PAPER No. 204.

THE DEG DIVERSION AND THE COMBINED HEAD FALL AND AQUEDUCT.

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Introduction.

The work of Deg Fall and Aqueduct which was constructed in the summer of 1935 at a cost of Rs. three lacs and twenty thousand may not perhaps be considered large enough to form the subject matter of a paper for discussion at this Congress. But both the scheme of the Deg Diversion, which has a history behind it, and the technical and constructional details of the work itself, have a significance for the practical Engineer far exceding the importance of the work from considerations of its magnitude and this is the reason for the presentation of the paper.

The Upper Chenab Canal, Main Line Lower, generally runs through low lying country and is in heavy filling. This has resulted in extensive waterlogging of the lands adjoining the canal. Various antiwaterlogging measures, some of them highly expensive, have been resorted to from time to time and the Deg Diversion Scheme, which by diverting the main canal at its 57th mile into the Deg Diversion channel leading to the Ravi River by way of Sharakpur will greatly, if not completely, eliminate waterlogging of the country in a length of nearly thirty-six miles from the point of diversion to the present outfall of the canal into the river at Balloki, is the culmination of those measures.

The scheme was first conceived by Mr. A. Murphy then Superintending Engineer, and the Deg Fall at a cost of Rs. 1,20,000 was constructed in the year 1928 about a thousand feet above the present Fall and Aqueduct. Just however as the work came into action it collapsed under a head of only 5·0 feet although it was designed to stand 13·0 feet. A plan of this work is shown in plate II.

At a subsequent inquiry, the failure was attributed to fine sand fill in a scour hole at the site, but it is now obvious that this was due to the absence of a downstream pile line or cut-off. This was the first work of any magnitude constructed in the Punjab without this protection in sand foundation and the first to fail. Simultaneously with the failure, the Right upstream wing wall cracked as seen in the photograph opposite, thereby reducing the length of impervious floor by nearly a half. This

defect has been rectified in the present construction, and the concrete foundations have been carried 20 feet below the spring level and lines of piles enclosing the work go another 15 to 20 feet.

The new work has now been in operation for over a year, and although the 8-feet Fall for which the work has been designed will take some time to develop, as the scour in the bed of the Deg Diversion Channel works upwards towards the Fall, the work will be quite safe under the worst conditions of its working.

The Fall carries over it an R. C. Aqueduct 10′×7′ for the Sikhanwala distributary crossing, with a 10-feet roadway over the top of the Aqueduct and provision has been made for fitting turbines between the Aqueduct and the crest wall, for generation of electrical energy as explained later on.

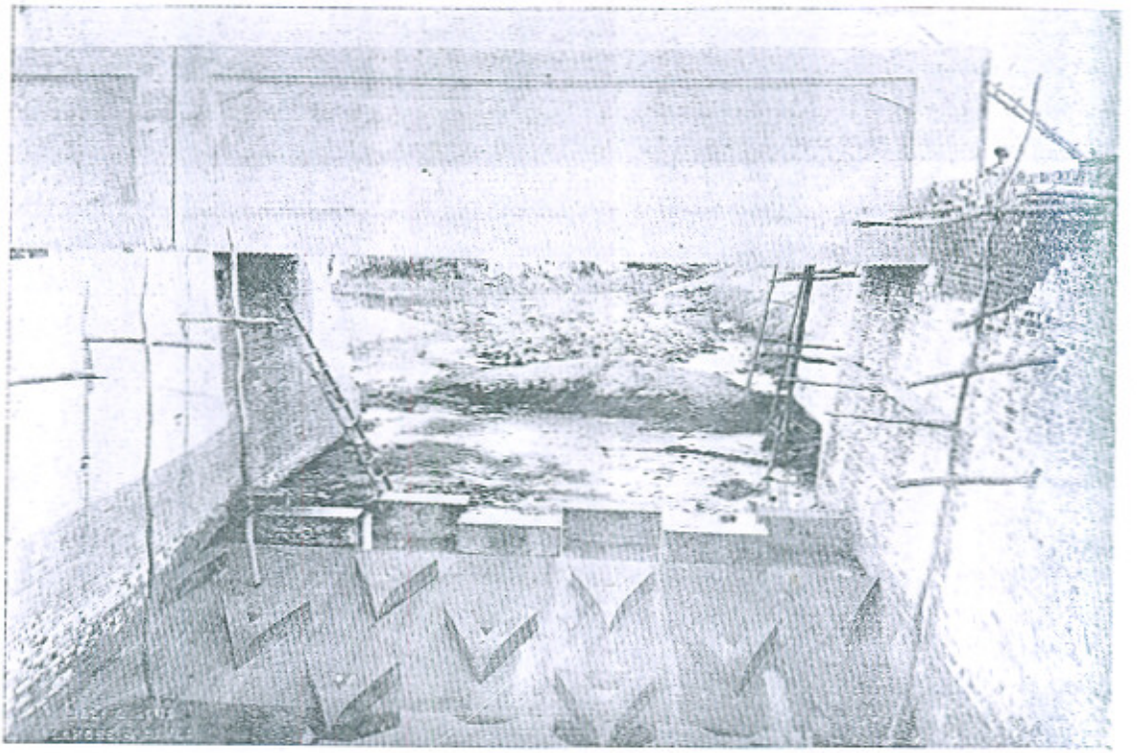
Selection of site for the combined Head fall and Aqueduct.

The site at which the Fall has now been built was finally selected in January 1935 after many years of study, investigation and correspondence. This site is only 1,000 feet below the head originally constructed as shown on Fig. 1, Plate I.

Borings were first taken during the year 1932 and trial bores 500 feet apart 30 feet deep were made over the whole area on the left of the canal from R. D. 2,81,000 to R. D. 2,87,000 for a width of 2,000 feet so as to get a deep clay layer on which to locate the work. The borings revealed that the strata consisted on the top of a layer of stiff clay impregnated with Kankar nodules varying in thickness from 5 to 30 feet, below which there was a bed of sand getting coarser as the borings were taken deeper.

