PAPER No. 195.

RECONSTRUCTION OF THE KHANKI WEIR.

By Rai Bahadur A. N. Khosla, B.A., I.S.E., M. Am. Soc. C. E.

The Khanki Head Works are situated near the Khanki village, some nine mile downstream of the Alexandra (Wazirabad) bridge. As originally constructed, the head works consisted of a weir, across the river Chenab, of 8 spans of about 500 feet divided by 10 feet wide piers; a set of 12 undersluice spans of 20 feet on the extreme left; and the canal head regulator with 12 spans of 24·5 feet.

The Lower Chenab Canal, which takes off from this weir, comprises the most extensive and successful irrigation system in the country and perhaps in the world. It irrigates two and a half million acres each year and brings in an annual gross revenue of two crores of rupees.

The annual value of crops runs into some crores. The prosperity of some of the richest districts of the province is so intimately linked with the safety of this canal and its parent works, that any mishap to the latter would spell ruin to an enormous population and cause serious loss to the province.

The construction of this weir and the appurtenant works was completed at the end of 1891 and these came into operation with the first opening of the canal in March, 1892. The section of the weir as originally constructed is shown in Fig. I, Plate II and, as will be noticed, is extremely flimsy. It was done under conditions of extreme economy. The very next year, the floods in the river were severe and considerable damage occurred. A portion of the weir crest about 55 feet, in length subsided in Bay No. 5 due to the formation of a deep hole upstream of the crest. In January 1895 about 100 feet of the weir crest in Bay No. 1 subsided about 2 feet. The damaged portion was dismantled and rebuilt and the upstream floor extended before the flood seasons of that year.

A further subsidence of the crest, however, took place in October and November of that year.

The Khanki weir was the first weir in the Punjab to be founded on the alluvial sandy bed of a river and this incidence of failures, so soon after the original construction, led to the classic experiments at Roorkee of Col. Clibborn, Principal, Thomason Civil Engineering College, with Khanki sand, to investigate the laws of flow of water through subsoils below hydraulic works. In 1896 Col. Clibborn put forward, for the first time, that the Hydraulic Gradients along the path of flow should

form the basis of design. The history of failures, repairs and remodelling of this weir is, essentially, the history of evolution in design of weirs on sand foundations, and is, therefore, of special interest.

The repairs indicated as a result of Col. Clibborn's Experiments were carried out from November 1896 to May 1897 in Bays 1 and 8 and part Bay No. 7 (Fig. 2, Plate II).

Towards the end of 1897 further leaks and springs, throwing sand, were observed downstream of the masonry floors of Bays 4 and 5 and it was concluded that the work was further undermined.

Protection similar to that added in Bays 1, 7 and 8 was added to the remaining bays in 1897-98 (Fig. 2, Plate II).

After the additions of the upstream puddle floor protected at the ends by a line of wells or sheet piles and on the top by masonry blocks the weir gave little trouble and did not need any but the periodical repairs and renewals.

The cost of the main work to the date of completion in 1892 was :

	Rs.
Weir	13,17,436
Undersluices	4,98,355
Regulator	1,78,120
Protection Works	6,61,236
Railway line	1,83,590
Total	28,38,727

The cost of additions and heavy repairs to the weir up to 1920-21 was Rs. 11,65,241 in addition to Rs. 31,23,000 spent in works for silt control, *viz.*, raising the crest by 4 feet, constructing the subsidiary regulator, raising the regulator crest, remodelling of the main regulator, constructing the guide bund, paving the flume etc.

The crest of the weir was raised by 2 feet in Bays 4, 5 and 6 in 1910-11 and in Bays 1, 2, 3, 7, 8 in 1916-17. It was again raised by a further 2 feet, to the existing crest level, in 1919-20.

After the record flood of 1903 (7,12,000 cusecs), proposals for extensive additions were put forward by the Chief Engineer, but were turned down in the Government of India.

From 1903 onwards, while there was no repetition of the failures of 1893 and 1895, the undermining of the impervious floor of the weir bays continued in varying degrees. Springs appeared in Bay No. 8 in 1904-05 and extensive grouting had to be done. In 1911-12 cracks appeared in the impervious floor of that bay and further grouting was

done. In 1923-24 springs appeared in Bay No. 7 and were stopped by grouting. In 1927-28 Bay No. 8 had to be grouted yet again. In 1929-30 springs appeared in all the bays from 1 to 7 and were closed by grouting. In 1929-30 all the weir bays were again grouted and the process was repeated in 1930-31.

With these periodical repairs the weir was kept going but the process of undermining was not arrested and large cavities continued to form but remained undetected and ungrouted.

Need for reconstruction.

The years 1928 and 1929 witnessed, in succession, the two highest floods on record, with discharges of 7,50,000 and 7,47,000 cusecs respectively. In addition to the damage to the training works, the downstream glacis of Bays 3 and 5 had part of the grouted pitching displaced and signs of disintegration of mortar of the top masonry were apparent at many places.

In July 1932, serious damage occurred to the protection downstream of the undersluices. In the right half the block protection was completely carried away and a scour hole formed which extended 27 feet below the floor level and ten feet below the bottom of wells. The scour hole was immediately filled up with stone and repairs of a permanent nature were carried out later.

Serious damage also occurred in August of that year in Bays 3 and 4. The damage was discovered by a head boatman stumbling into a hole in the glacis while attempting to catch fish. This was also disclosed by the distorted appearance of the standing wave.

In Bay 4, in almost half the length, the curtain wall below the impervious floor was carried away—the rubble masonry in a very large area was shattered almost right up to the crest. In Bay No. 3 the downstream curtain wall was undermined and a length of 30 feet of it stood as a beam. A patch of rubble masonry above it was gone.

While the damage to rubble masonry and grouted pitching was extensive, the loose stone protection remained practically intact. The damage, therefore, could not be attributed to the retrogression of levels downstream. It appeared to have been a *clear case of undermining of the subsoil by piping*. A series of pressure pipes, which had luckily been inserted in the weir before the flood season, showed practically no resistance below the crest in some of the bays. This, coupled with the fact that extensive grouting had had to be done in previous years in almost all the bays, confirmed the belief that the undermining of the subsoil under the weir had progressed to a dangerous extent. Large cavities had evidently formed under the weir which threatened its stability. This if not definitely and immediately secured would spell ruin to the millions of acres of cultivated areas on the canal dependent for their irrigation on the safety of this most important weir of the province

In a case, where the stakes were so big, a halting policy of wait and see and any stinting of expenditure might have resulted in disaster, in excessive expenditure as unseasonable times and in bitter regret that work had not been taken in hand earlier.

Such were the reasons which led to the Government's sanctioning the comprehensive scheme of reconstruction put forward by the Superintending Engineer, the late Mr. H. W. Nicholson, C.I.E.

The state of the weir floor and foundation subsoil as disclosed during dismantling and excavation showed beyond doubt that the work of reconstruction had been undertaken none too soon.

Scheme of reconstruction.

The scheme of reconstruction, which forms the subject of this paper, aimed at securing firstly, the safety of the weir and secondly, the exclusion of harmful silt from the canal.

Undersluices.—The damage to the downstream floor of the undersluices was thoroughly repaired. New 17 feet gates were installed in the undersluices instead of the old 14 feet ones.

