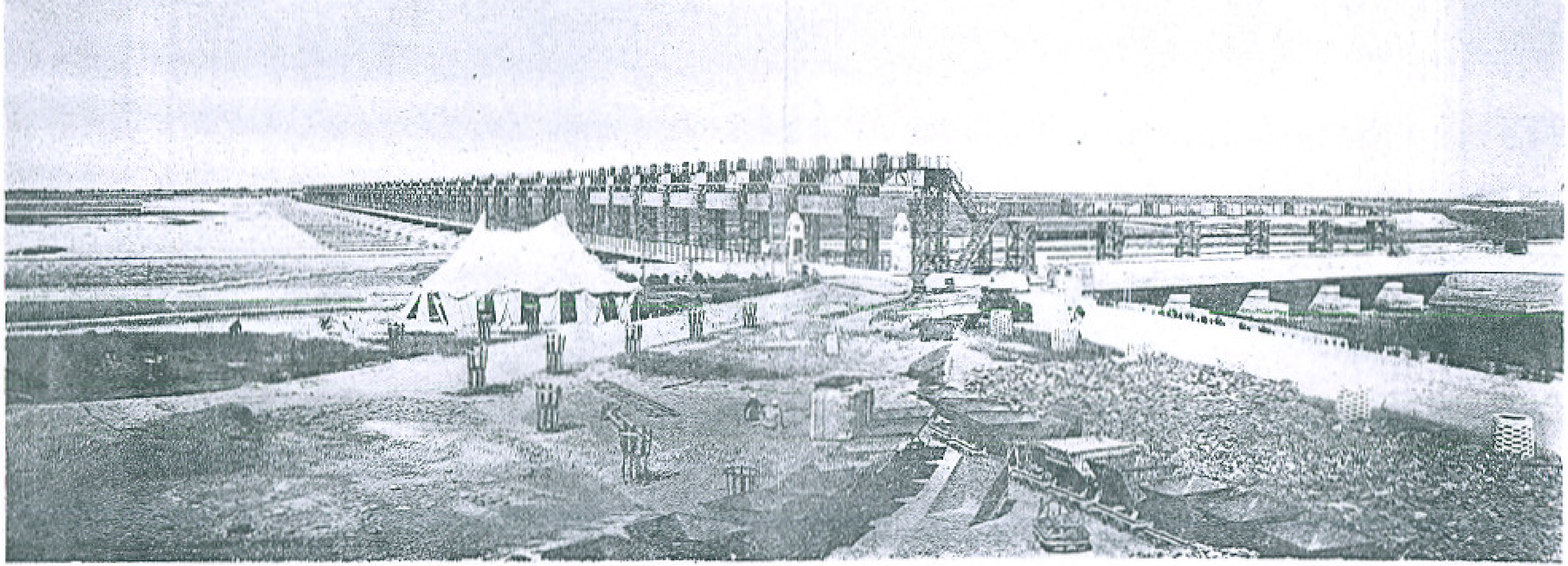
ERRATA FOR PAPER No. 244

Serial No.	*Page*	*Line*	
1	69	6	*Add* after confluence, "although an alternative site existed 7,000 ft. higher up."
2	69	29	Instead of "of he two alternative" read "of the two alternative."
3	72	6	Instead of "The quality of" read "The quantity and quality of."
4	72	25, 26	Instead of "Survey village canals" read "Sutlej Valley Canals."
5	76	36	Instead of "started" read "produced."
6	77	28	Instead of 469.5 read 469.0 and instead of 490.1 read 490.5.
7	79	36	Instead of R. L. 488.0 read 468.0.
8	81	14	Instead of "immediately proposed" read "proposed immediately."
9	92	31	Read 1.17 $R^{3/2} = 213$ or $R^{3/2} = 182$.
10	95	3	Between the earth pressure and plus *add* "of saturated sand has been taken as full hydrostatic pressure."
11	95	17	Instead of .271 h at the end read 27.1 h.
12	95	19	For $27.\frac{1x\ h^3}{6}$ read $\frac{27.1\ h^3}{6}$.
13	99	25	*Between* 7 tons *and* for *add* "for computing bending moments or 10 tons."
14	107	2	Instead of 483.0 read 483.5.
15	108	16	Instead of 100 lbs. read 1,100 lbs.
16	109	6	Instead of $2.\frac{5}{3}.08$ read $\frac{2.5}{3.09}$.

PAPER NO. 244



THE BARRAGE AS IT STANDS

PAPER NO. 244

CONSTRUCTION OF THE EMERSON BARRAGE AT TRIMMU

BY SOM NATH KAPUR, I.S.E.

History

The Haveli Project, for which the Emerson Barrage provides river control on the combined Chenab and Jhelum rivers, was first conceived in 1915 when, estimated at a cost of Rs. 170 lakhs, it provided for a weir with shutters below the junction of Chenab and Jhelum and two canals offtaking one from each bank. The right bank canal was intended to provide non-perennial irrigation to the low areas bounded by the river Thal and Karam Canal. The left bank canal tailed into the Ravi immediately above Sidhnai Headworks whence another canal was to offtake to link up inundation canals taking off from the left bank of the Chenab below the junction of the Ravi.

The project was calculated to show a return of 5.63 per cent. The Bahawalpur Darbar, however, objected to its execution on the ground that it would adversely affect the inundation canals of the State. The low productivity, coupled with this objection, was responsible for its not being sanctioned by the Government of India. With the construction of the Panjnad Weir for the benefit of the Bahawalpur Canals, the question was reviewed and the project revised in 1932 when it also included areas served by Ganesh and Talhiri Canals of the Muzaffargarh District for the non-perennial supply on the right and the area lying between the river Chenab and the irrigation boundary of the L. C. C. on the left. The examination at this stage, however, showed that a new Headworks would have to be built on the Ravi river in place of the existing one at Sidhnai and allowing for the cost of this and basing perennial irrigation figures on a winter discharge of 1,250 cusecs and a summer supply of 7,750 cusecs, the project resulted in a financial loss of 18 lakhs a year.

In 1935 a Committee was appointed by the Government of India to define and determine the relative interests of the Punjab, Sindh, Bahawalpur, Bikaner and the N. W. F. P. on the waters of the Indus and its tributaries and this Committee recommended a withdrawal of 2,750 cusecs in the *rabi* and 7,750 cusecs in the *kharif*.

The Lower Bari Doab Canal has no claim on the Ravi river during the winter and all available supply at Balloki used to be utilised in the Sidhnai system. The Sidhnai requirements having been met from the Chenab, the Ravi water below Madhopur was thus made available. A part of this saved water was to be utilised out of the Chenab river to convert 1,42,000 acres of the non-perennial area on the Burala extensions to perennial, and the balance was to be used to supplement the supply of the Pakpattan Canal which was to be passed through the Lower Bari Doab Canal and then *via* a link of 700 cusecs capacity connecting the Lower Bari Doab Canal with the Pakpattan Canal where an area of 1,20,000 acres was expected to be irrigated annually (see Plate I).

Thus the revised project of 1935 became productive by utilising the winter supplies in the Ravi below Madhopur and 2,750 cusecs out of the Chenab and the Jhelum. It is essentially a project of inter-connecting rivers and utilising excessive supplies in one to make up for shortages in the other.

The project as framed in 1935 was carefully examined by the Government of India and sanctioned by the Punjab Government under the New Constitution in 1937.

Description of the Barrage

The Barrage as constructed consists of 37 weir bays of 60 ft. span in the middle and two sets of undersluices with eight bays on the left and six on the right of 30 feet span each, separated from the weir by long divide walls 360 feet long on the upstream and 400 feet long on the downstream. Two fish ladders exist, one alongside each divide wall.

The left pocket is further divided into two halves on the upstream by a central divide wall 570 feet long, the left half containing a silt excluder for the Haveli Canal which, with an F. S. irrigation discharge of 5,249 cusecs, takes off from the left. A silt excluder of the Khanki type exists on the right to serve the Rangpur Canal which takes off from the right with an F. S. discharge of 2,710 cusecs. The barrage is provided with guide banks of Bell's type on the upstream and expanding at 1 in 5 on the downstream. Marginal bunds are provided, the right one being linked to Thal *tibbas* and the left one to high land, not so far inundated. Two silt ejectors have been put in the Haveli Canal at R. D. 1,200 and 2,000 to escape 1,200 and 1,000 cusecs and provision for one has been made on Rangpur Canal although it has not been constructed. The weir is connected with the North Western Railway by a broad gauge railway line. The arterial road (Lahore to Bhakkar) has been diverted to cross the river at this site and a suitable bridge has been provided.

Selection of Site (See Plate IX)

The barrage was to make use of the infiltration waters of both the Jhelum and the Chenab and as such it was to be located below the confluence of the Chenab and the Jhelum rivers. The limited drop available in the water levels between the confluence of the Chenab and the Jhelum on one side and the Ravi at Sidhnai on the other whence the canals were to offtake did not permit the shifting of the Headworks to any considerable distance below the confluence site known as Trimmu. The weir site was finally selected in March, 1937, and was located about two miles below the confluence. As the pond level was fixed from considerations of supply into the left (Haveli) Canal, the lower site meant higher gates and heavier embankments but this increased expense was more than justified because it gave a valuable storage capacity of 10,000 cusec days which could be utilised during the sowing period of the *rabi* and which could be further replenished and utilised with advantage during the winter freshets.

The river at this site during the floods used to spread out to a width varying from three to four miles but was generally confined between two *belas* about 1½ mile apart.

It was therefore evident that work was to be constructed outside this 1½-mile width in an area protected from floods and the river diverted on to the completed work. The barrage was, therefore, located on the *bela* to the east and its far end was so placed that it intercepted a major creek which was later to become useful for the diversion of the river and the right flank was to be about half a mile from the main river bank, so as to be quite safe from river attack during the construction period. The above were thus the considerations which determined the final site.

Preliminaries

N. W. R. Connections.—The site having been selected and the preliminary organisation started, the first need was the selection of a railway connection for transport of material. Of he two alternatives Jhang was 14 miles and Mudduki flag station eight miles off (Plate I). Mudduki, however, involved the cost of the conversion of the flag station into a "C" Class station, extra sidings and extra housing accommodation for the railway staff which, being an essential condition prior to the receipt of material, had to be completed before the work on the canal sidings could start. In case of Jhang more B. G. Track would have been needed, but the connection being available, work could start earlier. The decision was made in favour of Mudduki where work started on the 18th June, 1937. The question of providing assisted sidings was next taken up. Three sidings, one for a loaded rake, one for empty and one for miscellaneous traffic were the minimum requirements. They took off with points having a throw of 1 in 8, the shortest siding being 1,450 feet and long enough to take a full rake. A canal railway station was constructed on the offtaking siding where two more sidings were provided. A slightly improved design was adopted for Sikhanwala Quarry, the sidings of which are shown on Plate II.

Station Area Site.—After this the selection of the station area site was taken in hand. There were two alternative sites with good soil. The present site, which is lower but closer to Headworks, and another about a mile-and-half towards Mudduki along the railway line but with N. S. about pond level. The final pond level was 4 to 5 feet higher than the natural surface level of the lower site, and this evidently meant the construction of a ring bund round the station area to protect it from floods both during and after construction. Later, the effects of water-logging would be quickly visible, resulting in the construction of drains and a recurring expense on pumping. Its proximity to the river, however, ensured a quicker drop in the subsoil with a drop in the river, thus shortening the period of pumping. The great advantage which led to the final decision in favour of this site was that the workshops, powerhouse, the stores and the housing accommodation of the establishment would be closer to the works which would result in savings on transport of material, transmission of power and a better control on works which would be accessible at all times. The other site, although higher than the proposed pond level, was closer to the marginal bund and, therefore, the subsoil would have remained high for a much greater period, resulting in the appearance of *kallar* on the surface. It may be argued that the selection of a permanent station area would have been better if placed at the higher site but it would have involved much heavier cost by way of two station areas.

Railway Line and Sidings.—Before a decision could be taken as to the exact alignment of the railway line, the kiln sites had to be decided. The area in the proximity of the station area and Headworks was fully surveyed and soil samples taken and contents analysed. The range of selection was not very wide and the bed of an old creek had to be finally decided upon. The capacity of various sites as to the production of bricks was then worked out by making trial pits about 100 feet apart and thus working out the total good earth contents of the area. Instead of taking separate sidings from the station area for brick-kilns, the railway line was aligned in such a way that it crossed the kilns on its way from Mudduki to Trimmu (Plate IX). The only other factors which determined the final alignment of the railway line were avoidance of wells, huts, etc. The main cost in the construction of a railway line is of the track and, therefore, to avoid its extension, the alignment was determined by the shortest length between the control points. The formation level was determined from the high flood levels as this was to serve as a marginal flood embankment for the time being and could later form a second line of defence in the case of a breach in the left marginal bund.

