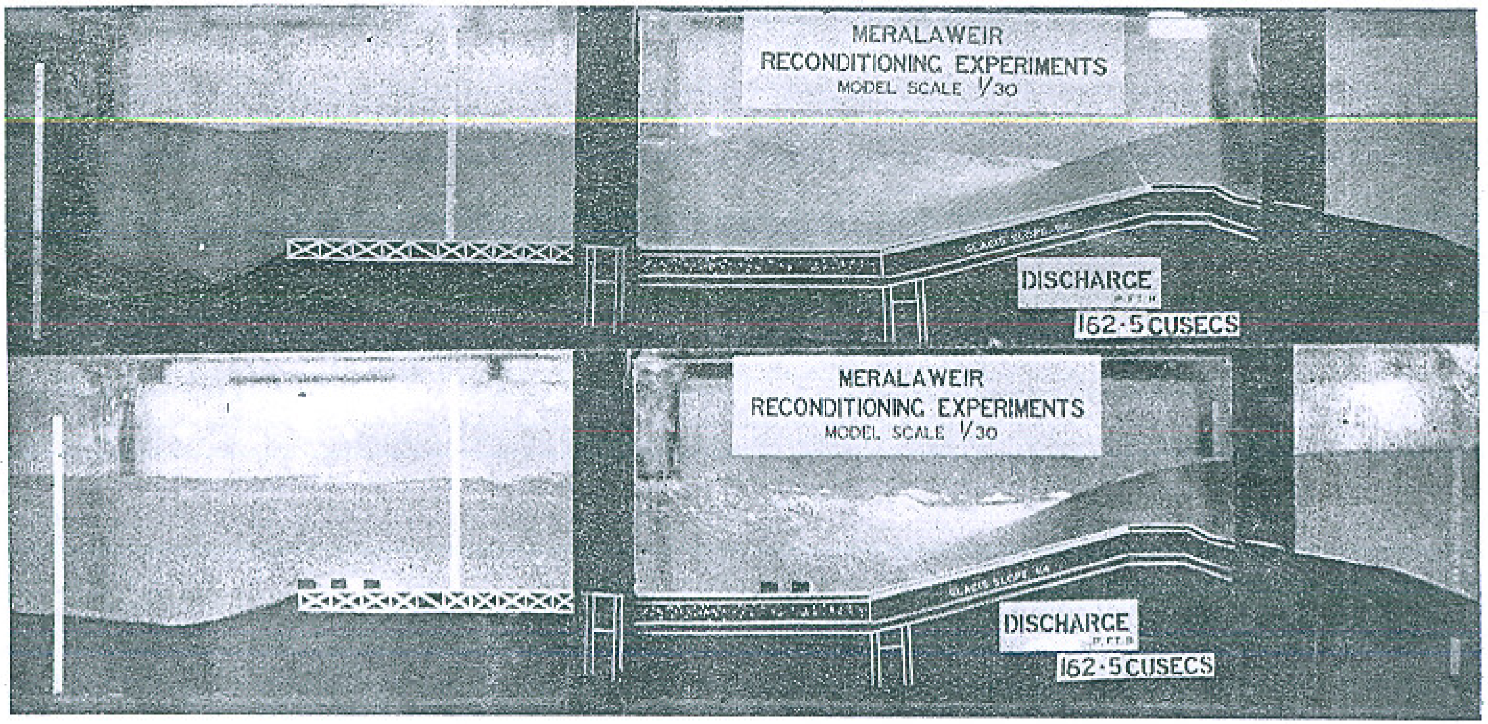

MERALAWEIR
RECONDITIONING EXPERIMENTS
MODEL SCALE 1/30
DISCHARGE
162·5 CUSECS
MERALAWEIR
RECONDITIONING EXPERIMENTS
MODEL SCALE 1/30
DISCHARGE
162·5 CUSECS

Photo No. 17.

would be greater than for a model. Therefore, when the Lahore Research Institute made their model they were almost certainly modelling a much smaller discharge than they thought they were. 10% error would prophesy the wave yet another 15 feet up the glacis.

Obviously the collaboration of officers of various experience was needed in model work as well as in final design, if errors were not be perpetuated.