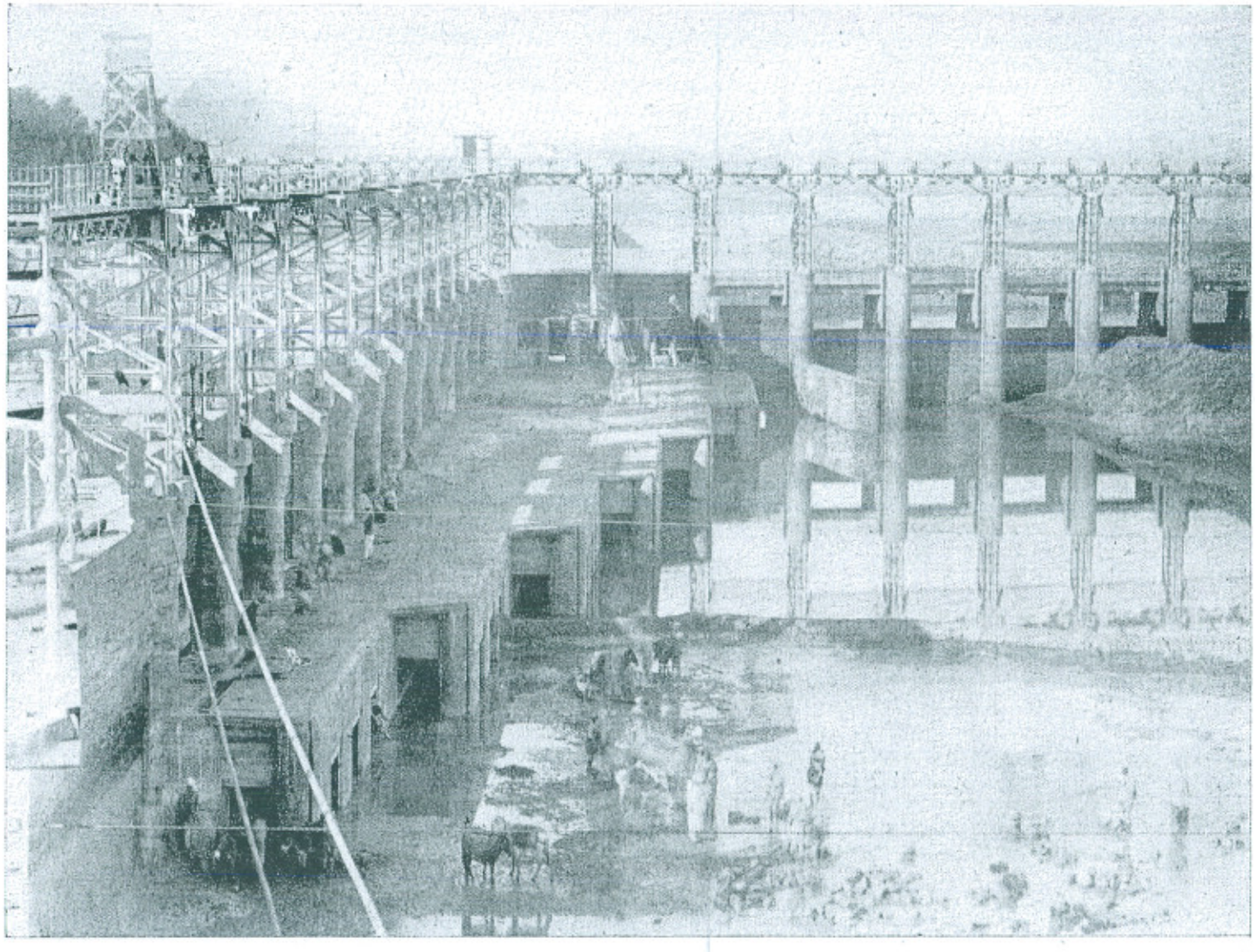
Weir Bays.—In the six bays 1 to 3 and 5 to 7, constituting a length of over 3000 feet of the weir, the entire impervious floor—with the exception of the crest block and part of the glacis down to the first toe-wall—has been reconstructed (Plate III, Fig. I) and protected at the upstream and downstream ends by a continuous line of interlocked steel sheet-piling. The floor below the downstream pile line upto the block area has been reconstructed as an inverted filter. The block and stone protection has been repaired and relaid, as necessary, along the entire length.

Central and Right Undersluices.—Bays 4 and 8 have been entirely reconstructed (Plate II, Figs. 2 and 3) with the crest depressed 12 feet below that of the weir so as to work as additional undersluices with 18 spans of 20 feet. Each sluice is fitted with counter-balanced steel gates 18 feet high. The additional water-way provided by these depressed bays will relieve strain on the weir during very high floods.

The old groynes No. 3, 4 and 7 have been reconstructed as divide groynes, to separate the high floor of the weir bays from the depressed floor of the Central (Bay 4) and Right (Bay 8) undersluices.

Bell's Bunds.—The aprons of the right Bell's Bunds have been depressed generally 14 feet to fit in with the newly depressed floor.

Silt tunnels.—In addition to the above, a set of six silt Ejector Tunnels have been constructed in front of the Main Regulator, for silt exclusion. (Photo 2.)



An experiment in silt exclusion from canal.

Three of the tunnels were constructed in 1932-33 December and January closure and these were extended and three more added in 1933-34 closure. These tunnels are stepped in plan. Each tunnel is intended to serve the two bays opposite its top end. They are 10 feet high and 7 feet wide inside and have one opening each 5′×5′ in front and three openings of the same size on the side, in order to extract silt from the bottom water approaching in front or on side. The roof of these tunnels consists of precast reinforced concrete units, assembled and strengthened to take the full load by addition of suitable depth of concrete and stirrups at site. By the use of precast units, it was possible, in addition to the construction of floor and partition walls, to roof a 1000 feet length of tunnels covering over 8000 square feet of area, in 21 days closure inspite of rains and bitter cold in the months of December and January.

These tunnels have proved to be quite effective in silt exclusion. Their efficiency is being carefully tested since 1934.

Silt exclusion.

The silt trouble on this canal is as old as the canal itself. It became acute in 1907 and 1909 when large sums of money had to be spent in silt clearances. In 1910-11 the control of the river was lost. The canal supply passed over the weir on the right flank, along the downstream of the weir and back through the undersluices into the canal Regulators. The weir crest was consequently raised by 2 feet in bays 5, 6 and 7 and the subsidary regulator of 6 bays of 24·5′ was added to the main head regulator. The cills of the former were made at R. L. 721. Those of the main regulator, which were originally at R. L. 717, were raised to R. L. 721 in 1911-12.

The *still pond system* of regulation, which had proved so successful at Rupar, had been in force at Khanki till 1910-11. But sufficient attention was, apparently, not given to the periodic and adequate scouring of the pocket flume with the result that the approach was silted up and the control of the river was lost as stated above. This method was consequently discarded in favour of the *open flow system* according to which a suitable quantity of water had to be continuously escaped through the pocket to keep the latter reasonably clear of silt while the canal was in flow.

In 1916, however, the trouble again became acute. The still pond system was again to be introduced but could not be tried due to lack of command in the river. The weir crest was consequently raised by 2 feet in 1916-18 in the remaining 5 bays.

Despite this raising of the crest, the silt trouble in the canal still remained acute and a further raising of the crest by 2 feet was done in 1920-22. The crest in bays 1 and 2 is at R. L. 726·5 and in the remaining bays at R. L. 727·0.

The regulation continued to be done on the open flow system and contrary to the original intention, unduly high discharges, generally up to 30,000 cusecs, were passed through the undersluices. This resulted in undue development of the left channel and in almost complete choking off of the right channel. Large quantities of silt found their way into the canal during high supplies. Similar trouble has been experienced at the other headworks when the main river flowed through the left channel—the supply channel for the canal.

Water flowed in high floods from the left to the right parallel to the weir. During the record flood of 1928 the river level over the left was three feet higher than that on the right. The upstream groynes of as many as four piers failed due to parallel flow.

In 1931-32 a start was made to develop the right channel by linking the middle of Bay No. 5 by means of a bund with the upstream *bela* (dividing the two channels) which extended for 2 miles. By thus preventing parallel flow and introducing steeper slope in the right channel, the latter did develop somewhat but not to the extent desired due to dissipation of head round the head of the guide bank. In 1932-33 the existing right channel was closed off by means of a bund near Spur E and a straight cut, in line with the Guide Bund Head and 60 feet wide, was excavated. With the rising supplies this took charge and developed into a width of 1200 feet so that it carried very nearly the same supply as the left part of the weir.

This development of the right channel created benign conditions for controlling the entry of silt into the canal. The left channel at the bifurcation above the '*bela*' takes off on the outside of the curve. The major part of the silt charge was thus carried into the right channel which took off on the inside of the curve.

Similar conditions of curvature of flow were sought to be established above the approach to the pocket so that the latter could get the relatively silt-free supplies from the outside of the curve. That could be secured only by depressing a bay in the middle of the weir. *The Bay 4 was accordingly converted into a depressed and gated bay—the Central Undersluices.* With the construction of Bay 4, the supplies for the flume would undergo a double decantation before entering the canal, firstly at the bifurcation of the left channel at the nose of the 'bela' and secondly, at the bifurcation of the approach channel to the pocket flume from the one leading to the Central Undersluices.

To maintain the curvature of flow at the nose of the bela, the draw in the right channel must necessarily be maintained. This could only be done by depressing and gating a bay on the right. The *Bay* 8 was the most suitable for the purpose and this bay *was consequently depressed and gated as the Right Undersluices.*

For efficient functioning of the above scheme, it is also necessary that the bela should remain stable as far as possible and the nose held in its present place. This could only be secured with doubtful certainty—by extensive protection which was found to be prohibitive in first cost and in subsequent maintenance. This part of the work was, therefore, dropped. A shifting bela necessarily adds to the silt trouble in the canal.

The ideal condition of curvature would be if the river were to come from the right along the weir and feed the pocket past the divide groyne. In this case, the weir bays will all abstract from the inside of the curve so that there will be decantation at every bay and the maximum amount of silt will be excluded from the water before it enters the pocket. Such a condition exists at Marala at present.

Having arranged for the maximum exclusion of silt from the water entering the flume, the next step was to exclude still more silt from the water entering the canal. For this purpose the 12 main regulator bays have been equipped with 6 tunnels with roof at R. L. 726 or 5 feet above the raised crest level of 721. The crest of the main regulator has thus been permanently raised to R. L. 726. The subsidiary regulator has its crest still at R. L. 721.