Plate II shows the layout of the railway sidings in the station area which, briefly, provided a triangle, sidings for workshops, loco shed, powerhouse, weighbridge, a stone siding to be used for stacking reserve material along it during periods of high supply and low demand, a sick siding, a loop and a siding for stone-crushers and the station area yard with sidings enough to accommodate a loaded rake, an empty rake, miscellaneous trucks and a rake ready for shifting to the weir works. The sharpest curves were of six degrees and therefore no check rails were provided.

Marginal Bunds.—There were two alternatives for the marginal bunds; firstly, to align them far out to avoid river attack and give maximum *sailab* to the zamindars and, secondly, to keep fairly close to the main river with provision for spurs to control the river course. A bund well set back would have been cheap in initial cost but would have meant greater compensation for land which came under the pond, much less control on the behaviour of the river and its course during floods with a possibility of oblique entries and subsequent *bela* formations. A bund controlling the river course with spurs would have been more expensive in initial cost and subsequent maintenance but greatly helpful in keeping a straight flow on to the barrage resulting in a better control. It would save in cost of land under the pond but reduce pond storage and would exclude a larger area from the benefits of the *sailab*. The final alignment, therefore, took a middle course so as to be most economical and least harmful. The right marginal bund was connected to the Thal and the left marginal bund went out and was connected with the high land which had been free from river inundation so far. The slope of the river during the maximum floods had been 1 in 5,000, but marginal bunds were given a slope of 1 in 8,000 because it was contemplated that with afflux caused at the barrage site the slopes would flatten out and if, at a later stage, raising of banks was found necessary, it could be done.

Quarry Investigation

The total approximate quantity of stone which had to be supplied for the construction of the barrage was of an order of 1,30,00,000 c.ft. Mr. Khosla was deputed on special duty, in January, 1937, to investigate the resources of various quarries and the speed at which they could supply. The only quarry working at that time in the Department was Baganwala, but it was situated about 150 miles from the proposed Headworks site. There were several outcrops near Hundewali, known as Kirana Hills, and the possibility of their working was fully investigated. Although at any one of these places new sidings would have to be laid and new organisation made involving water supply, staff, labour and sanitation arrangements which meant a capital expenditure of rupees four to five lakhs, it gave an advantage of lesser time in transport and savings in railway freight. On working out the cost of the various alternatives, it was found that these outcrops would be better placed from the point of view of economic supply. Hundewali Quarry had previously been worked by the North Western Railway and, although according to the civil records, it is the property of the Punjab Government, its possession was disputed. The railway authorities were not working this quarry but would not agree to part with it unless the Irrigation Branch paid a royalty on the stone quarried which made its working uneconomical. The negotiations took some time before they broke down and set back the quarrying operations by two months. Small hillocks around the place were then investigated. The quality of stone was tested by making abrasion tests on the various samples of stone and compression tests on the concrete with quarry ballast. The Sikhanwala Quarry (see Plate II) offered prospects of high progress on account of long working faces as compared with Baganwala or Hundewali. The quality tests having proved satisfactory both for ballast and pitching stone, this ouarry was finally decided upon and the work of assisted sidings at the railway station and sidings along quarry faces was taken in hand. The main consideration in the layout was to take sidings as close to the working faces as possible so as to involve the minimum lead for the labour—the main factor to determine the progress and, therefore, the final cost.

Preliminary Works (Railway Line)

Material for the railway line could not be received till Mudduki station was opened to traffic and this naturally took time. By the time Mudduki railway station was opened (on the 11th July, 1937) the formation of the railway line Mudduki—Trimmu was completed and arrangements for collecting track material started. The B. G. rails and fastening were obtained mostly from Nalagarh—an old abandoned quarry of the Irrigation Department—and from the Survey Village Canals. The total track material was estimated as arrangement for purchase of sleepers had to be made through the Sleeper Pool under the control of the Railway Department.

A total of 76,000 sleepers had to be purchased and selected, but in order to control the rates and quality, tenders were called for smaller quantities at a time. A survey of the timber market soon revealed that the total number of sleepers required would not be available in the market by the time the work was required to be completed. It was, therefore, proposed to put 3-foot *chil* pieces under each rail in the place of alternate sleepers. This was, however, only tried on straight portions and where there were no grades. After the tenders were called, inspection of sleepers was started by an officer of the Sleeper Control and the author and in all about 50,000 sleepers were inspected at a rate of about 2,000 per day out of which about 30 per cent. had to be rejected.

Sleepers purchased were mostly of deodar and a small percentage of *chil* creosoted and were all to the Standard Specification of First Class Railway Sleepers. The supply of 3-foot *chil* pieces solved the problem of sleeper shortage and experience showed that not only were they cheaper by Rs. 2/12 per sleeper but quite satisfactory for the possession was disputed. The railway authorities were not working this quarry but would not agree to part with it unless the Irrigation Branch paid a royalty on the stone quarried which made its working uneconomical. The negotiations took some time before they broke down and set back the quarrying operations by two months. Small hillocks around the place were then investigated. The quality of stone was tested by making abrasion tests on the various samples of stone and compression tests on the concrete with quarry ballast. The Sikhanwala Quarry (see Plate II) offered prospects of high progress on account of long working faces as compared with Baganwala or Hundewali. The quality tests having proved satisfactory both for ballast and pitching stone, this ouarry was finally decided upon and the work of assisted sidings at the railway station and sidings along quarry faces was taken in hand. The main consideration in the layout was to take sidings as close to the working faces as possible so as to involve the minimum lead for the labour—the main factor to determine the progress and, therefore, the final cost.

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Programme and Requirements

Before any further action could be taken, it was necessary to decide on a tentative design of the weir section and work out the approximate quantities of various items of work to be done. A rough programme of the work was then drawn up, considering that the work could start in full swing after the flood season of 1937 and would be completed by the end of 1939 when the river would be diverted. It was proposed that excavation should start in October and finish in four months. When about three-fourths of the excavation was done, piling should start and finish about the same time in February as the excavation after which the concreting of the floor should start and be completed by September when work on the superstructure was to start and be completed by January after which the remaining time was to be devoted to the steel erection.

The above were the limitations that determined the time available for each operation and, considering the usual factor that the labour in the first month would naturally be less and would go up to the peak in about the second or the third month, the total labour in each month was worked out for each type of work. Progress graphs of various items were plotted. This gave an indication of the approximate amount of labour and material required during each month, and further provided the basis for the supply organisation on which requirements were to be worked out. Roughly it indicated that about 3,000 donkeys and 6,000 coolies would be required at the peaks. In order to arrive at the above programme, the staff requirements were worked out on a rather conservative basis. This in turn formed the basis of housing accommodation to be provided in the station area, the approximate areas to be reserved for labour camps and the total number of supervision staff quarters to be provided. The population figures were thus obtained to form the basis of future sanitation, conservancy, medical and other welfare schemes which were to be taken in hand. Appendix "A" shows the various buildings constructed at Trimmu and should be considered as a very conservative guide for future constructions if the work has to be done during the same time. The scale of buildings, the number of officers and the construction organisation staff having thus been decided, the work of designing the buildings was taken in hand.

Brick-kilns

In the meanwhile tenders were called for burning two kilns for 2,500,000 bricks each and also for the local supply of 1,000,000 bricks. Incidentally, the exceptionally low rates quoted in these tenders showed the trend of future rates during the construction period. Although the final orders on kiln tenders were passed by the 12th June, the work could not be started till the beginning of the monsoons. The layout of the kilns depending on their capacity took some time to decide in view of the reluctance of the contractors to have bigger-sized kilns due to the low rates tendered. Contractors also tried to delay matters because of the impending monsoon season which was definitely disadvantageous but unavoidable, and an assurance had, therefore, to be given to make good the loss if kachcha bricks were spoiled before they could be loaded into kilns. By this time the approximate requirements of bricks on buildings were worked out and contractors were made to work bigger kilns which would permit a double set of chimneys to be used, thereby giving a progress of 30,000 bricks per kiln per day. Later experience showed that both the kilns kept on working practically continuously with a double set of chimneys, the total outturn of bricks being 155.2 lakhs, which should be a good guide for future.

Buildings, etc.

If the work was to be started in real earnest, the staff had to be housed before the winter set in and, therefore, the work of building had to be rushed. It took some time before the final details and design could be decided upon. The various alternative roof designs of temporary buildings were considered and, after some time, a neat type of roof design, although slightly expensive, was finally decided upon. This consisted of tiles spanning precast concrete troughs for spans over 10 feet and concrete battens with tiles spanning the spaces between for spans below 10 feet. Another difficulty experienced was that the station area site could not give sufficient area for kachcha brick-fields. The total quantity of kachcha bricks (90 lakhs) in buildings were worked out and, accordingly various alternative sites for kachcha brick-fields were selected and, in view of the long distance involved, extra payment had to be made for the carriage of kachcha bricks. The construction of buildings was started on the 10th September and the policy underlying construction was to provide accommodation in the following order which proved to be very satisfactory:

Offices, powerhouse, workshops, quarters for menial and clerical staffs, dispensary, godowns, telegraph office, mistris' quarters, senior clerks' quarters, subordinates' quarters, temporary officers' bungalows, permanent buildings, post office, bazar and schools. This ensured the maximum amount of accommodation with the minimum of expense and at the earliest period. Officers were apparently proposed to be housed last of all and, therefore, tent accommodation was provided for them. In winter these tents had kachcha walls constructed round them and the total expense incurred on kachcha walls, dry-brick floors, a cook-house and a bathroom did not exceed Rs. 300 per tent, which is a very moderate figure. Actually the officers could not shift to their bungalows before March, 1938. Before the construction of quarters, all establishment was housed either in *shouldaries* or under some sort of canvas cover and tent equipment for this purpose was collected from all over the Department.

Miscellaneous.—The other details of the preliminaries which involved a considerable period of time were the preparation of a working schedule of rates after comparing the rates that prevailed at Panjnad, Khanki, Marala and Deg, where works had recently been constructed, selection and arrangement of staff, settlement of major points in designs and the arrangements for the supply of material. After the schedule of rates was prepared, tenders were called for on the basis of these and contractors selected.

Plant and Machinery

When the project was under preparation, Mr. B. L. Sakuja had been deputed to inspect and reserve the plant and machinery available in the Department and at the Sukkur Barrage. This was purchased from within the Department as soon as the project was sanctioned. A large number of pumps, mixers and piles were purchased from the Sukkur Barrage Division. B. G. Rails were obtained from within the Department at one rupee per foot. The plant which had to be purchased from outside consisted of three electric generating sets for the powerhouse, necessary switch-board, lathes and other workshop machinery, B. G. Locos and Trucks, N. G. Trucks and track. To ensure the supply of material within the specified period and in accordance with the proposed programme, a minimum of two rakes of 47 trucks quarry material had to be handled in a day and at times of maximum demand almost a full rake of cement would be required per day. Two B. G. Locos would, therefore, be required to work at the quarry and five at Trimmu,—two for stone rakes, one for the receipt of miscellaneous materials and two for local shunting. The collection of B. G. Plant was, therefore, an immediate and urgent matter to be considered. Negotiations with the North Western Railway for the supply of the same failed as in the case of the quarry and, therefore, six B. G. Locos were purchased from the Bengal-Nagpur Railway and one obtained from Rohri and 102 B. G. Trucks were purchased from the Central P. W. D., Delhi. All possible B. G. Trucks in the Department were requisitioned and this gave a total of seven Locos and 500 B. G. Trucks. Two generating sets were available at Lyallpur and were taken over. An order for a new generating set was placed with the Indian Stores Department who obtained tenders and placed orders but this arrived after construction had been finished. Four Petter sets lying on the S. V. P., which were purchased in 1923 and which had almost served to the last were, therefore, overhauled and requisitioned. Three were installed and one kept as a reserve to serve for spare parts which were very frequently needed.

The telephonic and telegraphic connections took a long time in spite of the best efforts and co-operation offered by the postal authorities, because no such things had been foreseen and provided in the previous year's forecast. Statement I indicates the total number of telephones fixed during the construction period. The exchange system for the same could not be secured quickly. This therefore, is an item involving considerable delay which is likely to be overlooked and, for future constructions, should form one af the most important items to be tackled straightaway. Arrangements for opening a dispensary for the labour and staff were made and took about three to four months. The Post Office was opened in October, 1937.

