PAPER No. 251

THE KALABAGH BARRAGE

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INTRODUCTORY

The Thal Project

The Thal Project has had a chequered career. Time and again the scheme was relegated to the archives in the Secretariat, always to be taken out again and given a fresh dressing till it finally emerged as a reality in 1939. The idea of constructing a perennial canal off-taking at Mari from the River Indus to irrigate the Doab between the Indus and the Jhelum is said to have been first conceived during the sixties of the last century though a proper project was not submitted to the Government of India till 1919. This as well as the projects sent up in 1921 and 1925 were, however, turned down for one reason or the other.

In 1935, a committee was appointed to report on the future distribution of the Waters of the Indus and its Tributaries that may be acceptable and equitable to all parties concerned. According to this Committee, the Thal was allowed a maximum withdrawal of 6,000 cusecs at the head of the canal and the supplies in December and January of 2,000 cusecs, in February and March of 3,600 cusecs and in November 5,600 cusecs. The actual supplies were, however, expected to be appreciably more than the mean authorised. A fresh project was accordingly framed in 1936 which provided for the irrigation of the areas proposed in the 1925 Lesser Project excluding the uncommanded areas or those having high spring level. According to this, the total gross area for which irrigation was allowed, was 14·48 lakhs acres and the estimated cost was Rs. 5·53 crores.

In this Project, the main canal and branches were not proposed to be lined and it was estimated that the absorption losses would be about 1,500 cusecs thus leaving only 4,500 cusecs for utilization at the head of distributaries, which was considered just sufficient for giving irrigation to the western strip of the Thal.

As a result of the experience on the Haveli Main Line lining, it was decided in 1939, however, to line the Main Canal and Branches on the Thal Project utilizing about 1,350 cusecs thus saved for the irrigation of the Khushab Tahsil.

The work was approved to be taken in hand by the Punjab Government in accordance with the above and a revised project estimate was framed in 1940. In this project the Main Line was so designed as to permit its being enlarged to carry 10,000 cusecs in the event of additional water being required in the *kharif* and becoming available.

To demarcate the areas for this Project, a preliminary soil survey was carried out by the Director, Irrigation Research, which indicated that in a million acres of the flat land included in the 1936 project, there is a layer of salts in the soil crust and that these salts might come to the surface and cause thur when canal irrigation is introduced. In other areas which are comparatively free from salts it was envisaged that there will be difficulties in the construction and maintenance of watercourses on account of the high percentage of *tibbas*. The Director, Irrigation Research, suggested basin irrigation as a remedy for the salt-affected area and expressed the opinion that while most of the area required treatment of some kind it should ultimately be possible to cultivate the whole of it. In deciding whether the construction of watercourses would be practicable or not in the *tibba* areas the criterion adopted for the inclusion of land in the project was that the percentage of uncommanded *tibba* within a *chak* should not exceed 25%.

According to this investigation, the gross commanded area comes to 19·3 lakhs acres. The total estimated cost, which includes Rs. 50 lakhs for soil reclamation, is Rs. 7·72 crores, of which the headworks is estimated to cost Rs. 1·88 crores.

Description of the Barrage.

The barrage is 3,781 ft. long between abutments and comprises a central weir section consisting of 42 bays of 60 ft. with an undersluice section consisting of 7 bays of 60 ft. at each end. The abutment piers separating the two sections are 25′ wide and each contains a fish ladder. All other piers are 7′ wide. The crest level of the Weir is 678·0 with the U/S and D/S floor levels at 673·0 and 670·0 respectively. The undersluice levels are 3′ lower.

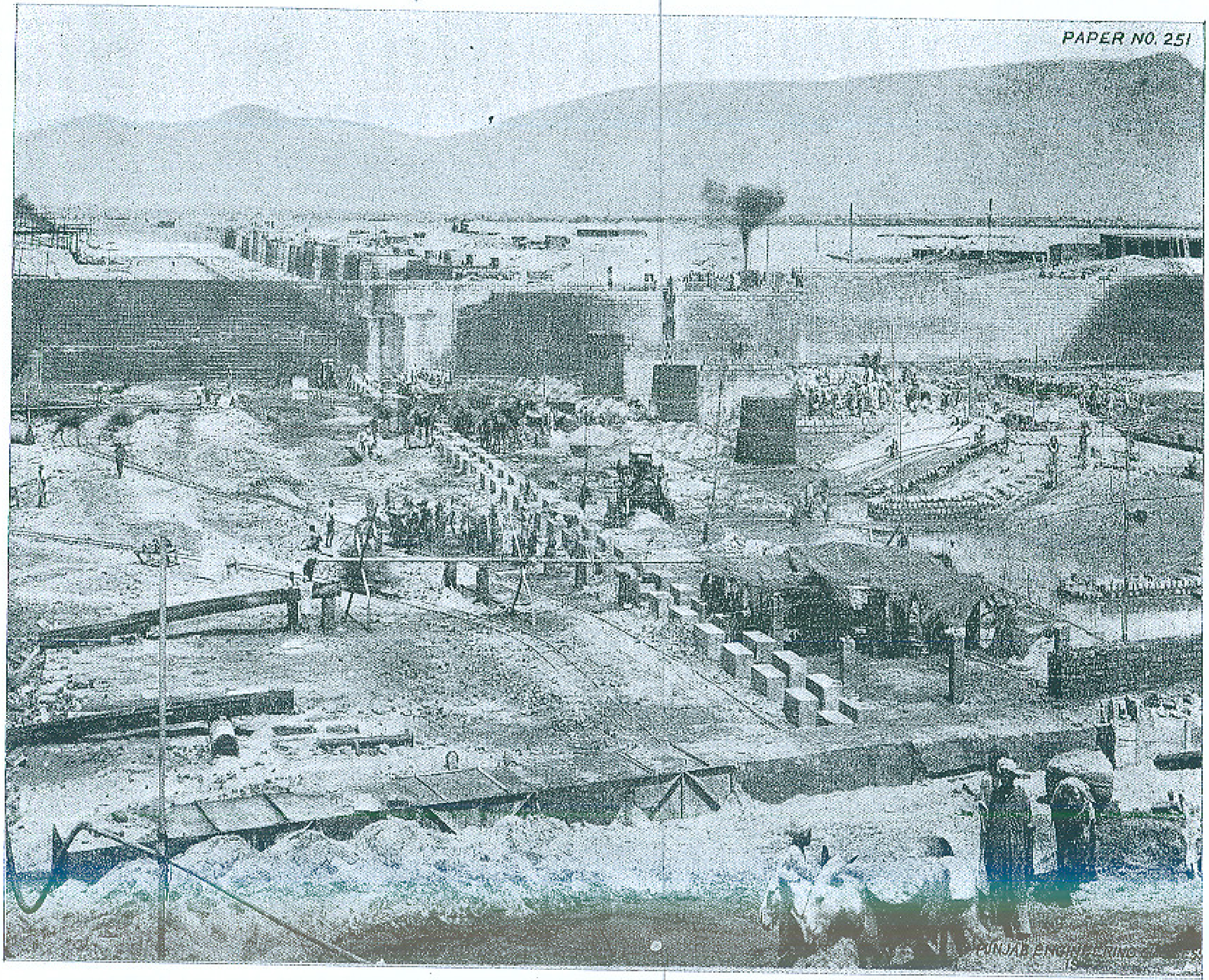
A canal with an ultimate capacity of 10,000 cusecs takes off on the left and a modified Khanki type Silt Excluder covering the two end bays is provided in front of the Head Regulator. There is a bridge 10′ wide with a 2½′ foot-path on one side over the barrage. The gates in the Weir will be 15′, in the Right Undersluices 18′ and those in the Left Undersluices 20′ high. Expanding guide banks of the usual Bells' pattern with marginal bunds on either side have also been provided.

A canal railway line connects the Barrage with Daudkhel Railway Station.

DESIGN

Selection of site

The river Indus emerges from a deep and narrow gorge just U/S of the Kalabagh Railway bridge, where its width between banks is 3,000′. Six thousand feet below this bridge, the width is 8,000′ and about 15,000′ downstream it is 16,000′ between Pakki Shah village



A GENERAL VIEW OF THE LEFT POCKET

(June 1941)

on the left and Kot Chandna on the right. A hill torrent, called the Jaba, joins the river on the left bank just below Pakki Shah. The river trifurcates about a mile downstream of the Railway Bridge just above a shingle bar. The main channel hugs the right bank and carries a discharge of about 15,000 to 20,000 cusecs in winter. The central channel, which is much wider but has a higher crest over the shingle bar, carries about 5,000 to 8,000 cusecs and the Pakki creek right carries 3,000 to 5,000 cusecs in winter.

The site of the Headworks was fixed in 1936, about a mile below Pakki Shah, the main points in its favour as compared to any possible sites upstream being :—

(*a*) Facility of construction and certainty of correct estimation of cost. As the width of the river at this site was about 3 miles, the area for the construction of the Headworks could be easily ring-bunded leaving ample waterway for normal floods. This necessitated the closing of Pakki creek which was not anticipated to present any difficulty. It was also considered that the slightly more expensive training works required at this site would be more than compensated by savings in the costs of pumping and temporary protection works during construction.

(*b*) At this site, the foundation of the work would have been on shingle but most of the excavation would be in sand.

(*c*) This site avoided the canal being taken through heavy expensive digging in the shingle bank, or heavy filling in the river bed open to constant river attacks.

(*d*) It obviated the necessity of providing a crossing for the Jaba, which was estimated to cost $4\frac{1}{2}$ lakhs for a discharge of 7,000 cusecs.

(*e*) The afflux, even with the highest flood, would not be sufficient to submerge any buildings on either bank.

(*f*) The Railway Line could be easily connected with Daudkhel Railway Station which was preferable to Mari, as it involved a saving of about 6 miles in carriage of materials over the N. W. R.

Any further shifting on the downstream side was not practicable from considerations of command.

Maximum flood discharge of the Indus

The Discharge Division of the Irrigation Department originally put the maximum discharge in the river Indus on 29th August, 1929, at 8,18,510 cusecs. The Executive Engineer, N. W. R., who was at the time in charge of the Kalabagh Bridge construction estimated it to be 11,76,000 cusecs, based on the mean of discharges by Kutters and Bazin's formula and by velocity curve and cross-section method. The data of the Discharge Division was scrutinized in 1936. The slope between Mari discharge site and the permanent gauge at

Kalabagh was fixed at 1/950. The rugosity factor for rivers in abnormally bad condition is taken in the neighbourhood of 0·035 and so for the highest flood of the year, the rugosity factor was fixed at a figure between 0·035 and 0·03. Discharges were then calculated both with Kutter's and Manning's formula with rugosity factors of 0·035 and 0·03, and the mean of the four, which came to 9,43,000 cusecs, was taken as the correct discharge.

It may be noted, however, that the flood of 29th August, 1929, which was due to heavy rains in the catchment did not synchronize with the bursting of a glacier dam, which is a not infrequent phenomenon on the Indus. Should the bursting of a glacier dam occur during a heavy flood, higher discharges might occur.

The Kalabagh barrage has accordingly been designed to pass a discharge of 9,50,000 cusecs with enough freeboard to pass a superflood of 11,00,000 cusecs with increased afflux, should this occur.

High flood level

The 1929 high flood mark on a house on the Weir cross-section line was found to be at R. L. 693·0. This also corresponded with a level observed 4,000′ U/S, allowing a slope of 1 in 2,000. This figure was accordingly accepted for the design.

Pond level and lowest water level

The canal section has been designed such that it can take 6,000 cusecs with a pond level of 692·0 and 10,000 cusecs with a pond level of 694·0.

The lowest supply recorded in the Indus is 18,642 cusecs with a level in the right-hand creek at the weir site of 674·25. With the canal in operation, a minimum discharge below the work of 16,000 cusecs, corresponding approximately to a level of 674·0, may be expected.

Retrogression and maximum head

Retrogression occurs in the D/S bed of a river in the first few years after construction and may affect the low river levels by 4′ to 7′ and the maximum flood levels by 1′ to 2′. This is explained by the fact that due to heading up at the weir, a portion of the silt is dropped on account of the flatter slope. The water passing over the barrage has thus less than its normal load of silt and causes erosion downstream, with a drop in the specific levels. In floods the water overflows its normal bed and occupies a section which is not so much affected by this process. When the pond silts up and provides a normal river section, after the lapse of a few years, the water passing over the barrage gets its normal silt charge, thus resulting in the restoration of the original slope and bed level.

The retrogression has thus the effect, during the first few years of construction, of increasing the head across the weir and steepening the exit gradient which has to be provided for in the design.