The method of regulation now in force, is what may be called the *partial still pond system*. The tunnels escape about a third of the discharge going into the canal. To get the best result, it was essential that the floor of the pocket should be free from obstructions and reasonably smooth so that heavily silt laden waters in the bottom layer could be escaped through the tunnels; any local obstruction in the bed would set up eddies, churn up the silt in the entire vertical section and render the tunnel ineffective. This clearing of the bed has been mostly done.

The original gates of the undersluices were only 14 feet high and with the raised crest made still pond regulation impossible. Those gates have been replaced by new ones 17 feet high.

The measures for silt exclusion which formed part of the scheme of reconstruction consisted of:—

(*a*) Development of the right channel and the construction of the Right Undersluices by depressing and gating Bay 8 to maintain draw in the right channel and curvature of flow at the nose of the bela.

(*b*) Construction of Central Undersluices by depressing and gating Bay 4 to maintain curvature of flow at head of the approach channel to the pocket flume.

(*c*) Construction of Silt Ejector tunnels which automatically raised the crest of main regulator by 5 feet, *i.e.*, to R. L. 726·0.

(*d*) Clearing the bed of pocket and paving the floor where necessary.

(*e*) Installing new 17′ gates instead of the old 14′ ones to enable still pond system of regulation to be worked.

With the construction of the above works and the partial still pond system of regulation it has been possible to effect a considerable improvement in the silt condition in the canal and if the anticipations are fulfilled, the silt trouble on this canal may disappear and the stable conditions obtaining on the Sirhind Canal may be obtained in this case as well.

It may be noted that during the high supply period (summer months) the subsidiary regulator is closed and the entire supply is fed through the main regulator. In the winter when there is no escapage through the tunnels, the supply is fed through the subsidiary regulator to keep the pond level down.

Before leaving the subject of silt exclusion, it may be stated that the withdrawal of silt from water by a canal from an alluvial river causes a change in hydraulic conditions downstream due to the fact that the lessened discharge below the weir has a relatively greater silt burden than was the case before the construction of the weir and the canal. The channel, therefore, adjusts itself to this condition by building up its slope and raising the levels of the bed and water surface downstream of the weir. But it takes some time before this increase of levels develops.

Upstream of the weir the effect of the pond, initially produced, generally dies out in course of time owing to the fact that the river endeavours to regain its original slope and, in so doing, increases its tortuosity.

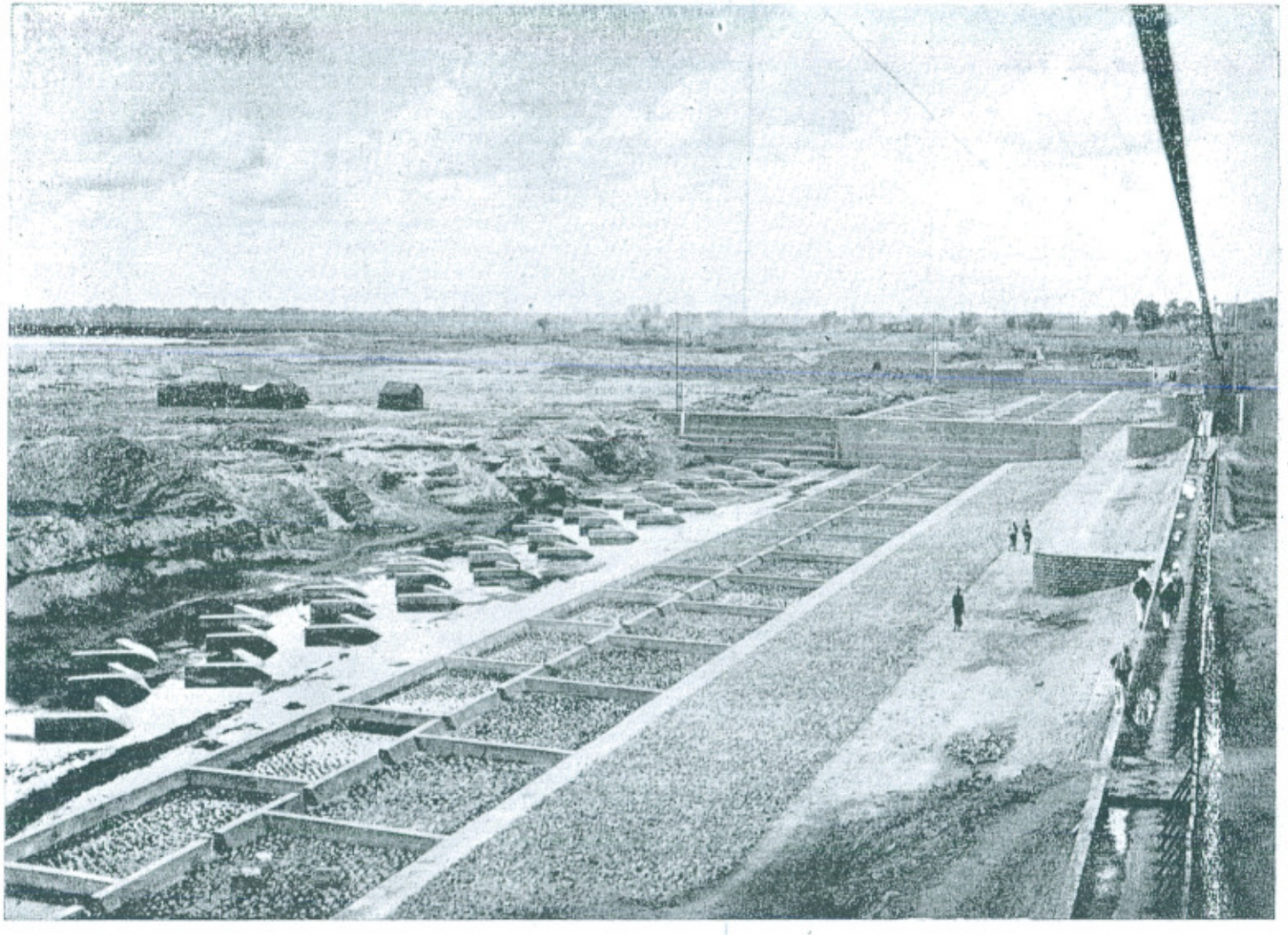
Owing to the rise below Khanki, the weir has been raised three times, as stated in the preceding pages and it is not unlikely that in course of time—perhaps a generation hence—the weir may have to be raised again.

Reconstruction of Weir Bays.

As has been stated earlier in the paper, the six bays 1, 2, 3 and 5, 6, 7 have been reconstructed as weir bays with crest level remaining unchanged. (Plate III, Fig. I, Photo No. 3.)

Impervious Floor.—Of the impervious floor, the crest block and the glacis down to the first toe wall has been left intact. The rest of it below this toe wall to the block area has been dismantled and reconstructed as impervious floor down to the second toe wall and as inverted filter below that wall.

RECONSTRUCTED WEIR BAYS DOWNSTREAM



Showing toothed glacis surface, stepped compartments and arrows.
Note the motor road way and gaps in groynes.

On the upstream side of the crest, a line of "Universal" interlocked steel sheet-piles has been driven 25′ to 30′ deep. In Bays 3 and 7, this pile line was driven 2′ upstream of the crest block but the vibration due to the driving of piles so close to the crest, through the adjoining stone (which still exists in most places) and clay substratum, gave rise to several cracks in the glacis and the crest. The cracks were specially pronounced where the foundation was hollow. Consequently in the remaining bays this pile line was removed 20′ from the crest block. The cracking of the floor in these latter bays was thus prevented.

The upstream pile line is connected to the crest block by means of a reinforce concreted pile cap 1.5′ thick which is tongued in to the crest block with the cross bars carried about 3′ further into specially jumped holes and grouted. (Plate III, Fig. 4 and 5.)

This pile cap which rested on sand is protected against cracking by a layer of 6″ to 1′ thick puddle covered with 1.5′ thick concrete blocks.

The newly constructed impervious portion between the first and second toe walls consists of a 9″ thick layer of 1 : 3 : 6 concrete resting on properly watered and rammed foundation sand, 21″ thick layer of coursed rubble in 1 : 4 mortar and top one foot of toothed kharanja in 1 : 3 mortar. The toothing extends about 3 inches above floor level. The bottom layer of concrete ensures perfect bedding on the sand which is extremely difficult to secure with stone masonry.

To prevent the various layers separating from one another, the surface of each layer was properly nicked, thoroughly cleaned with wire brushes and smeared with neat cement grout before the next layer was laid on. In case of masonry layers all joints were raked in addition. In each case the surface of stone or ballast, as the case may be, was bared and all loose mortar and laitance was removed.

In Bays 5, 6 and 7 vertical reinforcement bedded in the bottom layer was put in as an additional safeguard. Also binder or key stones were inserted in each layer projecting in the one above. With these precautions, *viz.*, nicking, cleaning, cement grout, binder stones and vertical reinforcement, it is hoped that the tendency of the layers separating from each other has been effectively checked. This feature is described in greater detail in connection with the two depressed Bays (4 and 8).