Land Acquisition

A serious obstacle in the preliminary work was the slow pace at which the land could be acquired, without which nothing could be done. Land acquisition is a subject so rarely dealt with on a large scale that few officers understand the difficulties involved. Firstly, unless the Department has a special officer of its own to deal with the acquisition, considerable delays are likely to occur on the part of the civil authorities on account of the nature of enquiries required to be made. This difficulty was soon overcome by appointing a special officer who was notified as a Collector in the various districts. As soon as the various layouts and alignments were finally decided, land plans were prepared from G. T. Survey sheets and Section 4 Notification issued. Action under Section 17 was simultaneously taken which dispensed with the lapse of long periods before taking possession. Thus, within a week, Notification under Section 6 followed. The disposal of land acquisition cases in later stages showed that there was a general tendency on the part of the local subordinate civil staff to pile up the rates and, therefore, in order to watch the interests of the Department, a special Senior Zilladar was deputed to go into all cases of mortgages and other mutations personally and thus to make sure that no fictitious or exaggerated transactions were started and, if produced, to be duly rebutted. This method has resulted in considerable savings and is one to be followed in future.

Zamindars did not object to handing over possession but their experience has been very bitter in so far as they were rendered destitute and lost their means of living but payments for the land acquired could not be made which, as a result of the laborious process involved, caused considerable delay. The fact that they are neither compensated nor given any other means of living for the time being is a serious problem and has to be solved in the near future by way of appointing more L. A. O.s and, if possible, by way of making part advance payments at the time of taking over of actual possession.

Design of the Weir

General.—The entire design including the waterway sections of the weir and the sluices, protections, guide banks, etc., are in conformity with the general principles laid down by Mr. Khosla in C.B.I. Publication No. 12 by Khosla, Bose and Mckenzie Taylor.

River Discharge.—A high flood had occurred in September, 1929, on both the Chenab and the Jhelum. The Haveli Project was being prepared at the time by Mr. Khosla, Officer on Special Duty, and the latter made very detailed investigations of the discharge and levels which obtained at Trimmu. The discharges are regularly observed at Chela on the Jhelum and the Rivaz Bridge on the Chenab which are about 18 miles above the Headworks site and also at the Trimmu Ferry site about two miles above the Headworks but, unfortunately, the gauge at this latter site was washed off and arrangements were insufficient to gauge a discharge of this magnitude. Mr. Khosla, therefore, resorted to the observation of the river cross-sections and water-levels as obtained from flood marks. The figures thus worked out approximated to 6,43,000 cusecs. This was also found to be correct after considering the discharge which passed at Mangla, Marala, Rasul, Khanki and Panjnad. The figure finally decided upon was 6,45,000 and this was adopted with a water level of 490.5 at the barrage site as the basis of design. The lowest discharge ever recorded at Trimmu was 805 cusecs in February, 1933, with water level of 471.4 opposite barrage site. River survey was carried out in 1936-37 and the minimum bed level at the barrage site was found to be nearly 469.0. A retrogression of 5 feet at low discharge and 1 foot in high discharge was assumed and thus the bed level downstream was kept at (469·5) 464.0 for undersluices and a flood level of (490·1) 489.5.

**Waterway.*—A discharge of 6,45,000 required a wetted perimeter of 2,140 feet according to Lacey, but in accordance with the practice which obtained on the previous Headworks, a length of 3,025 feet of the Barrage was decided upon. The statement below shows the required wetted perimeter and the actual length of the barrages as obtained at various places :

Headworks	Discharge	W. P.	Width of Barrage	Looseness Factor
Balloki ..	1,13,000	930	1,547	1.67
Ferozepur ..	4,50,000	1,780	1,956	1.1
Suleimanki ..	3,25,000	1,520	2,223	1.47
Islam ..	2,75,000	1,400	1,621	1.15
Panjnad ..	7,00,000	2,230	3,400	1.56
Sukkur ..	15,00,000	3,260	4,725	1.45
Trimmu ..	6,45,000	2,140	3,026	1.42

*Page 151, C.B.I. Publicatien No. 12.

Design of the Weir

General.—The entire design including the waterway sections of the weir and the sluices, protections, guide banks, etc., are in conformity with the general principles laid down by Mr. Khosla in C.B.I. Publication No. 12 by Khosla, Bose and Mckenzie Taylor.

River Discharge.—A high flood had occurred in September, 1929, on both the Chenab and the Jhelum. The Haveli Project was being prepared at the time by Mr. Khosla, Officer on Special Duty, and the latter made very detailed investigations of the discharge and levels which obtained at Trimmu. The discharges are regularly observed at Chela on the Jhelum and the Rivaz Bridge on the Chenab which are about 18 miles above the Headworks site and also at the Trimmu Ferry site about two miles above the Headworks but, unfortunately, the gauge at this latter site was washed off and arrangements were insufficient to gauge a discharge of this magnitude. Mr. Khosla, therefore, resorted to the observation of the river cross-sections and water-levels as obtained from flood marks. The figures thus worked out approximated to 6,43,000 cusecs. This was also found to be correct after considering the discharge which passed at Mangla, Marala, Rasul, Khanki and Panjnad. The figure finally decided upon was 6,45,000 and this was adopted with a water level of 490.5 at the barrage site as the basis of design. The lowest discharge ever recorded at Trimmu was 805 cusecs in February, 1933, with water level of 471.4 opposite barrage site. River survey was carried out in 1936-37 and the minimum bed level at the barrage site was found to be nearly 469.0. A retrogression of 5 feet at low discharge and 1 foot in high discharge was assumed and thus the bed level downstream was kept at (469·5) 464.0 for undersluices and a flood level of (490·1) 489.5.

**Waterway.*—A discharge of 6,45,000 required a wetted perimeter of 2,140 feet according to Lacey, but in accordance with the practice which obtained on the previous Headworks, a length of 3,025 feet of the Barrage was decided upon. The statement below shows the required wetted perimeter and the actual length of the barrages as obtained at various places:

Headworks	Discharge	W. P.	Width of Barrage	Looseness Factor
Balloki ..	1,13,000	930	1,547	1.67
Ferozepur ..	4,50,000	1,780	1,956	1.1
Suleimanki ..	3,25,000	1,520	2,223	1.47
Islam ..	2,75,000	1,400	1,621	1.15
Panjnad ..	7,00,000	2,230	3,400	1.56
Sukkur ..	15,00,000	3,260	4,725	1.45
Trimmu ..	6,45,000	2,140	3,026	1.42

*Page 151, C.B.I. Publicatien No. 12.

A tight section of the barrage would mean a low crest level which may not permit the accurate gauging of the discharge during floods. Further, it would mean deep gates involving a heavier cost and the greater concentration per foot run of discharge involving heavy initial cost on protective aprons and consequently heavy annual maintenance. The difficulties of loose sections are also obvious because it is once in 10 years that a heavy flood may be experienced. The extra cost on gates and protection may, therefore, be justified by the cutting down of the width. Further, as the discharges normally to be run are not very high, a loose weir would encourage a quick growth of *belas* which would be difficult to control even by regulation. A very tight section would, on the other hand, provide no factor of safety against oblique entries which, on account of their direction, would reduce the waterway.

Working, therefore, on the past practice, the length of about 3,000 for barrage was decided upon which, allowing 10 per cent. for pier width, would mean a water-way of $3{,}000 \times \frac{9}{10}$ or a discharge of $\frac{645{,}000 \times 10}{9 \times 3{,}000} = 240$ cusecs per foot run. This required a head of nearly $\frac{(240)^{\frac{2}{3}}}{3 \cdot 1} = 18 \cdot 3$. With the type of section proposed for the weir, a drowning ratio of 85 per cent. nearly could be permitted and this meant an afflux of three feet and a high flood level of 493.5.

Undersluices.—There appears to have been no definite formulæ to determine the size of undersluices in the past. The object of these generally is to provide scouring pockets so as to maintain wide approaches for the off-taking discharges to facilitate silt control. Again, since the offtakes are generally perpendicular, they should be wide enough to permit drawing in of surface water from a comparatively wider pool without disturbing the silt distribution in pockets. For this reason generally the width of the pockets has been about $1\frac{1}{2}$ times the width of the off-taking canal. The second object of the undersluices is to permit river diversion without unnecessary heading up. The downstream floor of the undersluices was decided at R. L. 464. It would permit a retrogression of about five feet. In view of the heavy pumping involved, it was not considered advisable to lower the floor level further although it would have been advantageous against retrogression. The undersluice crest level was then determined so as to give the maximum discharge during the high floods, but consistent with the depth on the downstream floor being sufficient to support the wave. This gave the crest level of 472.0. To suit the greater height of the gates, 30-foot spans were decided upon for the undersluices and 7-foot piers in between. The undersluices would thus have a downstream floor level of 464.0, crest level of 472.0 and 30-foot spans. If the river was to be diverted through both undersluices a discharge of about 20,000 cusecs would be encountered in February which would require a waterway of about 400 l.ft., provided excavation could be done to deep levels. In accordance with the above, therefore, 14 bays of undersluices were decided upon, of which six were to be on the Right and eight on the Left. This stipulation, however, was never fulfilled as by speeding up the work and arranging the programme differently, the diversion was done through right undersluices only and the low level of the undersluice crest was of little use. The total barrage width had been proposed at 3,000 feet as above. Two fish ladders 11 feet on right and 10 feet on left with 7-foot piers were proposed between the weir and undersluices. The undersluice and fish ladder, therefore, accounted for a waterway of (14×30) plus (13×7) plus (11 plus 10) plus $(2 \times 7) = 546$ feet, leaving about 2,500 feet for the weir. For the weir bays a 60-foot span was proposed as a result of past experience and permitting a 7-foot width of pier, 37 spans of weir bays could be accommodated.

The Barrage thus consisted of 14 undersluice bays of 30-foot span and 37 weir bays of 60-foot span (Plate III).

Weir.—A velocity of 10 feet per second was assumed during the floods. This would give a velocity head of 1.5 foot (nearly) and an upstream energy level of 495.0. To pass a discharge of 6,45,000 cusecs with an H. F. T. E. level of 495.0 through 14 undersluice bays with a crest level of 472.0 and 37 weir bays the crest level of the weir was required at 477.5 which was adopted. The section was then tested with a 20 per cent. concentration of discharges to provide for the uneven distribution of the discharge or the obliquity of the flow during high floods and it was found that in undersluices the wave would still be retained in the cistern. In the weir section, the weir cistern level was required to be kept at R. L. 470.0 which meant a 6-foot drop from the weir to the undersluices. As pumping to a lower level had to be done for undersluices and the depressing of the weir floor was possible without any expense, it was kept at 468.0. This guarded against possible future retrogression, resulting in low spring levels which would otherwise have needed deeper piles to ensure a safe exit gradient and also increased the flexibility of control of the barrage in low supplies.

Profile (*Plate IV*).—The upstream floor level was determined from the consideration of probable unwatering difficulties and was fixed at 470.0 for the weir and 488.0 for the undersluices.

A 6-foot wide crest was proposed to be kept. This would permit cill girders and gates but was relatively narrow to give a high coefficient of discharge. Before deciding the exact profile of the glacis upstream and downstream a series of model experiments were conducted in the Irrigation Research Institute embodying slopes of 1 in 5.7 to 1 in 3 and circular and transition curves. Finally, a slope of 1 in 4 was decided from the consideration of ease in construction and laying of concrete on slopes and was adopted for both upstream and downstream. The length of the downstream floor was kept at five times the height of the standing wave to ensure that the turbulent action would not go beyond the impervious or pacca floor. Assuming high flood conditions with concentration of discharge, this gave lengths of 66 and 80 feet for the weir and undersluices respectively. Friction blocks were provided on the downstream floor of both weir and undersluices as shown on Plate IV to kill the turbulence. Their exact position was determined as a result of experiments carried out by the Research Institute on the models there and were tied down to the raft reinforcement.

**Pile Lines.*—A pile line was then introduced on the upstream end to kill the pressure but its depth was determined from scour considerations. The loose protection upstream of the weir would always settle in front of the pile and would prevent its being exposed. The question of its failure by spewing could, therefore, not arise and hence the piles were taken down to the level of the probable deepest scour and no further. Another pile line was introduced at the downstream end and was taken deep enough to permit a safe exit gradient which was given a factor of safety of about 4.5. In the case of undersluices the exit gradient consideration, however, required extending the upstream floor which was done to 52 feet beyond the toe of the upstream glacis. Another pile line was introduced at the toe of the downstream glacis with the object of partially reducing pressures under the downstream floor and also to act as a line of defence in case the downstream floor failed. In the case of undersluices a fourth line was added at the toe of the upstream glacis as an additional factor of safety in case the upstream floor failed which, not being subjected to any heavy pressures, had to be relatively light.