In view of the underlying shingle bed and the smallness of the pond, it is not anticipated that there will be any extensive retrogression of water levels downstream of the weir. This figure may thus safely be assumed to be 2′ against 4′ to 5′ for low supply on other Headworks. This gives a minimum downstream level of 672·0, and a maximum head of 694—672=22′, for which the barrage is designed. The retrogression assumed for high flood conditions is 1′.

In actual practice, however, it will be many years before a pond level higher than 692·0 is needed, as the additional withdrawal of 4,000 cusecs is not likely to mature for many years, by which time the retrogression provided for will have passed. In addition, when the minimum downstream level of 672 occurs, it will not be necessary to maintain the maximum pond level and the head can easily be limited to a figure smaller than 20′ even, if necessary.

Length of Works

The length of the barrage was fixed in the light of experience on other Headworks, using Lacey's formula $Pw = 2{\cdot}67\sqrt{Q}$, as the criterion. In the case of a river in flood Pw may be taken as equivalent to the surface width.

The ratio of the length between abutments and Lacey's Pw for various Headworks is tabulated below :—

Work	Discharge lakhs	Lacey's Pw	Width between abutment =W	W/Pw
1. Balloki ..	1·13	930	1,547	1·67
2. Islam ..	2·75	1,400	1,621	1·15
3. Rupar ..	3·15	1,500	2,663	1·7
4. Suleimanki ..	3·25	1,520	2,223	1·47
5. Ferozepore ..	4·4	1,780	1,956	1·1
6. Sarda ..	6·0	2,070	1,964	0·95
7. Trimmu ..	6·50	2,150	3,026	1·42
8. Panjnad ..	7·0	2,230	3,400	1·56
9. Marala ..	7·18	2,263	4,475	1·98
10. Khanki ..	7·50	2,312	4,414	1·91
11. Rasul ..	8·75	2,501	4,400	1·76
12. Sukkur ..	15·0	3,260	4,725	1·45

In the case of Trimmu, the high ratio was adopted to keep the intensity down to a reasonable figure.

A high intensity of discharge would mean a heavier section of the floor and loose protection as well as higher gates due to the increased depth on the crest.

The figures adopted at Kalabagh are based on the Trimmu figure and are as follows :—

Discharge lakhs	Lacey's Pw	Width between abutments = W	W/Pw
9·5	2,600	3,797	1·46
11·0	2,800	3,797	1·36

This gives an average intensity of 250 and 290 on 9·5 and 11 lakhs discharge respectively.

It was suggested that if the shingle downstream did not scour out, the downstream water levels might take control in high floods thus drowning out the Weir, which would result in a very uneven discharge at various points and may possibly cause outflanking. No increase in the waterway on this account was however considered necessary, as an allowance for about 20% concentration with an eleven lakh flood was made in the design, although experience at Marala, where conditions of flow were similar and there was a shingle bed some distance upstream, showed that there was no likelihood of this concentration of flow. Moreover, it was agreed that this would simply have the effect of increasing the velocity over the shingle downstream and in any case the upstream bunds had ample freeboard.

Details of waterway

The total length between abutments is comprised as follows :—

56 spans of 60′	..	=	3,360′
53 piers of 7′	..	=	371′
2 Divide walls of 25′	..	=	50′
Guide bank slopes	..	=	16′
	Total	=	3,797′

Afflux

In order to obtain a small-gated area and control of the river during all conditions, it is necessary that the waterway at the weir should be constricted to give modularity of flow under all conditions. The heading up resulting from this during maximum flood conditions, which should be the minimum consistent with modularity, is known as afflux. Its value was obtained as follows :—

The average intensity for 9,50,000 cusecs would be $\frac{9,50,000}{3,360} = 283$ cusecs.

Assuming a coefficient of 3 in the formula $Q = CH^{1.5}$, we have $H = 20{\cdot}8$

With the gentle expansion proposed in the downstream glacis, an afflux of 15% or 3′ on the normal flood level of 693·0 was considered suitable, giving a H. F. L. upstream of 696·0.

Velocity head

The average intensity between guide banks for 9·5 lakhs is $\frac{9,50,000}{3,797} = 250$ cusecs/ft.

Taking $f = 4$ for shingle

$$R = d = \cdot 9\left(\frac{q^2}{f}\right)^{1/3} = 22\cdot 5 \text{ (From Lacey's formula)}$$

whence $V = \frac{250}{22\cdot 5} = 11$ and $h_a = \frac{V^2}{2g} = 1\cdot 9$, say 2′

It would be sufficiently accurate to use this figure for all high flood conditions.

The normal average H. F. T. E. L. upstream will therefore be 698·0 and the corresponding level downstream will be 695·0.

Undersluics

It was considered necessary to have a length of the barrage with its crest at a low level on account of the following considerations :—

(*i*) To facilitate the diversion of the river over the completed barrage

(*ii*) To facilitate the unwatering of the Weir for subsequent maintenance and inspection.

(*iii*) To facilitate silt control into the canal by the formation of a deep channel near its off-take, in which low velocities of approach could be secured.

It would thus be seen that the total length and crest level should be suitable for the diversion and the width on the left should be greater than that of the off-taking canal to attain the last object.

It was anticipated that at the time of the diversion at Kalabagh, the river discharge might be as high as 60,000 cusecs. The water level at the shingle bar, where the diversion bund has to be put, is 686·2 for this discharge. Allowing a slope of 1 in 3,333 on the 10,000 feet length to the weir site, the level there would be 683·2, and 60,000 cusecs could be passed by 840 ft. of waterway, with a crest level of 675·0. The undersluice could thus consist of 14 spans of 60′ each.

Weir Crest Level

With an U. S. T. E. L. of 698.0, the undersluice section would discharge 2,77,000 cusecs. The intensity on the rest of the waterway would thus be $\frac{9,50,000-2,77,000}{2,520} = 267$.

Using a coefficient of 3, this gives $H = 20'$, and a corresponding crest level of 678·0.

Minimum Supply conditions

As mentioned already, the downstream water level corresponding to a minimum discharge of 16,000 cusecs, after allowing 2′ for retrogression, has been assumed as 672·0.

The T. E. L., allowing 1′ for head due to velocity of recess, may thus be taken as 673.0. Under these conditions, the pond level may be taken as 688, giving an afflux of 15′.

16,000 cusecs spread over 840′ of waterway give an intensity of 19 cusecs per ft.

The depth and cistern level required for varying discharges per ft. run is as follows, as obtained from Mr. Montagu's hydraulic diagrams (reproduced as plates XI.1 and XI.2 in C. B. I. Publication No. 12) :—

Discharge ..	12	16	19	21
Depth (Ef_2) ..	5	6	6·4	7
Cistern level ...	668	667	666·6	666

Allowing for friction losses and the action of the friction blocks a discharge of 19 cusecs per ft. could thus be passed with a cistern level of 667.

Cistern level required for different conditions of flow

The D/S bed has to be protected from the action of the turbulence caused by the liberation of energy of the falling water. Most of this is dissipated by the formation of a standing wave when the change from the hypercritical to the subcritical flow stage takes place.

The Weir has thus to be provided with a cistern at a low level to ensure that any hypercritical flow set up by the weir cannot pass beyond it.

The following statement gives the discharge data for different conditions of flow neglecting friction losses and allowing 1′ retrogression downstream for high flood conditions :—

Condition of flow	Crest level	H	U. S. T.E.L.	q	Afflux H_L	D. S. T.E.L.	Ef_2	Required cistern level
Average supply ..	675	23	698	330	4	694	27	667·2
Concentrated supply	675	26	701	393	4	697	29·8	667·2
Minimum supply ..	675	13	688	19	15	673	6·4	666·6
Super flood (distributed)	675	25	700	375	4	696	29	667
Super flood (concentrated) ..	675	28	703	440	4	699	32	667
Average supply ..	678	20	698	270	4	694	23·7	670·3
Concentrated supply	678	23	701	330	4	697	26·8	670·2
Super flood (distributed) ..	678	22	700	309	4	696	25·7	670·3
Super flood (concentrated) ..	678	25	703	375	4	699	29	670

The cistern levels required in the undersluices and weir portions are thus 667 and 670 respectively, which as can be seen above will not impose any undesirable restrictions on regulation even with a concentration of about 20%. The floor level could have been kept higher if allowance had been made for friction losses, but this would have impaired the flexibility in the operation of the barrage, besides necessitating an increase in the depth of the pile line which would have in turn increased the pressures under the floor when the downstream water level was at floor level. It is thus desirable to keep the downstream floor as low as conveniently possible, bearing in mind the difficulties of any additional unwatering during construction.

The flexibility of control of the barrage in operation with low supplies was also studied by means of a graph (Plate I), on which are plotted against downstream levels :

(*a*) Normal river discharge.
(*b*) Capacity of the Undersluices.
(*c*) Capacity of the Weir.
(*d*) Capacity of the Barrage.

The curves are plotted for pond levels of 688 and 692 as limited by the condition that the downstream depth should be sufficient to support the wave.

The proportion of discharge to capacity at any level indicates the minimum proportionate length of the work required to be used.

Length of Cistern

This is calculated for concentrated high flood conditions as below :

Q	Crest level	U. S. T.E.L.	Afflux	Ef_2	D_1 U/S depth of water	D_2 D/S depth of water	D_2-D_1
330	678	701	3	26·2	10·3	23·0	12·7
393	675	701	3	29·2	11·5	25·5	14·0

As it has been found from experience that the turbulence subsides in a length equal to five times the height of the waves, the length of the cistern is generally kept as $5\,(D_2 - D_1)$.

The length required for the weir portion is thus

$$5\,(D_2 - D_1) = 5 \times 12{\cdot}7 = 63{\cdot}5'$$

A length of 70′ was kept to be on the safe side.

For the undersluices, the length required is $5 \times 14{\cdot}0 = 70'$.

A length of 75′ was similarly kept in this case.

Considerations governing profile

On a level floor with low friction the position of the standing wave is unstable, whereas on a sloping glacis its position is definite and can be closely predicted. It has been found that the flatter the slope, the more intense the wave and the greater the range of the trough requiring heavy thickness of the floor. The steeper the slope, the less the range and depth of the trough of the wave and therefore the shorter the length and thickness of the glacis. A slope of between 1 : 3 and 1 : 5 for the glacis is thus considered to be the most suitable both for the maximum dissipation of energy and economy.

A series of experiments were conducted in the Research Institute with different slopes and as a result of these a slope of 1 : 4 was adopted on the upstream and 1 : 3 on the downstream side, as it was considered that laying concrete at 1 : 3 would not present any constructional difficulties.

The width of the crest was to be kept sufficient to accommodate the gates and it had also to be relatively narrow so as to secure a high coefficient of discharge. A width of 6′ was accordingly adopted.

The total length of the Weir floor thus came to 140′ and that of the Undersluices 150′.

Stone facing

Good concrete can stand very high velocities if the water is clear and medium velocities are harmless, provided the water contains nothing heavier than sand. It cannot, however, stand the movement of pebbles and shingle over it as would be the case at Kalabagh where the river has a shingle bed for several miles above the weir site. It was therefore decided that in that part of the section which will be subject to the action of shingle moving at high velocities, 18″ quartzite setts should be used instead of 1 : 2 : 4 concrete. The piers, flank and divide walls were also to be faced with stone setts for a height of 2′ above the profile for the same reason. In the silt excluder tunnels, however, the height of the stone facing was kept as 3′.

Friction Blocks

Experiments were conducted in the Research Institute to determine the best form and position of friction blocks so as to reduce the downstream erosion. As a result of these, it was finally decided to have two rows of trapezoidal staggered blocks 5′×4′×2′, starting 10′ downstream of the toe of the glacis and another two rows of staggered rectangular blocks 4′×2′×2′ five feet clear of the downstream end of the floor.

Cut-offs

It was originally decided to have three lines of sheet piles, *viz.*—

(*a*) At the upstream end to guard against the undermining of the floor by the dynamic action of water in case there was a failure of the flexible protection. This cut-off has little influence on the uplift pressures under the D/S floor.

(*b*) At the toe of the glacis to reduce the uplift pressures under the cistern floor and to act as a second line of defence in case of failure of the downstream pile line.

(c) At the downstream end so as to have a reasonable factor of safety in the value of the exit gradient and to prevent failure by the slipping of the subsoil into the scour hole by earth pressure.

On account of the War, however, it was found impossible to import the required length of piling and the design was amended so as to provide for only one line of 7.5′ piles on the downstream end, as this quantity was obtainable.

The pile line was extended for some distance under the abutments and was also carried under the flanks.