The Speaker acknowledged the idea of change of state of the earth beneath a weir, due to repeated load, as being that of K.B.M. Iqbal Hussain.

Dr. **H. L. Uppal** of the Irrigation Research Institute said that he wished some space had been given in this Paper to model tests. Before the adoption of the final design for Marala Weir, a number of model experiments were carried out on different proposed sections of the weir. It was originally proposed to construct a subsidiary weir (tailing into a shallow cistern) at a distance of ten feet from the shutter line. The different crest levels of the subsidiary weir were investigated and the various bed levels of the cistern were also tested. After a number of trials it was found that the subsidiary weir did not work satisfactorily. No standing wave was formed in higher discharges as was shown in Photo No. 16. In the case of low discharges, three standing waves formed at unsuitable places. Construction of the subsidiary weir was therefore, given up. In the next design a 1 in 4 glacis from the shutter line and extending up to the B line of wells was tried and the downstream floor and the block portion was lowered by four feet. Different discharges were tested. It was found that the standing wave formed well upstream of the toe of the glacis under all conditions of flow. This was shown in Photo No. 17. The lowering of floor by four feet was very necessary to ensure that the standing wave was in a safe position.

Mr. Blench had just said that the model experiments were not trustworthy and therefore actual construction and design should not be based on model tests. This was far from the truth. Laws of hydraulic similitude had been established, as far as the weirs were concerned, from very early times. Castel, Boilleu, Lesbros, Poncelet and Francis' experiments performed during 1834 to 1850 had shown that results from small scale experiments were comparable to similar and larger scale models. Gibson afterwards had shown that the laws of similitude held good in the case of weir experiments. A number of experiments were carried out in the Irrigation Research Institute, Lahore, on similar models made to different scales and the above statement regarding hydraulic similitude was actually verified. Before starting the Trimmu Weir experiments, it was considered essential to prove or disappove the laws of similitude from similar models built to different scales. It was shown that a model to a scale of 1/20 gave results similar to those which were obtained from 1/30. The following measurements were actually obtained and showed that there was close agreement in the coefficient of discharge when using the broad crested weir formula of $Q=CBH^{\frac{3}{2}}$:—

Discharge.	Model No. 1 (scale 1/30) Value of C.	Model No. 2 (scale 1/20) Value of C.
125	3·174	3·172
100	3·163	3·146
75	3·090	3·100
50	3·071	3·083

Similar models of three falls built on the Khadir Branch of the Pakpattan Canal, namely Trikhni, Chit Drain and the fall at R. D. 2,41,000 were tested with different scales. Results obtained showed a remarkable agreement between the two sets and are given in Plates XVI and XVII.

Mr. **G. R. Sawhney** said he thought this was a very thorough and elaborate Paper and the organization of the work that he had witnessed at Marala was just as elaborate and thorough and the Joint Authors, he said, deserved to be congratulated for their success.

He asked the Authors whether we were to understand that with all the repairs which had been carried out these weirs on sand foundations were still only a make-shift arrangement and, not even now, sound undertakings.

How were cavities at any particular spot to be detected, he asked, when the Authors themselves did not know anything about a big cavity which existed just near a pressure pipe? If the only guide was the blistering of the floors, then finding out cavities was not of much help and in such cases it would be best to get on with job and rebuild the downstream floors, etc., without any further probing into the floors and by losing time, make things much worse. Some sort of cavity diviner needed to be invented and until this was done, he thought that our troubles would not cease. The cavity found in bay 7 of Rasul Weir might or might not have been fourteen years' old. This period should not be assumed to be safe for leaving cavities alone.

There was no doubt that engineers all the world over were inventing very wonderful machines and works everyday, but in other branches of engineering the mistakes found were neither so glaring nor discovered so soon as seemed to be the case with our hydraulic theories and the results of their applications. It was clearly proved that we accept and follow such theories far too quickly and also that there was much room for early and well considered research on a sounder basis, regarding our works.