Floor.—On account of the high head the subsoil pressures under the floor, when worked out, were found to be unusually high. A gravity section would have necessitated a floor thickness of as much as 12 feet in places. This would have resulted not only in heavy costs of excavation and pumping but considerable constructional difficulties and the increased quantity of ballast required would have meant delay in quarrying and completion of work. An alternative put forward by Mr. Kanwar Sain was to design the floor as a R. C. raft with the weight of piers, etc., utilised to counteract the subsoil pressure. There were only two difficulties in the case of a R. C. floor, the possibility of the rusting of the reinforcement due to the water getting in through the temperature or shrinkage cracks and the doubtful quality of work on wet foundations. To guard against the latter, a 9-inch thick 1 : 3 : 6 layer of cement concrete was first laid which ensured work to be carried out in the dry. Sufficient reinforcement was proposed to be provided which would avoid anything but superficial cracks and thus rusting possibilities would be avoided. The fact that the floor would invariably be submerged made the rusting a remote possibility. The floor was kept 3.5-foot thick which

*C.B.I. Publication No. 12.

would be strong against dynamic action. Further, to reduce bending moments in the 60-foot weir span, an intermediate groyne was introduced with the top at weir crest level to avoid turbulence from obstruction.

Pockets.—The designs of divide walls and flanks appear separately. The upstream ends of the pockets were given a pile line foundation so as to form the first line of defence against scour action and the pockets were paved with 1.5-foot thick concrete which added to the floor area and provided further safety to the undersluices against scour and dynamic action and also gave a quiet and smooth flow in the pockets, which is a great advantage.

Protection.—As velocities obtainable during high floods were likely to exceed 10 feet when stone could be picked up, block protection was immediately proposed upstream and downstream of the floors. Beyond this, loose stone aprons were provided which were designed in accordance with Spring's method and C. B. I. Publication No. 12, page 153, and were kept 50 feet upstream and downstream except in the case of undersluices and round divide wall noses where they were 60 feet and 80 feet respectively.

**Pressure Pipes.*—The barrage is further equipped with a complete set of pressure pipes to study its actual behaviour under high ponding conditions. All these pressure pipes consist of $1\frac{1}{2}$-inch filters driven vertically in the case of deep ones and laid horizontally, covered with *bajri* and wrapped in a bag in the case of those just under the floor. These are then connected by $1\frac{1}{2}$-inch G. I. Pipes to points vertically below pier tops and then taken up vertically (see photographs) so as to be accessible from the tops of piers. The horizontal pipe connections have been laid at a slight upward incline to avoid the possibilities of air locks which are likely to vitiate the pressure observation readings. At the top these pipes are provided with plugs contained in countersunk boxes which can be opened only by keys and thus the chances of their being blocked from the top are remote. The observation points are located at places which are not accessible except to those on duty and hence there are no chances of their being tampered with.

Design of Raft

The profile of the weir and the raft type of floor design having been determined, the investigation of the loading and pressures was done on the following lines :

(*a*) Subsoil pressures under the floor at various points were determined in accordance with Khosla's method of independent variables (C. B. I. Publication No. 12—" Design of Weirs on Permeable

*C.B.I. Publication No. 12 Chapter V.

Foundations ") and the condition taken was when the pond upstream would be maximum and the downstream dry.

(*b*) The uplift pressures underneath the glacis portion of the weir floor due to the formation of the standing wave trough as shown below (Fig. 1) in the maximum flood conditions may, in certain cases, be greater than those caused by the seepage flow as worked out in (*a*). These were investigated but the uplift pressures worked out as above (*a*) were found to be much severer than under (*b*) and hence these latter were ignored.

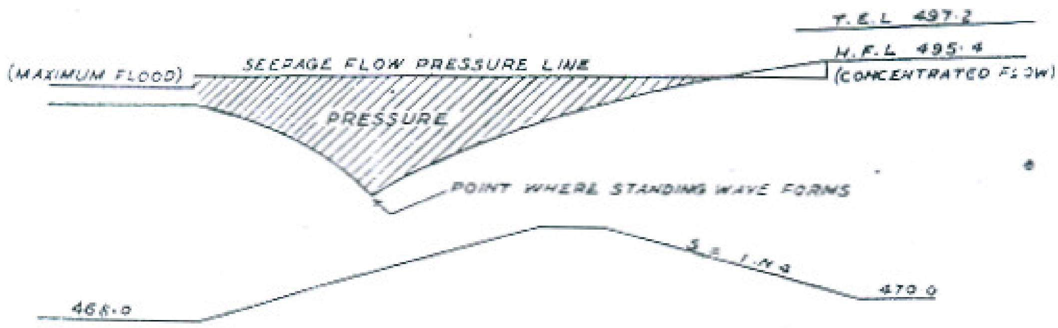


FIG 1

(*c*) The raft has been divided into various sections and the pier load on each section has been worked out independently and the load intensity per square foot determined.

(*d*) Although the standard practice is to take 50 per cent. of the total live load for the design of foundations, in view of the importance of the structure, three-fourth of the live load for which the bridge is designed was considered for the design of the raft. Impact has not been considered at all because the vibrations of the superstructure are entirely dissipated before the stresses transmitted reach the foundation.

(*e*) The dead load of the bridge was worked out and, to find out the load distribution on account of the dead and live loads of bridge, their maximum concentrated effect had been taken into account. The load is distributed through the masonry at an angle of 45 degrees and load intensities on each section were worked out in view of the eccentricity of the load.

(*f*) Similarly, the distribution of the superstructure load of 70 tons on various sections was worked out in view of the eccentricity of the same.

(*g*) Wind pressure was taken at 40 lbs. per square foot. Two conditions were taken, firstly, when the pond would be nil and the gates would be up. In this case the moments about the mean weir profile level were taken and taking the figures of wind load causing moments to be the same with either direction of it, the pressure intensities on various sections were worked out.

(*h*) Wind pressure was again considered with full pond conditions. In addition to the moments due to the wind pressure against the handrailing, the overbridge, the column, the counter-balance and the road bridge, the moments caused by the water pressure against the gates was taken into account. Both cases of the wind blowing upstream and downstream were considered.

(*k*) The accumulative effect of (*c*) to (*h*) was then worked out for both dry as well as full pond conditions and the maximum intensities for which raft was to be designed determined.

The raft is thus subjected to two systems of loadings, firstly, the hydrostatic pressures due to seepage flow as determined in (*a*) from below. These would vary all along the profile but would be of uniform intensity across the bay at each point, and secondly, the vertical load at pier points as determined in (*k*).

The weir floor transmitting the pier loads to the soil is subjected to the stresses of the same nature as those caused by hydrostatic uplift pressures. Before analysing the stresses the pressure distribution under the slab has to be investigated.

Although rigid and thick foundation slabs would transmit load uniformly, but under foundation slabs of thin R. C. sections the pressures vary, being maximum under the load and decreasing towards the ends, the variation depending upon the rigidity of the slab and the nature of the soil.

The soil at Trimmu being sand of considerable depth (an elastic material), it was quite rational to assume that the settlement caused in the sand would be linear function of load or soil reaction.

The modulus of soil reaction given by the ratio of soil reaction to settlement which is zero for clay in a state of flux to about infinity in case of rock is about 15 to 30 times the safe ground pressure which is about .5 tons per square foot. To be on the safe side, the lower figure was used which came to 7.5 tons per square foot. An experiment was done at Trimmu to determine its value and, although on account of sand not being boxed and there being every possibility of a lateral flow underneath, the results came to six tons per square foot.

The deflection caused is given by the general equation

$$E\,I\,\frac{d^4\ y}{d\ \ x^4} = -b\,(p-w)$$

where b is the width of beam, $p =$ the upward pressure and w the downward loading which is zero.

$p/y = E_I$ the soil reaction.

By solving the above differential equation, the bending moment and soil reaction can be obtained.

As long as the hydrostatic pressure from below is less than the weight of the floor, the above moment and pressure diagram would hold, but actually the net uplift would cause deflection and stresses, the maximum being at the centre. Since soil reaction is directly proportional to the downward deflection of the slab, the result of the upward deflection at the centre would be to reduce the intensity of soil reaction. The total area under soil reaction curve must, however, remain constant and, therefore, the soil reaction curve would be altered as given by Curve B.

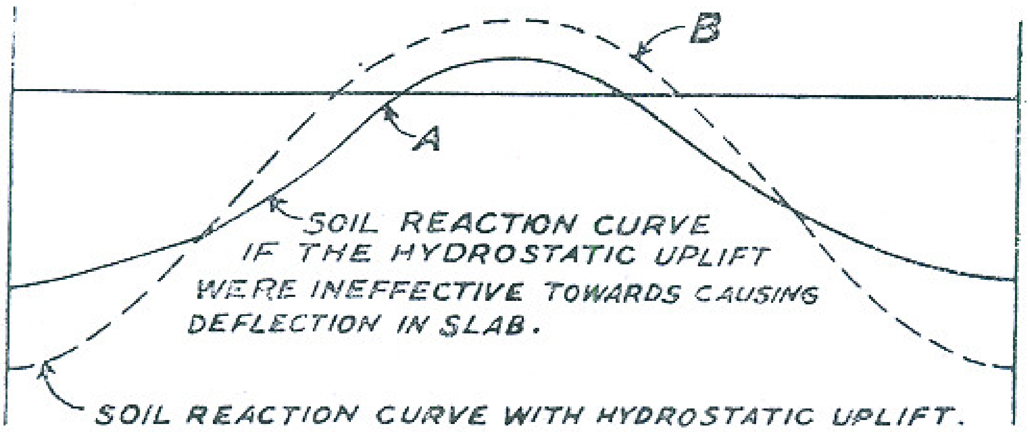


FIG. 2

The mathematical treatment of Curve B would be very complicated, hence Curve A was finally taken for B. M. diagram. This erred on the side of safety because Curve A, which meant a greater load concentration towards the centre than B would give greater bending moment.

The bending moments were worked out under the various loads as obtained above and their accumulative effect determined, which formed the basis of design.

For the alignment of cill girders of grooves it was considered desirable to leave a gap in the piers and also to partially concrete the portion of the floor as shown in the sketch where cill girders were to be installed.

Reinforcement was provided to join the two portions of pier masonry thus separated and was sufficient to transmit the load of the superstructure and bridge from one pier portion to the other. In the crest before concreting of cill girders, portion B (Fig. 3) was designed to transmit all load from upstream to downstream and enough stirrups were provided to take shearing force.

Provision for Temperature.—The sand boxed behind the flanks exerted considerable restraint on the free movement of the raft under temperature effects. Further, as the temperature range was likely to affect 9-inch 1 : 2 : 4 concrete on the surface, provision for temperature reinforcement in the transverse direction was made at .3 per cent of the volume of concrete in the top 9-inches. In the longitudinal direction the stresses in the steel worked out to about 11,000 lbs. per square inch maximum in floor and 14,000 in crest and glacis and hence, in view of these reduced stresses, no temperature reinforcement was provided. The shrinkage stresses in actual practice were minimised by properly damped sand curing extending over an adequate period. The maximum variations occurred during the construction period as, after construction, the leakage would always keep the crest wet and thus reduce the temperature range.

The undersluice section was similarly designed as above. The design of other parts is discussed in the following paragraphs which describe their construction.

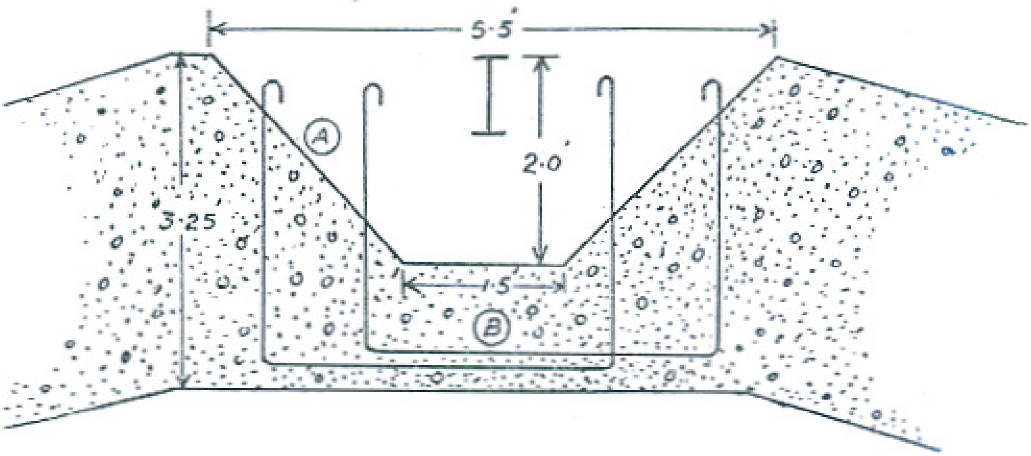


FIG. 3

Programme of Construction

The general time-limits had already been determined as per page 71. The study of hydrographs showed that the river was generally at its lowest in December and consequently this determined the time for river diversion which was apparently a few months in advance of the final completion of the work programmed to be completed by March, 1939. As the main river creek intercepted to be utilised for diversion lay to the right and as the river was also on the right, an early diversion through the right side was not likely to interfere with the completion of the remaining work. The diversion of the river through the left would have necessitated a temporary railway bridge for the transport of materials, etc., heavier pumping and a possible risk of interruption in the case of a sudden big rise in the river. It was, therefore, programmed that the Barrage be completed from the right side, advancing towards the left. The left undersluices were, however, to be taken up immediately so as to complete the deep work as early as possible and to avoid strain on pumping.