On the upstream side, a concrete curtain wall 4′ wide and 10′ deep was provided, the depth of the deepest possible scour at this site below the floor level being 12′ and 15′ for the Weir and Under-sluices respectively. This was considered to be enough as the flexible protection upstream of the Weir after settlement would prevent the full depth of scour occurring in the immediate vicinity of this cut-off.

An additional masonry cut-off about 8′ below the floor level was also provided just upstream of the crest. The purpose of this was to cut off flow through the sand pocket which generally exists under the crest and has a higher transmission constant than the shingle bed.

Cross cut-offs were also provided near the upstream and down-stream noses of Divide Walls as well as at three intermediate points in the Weir to enable the different compartments being bunded off and isolated whenever necessary. These would also be useful in allowing the various diversion cuts to be developed independently.

Exit gradient

The static head across the barrage which causes seepage flow through the subsoil threatens its stability by the undermining of the subsoil.

This undermining starts from the tail end of the work, if the pressure gradient at the toe is in excess of the restraining forces of the subsoil, *viz.* weight, internal friction, etc., which tend to hold the latter in position. This causes a movement of the bed, which is progressive, resulting in the failure of the work. This phenomenon is known as 'piping.'

If we consider flow in a stream tube of unit cross-sectional area, the failure will occur when the gradient $\frac{dh}{d}$ at exit is equal to the weight of the material after allowing for its submergence in water, *i.e.* $w\,(1\text{-}\Sigma)\,(P-1)$ where w = Weight of unit volume of water

P = Specific gravity of sand particles

Σ = pore space in unit volume

Taking $w = 1$, $P = 2{\cdot}65$, and $\Sigma = 0{\cdot}4$, failure will theoretically occur when the exit gradient G_E is 1.

In actual practice, however, there are several other factors which have to be taken in consideration. Some of these are non-homogenity of the subsoil, its angle of repose, the presence of faults, fissures or clay layers, scour at the D/S end and differential head caused by local waves or sudden ponding up U/S of the Weir.

In view of all these uncertain factors Mr. Khosla* has proposed a factor of safety of 4 to 5 for shingle, 5 to 6 for coarse sand and 6 to 7 for fine sand, to be allowed in the critical value of exit gradients.

Determination of the Exit gradient

The value of the exit gradient G_E is given by the formula

$$G_E = \frac{H}{d} \quad \frac{1}{\pi\sqrt{\lambda}}$$

where d is the depth below the river bed,

H is the total head,

and $\lambda = \quad 1 + \frac{\sqrt{1 + \propto^2}}{2}$, where $\propto = \frac{b}{d}$

The exit gradient for the Weir section, with H = 22, b = 140′ and d at the bottom of the filter (R. L. 664·0) = 8′, would thus be $= \frac{22}{8} \times \frac{1}{\cdot 105} = \cdot 289$, giving a factor of safety of 3·46, which is considered to be quite safe for a shingle bed. It may be noted here that when a filter is clear the loss of head in the seepage flow through it is negligible and the exit gradient has thus to be calculated at its underside for safety.

Similarly for the Undersluice section, having a floor length of 150′, G_E at the bottom of floor is 0·275, giving a factor of safety of 3·63.

The exit gradient at the surface (R. L. 670·0) for the Weir section would be 22/14 × 0·136 = 0·213, giving a factor of safety of 4·7. In the Undersluice section, this would be 4·9.

The following statement gives the factor of safety obtained in some of the Headworks :—

Work	Maximum head	Safety Factor	
		At floor level.	Below filter.
1. Sukkur ..	24	5·1	4·1
2. Panjnad Annexe ..	23	5·4	4·5
3. Khanki (Bay 8) ..	23	5·1	4·4
4. Marala ..	14	4·4	4·2
5. Trimmu ..	28	5·0	4·5
6. Ferozepore ..	22	5·0	4·3
7. Kalabagh (Weir) ..	22	4·7	3·5
8. Islam ..	21	3·7	There is only a shallow curtain wall for the D/S cut-off.
9. Suleimanki ..	20	3·3	

*Design of Weirs by Khosla, Bose and Mckenzie Taylor.

Pressures under the floor

The pressures at the various key points under the floor were determined by reading off from the curves based on the mathematical solutions for elementary forms (Plate VII. 6, C. B. I. Publication No. 12) and corrected for floor thickness, mutual interference of cut-offs and slope. The values thus obtained were as follows :—

	U/S Curtain wall.		Toe of glacis.	D/S sheet pile.
	ϕC (D/S junction of floor).	ϕE_1 (U/S Junction of floor).	ϕC_1 (D/S Junction of floor).	ϕE_2 (U/S Junction of floor).
Weir ..	79%	51%	49·9%	24·46%
Undersluices ..	79·5%	51%	49·88%	23·6%

It may be noted that when a filter is choked, the pressure at its bottom increases appreciably thus increasing the pressures at all points upstream. In calculating floor pressures, the downstream cover is thus considered to extend to at least the top of the filter.

The foundations of the barrage are generally on shingle but under the crest the underside of the floor is generally at a level higher than the top of the shingle and there is consequently a pocket of sand.

Sand containing a proportion of shingle, which is really what the shingle layers at Kalabagh comprise, has obviously a lower transmission constant than that of pure sand. As it was doubtful what the effect of the low resistance pocket at the upstream end of the floor would be, the Research Institute was asked to carry out experiments to determine it. The experiments did not show any serious departure from normal pressures and the only action taken in this respect was the provision of a 9″ brick cut-off wall under the crest.

Gravity section

The stability of a barrage is also threatened on account of uplift due to pressures under the floor being in excess of the weight of the floor.

The depth of the floor in the various sections was accordingly designed so as to contain the normal pressures, assuming the specific gravity of concrete to be 2·25.

A gravity section was preferred to the Raft design adopted at Trimmu, as shingle was locally available for the cost of screening only, thus making the gravity section much more economical, even allowing for the additional excavation and pumping. Moreover, there might have been difficulty in getting steel for the raft section.

Inverted filters

To have an adequate cover for the downstream pile line, to safeguard against the steepening of G_E in case of scour downstream, a special pressure relief area is provided between the downstream pile line and the flexible protection. Its width has been kept as 27·25′, *i.e.* practically double the depth of the downstream cut-off. This consists of blocks $4' \times 2{\cdot}75' \times 4'$ overlying 2′ of inverted filter, which consists of 6″ fine shingle 3/16″ to ¾″, 9″ coarse shingle ¾″ to 3″ and 9″ shingle 3″ to 6″. There are jhiris 5″ wide on two sides of the blocks which are filled with fine shingle to allow free seepage flow without disturbing the subsoil.

As a precaution against the filter being choked by silt deposited after floods, pressure relief pipes have been left in alternate blocks which are provided wth silt traps at their exit. It was found by experiment that a fully blocked trap would blow under a head of 2 feet.

A curtain wall $4' \times 8'$ is provided at the end of the filter portion.

Flexible protection

There is a certain depth for bed equilibrium, depending upon the material of which the bed is composed, for a given intensity of discharge. The maximum depth required for this purpose has thus to be calculated and provided for both U/S and D/S and suitable protection afforded against bed scour where this depth cannot be allowed to develop.

This depth of non-scouring flow can be obtained from Kennedy's or Lacey's formula

According to Kennedy $V_o = 0{\cdot}84\ d^{\cdot 64}$

or $d = 1{\cdot}11\ q^{\cdot 61}$

and from Lacey's $R = 0{\cdot}9 \left(\frac{q^2}{f}\right)^{\frac{1}{3}}$

The flexible protection has been designed on the basis of the concentrated flood discharge, *i.e.* an intensity of 393 for the Undersluices and 330 for the Weir. Reducing these figures by 10% to allow for the piers we get 354 and 297. Corresponding with these figures, taking f = 4 for shingle, we get R, the depth of normal scour as 29′ and 25′.

The protection is provided so as to cover one and a half and twice the depth of scour on the upstream and downstream sides respectively, at a slope of 3 to 1. Concrete blocks are provided in about one third of the total width, as in this portion the velocities may exceed 10′ per second which may be able to move stone by erosion. This is calculated as shown below :—

Sections	Upstream side		Downstream side	
	I (Weir)	II (Under-sluices)	I (Weir)	II (Under-sluices)
Water level ..	699	699	695	695
Floor level ..	673	670	670	667
Depth to be covered ..	38	44	50	58
Corresponding level ..	661	655	645	637
Depth below floor ..	12	15	25	30
Length of blocks ($4' \times 2{\cdot}75' \times 4'$) $\frac{1}{3}$rd at 1 in 3 ..	11	16.5	29·33	32·25
$\frac{2}{3}$rd in 4′ stone at 1 in 3 ..	25×4	30×4	40×5	50×5

Deep pier foundations

The piers have been designed to carry a 20′ wide road bridge along with the gates and gearing. Their weight is consequently considerably in excess of the uplift pressure under the section on which they stand and the depth of foundations is worked out from Rankine's formula, after splitting the pier in different sections and calculating the depth required for each section independently under worst conditions. Thus the upstream portion will be subjected to the worst conditions when water is headed up to R. L. 694 and the wind at 40 lbs. per square ft. is blowing downstream. Similarly the downstream portion will have worst conditions when the river is dry, the weight of the floor is submerged and the wind is blowing upstream.

The piers consist of 1 : 4 : 8 cement concrete with 8% plums, faced with precast R. C. blocks, as these give a very good architectural effect and facilitate construction considerably.

Flank Walls

The flanks which have a gravity section have been designed as cantilevers, the two forming part of the floor.

The saturation level, which will correspond to the level of the pressure gradient line, has been assumed as 688 on the upstream and 684 on the downstream side. The various sections have been designed so as to withstand the earth pressure due to dry soil above the saturated level and saturated soil below this level. The pressure of saturated sand has been taken as the full hydro-static pressure plus 50% of the dry earth pressure, which is equivalent to the earth pressure due to the submerged weight of soil.

Originally it was intended to design the flank wall as a mass cement concrete section reinforced for the extra bending moments caused due to point of application of the resultant pressure falling outside the middle third. This design, though economical had, however, to be discarded due to uncertain steel supplies.

The portion with a vertical face was decided to be built in 1 : 3 : 6 cement concrete with 8% plums, faced with precast blocks, as done in the case of piers.

Two return walls have been provided, the upstream being 10′ and the downstream 8′ below floor level to prevent outflanking due to seepage flow, which if not given a suitable length and properly directed may undermine the work from the flanks.

The development of the flank from vertical to 1·5 : 1 slope is done in concrete blocks 5′×4′×2′ laid in situ and from 1·5 : 1 to 2 : 1, where it meets the normal guide bank section, it is done in 2·5 ft. dry stone pitching over 2′ shingle.

Due to uncertainty of flow behind flanks, it was considered desirable to provide weep holes 1½″ diameter spaced 5′ apart in the upstream as well as downstream development portion of the flank wall so that the difference of head across it remains less than the designed figure. The weep holes were fitted with strainers enshrouded by fine shingle to prevent the blocking of the weep holes.

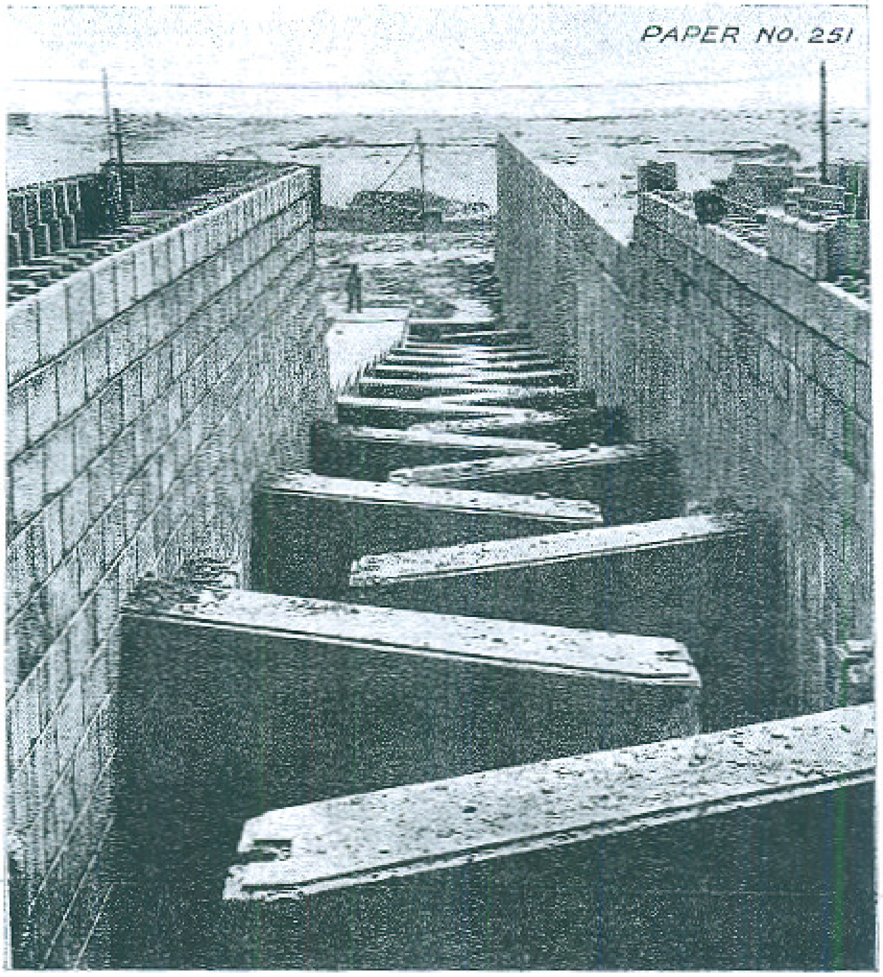
Two gauge wells have also been provided in each flank.