At the end of the pervious floor and tied into it by reinforcement is a line of interlocked steel sheet-piles (Ransome D) 15′ or less in depth depending on the depth of the clay substratum below the floor level. In all cases a minimum clearance of 2′ has been allowed between the bottom of the piles and the top of the clay substratum, in order to permit free departure of seepage water which may have entered the foundation

sand from the upstream. Where the clay substratum exists, the upstream pile line either beds into it or penetrates it. The upstream piles bedding into clay is an advantage as they cut off most of the seepage below the weir and thus largely reduced the uplift pressures due to seepage flow under the floor downstream of the crest. If the downstream pile line were also bedded into clay, any seepage flow from upstream will be dammed up above the downstream pile line resulting in abnormal pressures and blowing up of the floor. This point is of utmost importance where clay substratum is met with close below the floor level. At Khanki this clay substratum exists at or above the piles' bottom in the undersluices, left quarter of Bay 1, right three quarters of Bay 2, and the whole of Bays 3, 4 and 5. (Plate III, Fig. 8.)

The Inverted Filter.

Below the downstream pile line, the reconstructed floor is an inverted filter consisting of 1′ bottom layer of washed *kankar*, overlaid with 2′ depth of sand-grouted stone covered by 1′ deep toothed *kharanja* in 1 : 3 cement mortar. The relief is afforded by means of 1½″ diameter galvanised iron pipes bedded into the bottom *kankar* layer at 5′ intervals.

In Bay 3 the inverted filter follows the 1 : 15 slope of the original glacis. In the remaining bays it is in two steps with tops at R. L. 716.5 and 715.0 respectively.

The idea of having the filter in two horizontal steps instead of on a slope of 1 : 15 is that in the latter case the sand from the higher portion may get worked into the *kankar* in the lower portion thus disturbing the filter and causing cavitation in the former. This also explains the necessity for sand grouting the 2′ stone layer above the *kankar*.

The toothing of the *kharanja* over the inverted filter is intended to dissipate the energy of the bottom filaments of water which do most of the damage to the protection lower down. The *kharanja* stones were placed vertically with wide joints which were very carefully and thoroughly filled from top in 1 : 3 cement mortar. This vertical filling of joints has been found to be the most effective remedy against hollow joints.

It should be borne in mind that the inverted filter area is pervious but not flexible.

Flexible Protection.

Below the inverted filter is the flexible protection of 40′ length of blocks followed by stone apron 60′ long by 4′ to 5′ deep. In this area damaged portions have been repaired. Where blocks have been relaid on any large scale, they have been given an under layer of spawls and stone 2′ thick. Where the stone apron has been relaid, an under

layer of spawls 1′ thick has been added below the 4′ depth of stone. It may be mentioned that on excavation only 40 % of the supposed quantity of stone in the original apron was recovered.

An underlayer of spawls or ballast below aprons and under slope has been found most desirable as it prevents the foundation sand being sucked out due to local surging or wave action.

Energy Dissipation.

A series of arrows (Photo No. 3) have been constructed on the block area. These are tied into the blocks by means of vertical reinforcement. The idea of arrows originated with the late Mr.H. W. Nicholson, C.I.E. These have proved very successful so far. There was practically no damage to the talus during the last two flood seasons.

These arrows throw up and deflect the bottom high velocity jets to the top and reduce action on the protection downstream. The toothed surface of the floor, the stepped compartments and the arrows form an excellent combination for dissipation of energy of the bottom jets.

Cavities & Grouting.

During the course of dismantling, cavities were found, more or less, under the floor in all the bays. In some cases lengths up to 100′ and 200″ had clean hollows 2″ or so deep, the floor apparently standing as a slab on sand columns. Deposits of clay upto 1½″ in thickness were quite common under these cavities and at places there were more than one layer of such clay. Elsewhere the clay had subsided leaving a hollow between it and the floor. These clay bands and cavities extended right down to the second toe wall which had consequently to be dismantled in considerable lengths. The loose stone under the glacis downstream of the second toe wall was packed with sand and clay.

The continuous inverted triangle of loose stone which runs across the weir downstream of the crest block has been in most cases found packed with clay.

All these point to the fact that there has been more or less open flow, as distinct from seepage flow, under the various bays of the weir.

The existence of the inverted triangle of loose stone must have materially accelerated the process of undermining. Effective cut-offs have now been made in the centre of each span where the floor above the first toe wall was opened to sand level, loose stone removed and the trench filled up with cement concrete resting on foundation sand and butting against the masonry of the crest block. Incidently these cut-offs afford foundations for intermediate central piers should they have to be constructed at any future date to enable, by reducing the spans to half, heavier loads to be carried across in the aerial trollies.

The biggest cavity discovered was under Pier 5 (Photo 1 frontispiece). This cavity extended at least 20′ on one side of the pier and 13′ on the other and was any thing upto 13″ in height. The bed underneath was in crests and furrows with occasional heavy clay bands indicating considerable amount of open flow. Vigorous springs had been previously noticed in Bay 6 along Pier No. 5 downstream. The cavity was most pronounced under the glacis above the first toe wall. It took 900 cft. of cement and sand to fill up. Other similar cavities took any thing upto 350 bags of cement grouted under pressure.

While the floor below the first toe wall was rebuilt all afresh, the floor upstream of this had been opened only in a few places, generally in the middle of each bay. Some of the cavities in this area were discovered and grouted but there was no means of ascertaining the state of the foundation under the entire length of the weir. As a first step holes were made at 10′ intervals just upstream of the first toe wall, and cement grouted into the foundation below the floor pressure. Where no cavities existed close to the grout hole, no cement could be forced in, but, where it was otherwise, several hundred bags of cement went in. Before grouting, all holes were tested for leakage into cavities by pouring water by cans. Where cavities existed the water disappeared so much so that in many cases 1½″ pumps were worked for considerable periods and the pumped water just vanished somewhere.

Except Bay 1 which took just over 200 bags, all the other weir bays took between 1000 and 1200 bags of cement each. In this way the cavities have been possibly filled up just upstream of the first toe wall but there is no certainty that the entire glacis above this has been grouted into a sound condition. Isolated cavities in this area cannot do much harm specially as the foundation is boxed in between the sheet piles and the bulk of the floor is new.

The fact has, however, to be kept in mind that cavities possibly still exist in the portion not reconstructed and a programme of grouting this area appears to be called for. The expenditure involved will be small.

The grouting was done by the grouting machine, worked by men like a contractor's pump. This forced the grout at about 100 lbs. pressure. It can work satisfactorily only with neat cement or a very small admixture of sand. To avoid clogging of the flexible hose in cases where a mixture of sand and cement was used, the hose was replaced by a short length of G. I. pipe which was connected to the grout pipe by a union.

In the second season, a Canif grouting machine was obtained from the Hydro-Electric Department. With this machine they had done the grouting of the tunnels. It is worked by an air compressor and is very quick and effective. Mixtures upto 2 sand to 1 cement were successfully forced into the cavities. The working costs were heavy but it worked out cheaper in the long run as it replaced so much cement by sand. The

use of this was, however, given up as it was found that the pressure under which the grouting was done tended to lift the 3″ to 4′ thick floor.

Under each groyne separating the weir bays, a line of piles 15′ deep has been driven from the downstream wooden crates to the first toe wall. This is to segregate the various bays.

Central undersluices and right underslueics or depressed and gated Bays 4 and 8.

The Bay 4 was depressed and gated in 1933-34 and the work completed early in June 1934. Bay 8 was done in 1934-35 and the work completed by the end of May 1935.

The design in the two bays is similar. The depths of pile lines and length and thickness of floor were determined in accordance with the author's recommendations contained in P. E. Congress Paper No. 162 (Pressure Observations at Panjnad Weir). The section of floor is designed for a maximum head of 19′ *viz.*, upstream pond level at 733 or top of gates and downstream water at R. L. 714.0.

In Bay 4 a substratum of clay was met, with top at approximately R. L. 696.0 and 2′ to 4′ thick. In Bay 8 the subsoil is all sandy down to the bottom of the lowest pressure pipes at R. L. 680 except at one or two local patches. One bore hole was carried down to clay at R. L. 659.

These two bays have been depressed with crest at R. L. 715, *i.e.*, the same as that of the left undersluices and 12′ below that of the weir. The upstream and the downstream floors have been laid at R. L. 710.

The section of the floor consists, from upstream down, of :—

1. 34′ ×4′ of stone apron, the first 4′ being in wire crates ; top R. L. 710.
2. 50′ of 2′ deep cement concrete blocks over 2′ of puddle and sand-grouted stone. This is at R. L. 710 in Bay 8 and slopes from R. L. 710 to 715 in Bay 4.
3. A line of old wells with bottom R. L. 701.0 in Bay 4 with intermediate spaces (*jhiris*) filled up now with cement concrete down to R. L. 704 ; and a line of 15′ deep steel sheet-piles in Bay 8 with bottom R. L. at 691.0.
4. A width of 156 feet of impervious area in cement concrete with 1′ thick reinforced concrete layer on top. The reinforcement consists of $\frac{5}{8}$″ bars 9″ centres 2$\frac{1}{2}$″ below the surface. Under the piers and some distance on either side, a second grid of $\frac{5}{8}$″ bars 9″ centres is laid 9″ below the top. The upstream floor in Bay 4 is at R. L. 715 and in Bay 8 at 710, rising to the crest in a slope of 1 : 5. On the

downstream, the glacis slopes from the crest at 1 : 5 in both bays and is followed by the horizontal floor at R. L. 710.