The Speaker said he had come to the conclusion that, usually, our works were designed for so many cusecs per foot-run, the process being to divide the maximum discharge anticipated by the total length of the waterway, but in practice it was found, that, whether due to incomplete data having been collected in the first instance or through faulty regulation or from natural causes, some of the bays or portions of our weirs had, at different times, to pass much bigger discharges and were thus subjected to much higher action and pressures then they were designed for; and the result was a failure which might be known then or years afterwards. Hence to minimize such failures, the action and directions of the main streams both above and below our weirs needed to be noted, discharges through the part of the weirs over which this main stream passed to be properly estimated or measured and taken as the basis of our future calculations; and not to take the average as was done at present. Clearer regulation rules should also be framed for the various sites.

The conclusions arrived at in the last sentence of para. 2 page 161 were, in the Speaker's opinion, not convincing. The damage might easily have been done during 1928 or 1929 or at some other time and the washing away of the bay in July 1930 caused by a comparatively small flood might have happened simply because the damage had been done already and this flood just become the last straw on the camel's back!

The holes by a Calyx drill might help and might not; a good deal depended on luck as to the spot at which it was used.

Cement grout might close up a small inlet or passage leading to a big cavity, without filling the cavity, or it might pass down and across the floor in a big quantity through a small but straight passage, and without having filled a big cavity. Hence the quantity of cement grout used should not be taken as a direct proof of the absence or presence of big cavities.

Mr. **Madan Lal** complimented the Authors on their masterly review of the design of weirs that had either failed or been giving trouble from time to time and he found the detailed discussion on the design of Marala Weir and its reconditioning of considerable interest to future designers of similar structures.

As a student of engineering designs, he felt inclined to discuss the science of foundations and the possibilities of reinforced concrete as a structural material. Research intc soil mechanics had been of recent origin but it had long been known that sand, if held laterally, was the best material for foundations in the absence of rock. From the considerations of subsoil flow, as discussed at very great length in a recent Central Board of Irrigation Publication, "Design of Weirs on Permeable Foundations" by Rai Bahadur Khosla, Dr. Bose and Dr. McKenzie Taylor, it would be imperative to box the sand by deep sheet piles upstream and downstream of the crest of a weir. With sheet piles properly capped by concrete, the possibility of an undermining of a weir would be very remote since the depth of sheet piles would ordinarily be fixed from the considerations of maximum scour and safe exit gradient.

Sand boxed in this manner and if of unbounded depth would behave as an elastic material and there would exist a linear relationship between load and settlement till the yield point of the soil was reached.

If p (lb./sq. in.) be the pressure and y (inches) the settlement of the block stressing the soil, then

$$\frac{p}{y}=E_1 \text{ (Constant).}$$

This constant E_1 was termed the Modulus of Soil Reaction and its value for average sand in a dry condition was about 600 lb. 1 in^3. Its value for sand under weirs had been worked out to be 79 lb. per in^3. by experiments carried out at Trimmu under the supervision of the Director, Central Designs, Irrigation Secretariat, Lahore. The load settlement curve as obtained from the observation had been shown in Fig. 2.