Position of Undersluice-Bays

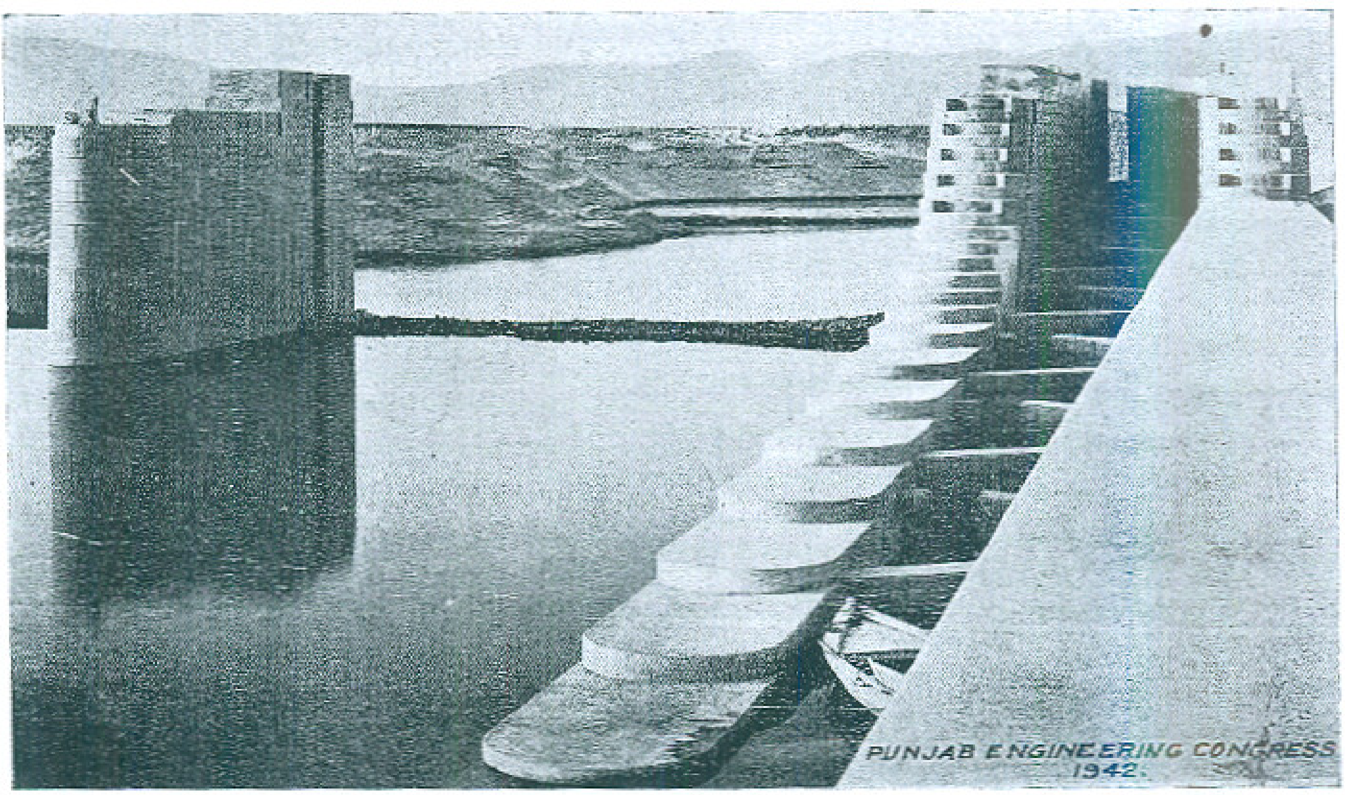
As the river after leaving the Railway Bridge hugs the right bank, the desirability of putting all the 14 bays on the right was at first considered. This idea was, however, dropped as it was thought that it would be better to spread the diversion cut across the barrage and to put half the Undersluices on the right and half on the left, so that, if necessary, one set of undersluices could be bunded off, unwatered and repaired. Undersluices on the left were also desirable in the interest of silt exclusion.

Regarding the 7 Undersluice-bays to be put in on the right, it was suggested that they should start 7 bays from the right with a view to draw in the main current of the river away from the Right Guide Bank. This would have reduced the action along the shank, but in view of the conditions at site, it would not have mitigated the river's direct approach on to the nose of the guide. Moreover this would have resulted in cross flow from either side and there would have to be the additional expense of providing against damage at two points, where the Undersluice and Weir floors with different levels join. It was accordingly decided to locate the right Undersluices along the Right Flank.

As regards the position of the remaining Undersluice-bays, at least two would be necessary against the left bank for an Extractor. With regard to the remaining 5 bays, it was suggested that they



FISH LADDER ALONG THE LEFT DIVIDE
WALL UNDER CONSTRUCTION
(June 1941)



A VIEW OF THE FISH LADDER ALONG THE RIGHT DIVIDE WALI should be put at a sufficient distance out to produce rotary flow if a subsidiary channel forms or if, as at Marala, only one channel persists which approaches the river from the right and flows along the face of the Weir into the Pocket, as in this case, these depressed bays would draw shingle away from the main stream. In the case of Khanki, these depressed bays are 1,500′ away, as a result of which Bay No. 4 did not function satisfactorily for the purpose for which it was designed. The situation has, however, improved since due to a change in the regulation rules effected in 1940. Moreover, the objections outlined in the case of the right Undersluice removed from the flank applied equally in this case. This proposal would have thus meant more initial expenditure, recurring additional cost and doubtful benefit from the point of view of river control. The Left Undersluices were thus also located along the flank as this would give better control of river as well as of the silt and shingle entry into the canal.

Divide Walls

The main functions of a Divide Wall are :—

(*i*) To separate the undersluices from the weir bays to avoid the heavy turbulent action which would otherwise result on account of the water in the two portions being at different levels.

(*ii*) To help in keeping parallel flow, which would be caused by the formation of deep channels leading from the river to the pockets removed from the floor of the barrage, thus avoiding damage to the upstream flexible protection.

Its length on the upstream has to be such as to keep the heavy action on the nose well away from upstream protection of the sluices. Similarly, on the downstream side it should be sufficiently long to guard against action set up by the undersluice discharge damaging the weir flexible protection.

On the Right side, a fish ladder 238′ long was provided with a divide wall 341′ long in continuation of one wall of the fish ladder on its upstream side and a divide wall 152′ long in continuation of the other wall of the fish ladder on its downstream side. These lengths were considered to be good enough to satisfy both the conditions specified above. The upstream nose was sloped at 1 in 3 in regular steps in a length of 81′, since this form spreads the turbulence thus reducing scouring action on the bed. On the downstream side, however, the nose was kept vertical to locate the action as far downstream and thus away from the floor, as possible.

The divide wall was designed to withstand a differential head of 3′ in the superflood condition and the fish ladder, which had steps of 1′ each, was designed to be safe against all river conditions. The latter was founded on double curtain walls 10′ deep, their depth being worked out by Rankine's formula. A 26′ width of concrete blocks and 30′×4′ of loose stone protection on the upstream side and 50′×5′

on the downstream side was provided along the shanks and 26′ width of blocks with 70′ of stone was provided at the noses to guard against the maximum scour that could be anticipated. The noses were founded on wells which were taken to R. Ls 650.5 and 654.0 on the upstream and downstream sides respectively. The difference in level between the Weir and Undersluice floor was adjusted by sloping the block and stone apron from the floor level of the weir to that of the undersluices on the Weir side of the divide wall. ½″ through joints were left in the Divide Wall above the floor level, every 52′ apart, to avoid cracks due to shrinkage. The main section is of 1 : 4 : 8 cement concrete with 8% plums, reinforced for extra turning moments caused due to the differential head. The base was, however, kept of 1 : 2 : 4 reinforced cement concrete. Its thickness was kept the same as those of the piers, *viz.*, 7′, with a plinth formed by a 1 in 12 batter below R. L. 678·0. The top level on the upstream side was kept as 702·0, *i.e.* 3′ above the high flood level and on the downstream side as 693·0. Both the Divide Wall and the fish ladder, were faced with precast blocks 2″ thick and having their lugs projecting in the wall.

In the case of the left pocket, the divide wall would also influence the entry of silt into the canal.

Before fixing the length of the Left Divide Wall, therefore, it was decided to carry out experiments on a model with lengths of 600′, 450′ and 300′, the two positions to be considered being at the 4th and 7th piers of the undersluices. These were conducted by Dr. Uppal of the Research Institute.

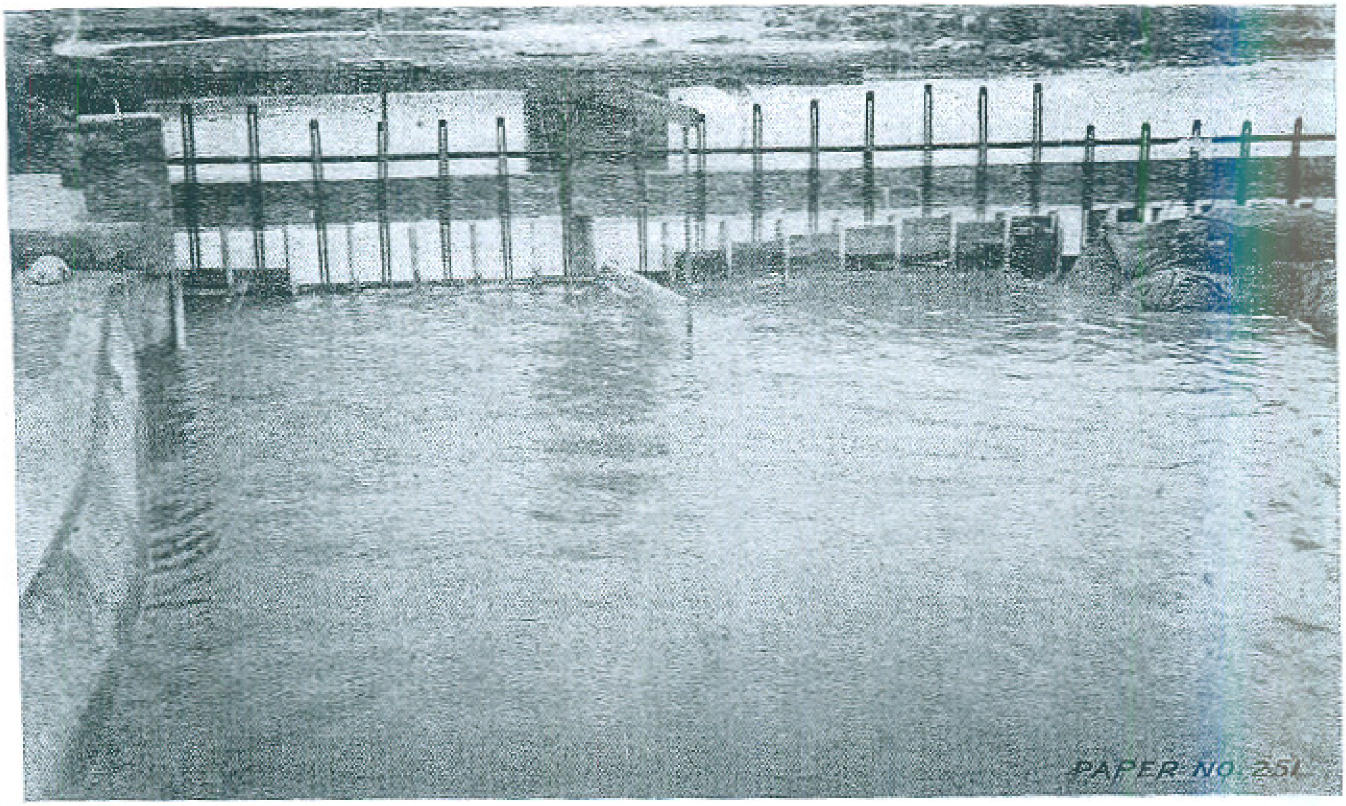
A definite quantity of silt was added in the river upstream of the Weir in the case of each experiment and discharges were run varying from 1½ lakhs to 2½ lakhs. The bed condition in the pocket and along the guide bank for a length of 3,000 feet were maintained the same by moulding the bed to its original condition after each test.