5. A line of Universal steel piles under the crest with bottom R. L. at 689.0 to 688.0.
6. In case of Bay 4, a second line of piles is driven 5′ below the crest line and is bedded into the clay substratum. As the crest piles penetrated the clay and there was danger of the pressure below clay being transmitted into the sand below the floor, the second pile line was considered necessary.
7. A line of piles at the end of the impervious floor and tied into it by vertical reinforcement. This pile line goes to R. L. 789.0 in Bay 8 and to 2′ above clay substratum in Bay 4. The clay substratum top is at about 696. 0.
8. A series of arrows on the impervious horizontal floor.
9. An area of 4′ deep blocks, 50′ wide in Bay 4 and 49′ wide in Bay 8, with some blocks projecting 1.5′ above the rest. The blocks rest on 1 foot of *kankar*-grouted stone on 1′ of washed *kankar*.
10. An area of loose stone 100′ × 6′ in Bay 4 and 80′ ×6 in Bay 8.

Both bays consist of 18 spans of 20′ divided by 5 thick piers. These are divided into 3 groups of 6 spans each, the divide piers being 7′ thick.

In both bays a cross line of piles runs under the divide piers linked at the upstream end to the crest pile line and at the downstream end to the downstream pile line. In Bay 4 this cross pile line beds into clay and has its bottom at about R. L. 695.0. In Bay 8 this bottom level is 696.0.

A double line of piles runs under both the left and right abutments in both bays. These are linked at the upstream end to the line of wells in Bay 4 and upstream pile line in Bay 8. At the downstream end they link on to the downstream pile line. The deep piles under the abutments extend 340′ under upstream groyne No. 4, 80′ under No. 3 and 130′ under No. 7 beyond the upstream line of wells or piles. Under the downstream groyne in Bay 4, these extend 120′ and in Bay 8,105′ below the downstream pile line. In the case of groyne No. 7, a line of cut pieces of piles about 8′ deep extends to the nose. A similar line of cut pieces has been driven under the developed blocks upstream.

It will thus be seen that the entire impervious area is boxed in by means of interlocked steel sheet-piles and cut up into smaller boxed-in compartments by the cross pile line under the divide pier. The use of double line of piles under the abutments boxes in the foundations of the

latter independently and was greatly helpful in speeding up construction. The original piers 3, 4 and 7 have their foundation at R. L. 714. The foundation level of the new abutments, close to the piers, went down to R. L. 708 or 6′ lower, within a few feet of the former. (Plate III, Fig. 6.)

The concreting of the new foundations several feet below the bottom of the foundation of existing work and within such small distance was attended with much risk of undermining. With careful planning and supervision this risk was entirely eliminated and sound work secured. The pile driving in the proximity of the old piers had to be done with great care. Pier 3 tilted 6″ out of plumb at the top, as the piles were driven at the full 100 lbs. pressure. In subsequent driving of piles close to Piers 4, 7 and right abutment the steam pressure was restricted to 70 to 60 lb to the square inch. A plumb bob suspended from the top of the adjoining pier gave indication of any tendency to tilt. Pile driving was thus successfully done next to piers 4 and 7 without the slightest tilt developing in the Piers. The progress was no doubt considerably cut down but that was of minor consequence.

This feature of the work is of special importance in cases where new work has to be linked to an existing structure and deep pumping and heavy vibration due to pile driving are inevitable. It demonstrates that risks of this nature can be taken with safety provided the necessary care, forethought and supervision are available.

Clay Substratum and Springs Bay 4.

The existence of the clay substratum in this Bay was responsible for a good deal of trouble and expense. The 30′ pile line under the crest had been driven in 1932-33 with its bottom at R. L. 689-88. The top few feet were cut in 1933-34 to suit the depressed level of the crest at 715. At bottom these piles penetrated the clay substratum and the indication was that there was a leak from below the clay along the piles into the sand layer above. The pressure under the clay was some feet above that in the sand above. So that if, at any future date this leak developed, the uplift pleasure under the floor resting on the sand would be some feet in excess of the designed and the floor would lift up. To ensure against such eventuality, a second pile line was driven 5′ downstream of the crest pile line with its bottom bedding 1′ into clay *and not penetrating it*. This second line was carried round along the abutment for the same reason. This explains the double line under the flanks in Bay 4. In Bay 8 the double line was put in more or less to function as independent lines of wells.

Bore holes were made in this Bay (4), some to investigate geological formation of subsoil and some for insertion of pressure pipes. Some of these bores went below the clay. The bore pipes were withdrawn and directly the sand was excavated to within 10′ of the top of clay

vigorous springs burst out from below the clay discharging upto 0.8 cusec with large volumes of sand in addition. (Photo No. 4.) The appearance of these springs caused a serious setback to progress of work and introduced other complications. In order to prevent indeterminate pressure acting on the floor, these springs had to be effectively sealed. For that purpose, the discharge level of springs was raised by ring bunds put round them. This way clear water continued to come out but the flow of sand was arrested. The discharge was then localised through a 3" or 4" pipe by means of a water jet clearing a straight passage to below clay. This pipe went below clay with top level such that clear water escaped and no sand. Puddle and sand was then heaped round the pipe so that the spring could not burst outside the pipe. This pipe was finally connected to a grouting machine with a regulated relief pipe. The cavity was grouted under pressure with cement. This way the springs were effectively sealed. The part of the grouted pipe above the foundation level was sawn off. There were six such springs and they took on an average 350 bags of cement each to seal up.

Divide Groynes.—The Divide Groynes No. 3, 4 and 7 have the base solid up to R. L. 726 ; above that to R.L. 738 they are made up of blocks with one foot spaces in between. These spaces are staggered along the length in the three layers of blocks at R. Ls. 726-30, 730-34 and 734-38. The laying of the top in blocks permits independent settlement of these blocks in case of damage. The spaces or perforations allow the water to flow from the weir to the undersluices' side thus relieving head across the groyne and reducing swirl action around it.

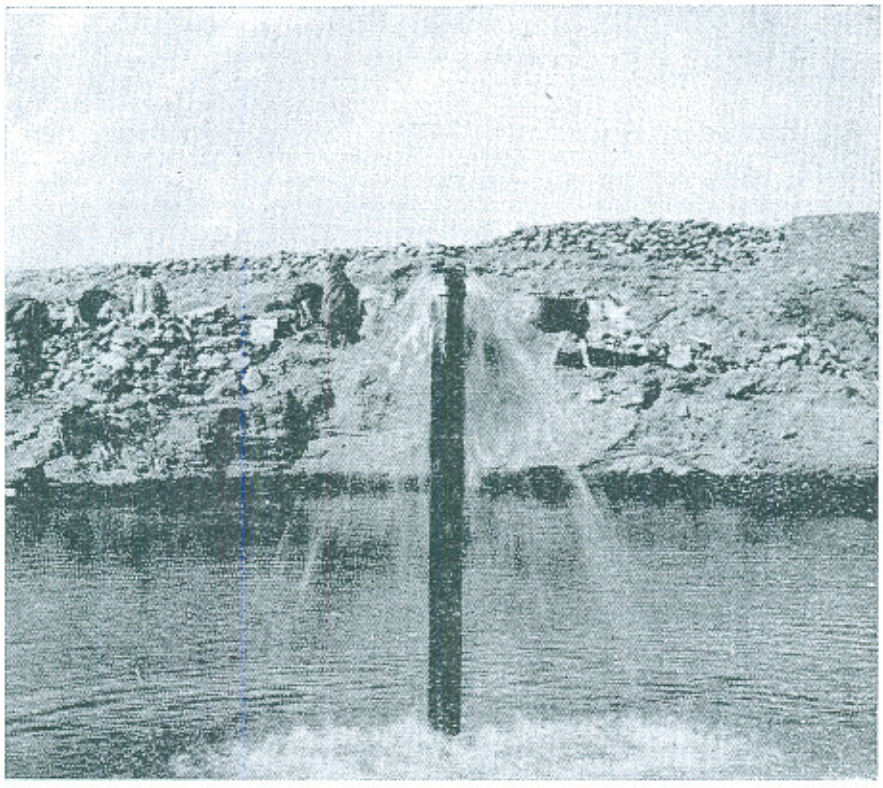
Developed Blocks.—The development from the vertical flank walls to the sloping groynes downstream in Bay No. 4 and to the Bell's Bund in Bay 8, has been done in stepped blocks. The wall of blocks is everywhere safe as a gravity section. Relief holes with ballast backing have been also provided. These developed blocks have proved most effective in maintaining smooth flow along the sides (Photo 5).