The work progressed as per programme as described below.

Excavation

The sections of the weir and the undersluices and the general layout having been determined, the work of excavation started on the 21st September, 1937. The excavations were proposed to the section shown in Plate V. On account of very deep foundation and heavy pumping involved, sloughing from the sides during the period of construction was the determining factor. It has generally been noticed that the bottom five feet takes a slope varying from 1 in 5 to 1 in 10 and after that the general pumping having dried up the surrounding area the slopes obtainable vary from 1 to 1 to $1\frac{1}{2}$ to 1. A certain amount of width has to be kept between the extreme ends of the aprons and the excavation edge to permit the laying of an N. G. track for concreting and also to provide space for concrete dumps and other materials. The excavation profile also made provision for bins, etc. It may, however, be pointed out that in actual excavation the weir profile cannot be retained as it would always wear out by seepage and dust storms and, therefore, generally some filling has to be done and consolidated under the crest later at the time of concreting. The subsoil water level was encountered at 10-foot depth. Further excavation was, therefore, done with the help of pumping which started on the 31st October, 1937 and which consisted of pumping from sumps suitably located and outside the floor area. To these sumps, surface drains carried the seepage water of the whole area. On account of the excavation from sides, these surface drains had to be continuously deepened which was ensured by the employment of manual labour helping the excavators. In front of all these sumps sufficiently deep pools of water had to be maintained to permit only water going into the sumps and to intercept sand which would otherwise choke the foot valves of pumps. As the excavation proceeded the water level on these sumps dropped and the suctions had to be continuously extended.

Excavation measurement below water table is complicated because, firstly, no deep borrow pits can be put in and secondly, any dislocation in pumping is likely to wash away all signs of borrow-pits. This was, therefore, done by observing N. S. level at which excavation started and finally measuring the level at which it finished. Where this was not possible, stack measurements were done outside.

The disposal of the spoil from the excavation was done in such a fashion that it permitted construction of a ring bund round the weir area to permit safe working during flood season. It also allowed enough berm to accommodate sand stacks for use in concrete later and the various B. G. sidings, etc. This ring bund provided enough berm and kept river water away during floods which otherwise, due to its proximity, would have caused such heavy seepage that pumping would have got out of control.

The sumps were generally effective in an area of 300 feet all round because water slopes in drains would not permit efficient excavation beyond. All these sumps were designed as rectangular masonry blocks with steining properly reinforced to function as a retaining wall against saturated earth outside. These sumps were laid on reinforced concrete curbs, details being shown on Plate V. A part of masonry was done in mud mortar which later, when dismantled with the drop of water, served as orifices permitting water into the sumps. During the course of excavation, borings were done at various sites and geological strata determined.

Piling.

Piling followed excavation. Plate VI shows the plan of the piling done in the barrage. In addition to the pile lines worked out in the paragraph under barrage design cross lines were introduced in the weir dividing it into five groups and under the divide walls and flanks.

Proposals were at first made to use Franki concrete piles but the idea was abandoned because, apart from their being costly, not only were they not capable of the speed required but the piling at corners and junctions was likely to be defective and also they would have resulted in congestion. A review of the piles available in the country disclosed that more piles were needed. An order was therefore placed for 1,528 tons of Larson Piles. *The section modulus of these piles was determined from the considerations of bending moment to which they would be subjected when about to fail by spewing. At such a stage the scour would be the maximum permissible and the pile would be subjected to the following forces:

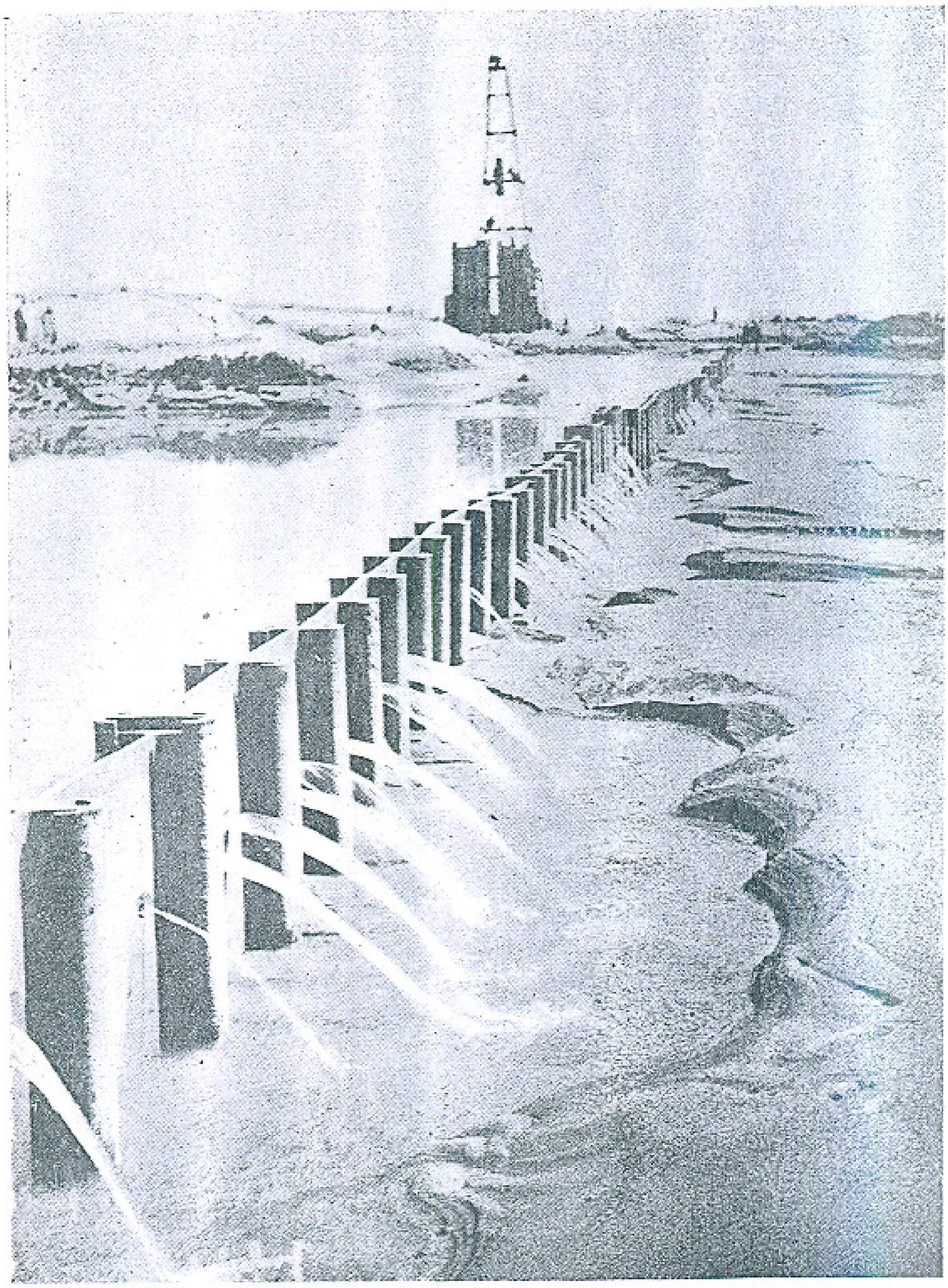
1. Bending moment due to end being fixed.
2. Reaction of floor at fixed end.
3. The earth pressure at the front and back of pile.
4. Effect of drop in subsoil pressure from top to bottom of pile.
5. Soil buoyancy.
6. Water pressure on both sides, taking into account the direction of seepage flow which would decrease or increase the specific gravity of soil by a figure equal to the pressure gradient.

*C. B. I. Publication No. 12. Page 143.

As soon as the excavation reached within a foot of the top level of the piles, the pile-driving was started on the 16th December, 1937 and the work was facilitated by the excavators digging a drain along the pile line. This permitted working ahead of the programme by about 15 days on all fronts. Four pile drivers with No. 7 hammer were in operation. As a rule, Larson Piles were all utilised in the first and second line and Universal and Ransome Piles were reserved for the last line and other such places where piles were required to resist heavy action. Experience showed that Larson Piles were rather weak to resist the heavy driving and their tops generally got bent and crushed which were then sawn at the top and a cap pile added on it to make up the top level. This was likely to give hollows under the pile caps. Further, it was observed, that there was considerable friction in grooves of some of the piles resulting in further driving of the pile already adjacently driven. The groove at places was out of shape and permitted disengagement. On the whole, the Universal Piles were the best and the Ransome's came next. As some of the Larson Piles were driven deeper than necessary due to friction in grooves as explained above, pile-caps had to be provided and were driven on the top of the previous ones. This, however, was a source of weakness which cannot be helped. The Larsons also required a greater number of correction piles than others and sometimes it was of an order of 1 in 10. A large number of correction piles was, however, prepared in advance. Eccentric driving was often tried to help them to maintain their verticality. Photograph shows the method adopted to finally close the pile line. A number of piles were erected in position and the dimension of the final closing pile thus obtained. The pile was, however, given slotted elliptical holes to permit slight adjustments lower down. The smaller piles were mostly driven by small hammers worked with steam fed from boilers of excavators, etc.

Concreting

Toe Walls.—With the excavation reaching the designed levels the laying in of toe walls started. The shuttering under water was generally done by properly stiffened iron sheets worked down in the sand to sufficient depth. The excavation inside was then done by manual labour. In order to allow for the possible slipping in due to the wet earth pressure behind, these sheets were kept apart generally a foot more than required, and at the top they were strutted by ordinary *ballies* spacers. The curtain walls and all other concrete walls were thus constructed with steel shuttering below water level and about six inches above it. Such remaining portions of these concrete walls as would not be visible were constructed by first building .4 ft. thick cement masonry walls with occasional headers protruding inside. With concrete placed inside these masonry shells, the walls would be completed, the masonry forming part of the wall.



Determining dimensions of closing pile.
Note heading up of subsoil by pile line.

Nine-inch Under-layer.—Laying of the lower nine-inch layer of concrete was then taken in hand to cover the wet area of foundation before placing reinforcement and to get dry working conditions on the saturated earth foundation.

An attempt was made on the upstream of the pocket to lay reinforcement on saturated soil but the reinforcement placed on stirrups sank about two feet by its own weight. The bearing area was then increased by giving masonry bearings to stirrups but these were also unsuccessful. Also the placing of scaffolding and staging prior to concreting would not be possible without a concrete base to rest on and thus this nine-inch lower layer of concrete was absolutely essential. It was desirable to cover the entire area with nine-inch concrete as early as possible to avoid cavities forming under it due to sand being sucked by pumping. At the same time, due to locking up on all sides by piles, it meant high pressures underneath, which in the beginning actually resulted in lifting. In some cases it cracked and thus gave relief to the pressure below. In such places holes were jumped in the concrete and pressure relief afforded. Local pumps were used to pump this water out. To avoid this cracking relief pipes with bottom of pipes encased in *bajri* were put in advance before laying this nine-inch concrete slab. In the undersluices pressures became so high on account of boxing by piles all round that it was not possible to lay this nine-inch slab without further dropping the local water table and this was done by installing tube-wells one in each bay of the floor and one on the downstream glacis. Where relief pipes had been put in either in advance or later these pipes were given T-connections, one leading to outside concrete area and the other vertically above floor. During the period of concreting and till such time as it had set properly, the pipe was kept discharging at the lower level. No sooner, however, the concreted floor was able to take the pressure the low-level pipe was plugged and the water rose in the vertical pipe. The pressures were generally high and relief was continuously afforded through these vertical pipes. Later, however, they were grouted under pressure and finally closed.