As the spring level near the canal was high and the thickness of the layer of clay uncertain, it was first decided to construct the work near R. D. 19,000 of the Deg Diversion Channel, where the spring level was 14·0 ft. lower than at the canal; and as the site was far removed from the latter, it was free from the effects of direct infiltration from it. This site, however, involved carrying the canal at a high level for 4 miles and as the Deg Diversion Scheme is fundamentally an anti-waterlogging measure, it was considered advisable to drop the supply level as close to the canal as possible. Moreover, with a regulating Fall 4 miles away it would have been very difficult, if not impossible, to regulate and control the supply in the early stages, while the Diversion Channel was scouring out. This site would also have necessitated the construction of a separate Aqueduct for the Sikhanwala Distributary near the Head of the Deg Diversion Channel, and the cost of the two works would have been much greater than if they were combined at one site.

In January 1935, the late Mr. H. W. Nicholson, C.I.E., Officiating Chief Engineer, after inspecting the site ordered the two works to be



ARROWS AND BLOCKS FOR DESTRUCTION OF ENERGY.



PUMPING ARRANGEMENT AT THE DEG FALL.

combined into one and to be built at or near R. D. 1,000 of the Deg Diversion Channel, if a site could be found, where the bottom of the clay strata was above the downstream floor.

Three sites were tentatively selected on the basis of the borings done in 1932 and the site finally chosen about a thousand feet below the old Fall was considered the most suitable and was adopted. (See Fig. 2, Plate I).

Tests were made on the soil of this site in the Irrigation Research Laboratory as a result of which it was found that the transmission constant depended on the fineness of the sand and not on the quantity of clay present. With very fine sand the condition known as "Dall Dall" resulted even with the smallest admixture of clay.

Design of the Present Work.

The work as finally sanctioned and constructed consists of 5 spans of 35 feet each, with 4 piers, each 4′ thick, giving a total width of 191·0 feet which is equal to the designed bed width plus depth of the channel.

A part plan of the work is shown in Plate III and the longitudinal and Cross Sections are shown in Plate IV.

The floor is designed for a total head of 20 feet. Upstream of the crest, there is a 20-foot length of pacca floor ending in a 3-foot deep curtain wall. Downstream, the floor is 87 feet long. The entire downstream floor is boxed in by 15′ interlocking steel sheet piles across the downstream end and on the flanks, while there are 20 feet long piles under the crest. The upstream wing walls are founded on 20 feet sheet piles which branch off from the crest line of piles and have the same bottom level. These wings are warped from 1 in 6 to $\frac{1}{2}$: 1.

The impervious floor upstream is given a 15-foot length of pervious protection at its up-stream end while the main impervious floor downstream is given a 50-foot length of pervious protection, the latter consisting of 3 feet stone pitching over 6 inches of Kankar ballast.

The special shape of the crest wall, a section of which is shown in Fig. 1 Plate IV, was designed to give the maximum co-efficient of discharge 3·7 with the minimum destructive action downstream. The arrows and blocks shown on the glacis of the cistern in the photograph opposite are designed to dissipate energy. The crest has been raised to give high levels upstream with low discharges to enable the Mangtanwala—Sayadwala distributary (500 cusecs) taking off upstream (see Plate I Fig. 2) to be fed without heading up. This raised crest will be dismantled if and when the proposed turbines are installed for generation of power. Plate IV, Fig. 3 shows the arrangement of turbines. The cistern below the crest has been put in to work as a draft tube for the turbine installation.

The length of the sheet piles, the pacca floor and loose protection downstream, the shape and position of the crest and the best method of destroying energy and reducing scour in the bed downstream, were determined by experiments in the Hydraulic Laboratory of the Punjab Irrigation Research Institute, a brief description of which will be found on pages 37 and 38 of the Report of the Institute for the year ending April 1935. The arrows and blocks provided as a result of these experiments are shown in the photograph.

The downstream floor is 4·0 thick and consists of 1 : 2 : 4 cement concrete, 1½" Pathankot shingle being used as coarse aggregate. The floor slab is designed as a continuous reinforced concrete footing to distribute the load of the piers, aqueduct and turbines, over the entire floor area, as this load is much more than the uplift pressure caused by the maximum possible heading up at the Fall with the downstream practically dry. A grid of 5/8" diameter bars at 12" centres has been provided at the top, for temperature and other stresses.

The piers are 4 feet thick laid in 1 : 3 cement mortar and the maximum intensity of load on them at the upstream end works out to about 4 tons per square foot against a safe crushing strength of 10 tons per square foot. It is usual to make the wing walls only 0·4 of the height, but the wing walls in this work have been designed to withstand wet sand with an angle of repose of 20° or an equivalent fluid pressure of 56 lb. as compared to water pressure of 62·5 lb. Therefore, the thickness of the walls at bottom which would have been only 8·5 feet in case of clay or moist sand had to be increased to over 13 feet for a wall 21 feet high surcharged with 5·0 feet of dry clay.