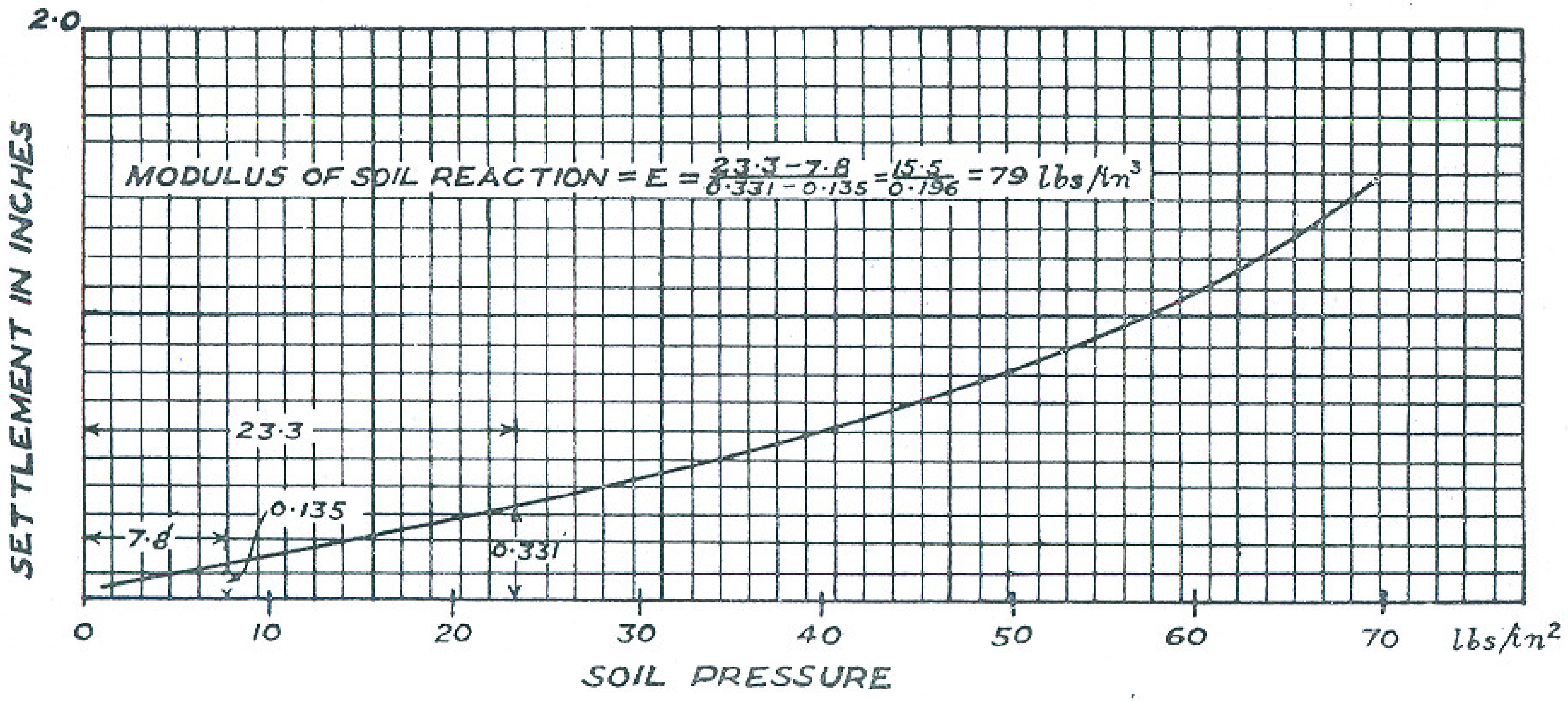
In the design of weirs a certain length of an impervious floor would be necessary from the considerations of surface flow and the surplus energy of the water to be dissipated. This would introduce the uplift pressures which were due to subsoil flow caused by the headed up water. The following methods of construction had been in vogue before the popularity of reinforced concrete had been established:—

(*i*) Lean mixture of mass cement concrete of a suitable thickness to weight down the uplift pressures.

(*ii*) Mass cement concrete, lean mix, covered with a 12" thickness of 1 : 2 : 4 cement concrete, reinforced for temperature and shrinkage stresses. Marala Reconditioned, Khanki and Sulemanke Headworks in the Punjab might be cited as places where this construction had been employed.

FIG. 2
PAPER NO. 215

SOIL PRESSURE TEST AT
EMERSON BARRAGE SITE
LOAD-SETTLEMENT CURVE



PUNJAB ENGINEERING CONGRESS.
1938.

The advent of the reinforced concrete era suggested the following modes of construction:—

(*iii*) A reinforced concrete raft (1 : 2 : 4 uniform mix) weighted down by piers and their extensions, as was adopted at Trimmu. The same raft would make the foundations for the piers.

(*iv*) A reinforced concrete raft held down by Franki piles, spaced at suitable distances apart. A small Headworks in the United Provinces on the West Bengal River had such a design.

Before concluding his discussion on the Paper the Speaker concurred with the Authors that the pressure pipes could not be the indicators of local cavities. On the contrary, they could give a clue to general undermining of the weir. Local cavities could best be ascertained from the dangerous cracks that formed in mass concrete sections.