Raft Construction

Reinforcement.—The reinforcement generally used in the raft was $\frac{5}{8}$-inch or $1\frac{1}{4}$-inch diameter bars (Tata's tested steel). To bend $1\frac{1}{4}$ inch diameter bars to proper shape and in hooks a bar-bending machine was first intended to be used but later they were all bent by hand. Heavy *kikar* wooden blocks were fixed deep in the ground and $\frac{5}{8}$-inch diameter bars inserted to mark the template boundaries. The bar was held in position well loaded and the end to be bent gripped with a clamp at the end of a long lever pulled by men who thus bent it to proper shape or hook round the fulcrum of sunk bars.

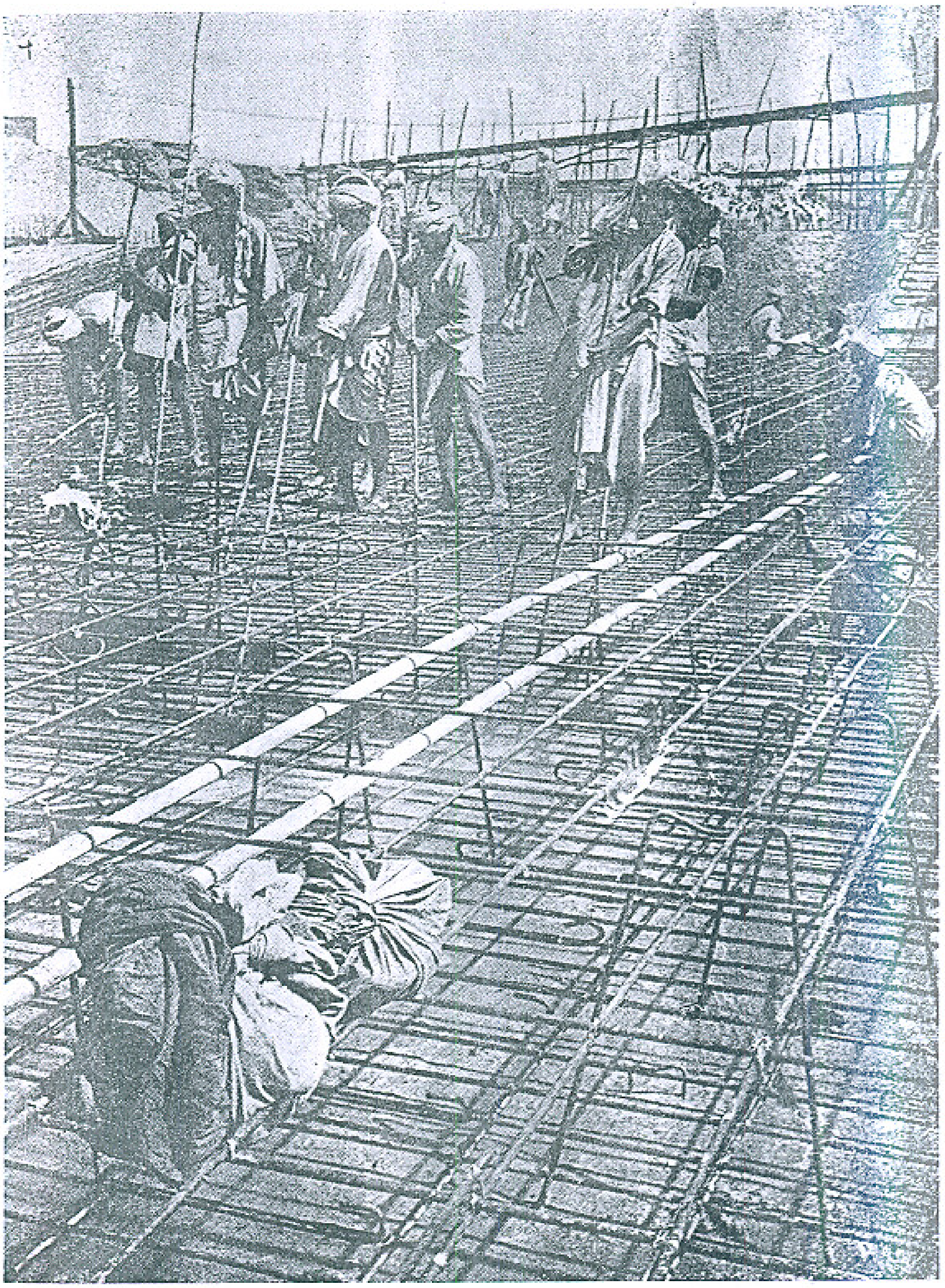
In the nine-inch layer or concrete laid at the bottom inverted *v* ( ) shaped stirrups were placed. The function of these was twofold.

Firstly they bonded the two concrete layers and secondly they formed a support for the bottom reinforcement. The top reinforcement was similarly placed on two-legged stirrups of the height of the floor. After the reinforcement was laid and tied the level of the top and bottom bars was corrected by adjusting the height of the stirrups. Before driving piles, holes were drilled into them about 15 inches below the top and bent bars were passed through these after they were driven with both legs projected up to be tied to the top reinforcement. Thus all reinforcement has been tied down to the piles. In the case of piles under divide walls holes had to be drilled at site to suit the position of reinforcement bars crossing from weir section to the undersluices and these holes were made by the use of an oxyacetylene flame.

In all 3,562 tons of reinforcement and 50 tons of G. I. wire for binding was used.

Concreting arrangements.—The general arrangement and organisation required for reinforced concrete floor is very much different from the ordinary one, and a passing mention of the same would therefore be not out of place. The concrete was mixed by mixers varying from $\frac{1}{4}$ cubic yard capacity to one cubic yard capacity. Sketch on Plate V shows the general arrangement of a mixer. The bins were all floored with dry bricks and sloped outside to permit the flow of all water from washing pumps. N. G. track was laid on the bin floor so that trucks could be run and mixers fed with the aggregate. The trucks were first filled with ordinary ballast up to a certain line marked on the inside of the trucks which was adjusted according to the quantity required for each kind of mix and then taken to the crushed ballast site for taking the requisite quantity in case of reinforced concrete. After this it passed in front of sand stacks from where the measured quantity of sand was loaded and then led to the mixer hopper. The water ratio required was determined from the workability of the mix and this had to be adjusted at different parts of the day depending upon the temperature. It was further dependent on the use of the type of the ballast and also the quarry from which it came. Slump and compression tests were carried out daily from the concrete of each mixer and the results were thus closely studied. High pressure two-inch pumps were used for washing the ballast and in the case of reinforced concrete there were separate pumps for the crushed ballast and ordinary ballast. The concrete from the mixers was led to M. S. Sheets on which it was dumped and from where it was carried by baskets to the site of the work. Scaffolding was erected in advance to permit coolies walking over these gangways before putting concrete at site.

Precautions Taken.—Before the work started every day the top of the nine-inch layer was washed with water jets from high pressure pumps and any water of this washing which did not find a natural



Getting ready for 1 : 2 : 4 concrete.

outflow was pumped out by suitable local pumps. The surface of the previous day's work was nicked and washed to afford good joints. (See photograph.) The reinforcement was checked, the spacing corrected, if necessary, and the laitance from the previous day's work, if any existing, was removed and, after all this had been done, the concreting for the day started. To ensure a proper covering over the bottom of the reinforcement, the concreting was done by two separate batches; the leading batch laid a foot deep layer at the bottom thoroughly wriggled to ensure its completely enveloping the steel before the second batch followed, completing the floor, 4 to 5 feet being the maximum distance permitted between the two batches. In no case was the lower layer allowed to set before the top one was laid on it. Men were continuously employed on cleaning the reinforcement and the cement mortar which might be sticking as a result of splashings and this ensured proper bonding of the concrete with reinforcement. To avoid any possibility of hollows at the bottom men were kept working under the reinforcement to keep on picking any loose ballast pieces which may be showing out whether at the bottom or on the sides as a result of the wriggling. It was observed that about 16 men were required for each mixer for wriggling with iron bars which were required to go down right to the bottom. In addition to this manual wriggling, mechanical wriggling was also done by vibrators which consisted of high-speed eccentrically driven cylinders by compressed air and thus gave a great throbbing motion to the concrete mass. To ensure efficiency the concrete was wriggled to an extent that it just flowed; over-wriggling was strictly guarded against. The day's work was generally ended under the piers in the case of reinforced concrete and small curtain walls were provided in the case of ordinary concrete to prevent sand escaping from under the concreted area. Separate supervisionary staff was employed to watch the mix, water ratios, washing of ballast and the working of mixers. At the site of concreting additional supervisionary staff was employed for each separate operation, gangways, washing, cleaning, concreting, wriggling, etc.

General.—The concreting was started on the 25th January, 1938, and by the 8th February, 1938, Bay No. 29 was got ready to enable the foundation stone being laid by His Excellency Sir William Herbert Emerson, the then Governor of the Punjab—4½ months after the first sod was broken. There were 16 one-cubic-yard mixers installed on the work (see Plate V) and although the average progress was about 30,000 cubic feet a day, a maximum of 54,000 cubic feet was recorded in a 12-hour working day on the 14th August, 1938. The progress in the case of reinforced concrete is limited on account of various conditions which have to be satisfied regarding cleaning and wriggling, etc., and cannot be the same as obtainable in the case of mass concrete where greater dumping space is available. The concreting of the floor was, however, completed on the 14th September, 1938. The total number of mixers in use was 16 of one cubic yard each, one of half a cubic yard and six of $\frac{1}{4}$ cubic yard each and the total horse power of the prime movers employed to drive these was 290.

Inverted Filters and Protection (Plate IV)

Between the end of the *pacca* floor and the downstream flexible protection, inverted filters have been laid which consist of six-inch crushed and graded ballast with 18-inch ordinary stone ballast over it. Over this two-foot ballast are concrete blocks five feet by three feet and four feet deep separated by $1\frac{1}{2}$-inch *jharies* from the adjoining blocks. These *jharies* are filled with $\frac{3}{4}$-inch *bajri*. The ballast below would permit the passage of the subsoil water to the top through *jharies* between the blocks and relief would thus be obtained without permitting the movement of sand, thereby ensuring an adequate cover for the down-stream line of pile at all points. The top surface of the inverted filter has been made proof against heavy dynamic action by putting an eight-foot deep curtain wall at the end which would hold these heavy blocks tight between the floor and this curtain wall. Beyond this curtain wall, further protection consists of blocks of the same size—$5 \times 3 \times 4$ feet lying over two-foot stone. These blocks are also separated by $1\frac{1}{2}$-inch *jharies* and would give a supplementary relief area for pressure and also protect the work against scour.

Stone Aprons.—The average discharge per foot run of the barrage is 645,000/3,026 = 213 cusecs; allowing 20 per cent. concentration for the upstream weir apron it comes to:

$213+42=255$. Now taking the value of $f=1$

$V=1.17\sqrt{R}$ therefore $q = 255=1.17\ R^{3/2}$.

Therefore, $R^{3/2}= 218$ or $R = 36$ ft.

Scoured depth upstream$=1.25\ R=45$ ft. (R. L. 48), *i.e.*, 22 feet below floor level.

In the case of the downstream weir apron,

$1.17\ R=213$ or $R=182$ or $R=32$.

Scoured depth downstream$=1.5\ R=48$ (R. L. 442) or 26 feet below the floor.

In the case of the undersluice the discharge has spread out at the apron site, hence the maximum discharge per foot run of 340 cusecs is assumed.