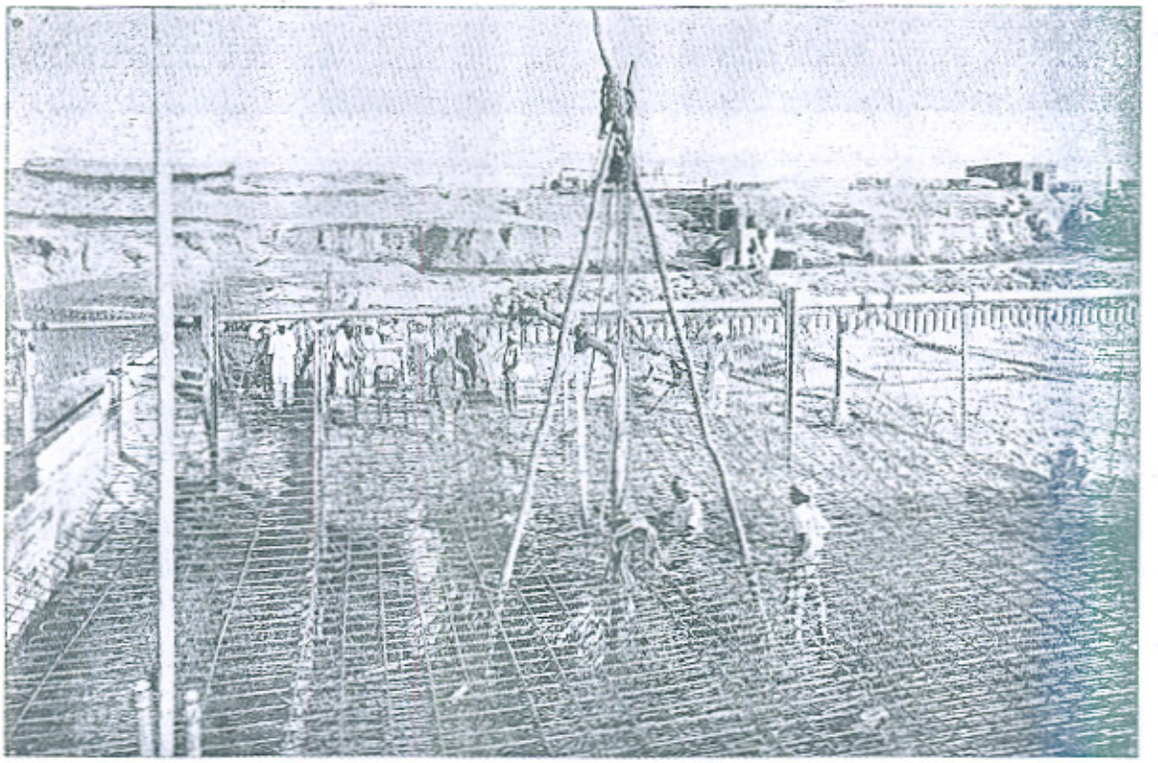
Arrangement has been made for temporary regulation by means of needles, which will be necessary until the channel down to R.D. 20,000 is capable of carrying its full discharge and a light foot bridge has been provided for the purpose.

Foundations.

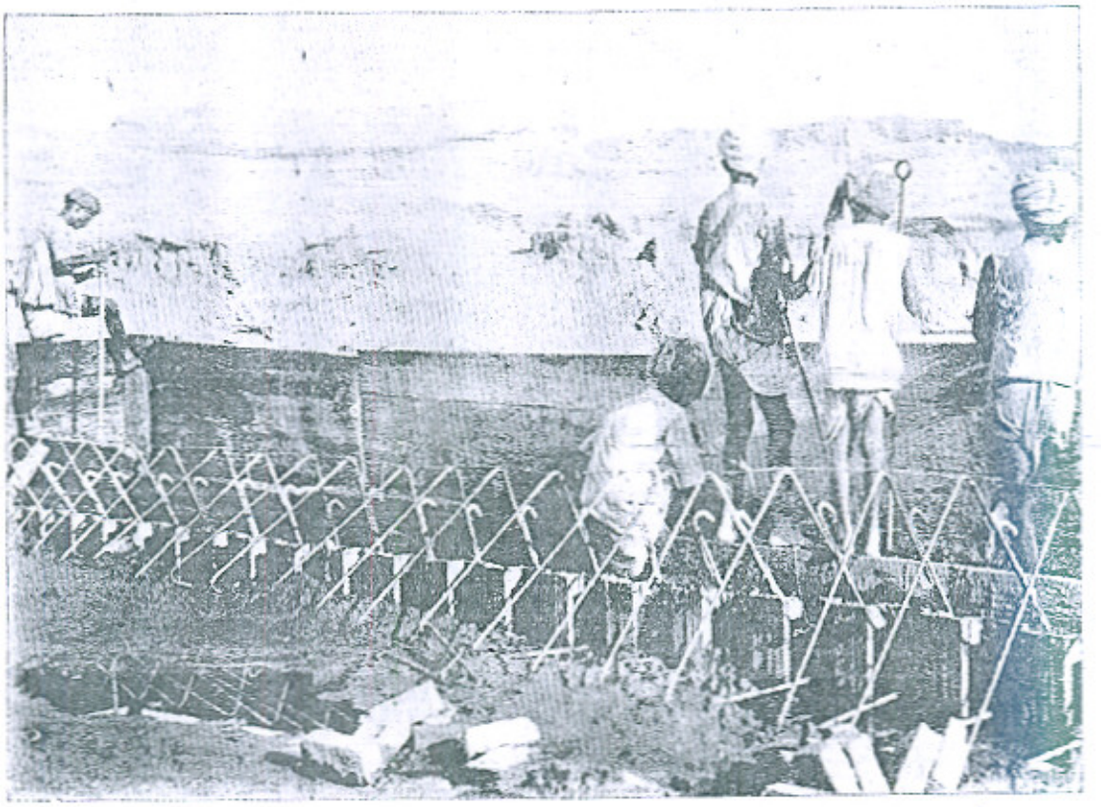
The most important feature of the construction of this work was the excavation of twenty feet deep foundations below the spring level and the laying of floor concrete.

The natural ground level at the site of this work was at R. L. 675·5, spring level at R.L. 671·5 and the lowest foundation level at R. L. 650·0 as shown in the section of the foundation pit in Fig. 1, Plate V. By connecting the pit to a drain dug in the bed of the diversion channel down to R. D. 13,000 of the Nikki Deg as shown in the site plan Fig. 1 Plate 1., the spring level was lowered down to R. L. 668.

After this nineteen tube wells spread over the area of the excavation where the foundation concrete has been laid, and interconnected by means of suction pipes as per attached plan Plate VI, were sunk into the



PLACING REINFORCEMENT IN POSITION.



SHOWING CHAIRS FOR BINDING PILES WITH CONCRETE.

ground. The upstream and downstream lines of suction pipes were connected to separate pumping units. In addition to these arrangements for pumping out seepage water from the foundation pit, three pumps installed at suitable points, were employed to pump out water from the sumps into which water flowed from open drains dug in the area of the excavation as seen in the photograph opposite.

All the pumps worked more or less continuously during the excavation of the pit and without a minute's break at the time of laying of the floor concrete, when absolutely dry foundations were obtained.