The Speaker further remarked that a 2·75 ft. thick floor between the *B* and *C* lines of wells sustained the severest standing wave conditions because of its having been pervious. It acted as an imperfect filter for the *B* line of wells.

Mr. **Nand Gopal** said he had read through the Paper with great interest. He thought the Paper was almost like a fairy tale from engineering world! Works and all difficulties, he said, were visualized like clockwork and the whole of the operations went on without a hitch. Such deep thought for all possible contingencies was rarely met with and it was no wonder that almost everything came off, as expected. The Speaker said that there was not much new information in the Paper for theorists and scientists, but for a construction engineer, it was the last word in organization and execution. The Paper would serve as a guide for young engineers for a long time to come.

Mr. **Cox** in replying to the discussion thanked the members of the Congress who had taken part in the discussion and added that he had very little criticism to answer.

In reply to Mr. Montagu, Mr. Crump subsequently pointed out that by inadvertently deducting $E_{f2}=18\cdot8$ from the T. E. line above the wave, Mr. Montagu had arrived at the conclusion that the old downstream floor was low enough. To find the floor level, however, E_{f2} should, clearly, be deducted from the downstream T. E. line. Thus the critical R. L. of the floor should be $810\cdot43-18\cdot8=791\cdot63$ as against R. L. 789·83, the actual level of the downstream floor as reconstructed. The difference was, of course, all to the good and made certain that the wave would form well up the 1-in-4 glacis.

Mr. Blench's remarks had to a certain extent been replied to by Dr. H. L. Uppal. The note of caution sounded by Mr. Blench that not only geometrical but also dynamical similarity should be secured between the model and the prototype was, however, very opportune. In the problems with which the practical irrigation engineer was faced the flow was almost invariably turbulent, the Speaker said, and in such cases any attempts made to deduce the conditions in the original from those in the model, based on purely geometrical considerations, were bound to give erroneous results. As stated in the text (p. 188) arrangements had been made to determine the position of the standing wave on the reconditioned weir by photographing actual waves and when records covering a sufficiently wide range were available, they would furnish a very valuable check on the results as obtained from model observations or by calculation.

Mr. C. L. Handa had remarked that it would in the long run have expedited matters, had the work in the left half of the weir been done in one stage and not in two. The reasons why it was considered necessary to make an early start in bays 1 and 2 and complete the work in these bays as soon as possible had been given in the text. Mr. Handa's proposal if followed would have meant that work in these bays would have been in progress during the freshet season with all its attendant risks.

CORRESPONDENCE.

Mr. **Crump**, in a written communication, congratulated the Joint Authors on having produced a paper of outstanding merit and of very great practical interest and importance for those concerned with the design, construction and maintenance of river weirs on sand foundations. The matter which he wished to emphasize was what he believed to be the chief cause of initial failure, and in particular the nature of the phenomenon known as "blistering."

He recalled that in the summer of 1936, during the monsoon tour of recessing officers, it had been his privilege to attend an informed meeting at Khanki at which the principal topic of discussion was the proposed reconstruction of Marala Weir. It was on that occasion that he had first expressed the belief that the general upheaval, during floods, of extensive sections of the downstream floor or glacis of riverain weirs occurred much more frequently than had hitherto been suspected; and had escaped detection because the "blister" subsided again after the flood had passed. He had been led to this belief by the failure and destruction, that monsoon, in two successive floods of unprecedented magnitude, of the whole of the downstream floor of the Jatli Level Crossing on the Upper Jhelum Canal. During the winter of 1935-36, this floor, which consisted of a 50-ft. length of $2\frac{1}{2}$ feet stone-in-wire crates, had been given a continuous covering of cement-concrete, one foot thick, which replaced the upper 1·0 ft. of stone and contained the surface wiring of the crates. This measure had been adopted because it was known that in medium and high floods the standing wave always formed on the crates, which were frequently displaced and damaged by the pumping action of the standing wave. A cursory inspection of the damaged work had indicated that, before they had cracked and been over-turned by the powerful dynamic action of the high-velocity jet, the concrete-covered crates had been lifted bodily by unbalanced pressure which, under the trough of the wave, must have been well in excess of the combined weight of the crates and overlaying water load.