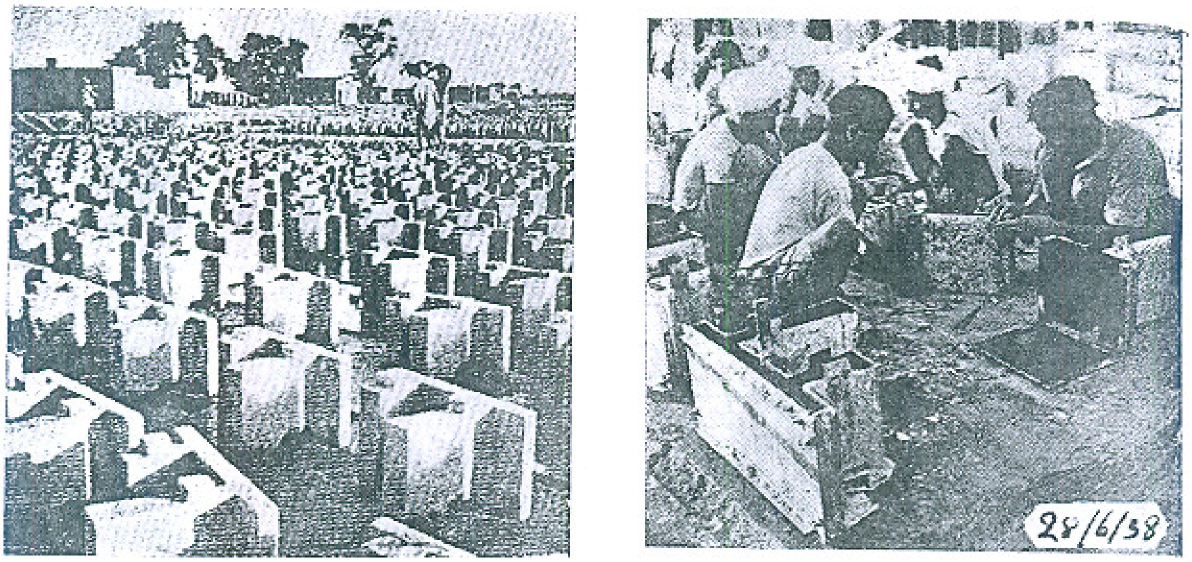
$R^{3/2}=340/1.17=290$ therefore $R=43$ and

scoured depth upstream$=1.25\ R=54$ ft. (R. L. 440)

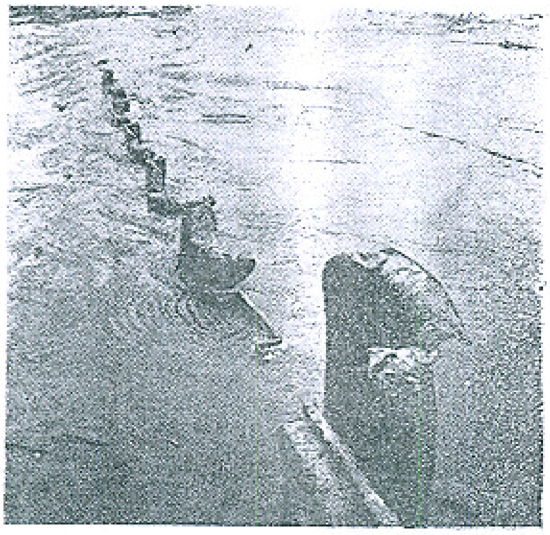
28 ft. below floor.

and downstream$=1.5\ R=65$ ft. (R. L. 424), *i.e.*,

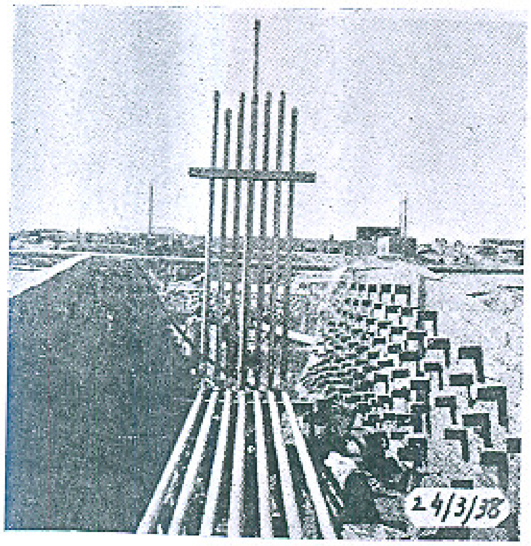
40 ft. below floor.



Precast concrete block moulding.



Driving of Larsen Piles. Note bent tops and unequal driving. (*See* para : "Piling.")



Erection of pressure pipes. Note brick masonry shuttering for pier foundations.
(*See* para : "Barrage Section.")

Observation Tower.
(*See* para : "Layout.")

The stone in the apron was kept which would be enough to line the scoured pit at a slope of 3 to 1 with 3-foot thick stone and hence the following quantities were worked out:

	Upstream	*Downstream*
Weir	$22\times9=198$ or 50×4	$26\times9=234$ or 50×5
Undersluices	$28\times9=252$ or 50×5	$40\times9=360$ or 60×6

Construction.—The blocks on the downstream side were constructed by building .4 foot thick cement masonry shells with odd bricks projecting inside and then concreting within the shells. The outer .4 foot thick masonry shell not only provided a shuttering but also a part of the blocks (see photograph).

On the upstream side the blocks were to serve as a protection against scour only and, therefore, there was no object in leaving any spacing between adjacent blocks. Masonry shells for alternate blocks were, therefore, prepared and concreted. The outer surface of adjacent shells thus gave the shuttering for the intermediate blocks which was mud-plastered to ensure their freedom from each other and concrete was dumped in. The work on stone aprons was carried on as stone became available from other important works, the only precaution taken being that the apron area was covered with a one-foot layer of stone as early as possible to avoid the filling up of the excavation by sloughing and to reduce the seepage and the pumping.

Superstructure

In the previous constructions generally the piers and other superstructure were constructed with stone masonry. This, however, was a laborious procedure and necessitated the employment of a large number of masons and resulted in slow progress. The superstructure at Trimmu had a novel feature in so far as the piers were to be constructed of concrete with suitable architectural effects as envisaged and tried during Khanki reconstruction.

Precast Blocks.—If the shuttering of each pier had been independent it would have again been a laborious and expensive proposition. To avoid this it was proposed to employ precast cement concrete face blocks and within these blocks hearting was to be done in concrete. This method of construction permitted the manufacture of such face blocks in advance and a lesser number of masons were required to set these blocks in proper position as the rest of the superstructure work was to be done with coolie labour as routine concreting. About 700 steel moulds were manufactured to prepare these face blocks and at times two to three shifts were obtained in a day. The total number of such face blocks was 1.71 lacs.

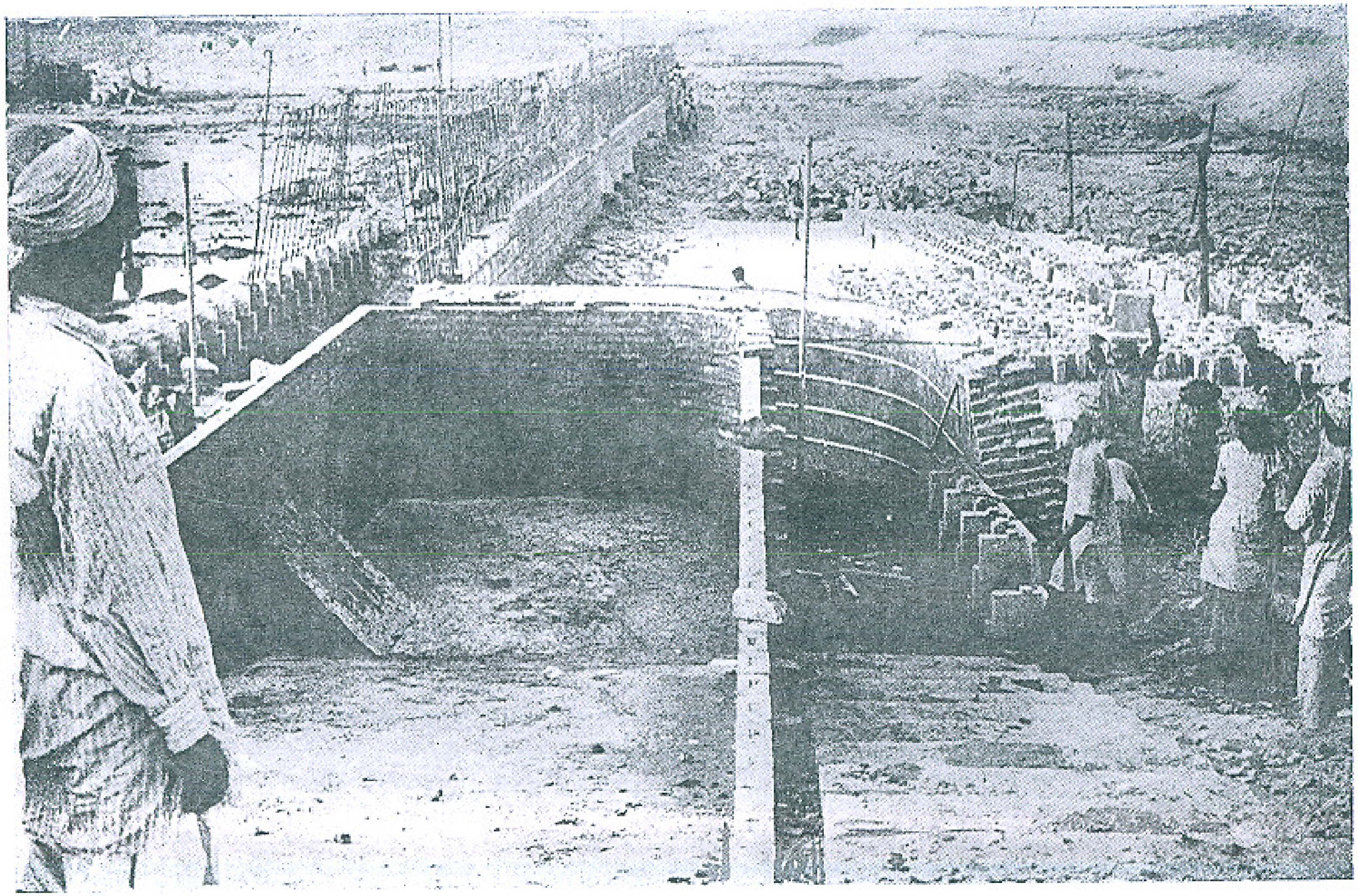
Tapering lugs were provided to these blocks, as shown in the photograph opposite, so as to get proper bonding with concrete. These blocks could be erected to a height of four feet at a time although two feet was the usual workable height and the hearting of the concreting was done with plenty of plums. On an average a pier could be raised by one foot per day and the progress could be multiplied as much as desired. The use of these precast blocks resulted in an economy of Rs. 2,40,000 in the Barrage besides enabling the work of the superstructure to be completed within a period of about $1\frac{1}{2}$ months from the completion of the floor. The precast face blocks were manufactured in two sizes, 2 ft. × 1 ft. and 4 ft. × 2 ft.

In Situ Work.—Where a large number of curved blocks required were of the same size, precast units were manufactured but there were places where precast blocks, if manufactured, would not have been repeated, and as such the cost of special moulds would have been excessive. For such places the shuttering of brick masonry with cement plaster inside was prepared and wooden *chapties* fixed on to it as shown in the photograph opposite to give the desired joint effect. The concrete on the face portion used was 1 : 2 : 4, whereas the hearting was the same 1 : 4 : 8 with the filling of the plums. A solution of soap and alum was generally given to the cement plaster after rubbing it to ensure ease in dismantling cement plaster without sticking. Another method of providing shuttering for superstructure concrete was the manufacture of properly stiffened and strutted wooden moulds with $\frac{1}{8}$-inch M. S. sheet inside and *chapties* screwed on to the sheet to give the joint effect. These were set and aligned at site and concrete poured in. It was observed that the general progress of a mason was laying 18 to 20 blocks per day although the start was very low. All coping work and other heavy block work was done with shuttering at the site.

Curing.—The concrete was generally kept wet for a period of two months after it was laid. Various methods were used to effect this. The floor was generally covered with sand which was kept moist by water from a jet playing all the time over it. Later, when the superstructure work started, pipelines were laid from the deliveries of pumps and all pipe lines were interconnected as far as possible. Some pipelines were fed independently by pumps. A large number of connections were taken out of this network of pipelines and the curing of the various portions was done by water jets playing on the completed work. Thus the water for curing was obtained both by gravity and by pumping. On the copings wet gunnybag tarpaulins were laid and kept continuously wet.

Flank Walls

Design.—In previous constructions the flank walls have been designed as gravity section and have been costly. An economic design of flank wall would have been R. C. retaining wall counterfort type but as the facing of these was to have precast blocks for the sake of uniformity from architectural considerations the flanks have been designed as gravity-*cum*-cantilever retaining walls.

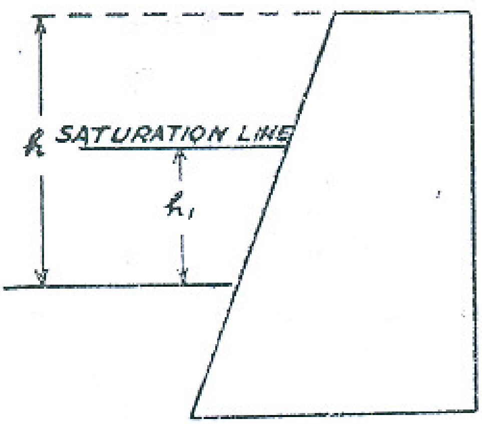


Divide wall construction showing shuttering for fish ladder entry.