In the beginning only small quantities of water had to be pumped out; but as the excavation progressed, pumping was accelerated, till towards the end of the work as much as 10 cusecs had to be dealt with, out of which open pumping from the sumps was responsible for over 6 cusecs. Three Ruston Oil Engines with ten 8″ centrifugal pumps, one 15 H. P. portable steam engine with a 14″ centrifugal and three old Donkey Pumps were on the work. The donkey pumps were far from efficient both in performance and running costs and the boilers used (taken from derelict excavators) were unsuitable, but in the interest of economy it was decided to use material available in the department for this installation. Rupees forty thousand were spent on pumping alone. An analysis of the cost of working the various pumping units is given in Appendix II.

The tube wells consisted of "Esbee" coir strainers (6″ inside and 7″ outside diameter) 25·5 feet long with the top of the strainer 5 feet below the bottom of the foundation, and with three lengths of iron piping about 5 feet each on the top to enable these to be removed as the excavation of the foundation progressed and the pipe line was lowered.

4 inches diameter suction pipes, each 18 feet in length were let into each of the 19 tube wells and these were connected to the suction pipe lines mentioned above, as shown in Fig. 1, Plate VI.

The maximum discharge obtained from the tube wells, with all the 3 donkey pumps and the centrifugal working was 4 cusecs, which was sufficient to keep the foundations dry with the open pumping dealing with all the seepage water outside the Coffer dam, formed by the pile lines.

The draw down at each well was only 2 feet and the curve showing the cone of depression as observed and shown in Fig. 2, Plate V, (in firm line) will be seen to have a close resemblance with the curve given in P. W. D. Paper No. 63 (shown dotted) and expressed by the equation—h, the depression below normal spring level at any point distance x from the well

$$=He^{-\frac{X}{20}}$$

where H is the depression head and x is in feet.

In an isolated test carried out at this work, the discharge obtained from a single tube well with a depression of 10 feet was 0·6 of a cusec, as against an actual discharge of 0·2 cusec per tube well obtained from the system, due to insufficient pumps and air leaks in the pipes, and due to the cones of depression overlapping.

Pile Driving.

After the excavation pile driving was started. It was so arranged that the circuit to be completed by the pile driver was shortest and the male end led throughout. The light running of the machine was thereby reduced to a minimum.

The piles were driven vertically and in a straight line, great care being taken in driving the first few poles. Turning the corners is rather difficult but considerable time and labour can be saved if it is correctly done, as described in Punjab Engineering Congress Paper No. 195, page 131.

In spite of all the precautions taken, the piles do get out of plumb. It is, therefore, essential that they should be frequently tested by means of a spirit level and the error adjusted by means of a suitable correction pile.

If a pile got out of line it was set right by keeping it pressed in the opposite direction while the driving was in progress. The error must, however, be rectified gradually, *i.e.*, in a number of piles otherwise a pronounced kink would result. Another important point in pile driving is the closing of a circuit. For this purpose a junction pile is used, as the ordinary pile of standard width of $14\frac{1}{4}''$ cannot usually cover the last gap. This is made out of two ordinary piles by cutting off one end of each and bolting the webs together. Slotted holes are provided for the bolts so that the pile is capable of adjustment in width to the extent of about 3". The maximum progress obtained was 100 piles 15 feet long in 2 shifts of 9 hours each, which is equivalent to approximately 1800 square feet. An analysis of the cost of driving of the piles is given in Appendix III.

The floor slab reinforcement.

Before laying the floor concrete, the steel reinforcement for the floor had to be fixed in position.

Tata steel was used for all reinforcement work.

A bending schedule was prepared showing the shape and length of the bars to be bent and the total requirements of bars of each diameter were worked out separately. Separate schedules were prepared for the

main floor slab and the Aqueduct, details of which are given in Appendix I.

Bars were obtained in lengths of 40 and 45 feet respectively to suit spans of 39 feet from centre to centre of piers and therefore cutting of bars and overlaps were reduced to a minimum.

The site of work being 8 miles from the Railway Station, special motor trucks had to be arranged for the carriage of these long bars.

Additional steel to the extent of 10 per cent. was obtained and kept on stock at site for emergencies which was subsequently used in the Aqueduct and other works.

It is important to arrange for the reinforcement to be delivered sufficiently far ahead to permit of the steel being bent before it is required in the work and to ensure that the mill scale will have weathered before the reinforcement is concreted.

Some authorities recommend that the steel should be watered twice or thrice a day to accelerate the peeling off of the mill scale, but this was not necessary in this case, as the steel left the mills about 1½ months before it was used on the work and the weather was wet.

The "junctions" or "bar crossings" were tied very carefully and rigidly so as to prevent distortion or displacement during concreting. In order to support the reinforcement in position, special chairs were devised, one of which is shown in Fig. 3, Plate V.

The bars of the bottom grid were supported on the two arms of the chair, 6" above the foundation level, while the top bars were supported on the saddle, 3½ feet above foundation level. These chairs were spaced roughly at 5' centres across the work and at seven feet centres longitudinally, this number being found just sufficient to give a rigid frame. The photograph opposite shows the chairs and bars in position.

In addition to these chairs, 6" cubes of cement concrete were used to keep the bottom bars at the correct height and pieces of Salballis were inserted under the top reinforcement whenever found necessary resting on the foundation. These were, however, removed as the concreting progressed.

The bottom grid had longitudinal bars 14¾" apart so that there was one bar opposite each pile 6 inches above the foundation level and all longitudinal bars were passed through the piles and hooked at the end.

Similarly all cross bars which were nearest the centres of the side piles were passed through the latter.

Chairs of the shape shown in the photograph opposite page 111 were provided at alternate piles for binding these to the concrete.

Laying of concrete in the floor. Concrete Materials and Construction.

Cement.—The cement was obtained from the Punjab Portland Cement Company, Wah.

Ordinary cement was used throughout the work, special care being taken to see that the cement was not stocked at the work for more than one week before it was used.

The importance of using fresh cement was fully discussed at the Congress in 1924 in a paper by Mr. Robertson.

All cement used on the work was measured by weight.

Sand.—The sand used in the floor slab was clean washed sand obtained from the foundation pit. The firm line on the graph Fig. 4, Plate V, shows the analysis of this sand. A graph for an ideal well graded sand is also shown dotted, for comparison.