Blocks.—Free use has been made of bricks' shells for concrete blocks both in the groynes and in the floor. These shells are half brick thick with odd bricks projecting in. They are in 1 : 4 or even in 1 : 6 mortar. This method speeds up work considerably, is quite sound and leads to economy.

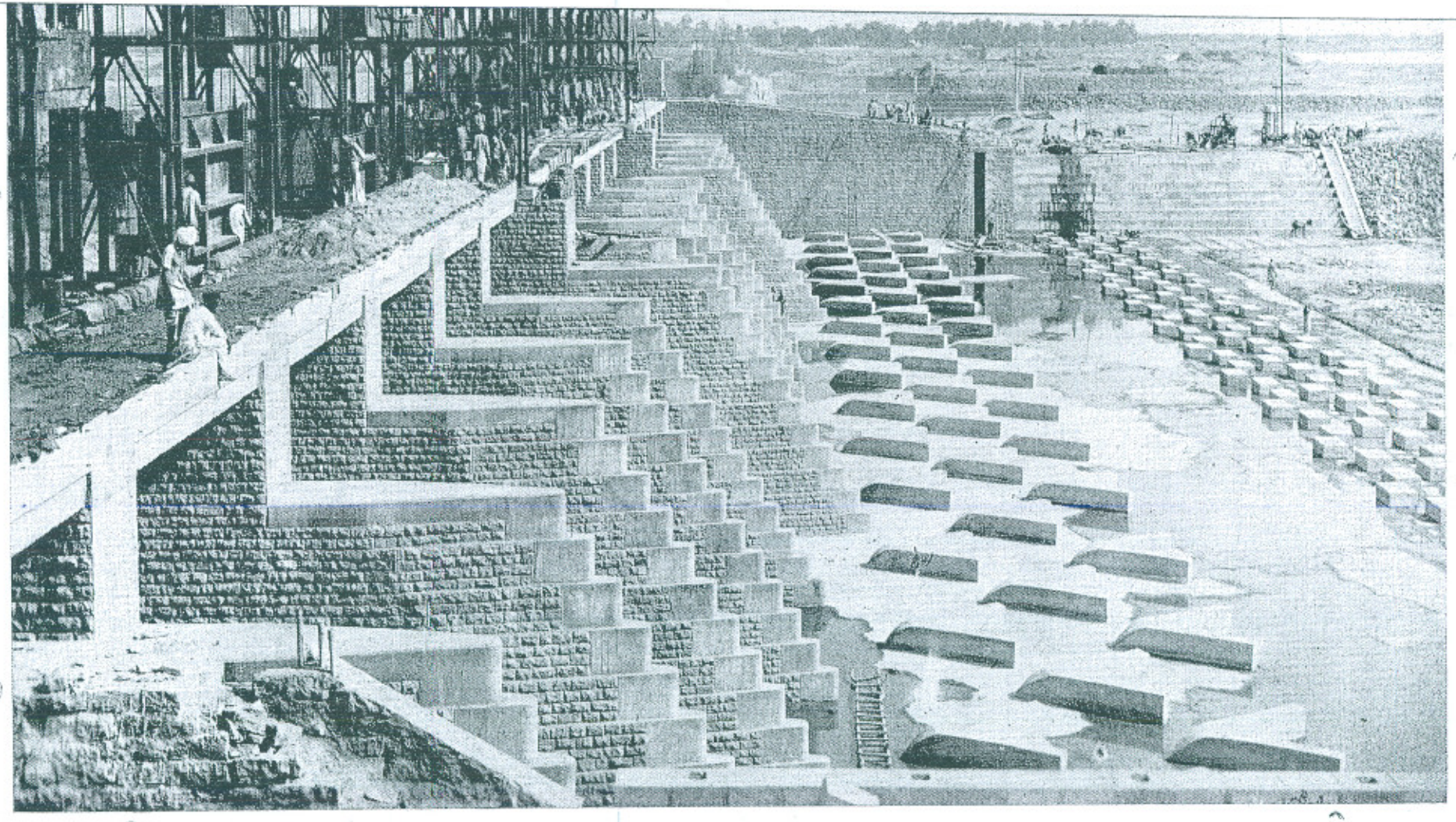
In the inverted filter area the blocks are 4' deep. They have 1" to 1½" *jhiries* on three sides and on the fourth they butt against the raised blocks (Photo 5). This arrangement gives effective relief and ensures against lifting up of blocks by local unbalanced head. At Islam and other places blocks have been known to be bodily lifted out and carried forward.

Dissipation of Energy.—The scheme of arrows on the impervious floor and the raised blocks on the inverted filter area is clearly shown in Photo 5. This combination has proved most effective in shooting up the

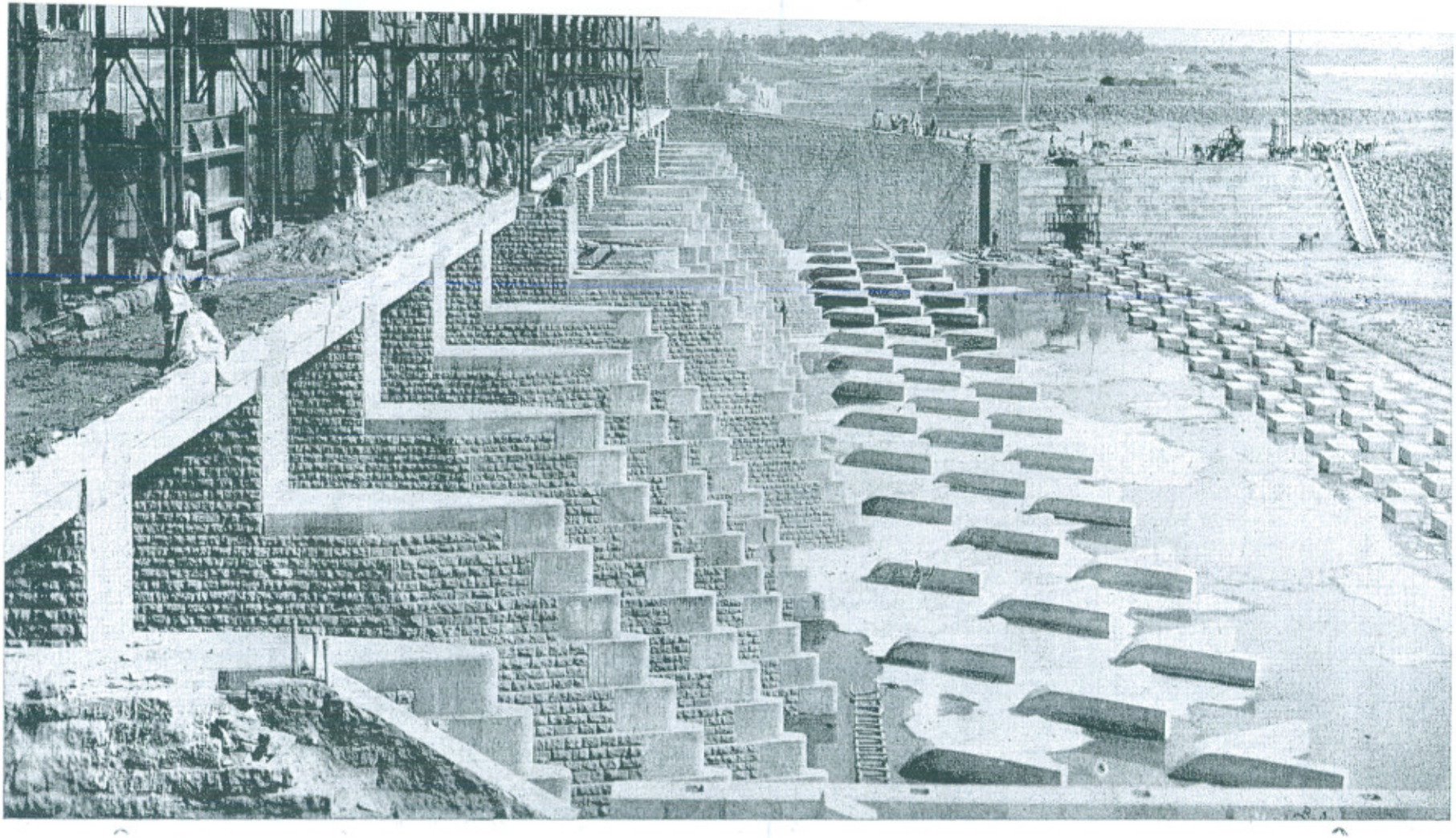
SPRINGS IN CENTRAL UNDERSLUICES



Six such springs appeared in Bay 4 from beneath the clay substratum, each discharging 0·8 cusec and volumes of sand.



Showing steel work, trough bridge, stepped fish tails of piers, arrows, raised blocks and development from vertical to 1.5 by stepped blocks.