From an examination of the data thus arrived at, it was found that a 300′ length of the divide wall gave the least silt entry into the canal. The silt going into the canal with the divide wall at the 7th pier was also much less than that obtained with the divide wall at the 4th pier. It was therefore decided to keep the left divide wall only 300′ in length on the upstream side, that is the same as on the right side.

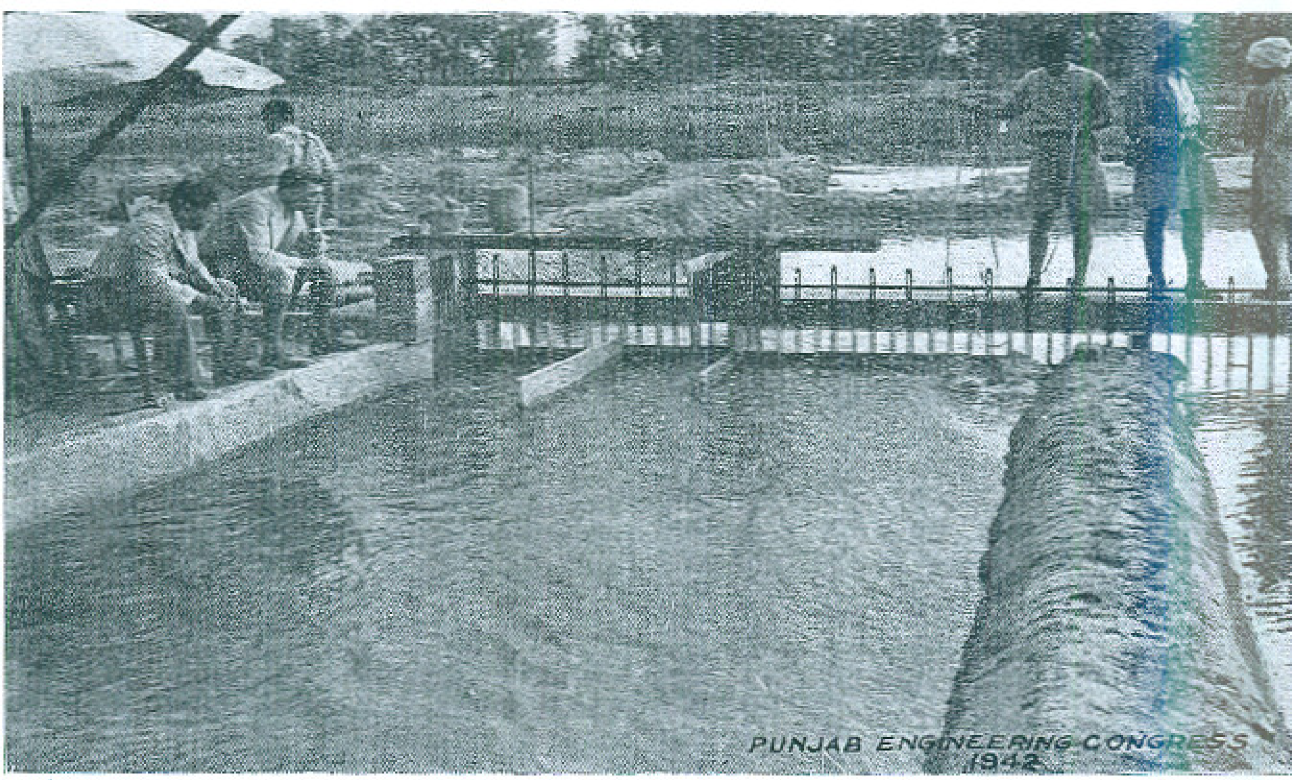
Left Pocket

In designing the pocket, the first point to be considered was the nature of the river bed when the headworks were in operation. This could only be a matter of guess as it could not be determined experimentally, since model technique has not yet developed to deal with varying grades of silt in the same model. It is probable that the shingle bed, which is at present about R. L. 670, may rise, due to the pond area being small and the runoff very large, till it is controlled

MODEL OF KALABAGH HEADWORKS



SHOWING THE CONDITIONS OF FLOW IN THE POCKET WITH KHANKI TYPE OF EXCLUDER AND 300 FEET LONG DIVIDE WALL AT PIER NO. 7 (AS CONSTRUCTED)



SHOWING CONDITIONS OF FLOW IN THE POCKET WITH TRIMMU TYPE OF SILT EXCLUDER COVERING 4 UNDERSLUICES BAYS AND 600′ LONG DIVIDE WALL AT PIER NO. 4 IN ADDITION TO 300′ LONG DIVIDE WALL AT 7TH PIER by the scouring power of the sluices. It was suggested that in such a case the still pond system of working would be impracticable, as the mean diameter of bed silt is 2″, corresponding to f=13, against silt of the grade of f=1 and 1.5 at Khanki and Rupar respectively. If this was so, there was no object in designing the pocket for this purpose, especially as the still pond system is objectionable on account of the periodic closures necessary and possibly unnecessary with efficient silt ejectors. From this consideration, the design was to be such as gave the most efficient silt exclusion, to be determined by experiments.

It may also be mentioned that another school of thought considered the still pond system to be quite suitable and possible in this case, as it was argued that at the time of heading up during low supplies, there would be no velocity in the pocket that could move the shingle.

Silt Excluders

It is very desirable to exclude coarse silt from the canal, one reason being that it is necessary to use flat slopes in portions of the Project in order to ensure command. Another reason is, that for economy it is proposed to use deep narrow sections for the Canal and Branches. With low supplies it will be necessary to head up at places to command off-takes, conditions likely to result in the deposit of silt which will have to be scoured out subsequently.

To determine the design which would give the best silt exclusion at the head it was decided to test the relative merits of the following two types :—

(*i*) The Trimmu type of Silt Excluder with a slab covering the first four bays and a divide wall 600′ long at the 4th pier.

(*ii*) The modified Khanki type covering only two bays of the undersluices without any additional divide wall.

Experiments were accordingly conducted by the Research Institute in the model at Malikpur. The model was run for four hours in each case with a discharge of 1 lakh passing in the river and 7,000 cusecs in the canal. The silt collected in the canal and the undersluices was measured and it was found that with the modified Khanki type the silt which passed into the canal was 44% of that which passed with the Trimmu type.

The experiment was repeated with a concentrated discharge equivalent to 40,000 cusecs passing in the first ten bays. This confirmed the previous results as with the modified Khanki type only 41 % of the silt passing with the Trimmu type entered the canal. It was also noticed that with the Trimmu type there was a considerable turbulence and surging action in the pocket. With the Khanki type, however, the conditions of flow smoothened out and a regular curvature of flow towards the Head Regulator was obtained. The Khanki type was obviously more efficient as it not only removed the coasrse silt contained in the bottom layers of water, but also tapped the heavy concentration of silt which occurs on the inner side of a curve. This type which was incidently more economical also, was accordingly adopted.

It may be noted, however, that the main function of this exclude is to prevent the entry of coarse rolling silt and a proportion of the suspended silt, in addition to keeping out submerged debris brush-wood, etc., from the canal. Most of the suspended silt, however, would be removed in the silt extractors provided in the canal where steady conditions of flow with normal silt distribution upstream of the extractors can easily be obtained.

Silt Extractors

A Regulator is proposed to be provided at R. D. 3,300 of the canal in order that pond level may be maintained in the reach above it. In this reach, a series of five extractors are proposed capable of escaping 1,440 cusecs.

This arrangement is a decided advance on the various methods of silt exclusion attempted in this Province so far. A number of extractors are proposed to be provided as the silt intensity increases very rapidly in the vicinity of the bed and it is accordingly much better to divide the supply available for escapage amongst several excluders instead of using it all in one.

According to Hazen's formula, if t is the time which silt of a certain grade takes to settle 1′ in still water and the extractor area is made t n Q, the percentage of that grade of silt extracted will be 86 and 95 for $n=2$ and 3 respectively. For $f=0{\cdot}8$, $t=13$ and for $f=0.7$, $t=20$. It is accordingly proposed to make the extractor reach $240' \times 3{,}000'$, which will give $n=3$ for $f=0{\cdot}7$ grade silt. This has been kept on the large side as the higher velocities in the upper part of the reach may interfere with its proper functioning.

The extractor reach has been taken to start at R. D. 200, *i.e.*, clear of the Head Regulator.

The first extractor flume will have to deal with all heavy silt entering the canal head, as the depth in the reach above it will adjust itself to move such silt close to the flume. This flume has therefore been designed to take a higher discharge, *viz.*, 480 cusecs, against 240 cusecs in each of the remaining four, to make it capable of dealing with the heavier grade of silt. This will be placed at R. D. 300 of the extractor reach (500′ from head) by which time it is estimated that all silt of grade exceeding $f=1{\cdot}5$ should be on the bottom. In order to provide an efficient settling reach for the fine grade silts, it is necessary to remove the heavier silts as soon as possible and as the time of settlement varies roughly as the square of the particle diameter, it was considered advisable to place the flumes in geometrical progression. The remaining four flumes are accordingly proposed to be kept at R. Ds. 535, 950, 1,700 and 3,000 of the extractor reach.

Each flume will take its supply through a slit 1′ high across the 240′ wide bed of the reach. Normally, the extractor will work under a head of 1′. It will, however, generally be possible to increase this to 4′, in which case the discharge can be doubled and f at entrance increased fourfold.

The extractor tunnels will be 8′×4′ each taking normally 120 cusecs with a velocity of 3·75′. No. 1 will have four tunnels and the others two each.

In the case of extractor No. 1, each tunnel will serve 60′ of bed width; and to ensure correct distribution over this width, a diaphragm is proposed separating it into two. The orifices are stream-lined and are 6′ apart, each capable of taking 12 cusecs under a head of 0·5′.

In the lower four extractors, one tunnel serves half the bed width and to get reasonably uniform distribution of offtake there are 3 diaphragms.

In addition to the above, provision has been made to carry an extra 720 cusecs in the canal below the extractor reach, so that additional extractors may be provided at R. D. 105,000 in case any further silt extraction is considered necessary at a later stage. This site was selected as there would be about 20′ of head available here for working the extractors, which may also be useful for Hydro Electric development.

Head Regulator

This was designed to take 6,000 cusecs for the canal, 1,440 cusecs for the Extractors at head and 720 cusecs for the Extractors near Mianwali, *i.e.*, 8,160 cusecs in all. With the pond level at 694, however, it has to pass 4,000 cusecs extra.

The crest level has been kept at R. L. 683·0, *i.e.*, 13′ above the floor, the maximum pocket depth being 24′. It consists of 7 spans of 24′ each with 5′ piers, which would give the following data :—

Discharge	..	8,160	12,160
Pond level	..	692·0	694·0
Canal F. S. Level	..	690·0	692·3
Waterway	..	168	168
Discharge per ft.$=q$	..	48·5	72·5
Depth over crest$=H$	..	9	11
Coefficient $C=q/H^{1\cdot5}$	..	1·81	1·98

The value of C has been kept about 2, so that there is practically no loss of head through the Regulator, and we have the maximum head available for the working of the silt ejectors, in the canal.

The minimum thickness of piers required for installing gates is 5 ft. and is therefore provided. These are extended upstream as stream-lined vanes so as to divert the flow in the Regulator with the minimum loss of head.

The cistern was placed low enough to support the wave under all practical conditions. The pressures under the floor and the exit gradient were determined as mentioned already and the length of the cistern was kept equal to 5 (D_2—D_1). A glacis sloping at 1 in 5 was provided downstream so that there is practically no loss of head. The breast wall was designed as a cantilever fixed in the diaphragm wall and its bottom was kept at R. L. 694, as by keeping it at 692, there would have been a loss of head of 0·35′ with the canal discharge of 10,000 cusecs. The downstream floor level of the Regulator being 680, the maximum head for which it was designed was 19′. Friction blocks were also provided downstream to dissipate energy.