The sand was rather fine, and the Principal, Rasul School of Engineering, advised that either some grit may be added to it or 10 per cent. extra cement may be used, but as the compressive strength allowed in the calculations was only 600 lb. per square inch, a 1 : 2 : 4 mix was used, the sand being measured by volume, after finding out the actual bulking every morning as explained later.

Bulking of Sand.—It is now universally recognized that cement should be measured by weight and not by volume, owing to the great variations of density, which can be obtained with the same cement according as it is loosely packed or thoroughly shaken; but the same is not recognized of sand in which a similar effect obtains, the variations in this case depending partly on the degree of packing or consolidation by shaking, but the main factor is the moisture content of the sand.

The results obtained by Oscar Faber, of a series of tests on the variation in value of sand under different conditions of packing and moisture content, which appeared in the 'Concrete and Constructional Engineer' for January, 1936, are shown in the form of a graph in Fig. 5, Plate V.

At the work under reference, the author found that bulking was sometimes as much as 30 per cent.

This clearly brings out the necessity of measuring sand by weight and allowing for the moisture content in the material. Although sand was not measured at this work by weight, the method followed was

equally accurate, as the following experiment was carried out at least twice a day. A sample of sand was taken from the sand heaps and was filled into a tall cylindrical tin flush with the top. The height of the tin had been carefully measured.

The tin full of sand was then heated till all the moisture had been driven out. It was then found that the level of sand in the tin had fallen. This reduced height was carefully measured. The ratio of the reduced height of the dry sand to the original height of the damp sand gave the extent of the bulking and thus the total volume of damp sand required to yield the correct volume of dry sand required for the batch could be computed. This volume of damp sand was then measured by means of measuring boxes from where it was tipped into the trucks. No undue tamping or shaking in the measuring boxes was permitted and the surplus was removed from the top by means of a straight edge.

Coarse aggregate.—Pathankot shingle passing through 1½" mesh was used in the floor slab.

A sample of the shingle supplied by the contractor was tested at the Rasul School of Engineering and was found to be structurally and mineralogically sound for reinforced concrete work.

The aggregate was washed by water raised by a portable Peter pumping set from the foundation pit as it was clear and free from impurities. The washed aggregate was measured by loading into tip wagons up to a certain mark which indicated the quantity required for each batch. Every effort was made to remove the clay from the sand and shingle. However, tests carried out in America show that materials containing fine clay to as much as 3 per cent. of the amount of total eggregates may be safely used in concrete construction (Engineering News Record, dated 23rd August 1934, pp. 233-34). The results also showed that clay gave a smoother mix and had a slightly greater beneficial effect upon the strength of concrete having an under sanded mix.

Mixing.—After loading the coarse aggregate, the tip wagons were railed to the sand stacks where sand was poured over the shingle in the wagons by means of 2 c.ft. measuring boxes. This mixture of shingle and sand was then tipped into the mixer-feed hopper and four bags of cement were added simultaneously. One batch was completely taken out before a fresh one was put in and attempt was made to turn out one batch every 3 minutes.

An analysis of the cost of 1 : 2 : 4 cement concrete on this work is given in Appendix IV.

Cement Water Ratio.—This subject was discussed at some length at the last session of the Punjab Engineering Congress, but it is so important that it will bear repetition.

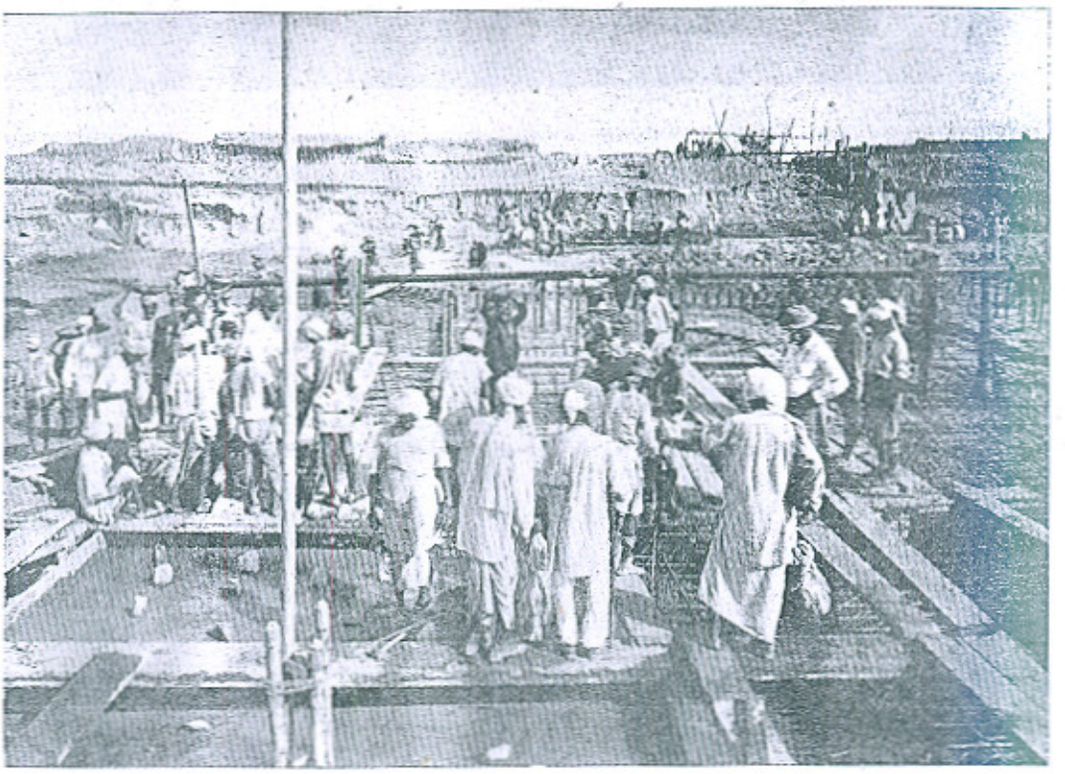
In practice, the amount of water required to be added depends on so many factors that it is impossible to fix any hard and fast rule for ensuring the correct ratio. It varies with the amount of moisture in the coarse aggregate and sand, and with the temperature and humidity of the atmosphere. The graph Fig. 6, Plate V, indicates the effects of temperature and humidity on the quantity of water used per bag of cement on this work.