The following assumptions were made in the design :

(C. B. I. Publication No. 12, page 138) :

1. The earth pressure plus full sand pressure due to the submerged weight of the fill with the same angle of repose as in dry conditions, which for sand was taken as 35 degrees. Thus the weight of the soil in submerged condition was taken as $W_S = W\ (1-E)\ (P-I)$ where E is the pore space $=40\%$ and P is 2·65 the specific gravity and as such $=62{\cdot}5\ (I-{\cdot}4)\ (2{\cdot}65-1)=62$ lbs. Similarly, pressure due to the fill in partially dry and partially saturated conditions is worked out as :



$$Wh_I + 100\ (h-h_I)\ \frac{1-\text{Sin}\ \phi}{1+\text{Sin}\ \phi} + W_s h_I\ \frac{1-\text{Sin}\ \phi}{1+\text{Sin}\ \phi}$$

$$=62{\cdot}5\ h_I + 27{\cdot}1\ (h-h_I) + 62\times{\cdot}271\ h_I = 52{\cdot}2\ h_I + {\cdot}271\ h.$$

The moments caused by above work out to :

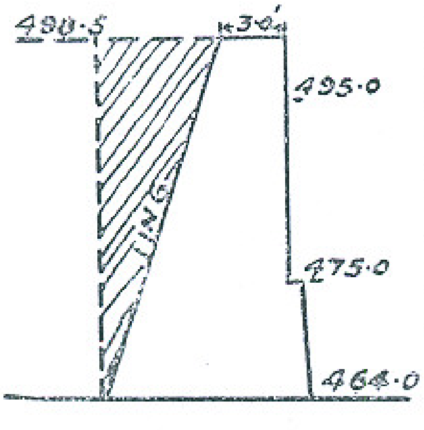
$$27.\ \frac{1xh^3}{6} + \frac{52{\cdot}2}{6}\ h_I{}^3 = 4{\cdot}5h^3 + 8{\cdot}7\ h_I{}^3.$$

2. 1 : 4 : 8 concrete will easily take tensile stresses up to 20 lbs. per square inch.

3. For the upstream flank walls the worst case arises when the pockets are empty and the saturation level is assumed as 485·0 although the pond level before would be 492·0 and canal water before closure would be 489·0

4. On the downstream side the saturation level has been taken as 480.0 : the worst case happens when downstream is dry at 464.0, canal water level at 489·0 and upstream pond 492.0.

The flank walls have a batter of 1 in 6 at the back but in calculating their stability the weight of the fill behind lying over this zone of batter has been neglected and adds to the safety. The reinforcement is designed to take balance of such moments exerted by the fill as cannot be taken by the wall without shifting the resultant outside the middle third. Analysis of actual steel provided shows the following :

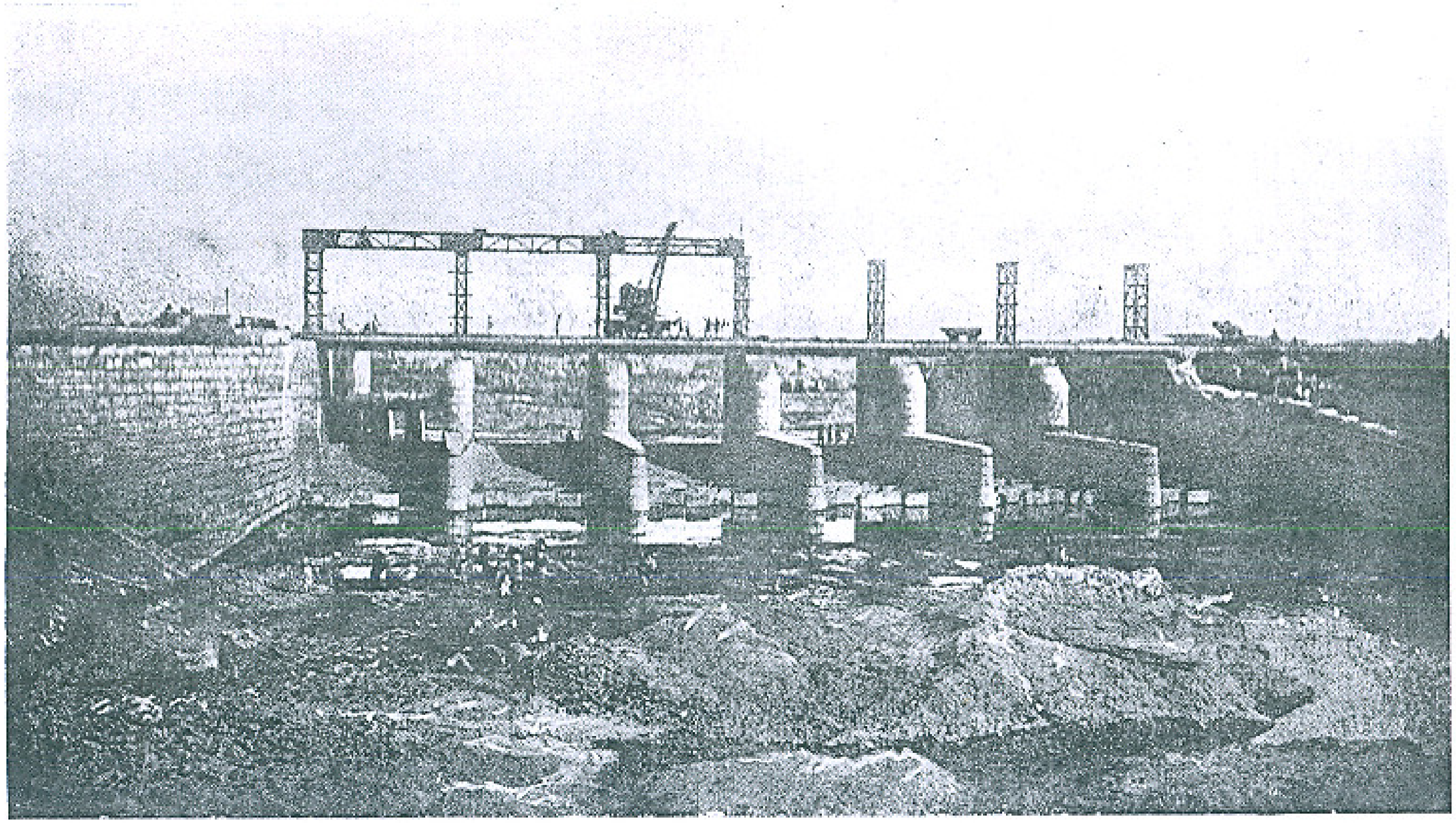


Upstream				Downstream			
R. L.	Rqd. Steel	Actual Steel	Stresses	R. L.	Rqd. Steel	Actual Steel	Stresses in steel
468	1.05	1.05	16,000	464	1.15	1.05	17,500
473	.64	.526	19,500	474	.48	.526	14,500
478	.34	.264	20,600	477	.358	.264	21,700
482	.17	*Nil*	Stresses in concrete 19 lbs. per sq. inch.	480	..	*Nil*	Stresses in concrete 19 lbs. per sq. inch.

The steel percentage in the section being very low, the excessive steel stresses have been allowed because in such cases the concrete in the tension zone contributes appreciably to the tension resistance. Taking the yield point stress in the case of Tatas tested steel as 44,000 lbs./sq. inch, the minimum factor of safety is about 2·0. During construction, when no bridge decking is resting on the abutments, it would behave like an ordinary flank section but later the worst condition would occur when there is no live load on the bridge, but there is heavy stationary axle load without impact close to the abutment. The maximum axle load of 15 tons on 18-inch wide wheel with $3\frac{3}{4}$-inch contact and 9-inch thick surfacing was assumed and pressure intensities worked out at various depths by taking the pressure spreading out and distributing itself at 45°. In considering the stability of the abutment section, the bridge dead load is also considered distributed at 45° till it spreads over the whole section.

Similarly, in designing the superstructure abutment, the worst condition has been taken when the gates and superstructure have been removed and the fill behind the abutment is subjected to the live load of the road traffic.

Footings.—Footings are required under the flank section which were available in the form of a raft on the downstream side and to which suitable reinforcement was added. On the upstream side, however, it was stepped out at the back by six-feet. Pressures due to soil reaction were computed by considering the weight of the flank, the weight of the fill on the footing, the saturated soil pressure from below and the bending moment on account of the pressure exerted by the fill from behind. The stepped-out footing was thus reinforced to take the stresses by the load of fill from the top and net soil reaction fom below.



Right undersluices from downstream showing developed blocks and silt-excluder.

The maximum pressure at the toe was also tested and came out to 6,250 lbs. per sq. ft. which was considered safe on account of the deep pile line under the flank wall which would keep the sand boxed and because of the ample supporting value of the sheet pile due to the skin friction.

Construction Details.—Precast blocks were used for face shuttering but at the back .4 ft. thick brick masonry shuttering was used which remained as a part of the flank section over and above that designed and gave extra cover to the reinforcement. In the flank wall as well as divide wall the reinforcement was to be placed at different batters. The correct spacing was ensured by the use of wooden spacers and the correct batter was ensured by fixing down the base of this reinforcement by about 4 No. $\frac{5}{8}''$ bars horizontally. These bars were raised as the height of the concrete rose.

Gauge Wells and Return Walls.—The flanks have been given return walls 40 ft. long going back into the filling and are founded on the pile line. Concrete gauge wells have been provided at the corners between the abutments and the return walls. The various walls have been designed in the same way as the flanks.

Development Blocks.—The flank ended vertically but had to be joined to the guide bank pitching at a slope of $1\frac{1}{2}$ to 1. To obtain this transition in the side slopes the developed block scheme used at Khanki has been attempted (see photograph). These consist of 1 : 4 : 8 concrete blocks 5 ft. × 4 ft. × 2 ft. placed one against the other and diverging out along the profile of development at each level. One-foot-thick stone and spawl filling has been provided at the back of these and weep-holes have been kept through these to drain the subsoil behind. These blocks are meant to settle in case of cavities behind. Concrete steps have been provided at the end of these blocks. The construction of the developed blocks was done by steel shuttering and that of gauge well and steps involved intricate *kachcha-pacca* brickwork shuttering.

Divide Walls

To feed the canals at all times, regular approach channels have to be maintained from the main river to the pockets and this is done by passing heavy discharges through the undersluices. Lest these channels should form close and parallel to the upstream weir apron and, with heavy discharges induced by the undersluices, cause damage, two 360-ft. long divide walls have been provided to keep these channels well away and to separate the weir from each undersluice. This gives upstream pocket dimensions which ensure the restoration of normal flow and silt distribution in the stream turning round the divide wall nose before it reaches the canal regulator. In addition to the above, another divide wall, 560 ft. long, has been put in in the left pocket, dividing it in two and creating a further pocket to stabilise the flow before it is met with the silt-excluder as shown in Plate III. The downstream weir floor is 4 ft. higher than the undersluice floor and on the downstream the deep channel is often to the undersluice side. To prevent cross-flow, 405-ft.-long divide walls have been provided, extending 275 ft. beyond the end of the weir floor from where it slopes down and provides a sufficiently long level stretch at the undersluice floor level along the weir side of the divide walls and thus avoid the probabilities of a turbulent cross-flow.

The noses of the upstream divide walls are generally subjected to heavy action as the flow turns round these. In order to spread the turbulence and reduce the scouring action the nose profile slopes down at 1 in 3 in steps of 1 ft. The nose of the upstream excluder divide wall is, however, kept vertical to give a longer reach for turbulence to subside before reaching the regulator faces. The noses of the downstream divide walls are also vertical which pushes the action further down (Plate IV).

The noses of the divide walls on the upstream and the extensions portion on the downstream are founded on wells which have been sunk to an increasing depth towards the end, the lowest level suck being 428.0 on the upstream and 424.0 on the downstream (Plate IV).

Various alternatives were considered for the divide wall sections gravity section with mass concrete, both for submerged and unsubmerged condition, R. C. cantilever section and R. C. box section sand-filled as at Ferozepur. The last was perhaps the cheapest but from practical and architectural considerations it was decided to adopt a mass concrete section of pier width and constructed with precast blocks and to provide reinforcement to make it stable against a head of 7 ft. with maximum pond on one side. 20-ft. footings have been provided where the wall is not founded on wells and this has been reinforced to spread the load.

Fish Ladders.—Two fish ladders have been provided, one along each of the weir divide walls. The total drop has been divided into falls of 1 ft. each, the floor starting from a level of 481.0 on the upstream and dropping to the undersluice floor-level downstream. The portion between the raft matresses and the floor slab of the fish ladder is filled with sand and the compartment walls are founded on this slab. The openings in the compartment walls are all 2-ft. wide and open to the full depth. The compartment walls and fish-ladder entry were all constructed with K. P. brickwork shuttering cement-plastered inside and with wooden *chapties* fixed to give the architectural effect of precast masonry (see photograph).

Well Sinking.—In addition to the sinking of the sumps, a large number of wells were sunk under the divide wall. Practically the whole lot was sunk by the use of Bull's Dredgers. The sinking of