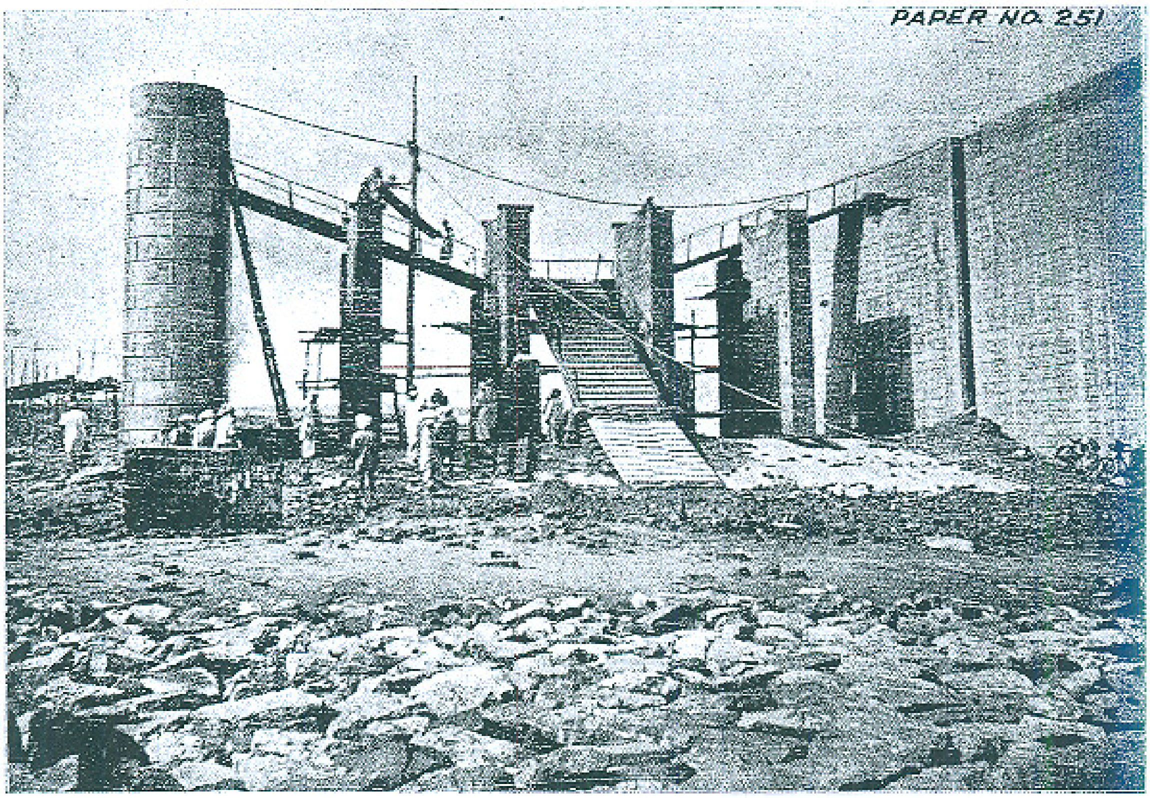
A 25′ roadway was provided over the Regulator. The alternative of constructing a 13½′ bridge which could be subsequently widened to 25′ was also considered but was dropped as the extra cost involved was only Rs. 10,000.

Road Bridge

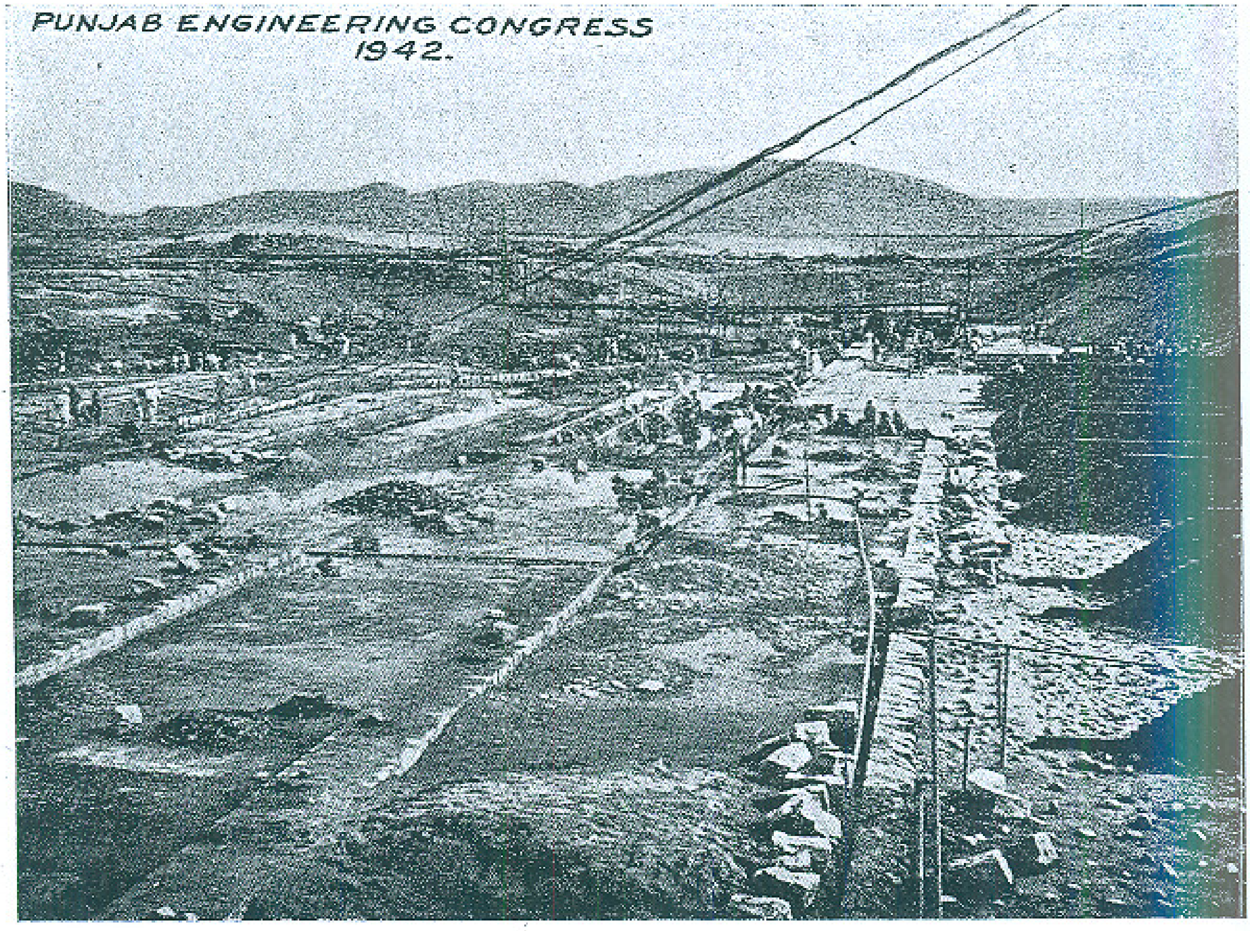
The construction of the Kalabagh Headworks afforded an opportunity for providing a road bridge which would serve as a link between Punjab and the N. W. F. P. The expenditure involved in the construction of an Arterial road bridge was, not justified as the traffic at this site was not likely to develop to such an extent as to warrant its construction. A bridge, however, was absolutely necessary from the point of view of the Irrigation Branch, in view of its convenience for transporting stone to any site required and its serving as a platform from which stone protection could be repaired. In addition, it would be helpful for erecting the gates and gearing.

Three types of bridges were accordingly considered and their cost worked out as shown below :—

Type of bridge.	Cost of bridge including railing.	Extra cost of erection of gates and steel work.	Total.
	lakhs	lakhs	lakhs
1. 10 ft. bridge, light traffic no foot walk ..	2·01	0·6	2·61
2. 10 ft. bridge, Arterial Road, one 2½ ft. foot walk ..	2·60	0·6	3·20
3. 20 ft. bridge, Arterial Road, two 2½ ft. foot walks ..	5·24	..	5·24



SUPPORTS FOR OVERBRIDGE FROM AND RAMPS FOR CONCRETING



SILT EXCLUDER UNDER CONSTRUCTION
(August 1941)

Out of these, the proposal of providing a 10 ft., Arterial Road Bridge with one $2\frac{1}{2}$ ft. foot walk was finally adopted, as it combined as far as possible economy in the present expenditure with provision for future requirements.

The design adopted was the same as that of the bridge on the Emerson Barrage, *viz:* a cantilever-cum-continuous type in which there was a simply supported span of 30′ on 15′ long cantilevers carried from the two adjoining bays. This arrangement provided against failure due to unequal settlement of individual piers.

A 10 units B. E. S. A. loading was adopted and the beams were spaced in such a way that the bridge could easily be duplicated later on, when required, after removing the railing on one side.

The bearings for the beams were of the same type as adopted at Trimmu and consisted of $\frac{1}{2}''$ thick curved steel plates resting on horizontal plates of the same thickness. In order to prevent progressive creep, due to expansion and contraction, the movement was restrained at alternative bearings by means of a rivet head fitting a hollow in the bottom bearing plate. Expansion joints were also provided at either end of the suspended spans.

Gates and Gearing

The gates and gearing were of the same type as adopted at Trimmu and were manufactured in the Central Workshops at Amritsar.

Guide Banks

It was decided to have expanding type of guide banks as recommended by the 1930 Islam Committee and adopted at Panjnad and Trimmu since the bottle neck type at Suleimanki and the parallel type at Islam have not been found to be very successful. In both types, besides the heavy action at the nose, a big shoal is formed along the shank, due to the stream being thrown away from the head. This contracts the channel which scours excessively and produces conditions of disturbed flow at the Regulator. The diverging types ensure a smoother entrance and are less expensive to maintain. They also protect a much longer length of the marginal bund and due to the longer distance apart between the noses are much less likely to be brought in action by any lateral flow.

Their alignment was based on the following principles :—

(*a*) They should be 1,000′ straight upstream of the barrage.

(*b*) The shanks should then splay at a general angle of 1 in 5.

(*c*) The heads should have a radius of 1,000′ from a centre so situated that the distance from the marginal bund plus the radius is twice the distance from the high banks.

(*d*) The head and the parallel portion of the shanks should be joined by a large circular arc tangential to both.

The distance of noses from the Weir line was kept as 4,116′ on the upstream side and 800′ on the downstream side in the case of the right guide bank and 2,962′ and 600′ respectively for the left guide bank. This distance in the case of some of the other Headworks is as follows :—

Headworks	Maximum flood discharge.	Width between abutments W	Width at U/S G. B. Noses.	Distance of noses above Weir line L_1	Distance of noses below weir line L_2	$\frac{L_1}{W}$	$\frac{L_2}{W}$
1. Ferozepore	4,50,000	2,006	2,080	2,700	600	1·35	1/3·3
2. Suleimanki	3,25,000	2,223	1,625	3,500	1,125	1·6	1/2
3. Islam ..	2,75,000	1,621	1,720	3,600	830	2·24	1/2
4. Panjnad ..	7,00,000	3,300	3,850*	3,000	600	0·9	1/5·5
5. Trimmu ..	6,50,000	3,006	4,028*	3,500	600	1·17	1/5
6. Kalabagh R	9,50,000	3,781	4,991*	4,116	800	1·09	1/4·7
L	..	..	..	2,962	600	0·79	1/6·3

*These are widths at the points where 1,000′ curves join the shank at Trimmu and Kalabagh and the 1/10 splay and 500′ curve join at Panjnad.

Spring recommends a ratio of $\frac{L_1}{W}$ as 1·0 or 1·1 and $\frac{L_2}{W}$ as 1/10 to 1/5.

The length in the case of Kalabagh was however fixed from a consideration of the length of marginal bund to be protected, being made approximately half this.

A freeboard of 6′ over distributed superflood was adopted, giving a level of 704 at the barrage, and sloping up at 1 in 2,000.

The top width was kept as 30′ in the case of the shanks and 60′ at the head. The slope on the inner face was kept as 2 to 1 and that on the outer face as 3 to 1 for the top 5′ and 5 to 1 thereafter. The river side was armoured with 1·3′ stone on 1′ spawls upto within 3′ of the top.

The main considerations in the design of the apron were :—

(*i*) that it should be possible to lay them with a moderate amount of pumping.

(*ii*) the amount of stone should be sufficient to provide for 2·25 R at right head 2·0 R at left head and 1·5 R on shanks, R being the maximum depth of scour.

With an average intensity of discharge of 250 cusecs, and f=4, we get R=23, from Lacey's formula.

It may be noted that in the case of a river in flood R is practically equal to d.