The humidity during the period was very high and variations from day to day small. It will be noted from the curves, however, that humidity varies considerably with temperature and so it was found that in the mornings when the temperature was low and the air damp, we used very little water. As the day advanced, the temperature rose and the sun dried out the sand and ballast stacks, we found that the water in the mix had to be increased considerably to keep it workable. Slump tests and personal observation are, therefore, the best guides, but it should be taken as a working rule "that the minimum amount of water that will produce a mix just sufficiently plastic to work into the moulds and round the steel after adequate tamping, without producing surplus water or an undue amount of slurry on the top of the wet concrete, should be used."

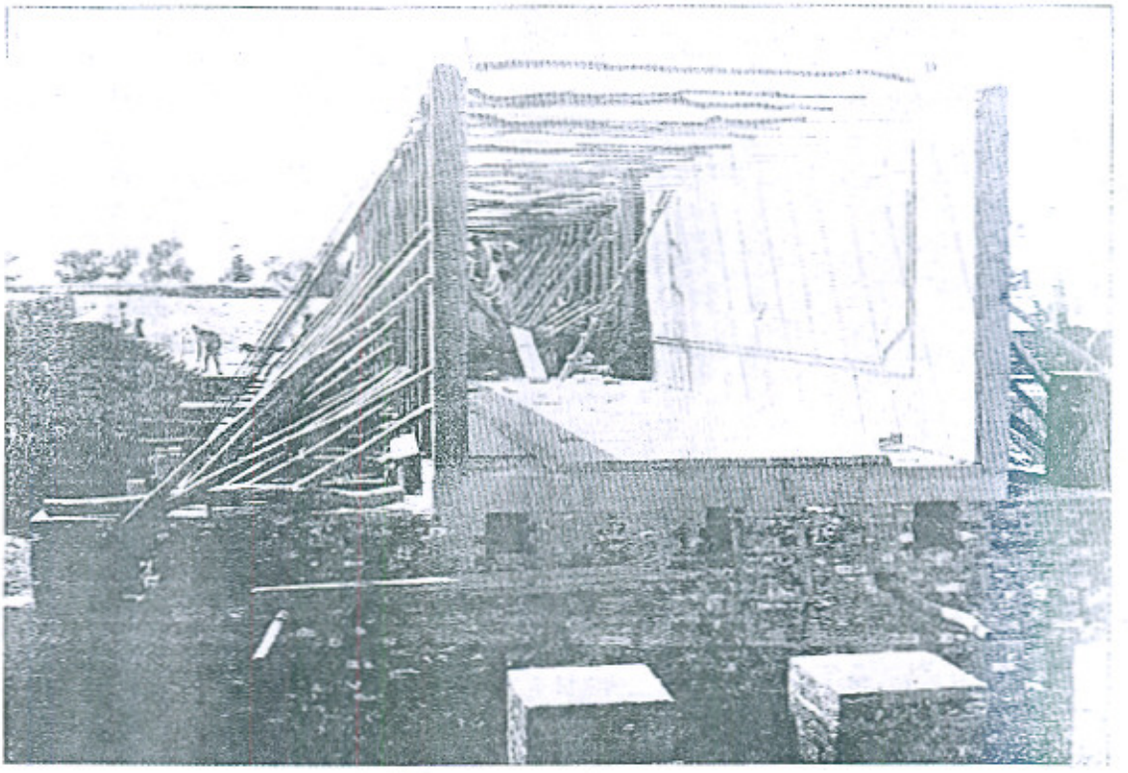
How serious an effect, deviation from the above rule may have on the resulting concrete is illustrated in the following table reproduced from the issue of the 'Concrete and Constructional Engineer' for April 1935.

Comparative effect of water content on strength of concrete.

Water content.	Water cement ratio.	Crushing strength lb. per sq. inch.	
Gallons per 112 lb. of Cement.	c.ft. water to c.ft. cement.	7 days.	28 days.
4·67	0·6	3,000	4,500
5·85	0·75	2,300	3,700
7·0	0·9	1,700	2,900
8·17	1·05	1,300	2,200
9·34	1·2	800	1,600



GANGWAYS OF LARSEN PILES FOR CONCRETING THE FLOOR SLAB.



AQUEDUCT SHUTTERING.

Thus a variation of the water cement ratio from ·6 to 1·2 reduces the strength to about one-third. On the other hand if the concrete is unduly dry, honey-combing will result and greatly weaken the structure. It must, therefore, be clearly kept in mind that both workmanship and workability are essential factors in producing good concrete.

On these considerations, the course adopted was a workable mix, the quantity of water being varied according to actual requirements as is evident from the graph Fig. 6, Plate V.

The water content to be added was fixed by the officer in charge of the mixing arrangements, being varied every 2 or 3 hours according to requirements by his personal observation, but at least one slump test was made for every 20 batches of concrete as it gives a true indication of the consistency of the mix and has also a valuable moral effect in emphasizing the working rule mentioned above.

The excessive loss of strength on account of too much water is not of unusual occurrence on works where mixing is done by manual labour. It is for consideration whether hand mixing should at all be permitted in the department for reinforced concrete work of any magnitude when cheap and efficient portable mechanical mixers of all sizes are available in the market. There will be practically no extra cost in concrete labour, and the adoption of greater working stresses would result in considerable savings.

Transport and Placing of Concrete.

The concrete was poured directly from the mixer into tip wagons which were railed to the site of work where it was tipped on to platforms covered with iron sheets and was thence removed to the work in baskets.

The whole operation of mixing, carrying and placing at site took about 12-15 minutes per wagon and therefore 4 gangs worked for carrying the concrete from the platforms to site, so that they could dispose of 4 wagons in 12—15 minutes which is equivalent to 1 wagon every 3—4 minutes.

The placing of concrete by baskets as compared to tipping it directly from the wagons is much more suitable in many respects and the extra cost involved is only -/8/- per 100 c.ft.

It has the following advantages :—

(1) Concrete deposited in small quantities can be wriggled sufficiently and so is well consolidated.

(2) It does not produce vibrations on the work which has taken its initial set.

(3) It does not displace or distort the reinforcement assembled in the work.