The Writer stated that a detailed examination and survey of the work in its damaged state, which was carried out at the end of the flood season of 1936, had fully confirmed his belief. In many places, crates which had escaped dynamic action, were found to be bulged upwards by several inches above their original position.

The Writer now wished to suggest that :—

(*i*) The height and location of bulges, or "blisters", detected when a damaged work became visible, was no indication of the maximum extent of upheaval that had occurred during floods. During floods, if the floor was impervious and was not cracked to a sufficient extent to afford pressure relief, the cavity formed by upheaval would be filled with water which would be extruded by the weight of the floor, when the flood subsided. A smooth-bedded floor would resume its original position leaving no trace of its movement during floods. In other cases, the displacement of odd masses of material from the underside of the floor would prevent the floor mating with its sand bed, so that portions where such obstruction arose would present the familiar "blistered" appearance. It was suggested that these blisters showed only what happened in the course of subsidence and not what happened when the floor was being lifted during a flood.

(*ii*) The static uplift pressure indicated by the conventional method of drawing the gradient of subsoil flow was not realized in those cases where the combined effect of these static pressures was in excess of the effective resistance of the floor due to (*a*) its own weight and that of the water above it, (*b*) flexural and shear resistances called into play in the process of upheaval. On the contrary, the actual pressure in the cavity formed by the floor lifting, adjusted itself to that particular pressure which gave an exact chance between the effect of pressure uplift and the resistances (*a*) and (*b*) considered above. This precise balance implied that the floor lifted so slowly that its acceleration was negligibly small. Stated more simply : the effect was exactly similar to the lifting of a load by a hydraulic jack of requisite strength but small pumping capacity : neglecting acceleration, the pressure on the plunger of the jack adjusted itself to that value which was just sufficient to balance the load to be lifted.

(*iii*) The flow-gradient from the surrounding sand into the blister cavity was determined by the pressure in the cavity which was, in turn, dependent as shown above upon the resistance to uplift of the floor. The *rate* or *speed* at which the floor was lifted, depended upon this gradient and the transmission-constant of the sand through which water flowed into the cavity. The latter factor was so small that the rate of lift was extremely slow. The maximum lift in a flood of 24 hours duration would probably not exceed one inch.

(*iv*) As time went on, owing to imperfect subsidence after each flood, the accumulative effect of successive floods created a permanent residual raising of the floor which encouraged "piping" and gradually resulted in the formation of small runnels, unfilled by sand, between the main cavity and the downstream end of the floor-protection. The existence of these runnels enormously increased the rate at which the blister-cavity filled during a flood. If these runnels were left ungrouted (either by cement or sand) the time would come when the lift of the floor during a flood would be enough to cause its destruction.

The Writer stated that he had succeeded in demonstrating the behaviour outlined above by means of a small model. Mr. Burkitt had done the same by inserting a loaded plunger into a floor where the measured uplift pressure was more than enough to raise the plunger, which had ingeniously been surrounded by a frictionless mercury seal. The rate of rise of this plunger was so slow that Mr. Burkitt was deceived and had drawn an incorrect inference.

The Writer considered that the Authors' description of instance of blistering and cracking, provided ample evidence to support and confirm his views. The formation of long transverse cracks definitely pointed to a uniform upheaval of large sections of a weir. Finally, he was of opinion that the very peculiar shape of the surface contour observed by Mr. Khosla and shown in Plate I of the Paper was quite incompatible with a glacis in its normal position. The contour could however be explained by accepting that a large section of the glacis between the crest and the B line of wells had moved round its upstream edge as a fulcrum, until it had assumed a practically horizontal position. The Writer believed that this was actually what happened.

Prof. **Th. Rehbock** thanked the Council warmly for the opportunity of contributing to the discussion of this Paper. He said that he had been concerned for many years with Indian Flat Weir problems and had expressed his views on these weirs to numerous engineers from India who had visited him. Since March, 1936, he had been carrying out special tests, directed to the protection of Marala Weir from scour, in the River Hydraulics Laboratory at Karlsruhe.