The aprons were proposed to be laid at a level of 675 and it was decided to have 10 c.ft. of stone per ft. of depth to be covered. The quantity of stone required thus worked out as follows :—

	R. Head	L. Head	Shanks
W. L. ..	..	696	..
Apron level ..	..	675	..
Depth required ..	52	46	35
Level of toe ..	644	650	661
Depth to be covered	31	25	14
Stone required ..	310	250	140
	say $70\times\frac{6+3}{2}$	say $60\times\frac{5.5+3}{2}$	say $35\times\frac{5+3}{2}$

Allowance was of course made for slope in calculating levels at any section.

The top level of the downstream guide bank was kept at 701 and the section adopted for the heads was the same as above but the stone in the shank aprons was kept a bit more *viz.*, $40\times\frac{5+3}{2}$ *i.e.*, 40′ wide with depth of stone ranging from 3′ at the toe to 5′ at the outer edge.

Marginal Bunds

The top level of the marginal bunds was kept as 706 *i.e.*, 2′ higher than the guide bank level at the barrage, so as to allow for the level at the guide bank noses. The top width was kept as 30′ with 3 to 1 slope on the inner side and a similar slope on the river side up to R. L. 700, below which the slope was 5 : 1.

A T-headed spur was also provided to protect the left marginal bund from the action of the Jaba and its outer face was pitched with stone in the reach likely to be affected by the torrent.

CONSTRUCTION

Organization

The work was placed in the charge of two Divisions, with five Sub-Divisions. The Kalabagh Division which started functioning on 15th May, 1939, was responsible for the construction of the Head-works, supply of materials, Railway and Quarry. The Power Division looked after the Power House and Workshops and was responsible for the upkeep of all pumps, mixers, compressors, etc., in the field.

In addition, it had to look after the experimental farms in the Thal as well as the maintenance of works in the defunct Mianwali Division, after the latter was closed on 31st October, 1940.

These Divisions remained in the charge of the following officers :

KALABAGH DIVISION

R. S. Duncan	..	15th May, 1939 to 28th September, 1939.
S. I. Mahbub	..	29th September, 1939 to 27th October, 1939.
T. A. W. Foy	..	28th October, 1939 to 15th September, 1940.
S. I. Mahbub	..	16th September, 1940 to 8th October 1941.

POWER DIVISION

B. L. Sakhuja	..	1st June, 1939 to 31st July, 1940.
T. A. W. Foy	..	1st August, 1940 to 31st August, 1940.
C. L. Handa	..	1st September, 1940 to 27th June, 1941.
S. I. Mahbub	..	28th June, 1941 to 30th June, 1941.

The Power Division was closed and amalgamated with the Kalabagh Division on 1st July, 1941. At the time of the writer's taking over in September 1940, the floor of the Right Undersluices had been practically completed and concrete had been poured in weir bays No. 35—49. Work was also in progress on the right flank and the right Divide Wall.

The Superintending Engineer in charge of the construction was Mr. F. F. Haigh till September, 1941, and was followed by Mr. E. L. Protheroe in October, 1941. Mr. E. O. Cox, M.B.E., was the Chief Engineer up to 30th May, 1940, followed by Mr. A. St. G. Lyster who handed over charge of the Thal to Mr. F. F. Haigh on 24th September 1941.

The design work as well as the preparation of Major Estimates was done in the Central Designs Office, under the direct supervision of the Chief Engineer. The charge of the Central Designs Office was held by Mr. J. M. Macintyre up to 31st October, 1939 followed by R. B. L. Kanwar Sain up to 1st July, 1940, and then by Mr. B. K. Kapur.

Rates

The Haveli schedule of rates for labour was suitably amended by Mr. R. S. Duncan and the author in the light of the experience gained on the Haveli and tenders were then invited by the Chief Engineer on the basis of this revised schedule. Over 300 tenders were received and the average of the rates tendered approximated very closely to the schedule. This schedule was accordingly adopted and the Executive Engineers were empowered to allot work on a work order basis without calling for any further tenders.

In accordance with the Haveli precedent, concession rates were obtained for cement and steel as well as for the railway freight of cement.

No special rates for harvesting or the Hot Weather period or any importation charges were allowed all through the construction although we were handicapped by the fact that on the closing down of work on the Main Line in September, 1940, no reserve was available where the labour could be employed during comparatively slack periods on the Headworks.

Site for the Colony

The site for the Canal Colony was selected along the proposed railway line leading from the Headworks to Daudkhel railway station and located so as to avoid any crossing over the Jaba between the Headworks and the Colony. All officers' quarters and permanent buildings were located on the Dhaya, to avoid any water-logging when water was headed up in the pond. The temporary buildings, Power House and Workshops were, however, kept below the Dhaya so as to be nearer the Headworks and were protected from floods by a suitable protection bund.

Land Acquisition

The first preliminary before any work could be taken in hand was the acquisition of land. Necessary notifications were sent up after the preparation of land plans and an effort was also made to obtain possession by Razinamas with the owners concerned. Possession of a portion of the land required for the temporary quarters and Power House was thus obtained on 5th June, 1939. The land for the railway line connecting the Headworks with Daudkhel railway station could not, however, be acquired till July, 1939, and that under the Headworks till September 1939.

This would show that a considerable time can be saved if action to acquire the land is taken as early as possible in all major projects.

The work of land acquisition was done by a special Land Acquisition Officer with a staff of 1 Naib Tahsildar, 2 Qanungos and 12 Patwaris. To watch the interests of the Department, a special Zilladar assisted by two Patwaris was also deputed for this work and his recommendations for rates were forwarded to the Land Acquisition Officer through the Executive Engineer.

The cost of land is one of the main items of expenditure on a work of this nature and a very rigid control in the matter is necessary by the Executive Engineer in charge, as the labour involved is more than amply repaid by the results obtained. To cite one instance, in one case of land acquisition on the Thal, the author was able to save Rs. $1\frac{1}{2}$ lakhs by collecting data which proved that the rate recommended by three successive Deputy Commissioners and accepted by the Financial Commissioner was four times as much as it really should have been.

Construction of Buildings

A few temporary buildings were started in June, 1939, and others were taken up as soon as the necessary land was acquired. Practically all the buildings were completed by December, 1939. These included, amongst others, Power House, Workshops, Rest House, Executive Engineer's quarters, 1 permanent and 1 temporary; Sub-Divisional Officer's quarters, 1 Pt. and 5 Ty; Divisional Office, 1 Pt. and 1 Ty; Senior Clerks and Subordinates quarters, 10 Pt. and 47 Ty; Junior Clerks quarters, 12 Pt. and 61 Ty; Mistris quarters, 8 Pt; Menials, quarters, 25 Pt; Married Menials quarters, 83 Ty; Unmarried Menials quarters, 280 Ty; Dispensary, Post Office, Godowns, Bazar shops, etc., as well as a few buildings at Daudkhel railway station.

One innovation in the buildings was the type of roof adopted for Officers' quarters, Divisional Office, Rest House, etc. Inverted T-beams as shown in (Fig. 1) were used with tiles on top thus giving an economical hollow roofing, instead of the R. C. troughs used at Trimmu. A statement showing the comparative cost at Daudkhel of the various types considered is given below :

Type of roof	Cost % s.ft. for room 18′×24′			Cost % s.ft. for room 14′×15′		
	Rs.	a.	p.	Rs.	a.	p.
1. R. S. beams with R. C. battens and tiles ..	82	5	0	60	8	0
2. R. S. beams with special R. C. battens and special tiles ..	96	14	0	73	13	0
3. R. S. beams with inverted T-battens and tiles ..	98	8	0	78	8	0
4. Special R. C. battens with special tiles ..	88	14	0	82	14	0
5. Inverted T-battens and tiles (as adopted at Daudkhel) ..	84	4	0	77	10	0
6. R. C. troughs with tiles (Trimmu type) ..	148	0	0	130	4	0

FIG. 1
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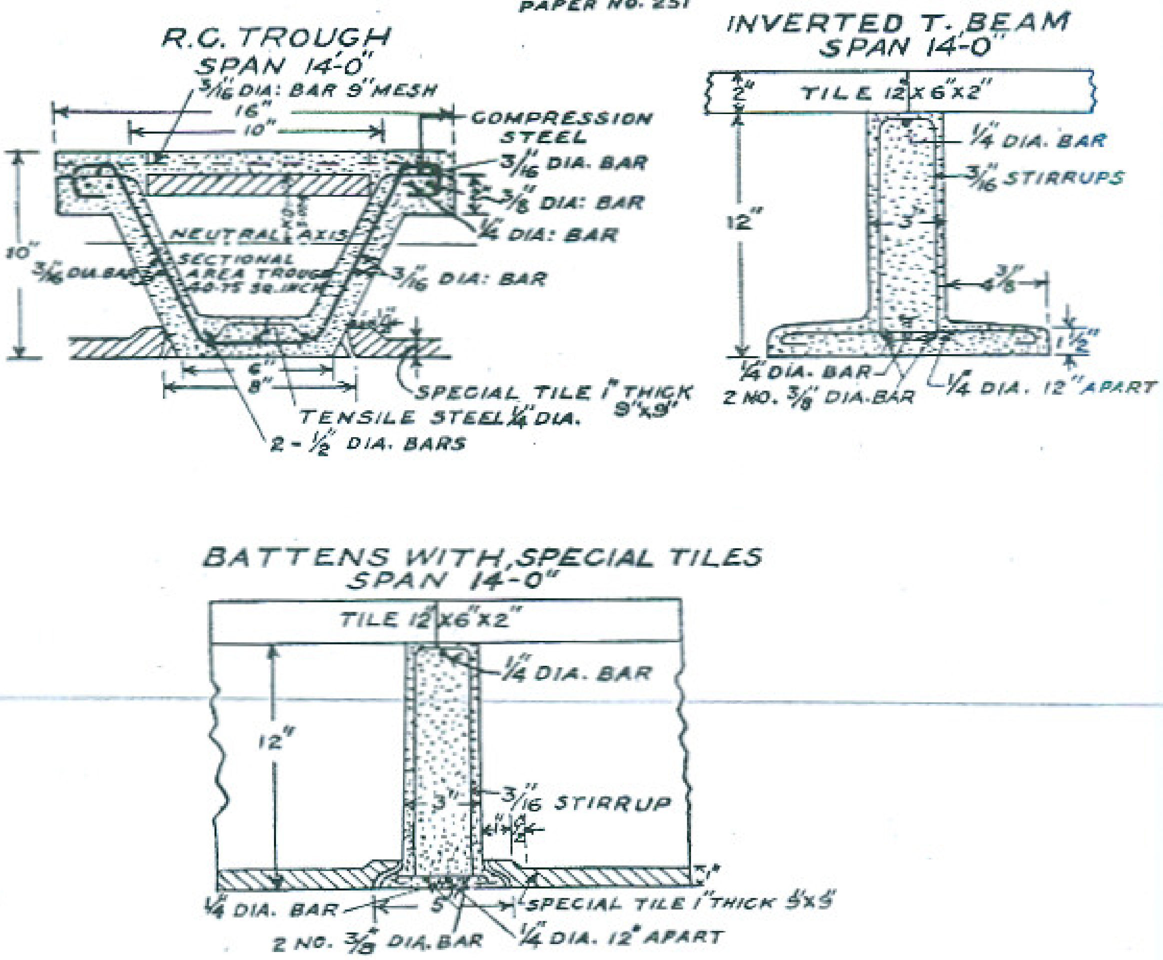


Railway Sidings

It was decided to have three assisted sidings at Daudkhel Railway Station, *viz.* one for a loaded rake, one for an empty rake and one for other miscellaneous traffic, the minimum length of the sidings being 1,450′ so as to accommodate a full rake. Work on these was commenced by the Railway Department in July, 1939, and was completed by September, 1939.

B. G. Lines were run both U/S and D/S of the Weir, on the U/S Ring Bund and on both the guide banks. The latter considerably facilitated the carriage of stone on to the guide banks. Sidings were also made for the brick kiln, workshops and Power House, precast site, weigh bridge, etc., in addition to the station yard where a sick siding and a loop along with sidings for loaded and empty rakes were also provided.

Supply of Materials

(*a*) *Pitching stone*

The total requirements of pitching stone, which is defined as good stone weighing from 60 lbs. to 200 lbs. each, were 77 lakhs c.ft.

To investigate the possible sources of supply, a survey of the hills about 2 to 3 miles to the east of Daudkhel was carried out soon after the opening of the Division and two good faces were found. The proposal of getting supplies from these, however, did not materialize as it involved expensive crossings in the railway yard at Daudkhel.

Arrangements were also made to obtain stone locally from across the river but it was felt that the supplies thus obtained would be only nominal as compared to our total requirements.