(4) It produces comparatively low stresses in the shuttering.

(5) It gives another turn to the concrete which somewhat segregates in the wagon during its movement from the mixer to the platforms.

The thickness of the floor slab being 4 feet, all gangways for labour were arranged 5 feet above the foundation level, so that they were clear of the finished top. Over a dozen men were constantly employed on wriggling and specially skilled men were employed for placing the concrete in position which was done under very strict supervision of one officer, 2 subordinates, and a lot of mates and mistries, who were in charge of the various operations such as mixing, tipping, turning the concrete over again at the platforms, clearing the platforms and removing the droppings, wriggling, cleaning the reinforcement, removing the concrete pats and timber supports from underneath the reinforcement, finishing the top surface, ramming the ends, and so on.

Some authorities recommend that the concrete should always be rammed against a rigid shutter preferably a corrugated iron sheet fixed vertically or at a slope at the end of the day's work, as concrete placed loosely remains unconsolidated at the slopes, and therefore it should never be allowed to "tail away."

As it was not possible on the work under discussion to provide such a rigid surface at the end of each day's work, the following precautions were taken :—

(1) The tailed away portion was stepped and rammed with wooden mallets.

(2) The surface was chiselled off for a depth of 1/2 to 1 inch and washed.

(3) Neat cement grout was poured all over the joint before starting fresh work.

(4) Steel rod hooked dowels were introduced to provide a definite mechanical bond at the joints.

The tools used for wriggling were short and long steel rods having a blade measuring about 5 inches by $1\frac{1}{2}$ inches at one end.

Construction joints.—The position of the construction joints was settled in advance and marked on a plan of the work. The whole work was divided into about 17 compartments, the concrete in each compartment being just sufficient for a day's work. The concreting was done monolithic and not in layers, as on account of the double grid of reinforcement with bars bent up to take the reverse bending moment near the supports, it was more economical and in every way better to do so.

Great care was taken to keep the fresh concrete free from vibration and men were not allowed to walk over it. As mentioned above, raised gangways were provided for the labour all over the work. These consisted of 30 ft. long Larssen piles on specially constructed angle iron tripods which, for stability, were fitted with detachable bracing that was fixed after placing the tripods in position through the reinforcement.

The arrangement is shown in the photograph opposite.

The concrete finished on the previous evening was covered over with wet sand to protect it from the effects of a hot sun and water was constantly sprinkled over it for 15 days, to keep it wet, as this increased the strength of the concrete, and reduced the tendency to the formation of shrinkage cracks.

The watering of concrete after it is placed in position is of much greater importance than is generally realized, so much so that it is worthwhile having a separate item for watering in the schedule of rates.

As regards the setting heat of concrete, a depth of 4 feet is not great and considering that the concrete is in contact with water below and with the atmosphere above, the maximum depth from outside is only 2 feet.

The subject of temperature effects in mass concrete is discussed at length in a paper by W. T. Halcrow read at the summer meeting of the Institution of Water Engineers in England in 1934. The results of actual experiments given in this paper show that there was a rise of temperature of only 20° between the surface and at a point 3 feet inwards and therefore at a depth of 2 feet the temperature rise would be about 15°. A temperature rise of 1° F. produces a change of stress of approximately 13 lb. per square foot and therefore the stress would be approximately 200 lb. per square inch at the outside which is not much.

The subject is also dealt with exhaustively in the Building Research Technical Paper No. 18, published by His Majesty's Stationery Office, wherein it is stated that the temperature gradient should not exceed 25° F. and it is suggested that in large dams the rate of placing layers would have to be restricted to about 6 inches per day, or to 3′—6″ placed at intervals of not less than seven days.

It is thus evident that it is quite safe to put in a 4 feet depth of concrete in one layer. It is also brought out that if concrete is placed in layers, sufficient interval of time should be allowed between placing the layers to avoid the effects of temperature stresses accumulating.

The Aqueduct and Road Bridge.

A reference to Plate VII will show that the Sikhanwala distributary which has a designed bed width of 33 feet has been flumed through a 10′ wide opening, the bottom of the flume having been depressed by 1·87 feet to give an increased area thereby reducing the loss of head.

The approach and delivery curves as well as the water levels expected in the flume and its approaches are also shown on this plate. These have been designed on the basis of the publication on fluming published by the Central Board of Irrigation. The approach and delivery curves are parabolic, the convergence at the approach being at 1 in 3 and the divergence at the exit being at 1 in 4.

It is generally recognized that neither a straight line nor circular approach and exit curves in masonry flumes give smooth flow. The reason is obvious. In both these cases the contraction as well as expansion is too rapid at some place in the profile, where a ripple must appear.

On the other hand parabolic curves are known to give very smooth flow, the water passing without a ripple both at the entry and the exit.

The writer has tried to analyze the cause of this improvement with a parabolic curve and has come to the conclusion that with a level bed parabolic approaches give a straight line curve for V^2. As resistance to flow depends on the square of the velocity, for smooth flow V^2 should be reduced in proportion to the distance from the flume which is achieved by a parabolic approach and exit. Consider a flume 10′ wide which is to expand into a channel 30′ wide, the discharge being 300 cusecs and depth 5 feet.

The velocity at the exit will be 6 feet and that at the full width of the channel only 2 feet.

With a divergence of 1 in 4, the horizontal distance will be 40 feet which may be divided into 8 equal parts. Thus for smooth flow the square of the velocity should reduce from 36 to 4 in proportion to the distance from the flume and therefore the velocity required is as shown against V in the diagram, Fig. 10, Plate V.

Knowing the velocity and the depth which is 5 ft. the required widths have been worked out. These when plotted approximate to a

parabola. The depths required for the Aqueduct flume have also been worked out by assuming the approach and exit curves to be parabolic and the result is shown in dotted lines in section on Plate VII for comparison.

This agrees very closely with the profile as worked out according to instructions in Publication No. 6 of the Central Board of Irrigation and presents a handy method of designing bed profiles.