Showing steel work, trough bridge, stepped fish tails of piers, arrows, raised blocks and development from vertical to 1.5 by stepped blocks.

high velocity jets of the bottom flow. Discharge upto 1,05,000 cusecs has been passed through Bay 8 without the least damage to the loose protection. It is hoped that under still higher supplies, these blocks and arrows will continue to prevent damage to the apron ; even the retrogression of levels has set in.

Concrete in layers or en masse.—The entire concrete in Bays 4 and 8 was laid in layers and not *en masse* for the whole depth. This is a point over which there has been and will be a good deal of argument. No expansion or contraction joints have been provided in the floor concrete.

If the concrete of the floor is laid in one mass in the entire thickness, it has to be done in compartments such that one or more of these compartments can be finished in one day. Between each compartment, fairly wide joints are left which are filled later. Where the surface is reinforced, the reinforcement from one compartment has to overlap on the reinforcement of the other compartment by the usual 40 or 50 diameters. Work done in this way is sound in so far that the whole depth having been laid in one mass there are no layers to separate, as they did at Islam and Ferozepore weirs.

Their drawbacks are :—

(*a*) The existence of vertical joints.

(*b*) The difficulty of clearing the deep vertical joints under the overlapping layers of surface reinforcement projecting from the adjoining blocks.

(*c*) The indeterminate internal stresses set up in a thick mass of concrete due to the setting heat of cement.

(*d*) Additional expense involved in the overlaps at short intervals. If the compartments were say 12′ in one direction, the reinforcement used be $\frac{5}{8}$″ diameter, and the overlap 50 times the diameter, the additional length used in the overlaps will be approximately 20 %. On the other hand, if the full 40′ or 45′ lengths were used this additional length due to overlaps will be only 5 %.

(*e*) The entire floor thickness has to be the same mix, *viz.*, 1 : 2 : 4 1 : 3 : 6 or 1 : 4 : 8. In thick floors this is likely to add considerably to the cost. On the other hand any attempt to put in cheaper mixes in the lower layers is bound to prove difficult and unsatisfactory.

The only objection to laying the concrete in layers is that if these are not definitely tied to each other, there is danger of layers separating from each other and thus failing to act as one slab. While the total thickness of the floor slab will be just enough to withstand the uplift pressure, the

weight of the separate layers will be inadequate against such uplift and there will be likelihood of their blowing up. This is a very serious matter. The separating of layers has been responsible for a good deal of damage and expense at Marala, Rasul, Ferozepore and Islam Headworks. The reasons for such separation of layers are :—

(*a*) The unequal expansion and contraction of the fresh layer and the layer below due to variations in atmospheric temperature.

(*b*) The presence of laitance, dirt and other matter above the bottom layer when the upper layer is laid. This is in most cases the real cause.

(*c*) Difference in the kind or in the proportion of the cement used in the two layers. The former was responsible for the separation of layers at Sulemanki during construction. The separated layers there were entirely removed and relaid after proper nicking and clearing of the surface of the bottom layer.

These objections to the laying of concrete in layers have been effectively overcome at Khanki without any additional expense. As a matter of fact considerable economies have been possible. The bottom most layer is 1·0 or 1·25′ of 1 : 3 : 6 concrete. This rich mixer was used to make the concrete in contact with sand water tight. The top one foot is 1 : 2 : 4 mix and carries the surface reinforcement. The intermediate layer or layers are of 1 : 4 : 8 concrete.

From the bottom layer of 1 : 3 : 6 concrete and bedded about 9″ in it, stand $\frac{5}{8}$″ vertical stirrups with tops at the level of the surface reinforcement (Photo 6). These were spaced at 5′ intervals and bind the various layers from top to bottom. The surface reinforcement was passed through these stirrups and held in position by them. In addition 2′ to $2\frac{1}{2}$′ long binder stones were bedded into the bottom and other layers projecting into the next layer or layers above. These were at 5′ or closer intervals and were staggered in the various layers. Stones with rough sides were selected. Between the vertical stirrups and the binder stones, enough strength is provided against any shear stresses that are likely to develop tending to separate the two layers.

In addition to the mechanical bond provided as above, the surface of each layer is thoroughly cleaned of all laitances and other foreign matter and a fine film of cement grout applied to it immediately before the concrete of the upper layer is laid.

With the bond between the various layers effectively secured in this way, the system of laying in layers has much to commend itself.

CONCRETING THE FLOOR

RIGHT UNDERSLUICES



Showing vertical stirrups and binderstones to bind the floor concrete which is laid in layers

(*a*) There are no through vertical joints.
(*b*) The progress is considerably accelerated as there is no restriction as to work being done by whole compartments only.
(*c*) The setting heat of cement in each layer is largely dissipated as the concrete is poured, as the layers are thin. Thus the indeterminate stresses inevitable in one thick layer, are eliminated.
(*d*) Considerable economy is possible by using cheaper mix for the intermediate layers which are merely to provide the necessary weight.
(*e*) The extra length used in stirrups is more than made up, as the wastage due to overlaps in the surface reinforcement is reduced to a minimum of 5 % against 20 % used in *en masse* work in compartments.

Concrete laid by baskets.

The entire concrete in the floors has been laid by baskets. The tram trucks brought it from the mixers and dumped it on to platforms made up of two 8′×4′ sheets held in on the sides by B. G. sleepers. These sheets bins were carried from place to place, as required. Thus no temporary platforms were made and at the same time no foreign matter was allowed to get into the concrete. The method of tipping the trucks direct into the work was tried for a couple of days and definitely discarded as it was impossible to keep off the sand on the wheels and other foreign matter getting mixed up with the floor concrete. Another drawback was that, in long leads, a good deal of moisture and laitance worked up to the surface so that when the trucks tipped the concrete, there was scegregation of mortar from the aggregate. Both these drawbacks were overcome by basketing the concrete from the bins to the floor. This, incidently, gave another turn over to the concrete. Each basket of concrete was riggled directly it was laid. Thus sound work was ensured, in each individual part of the floor. The additional cost on this account was 0-8-0 % cubic foot.

Setting Heat of Concrete.—Extensive experiments have been carried out by the U. S. Reclamation Bureau to investigate the setting heat of cements and of cement concrete in dams. The generation of setting heat and the accompanying rise of concrete temperature are directly due to the chemical changes that take place during the hardening process. The richer the mix the greater the rise in temperature. In neat cement pastes, this rise ranges from 118° F. to 208° F. in different brands of cement. The maximum rise in temperature in concrete does not ordinarily exceed 70° F. In the weir floors which are exposed on the outside to atmosphere or water and at the bottom to subsoil, the rise in a 5′ thick floor may be between

40° to 50°F. In 1′ or 18″ slabs, it will not exceed 15°F. or so. In this latter case, the internal stresses and resulting deformation will be small and almost negligible.

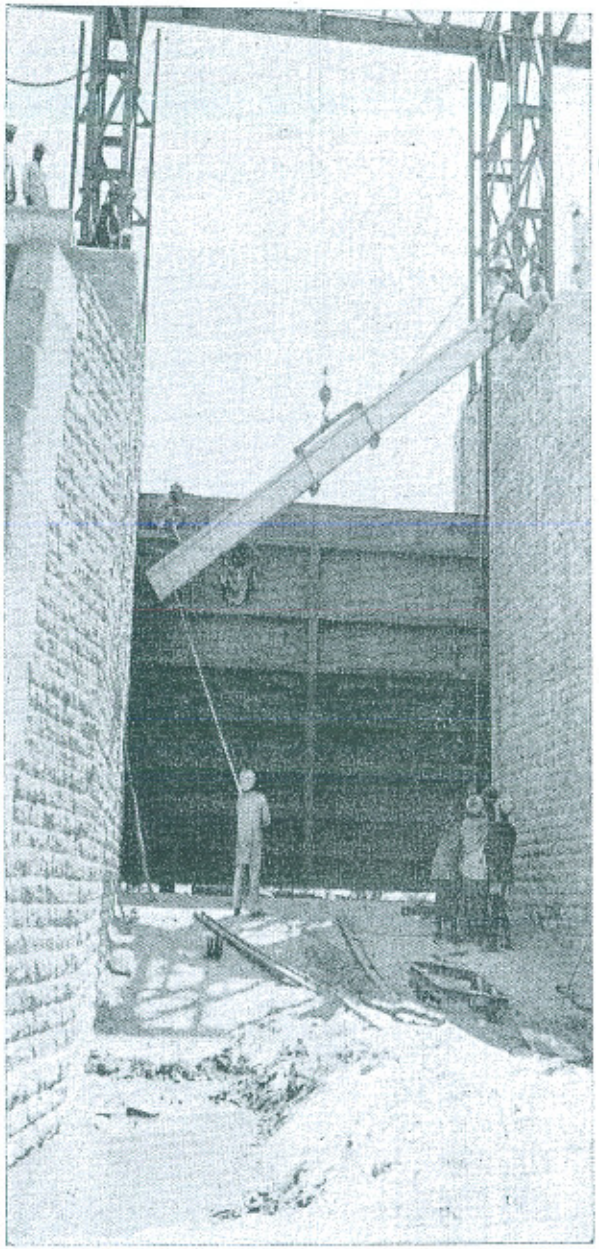
Precast Concrete units.—Another noticeable feature in this construction is the liberal use of precast units. This accelerated speed, saved in cost and resulted in better finish. All nose blocks (Plate III, Fig. 7) of the piers and fish-ladder walls and all copings were precast in 1 : 2 : 4 concrete. So were the troughs of the road bridges. The nose blocks were made hollow and laid break-jointing with bold V-joints. They present a very neat appearance. It should be possible, consistent with economy, to construct the entire piers with face-work in precast blocks and the hearting of cheap concrete, say 1 : 5 : 10. This should be an extremely rapid form of construction and perhaps no more costly than the one in stone masonry.