To the question of whether this re-construction on so large a scale was necessary, he said that, in his opinion, the use of dentated cills would have avoided altering the angle of the downstream slope of the weir and the lowering of the apron. On the other hand, it was absolutely necessary, of course, to fill up the voids under the floor, and certainly it was necessary to raise the insufficient margin of safety of the weir against such under-scour. The two lines of sheet piling were a proper means of doing this.

On the question of the safety of a weir against the washing out of the sand under the foundation (underscour, piping), he wished to make the following remarks:—

The opinion was widely held that a weir was safe against under-scour if the length of the path of the water that passed under the weir from upstream to downstream exceeded a certain number of times the amount of heading up. The permissible number of times depended on the quality of the soil and was first fixed by Bligh.

In arriving at those numbers, Bligh measured the length of the sub-soil path along the underside of the cross-section of the weir down and up the sheet piling and cut-off walls.

He believed this method of measurement was not correct, because the greatest part of the underground water would take the shortest path and only comparatively small quantities followed the up and down line of the weir. The length of this line was, therefore, of no great importance.

In the Writer's opinion, the shortest path possible would give a much more reliable co-efficient to determine the safety of a weir against underscour.

The factor of safety of a weir against underscour depended on the maximum exit-velocity at any point where the water came out of the sand into the open bed of the river downstream of the water. This velocity should not exceed a certain limit, which depended on the particular class of soil in the river bed. If the exit velocity exceeded this limit, the water could carry sand and this would produce, in course of time, cavities under the weir and might lead to its destruction.

Therefore, it was of the greatest importance to diminish the exit velocity by making the area of exit of the subsoil water as large as possible, which could be done by proper design at the downstream end of the apron, in addition to deep and watertight upstream sheet piling, solid with the weir.

According to his experience, the width of the apron in the direction of the stream should not be too great, but it should lie sufficiently low to allow a hydraulic jump with a surface roller to be formed. It should be provided at its downstream end with a second well of sheet piling not so deep as the upstream piling, and be furnished with a scour reducing cill at its extreme downstream end. The cill would ensure safety against washing out of the bed immediately downstream of the apron, and would create a sufficiently flat scour tangent angle. It also appeared important to the Writer to avoid all large water-tight blocks on the river bed downstream of the final cill, which hindered the subsoil water's emergence. A layer of gravel at this point was of advantage.

The lower layer of it should be material which was fine enough to prevent sand being forced through from under the weir, and the upper layer should be of a size large enough to prevent it being washed away, but neverthelss able to take up sufficient motion to form a smooth scour tangent slope. The danger of this valuable covering layer of gravel being washed away, would be further reduced if care were taken in the design of the cill to form a sufficiently large, but not too quick-revolving, ground roller downstream of the apron.

The lower surface of this ground roller moved upstream and would prevent the gravel being carried away. To create this valuable ground roller it was advisable to put the dentated cill on the extreme downstream end of the apron, and so to lay the layer of gravel that it fell gradually away from the downstream end of the apron at the desired angle of scour. Covering the river bed downstream of the water-tight apron with concrete blocks, often applied in India, he considered harmful, as they hindered uniform distribution of the exit of the subterranean water into the river bed. The concentrated exit of this water through the grooves between the blocks might permit sand to be carried through, and might lead to the weir being washed out from below.

Where the danger of uplift of the weir by pressure from below by the subsoil water, or by suction by standing waves occurred, this danger must be fought by suitable "training" of the water flowing over the weir. For instance, in the case of Marala weir in its original form, with the downstream slope 1 : 15, it would have been possible to reduce the 8 ft. high standing wave, which formed on the slope of the weir, to a fraction of its original height by putting a second cill with teeth on the slope of the weir. By this means, the dangerous suction could be almost eliminated.

If the above mentioned proposals, derived from twenty-six years of experience on models and observations in the field were adopted, then in the Writer's opinion, today, even in river beds of fine sand, weirs could be built at a much reduced cost of construction and of maintenance, which would fulfil all conditions of safety and eliminate in operation all danger of undermining by subsoil flow, of excessive scour downstream, and of destruction by uplift.

Some of these proposals could be adopted also with advantage in the conversion of existing flat weirs, and would lead to very material savings.