The main sources of supply which could be relied on to yield the desired quantities were thus :—

(*a*) Sikhanwala Quarry, an I. B. Quarry which supplied all the stone for Trimmu Headworks.

(*b*) Paikhel Quarry, a Railway quarry situated about 4 miles to the north of Paikhel, a railway station 11 miles from Daudkhel.

Out of these, it was more economical to get stone from Paikhel, as the Railway freight in this case was appreciably less. This quarry had, however, to supply ballast for the Railways also and as such could not meet our full requirements. The supplies from this quarry had thus to be supplemented from the Sikhanwala quarry.

To improve the supplies from Paikhel, two new sidings were laid in the Quarry at a cost of about Rs. 80,000 in addition to a siding that had to be provided at Paikhel railway station to accommodate a rake.

The conversion factor of the stone obtained from Sikhanwala as well as Paikhel was 24 c.ft. per ton, loaded in trucks.

(*b*) *Stone Setts*

About 260,000 s.ft. of stone setts were required for use in the portion of the floor and the piers which was to be subject to the action of heavy rolling shingle.

Tenders were accordingly called for this supply and work allotted to the lowest tenderer at Rs. 60 per hundred s.ft. F. O. R. stations of despatch.

The specifications of these setts were as follows :—

"The setts shall be of hard quartzite stone free from faults and flaws. These shall be between 16″ and 18″ deep and the top dimensions shall not be less than 8″ in either direction. At least 50 per cent of the setts shall have a superficial area of not less than 72 square inches per sett. The top surface and the four sides to a depth of 6″ from the top will be squared and hammer dressed. The lower portion of the four sides shall not project beyond the hammer dressed faces and the undressed section no where shall be less than one half the area at the top."

According to the original contract, the main supply was to be made from Jammu with not more than 25% of the total supply from Pathankot. The contractor's arrangements at Jammu, however, failed due to some deadlock with the state authorities and the main supply had thus to be made from Nowshera and Abbotabad where stone of good quality complying with the specifications was available.

The conversion factor of stone received from Jammu and Pathankot was 15 c.ft. per ton, whereas for the stone received from Abbotabad and Nowshera it was 15.8.

(c) *Shingle*

The total quantity of shingle required was about 78 lakhs c.ft. An attempt was made to obtain as much of this as possible from the excavation. All shingle excavation was heaped in separate stacks and screened. On some of the mixers no outside supply was permitted and the contractors had to replenish their bins from the local screenings. Any laxity in their arrangements for screening thus directly affected the work on their mixers and this kept them up to the mark. All shingle over 3″ in size was also removed and pieces larger than 6″ were selected for use as plums, the size used for the mass concrete being 3/16″ to 3″.

An amusing incident was once noticed when a donkey man was caught carrying the pit shingle, without the removal of sand, to a bin. The bin, however, showed no admixture of sand in the shingle. It was discovered that the sacks in which the shingle was being carried over the donkeys were of an open weave. As the lead involved was about 10 chains, the joltings of the donkeys, while walking, were enough to make all the sand leak out through the sacks, thus leaving clean shingle by the time they got to the bins, and dispensing with the need of screening.

No washing had to be done in the case of shingle obtained from the excavation as it was free of all dust and foreign matter and the average rate for this supply in the bins worked to about Rs. 2 per hundred c.ft.

As only 67 lakhs could be obtained from the local screenings, the balance of 11 lakhs was obtained from outside. This supply was arranged by having a shingle quarry about 2 to 3 miles from the Weir, from where the stuff was carted by hand-shunting in N. G. Trucks to a siding near the Power House, where B. G. Trucks were loaded and carried to the bins. The average rate in this case worked out to about Rs. 5 per hundred c.ft. at the site of the work.

Some carriage was also done by camels and donkeys direct from the quarry site.

(*d*) *Bricks*

The total quantity of bricks required was about 200 lakhs and of these about 80 lakhs were purchased locally from Mari Indus to facilitate an early completion of the buildings.

The balance of 120 lakhs was obtained from the government kilns which were located on the line connecting Daudkhel railway station to the Headworks, about 3 miles from the latter; this site having been selected after getting soil samples from various sites analysed in the Research Institute.

Work on these kilns was allotted, after calling for tenders at Rs. 7-7-0 per thousand Two sets of chimneys were worked on each kiln and the combined average daily output was about 30,000 bricks.

Special attention was paid to moulding as on this mainly depends the quality of the bricks outturned. The following points, which are sometimes apt to be lost sight of, deserve special mention in connection with the proper control of the manufacture of bricks :—

(*i*) The earth to be used in moulding should be allowed to weather, after being broken up to powder, for at least 24 hours and it should not be used for another 24 hours after its having been made into a plastic mass with the admixture of water.

(*ii*) The admixture of water should be just enough to make the clay workable and too much water should be avoided. A rough and ready method for testing the consistency of pugged earth is that a man should be able to stand on a heap without its settling underneath.

(*iii*) The surface of the moulding platform should be kept perfectly level and uniformly sprinkled with sand.

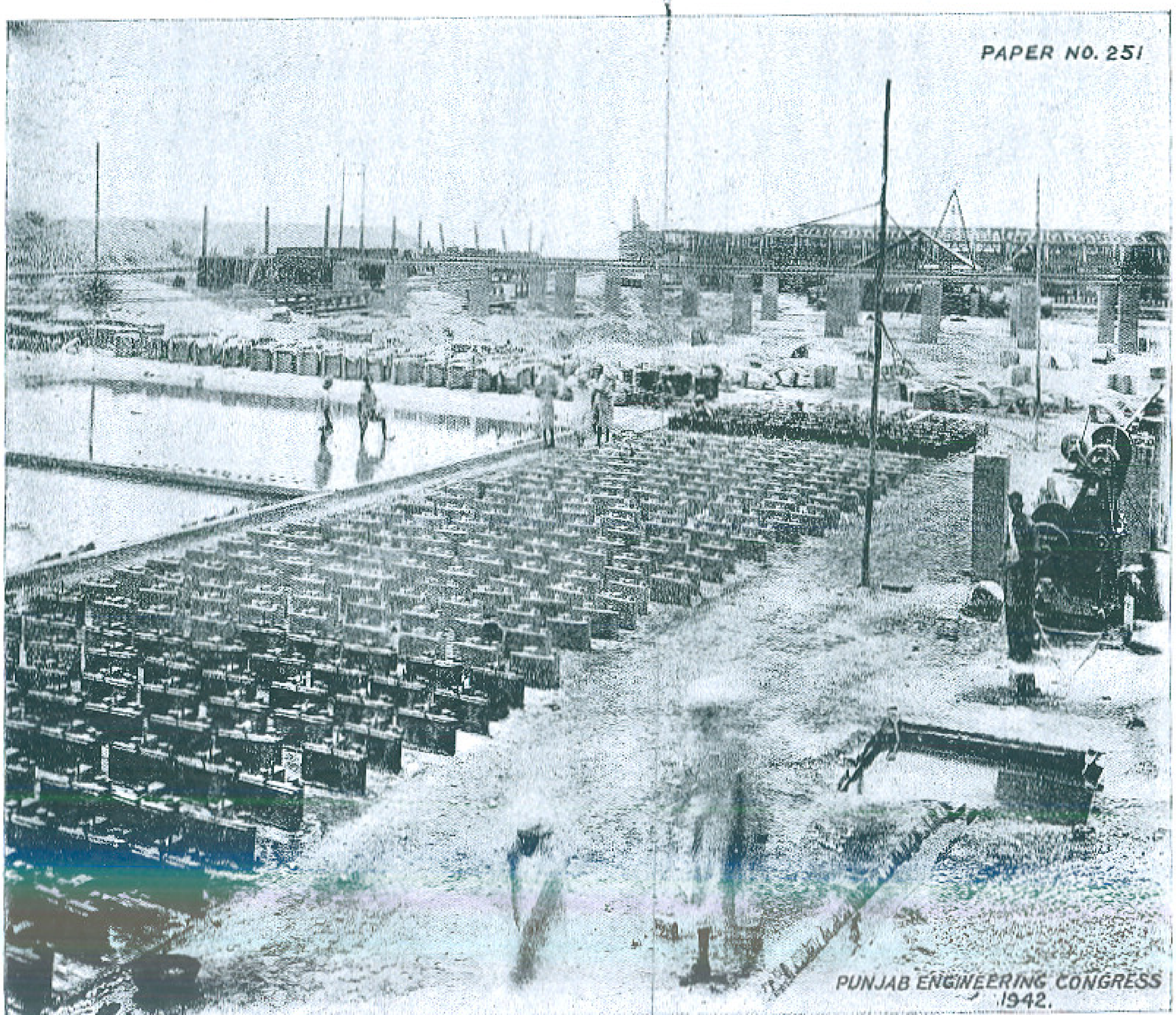
(*iv*) The pugged earth should be rolled on a gunny bag and not on the ground, without the admixture of any sand. Sand should be sprinkled round the *thapi* only when it is ready to be dashed in the mould and at no earlier stage. This would need special watching, at least in the early stages.

(*v*) All surplus earth removed from the mould should be thrown in a basket, provided for the purpose and never mixed in the clay being used for moulding.

(*vi*) All moulds should be kept clean throughout. These must be frequently checked by the Sub-Divisional Officer and every time the moulds are changed, the old ones must be invariably destroyed.

Precast Shells

The piers, divide walls and flanks were faced with 2″ thick 1 : 2 : 4 precast shells of the same type as adopted at Trimmu; the only difference being that after some time it was found possible to reduce



A GENERAL VIEW OF THE PRECAST WORK

the original reinforcement by. about 30 per cent., as shown in the Fig. 2.

FIG. 2
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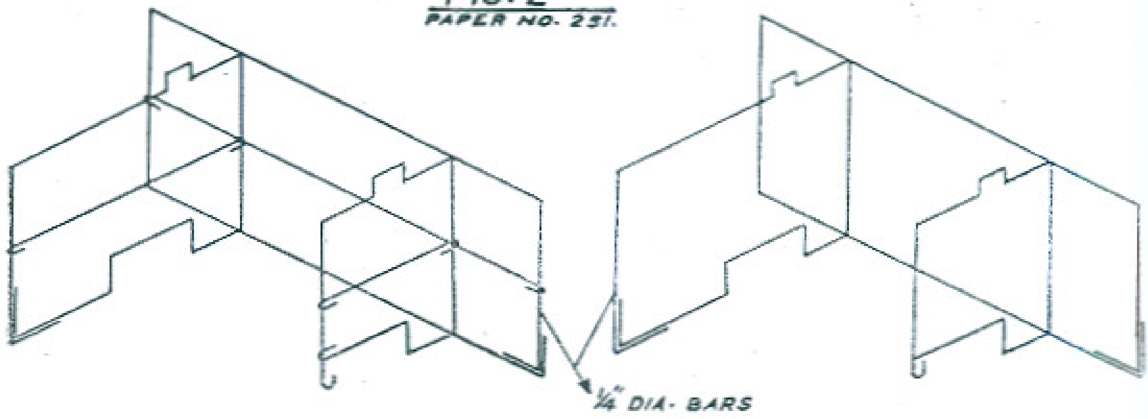

ORIGINAL REINFORCEMENT AMENDED REINFORCEMENT

These had two lugs of different lengths at the back for bonding in the concrete with projections on the upper edge and corresponding recesses on the lower to enable them to interlock with each other. The total number of units manufactured was 120,000.

Power House

The Power House which had to supply energy for all the electrical pumps, mixers, compressors, etc., started functioning on 6th March, 1940. This consisted of one 150 K. W. Ruston set, which was originally ordered for the Emerson Barrage but was received just in time for the Kalabagh Barrage, two No. 125 K. W. S. L. M. sets, which had been in service since 1928, and two No. 250 K. W. Crossley Diesel Generating sets which were specially obtained for this Barrage. Power was generated at 3,300 volts and stepped down to 440 volts at two sub-stations.

Workshops

To cope with the heavy repair work of all the plant in use, it was necessary to have a well-equipped workshop. This was fitted with 5 lathes ranging from 6′ to 20′ bed, 2 radial drilling machines, I wheel-turning lathe, a vertical drilling machine, shaping machine (12″ stroke), tool grinding machine, screwing machine (up to 4″ pipes), metal punching and cutting machine, rolling machine, hacksaw machine, Bandsaw, power hammer, Cupola, Blower fan, Roots blower, etc. The workshops started functioning on 9th December, 1939.

Other Plant and Machinery

A lot of plant was obtained from Trimmu and Sukkur but some new purchases also had to be made. The main plant available for