The stepped fish tails of the piers are another feature. The tail blocks were cast at site.

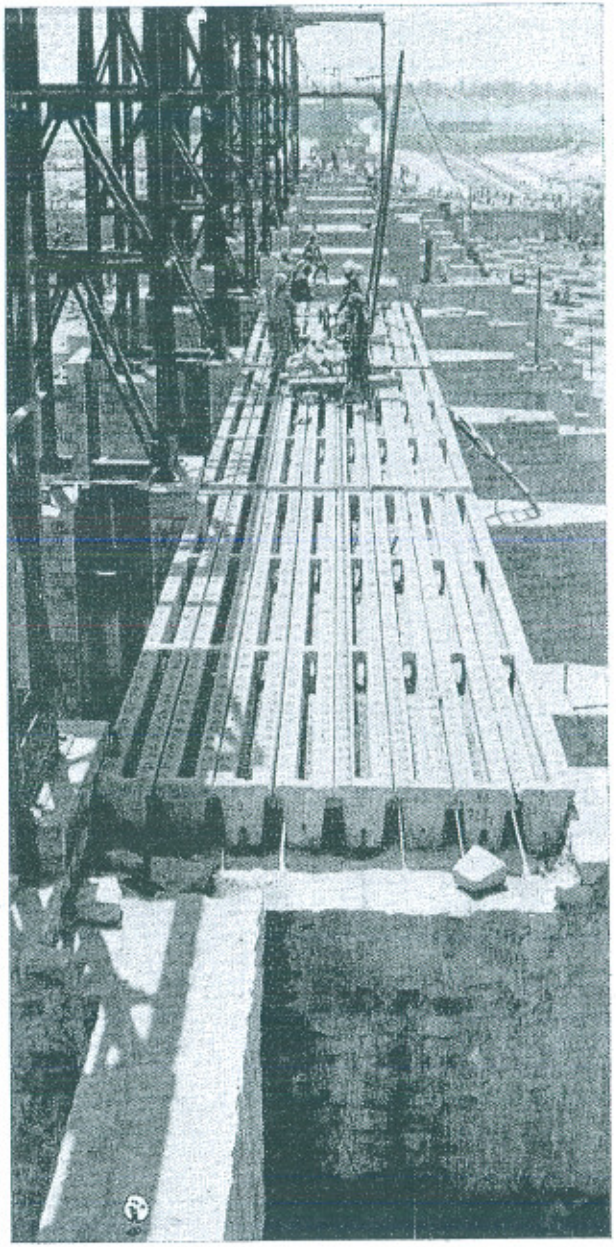
Use of Plums.—For economy liberal use was made of plums in mass concrete of groynes and blocks.

Trough Bridges.—On the glacis of the weir bays, a width of 12′ to 15′ has been finished in hammer dressed stone masonry instead of the usual toothed one, to provide roadway for motor traffic. Corresponding gaps have been left in the downstream groynes. (Photo 3.)

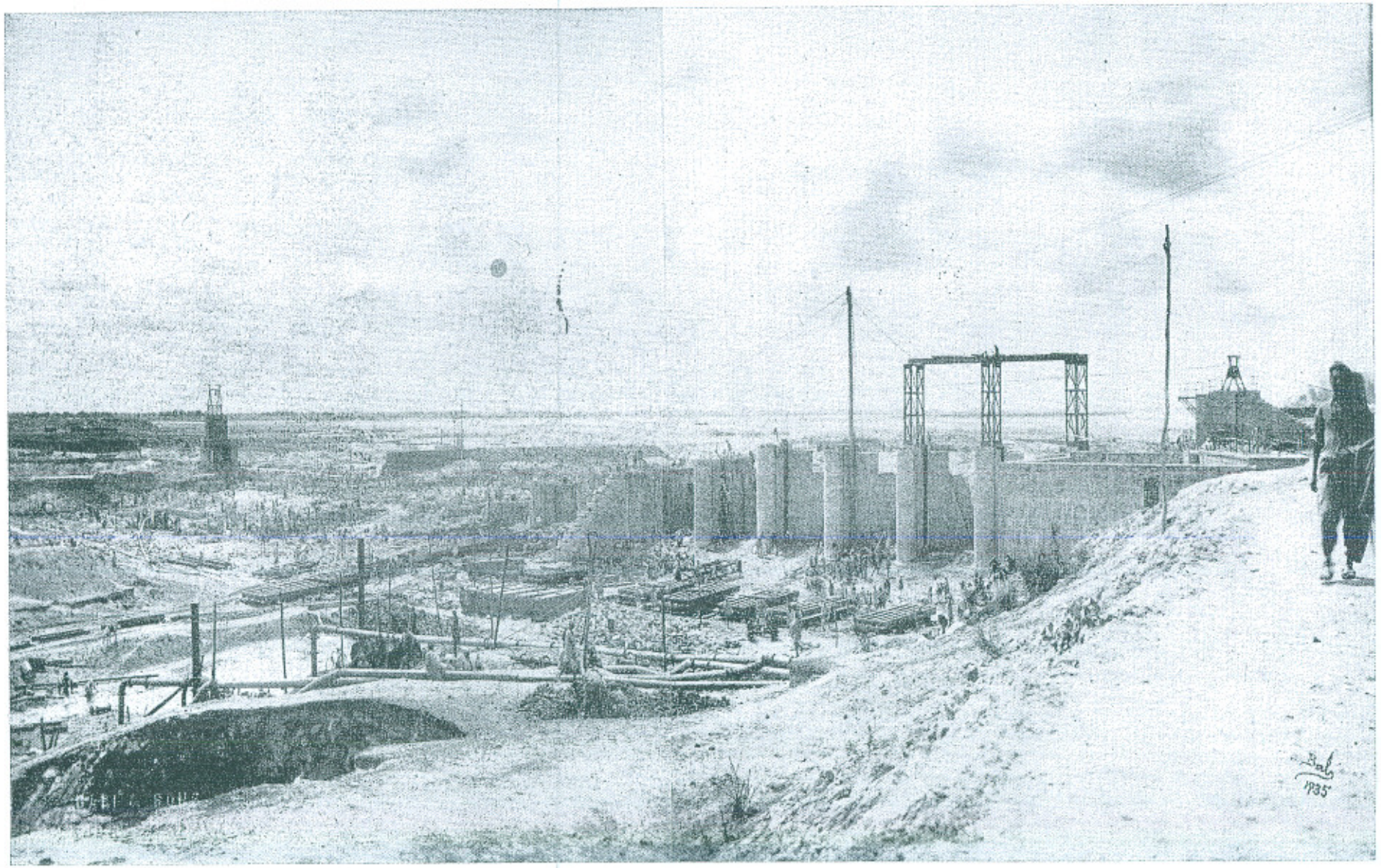
The Central and Right undersluices (Bays 4 and 8) have been provided with road bridges connected to the roadway on the weir by means of suitable ramps. These road bridges are designed for a 15-ton road roller. They are made up of precast troughs which are hoisted on to the piers (Photo 7 and Plate IV) and made up to full section by the addition of a suitable depth of concrete and vertical reinforcement at site. The tensile reinforcement is all provided in the precast work (Photo 7). Each trough is 24′ in length and weighs 1.7 ton. With rapid hardening cement it can be lifted 3 to 4 days after casting. The troughs were cast in bottomless moulds direct on to a platform of dry bricks lightly plastered over. The sides of the moulds were in portable sections. With rapid hardening cement the moulds were removed after 4 hours so that two troughs were cast per mould per day in the month of July and August. The troughs were left to cure on the platform till they were fit to be lifted. The wedge shaped interior in the precast troughs ensures mechanical bond between the precast and the insitu work. The stirrup projections allow extension stirrups to be added to take the diagonal tension. The inside of the trough is cleared of laitance with wire brushes when the concrete is nearly green. A film of cement grout is laid on the precast work immediately before the insitu concrete is added. These bridges have been already tested with a 10 ton portable and a full crowd load.



Each trough is 1·7 ton in weight and 24 ft. in length.



Showing precast troughs in position before the insitu concrete and steel are added.



Showing the 66 feet spar for erection, three trestles and cross girders up and the gates and trestles arranged in foreground.