The shape of the curve of divergence having been determined as described above, the actual splay and the length of exit transition will obviously depend on the volume of water to be dealt with and the velocity of flow through the flume. Investigations on this point have been carried out by Engineers both in India and abroad notably in America. The following from paper No. 1690 of the transactions of the American Society of Civil Engineers for 1928 on the Hydraulic Design of Flume and Syphon Transitions by Julian Hinds will be of interest :—

" It is desirable first to determine approximately the length that will be required for the transition. In the absence of specific knowledge this must be done arbitrarily. For structures of the type under consideration, the United States Bureau of Reclamation has adopted the rule of making the length such that a straight line joining the flow line at the two ends of the transition will make an angle of about 12½° with the axis of the structure."

A practical and cheap method of shortening the length of exit transition, and at the same time keeping the angle of divergence small is to build thin divide walls within the exit transitions as shown in plan on Plate VII. Even though the angle of divergence of the outer walls of the exit flume be quite sharp, inner divide walls have the effect of greatly turning down the rate of expansion of the jet of water issuing from the flume. The actual splay of the side walls at the exit of the flume is 1 in 4, but with the construction of two divide walls it has been reduced to something like 1 in 10.

Design.—As mentioned abve, the Aqueduct side walls are calculated to carry the combined load of the water and the road bridge.

The bottom slab is designed for a clear span of 10 feet with ends fixed to carry a load of 1·0 foot of silt and 6·0 feet depth of water.

The Aqueduct side beams are designed for a clear span of 35 and effective span of 37 feet freely supported to carry the entire weight of the road bridge and its load including the wheel guards, etc., together with the weight of the bottom slab, silt and water. The beams are 10 feet high from bottom of slab to top of wheel guards, but the calculated depth is 93 inches, the thickness being 15".

Shear and Diagonal Tension has been provided for by means of 'U' shaped stirrups,

The two usual methods of distributing the stresses caused by shear and diagonal tension, are described on page 246 of "Concrete Plain and Reinforced" by Taylor and Thomson. In the first method, the concrete is assumed to resist an amount of shearing stresses equal to the allowable stresses for plain concrete, and the steel is assumed to resist the balance. In the second method, concrete is assumed to resist one-third and the steel two thirds of the diagonal tension.

The combination of the two systems in the manner suggested above is not sound as the full tension of 16,000 lb. in.^{2} cannot develop in the steel without the corresponding strain, and at that strain the concrete will necessarily have cracked.

The method followed is, therefore, the one recommended by Oscar Faber in his book, "Reinforced Concrete Beams in Bending and Shear" which takes into account the stresses induced in the concrete. The reinforcement used as well as the stirrups and their spacing is shown in Figs. 1 & 4, Plate VIII.

Bending and binding reinforcement for aqueduct.—The reinforcement used is shown in Figs. 1 and 4, Plate VIII.

The main reinforcement consisted of 12 bars of 1" diameter which were placed in 3 layers separated from each other by 5/8" bars. Half of these 12 bars were bent up for reverse bending moment as shown in Fig 4, Plate VIII, 2 bars being bent at each point.

The spacing of the stirrups is also shown on the left half of Fig. 4, while the right half shows the reinforcement put in to withstand side pressure.

As it was not possible to get bars of the full length required in India, the pieces were so arranged that the joints were far removed from the middle where the bending moment is a maximum.

Next in importance to the main reinforcement are the stirrups. These were adequately secured at both ends, as otherwise, if fixed at the bottom and not at the top, the stirrups will not be fully stressed.

Great care was taken to ensure that the corner bends of stirrups were made to suit the bars which they encircled as shown in Fig. 7, Plate V.

After assembling, the reinforcement was kept in position by hanging from the top tie bars and *balli*s shown in Fig. 5, Plate VIII.

The vertical rods were spaced at the correct distance apart by means of short iron bars of the shape shown in Fig 8, Plate V which were tied by means of annealed wire.

The necessity of correct spacing and using spacers cannot be over-emphasized particularly in out-of-the-way places where labour is generally not sufficiently skilled.

Shuttering.—The shuttering was so designed as to be absolutely rigid. It was braced and tied so that it was able to withstand without displacement vibration or movement, the weight of construction, movement of men, plant, material and the maximum wind pressure. The most important part was the shuttering for the sides both inner and outer which was designed to stand the side pressure caused by 7 feet depth of concrete.

A section through the shuttering as erected is shown in Fig. 5, Plate VIII and the photograph opposite page 117 shows it completed in one span and in progress in another. Shuttering was made for 2 complete spans and for the bottom of a 3rd span, so as to finish all the 5 spans in about a month and a half.

The timber used had a minimum thickness of 2 inches and was butt jointed. The bottom planks were fixed on wooden beams which were placed 3 feet apart at right angles to the span and rested on girders 12″×5″ at 30 lb., 38 feet long of which 15 were obtained for use in the temporary gangway for regulation. The uprights for the sides rested on the wooden beams and were spaced similarly. The whole shuttering was made absolutely rigid by 2 systems of braces and ties as shown in the drawing, which is detailed and clear. An analysis of the cost of shuttering is given in Appendix V.

The shutters for the roof slab were supported on a series of light kacha pacca masonry pillars shown dotted in the Section (Fig. 5, Plate VIII). All angles were chamfered. The bottom shutters were cambered in the proportion of 1 300th of the span. Wooden wedges were placed between the girders and the beams so that they could be lowered gently when the shutters were struck.

The side shutters were erected plumb from the bottom and were kept in a straight line by stretching wires from one end to the other, the shutters being set to the wires so that any error could be easily detected and set right.

When the work was completed it was found that the maximum error out of line or out of plumb did not exceed half an inch. The concreting of each span of the Aqueduct was done in parts shown in Fig. 9, Plate V.

The bottom slab was completed first, a dowel being left in the position shown. After two days this was chipped and washed and covered over with neat cement slurry before starting the concreting of the side walls.

Steel dowels were also left in addition to the stirrups and other reinforcement shown in Fig. 1, Plate VIII, to make a good strong joint.

The side concreting was then completed upto 6" above full supply level, leaving another dowel as shown in Fig. 9, Plate V, after which the shuttering for the road slab was erected as shown dotted in Fig. 5, Plate VIII, and the concrete placed in position. The whole work in one span took about a week to finish.

In this connection it would not be out of place to mention the stresses developed by time shrinkage which takes place gradually upto an effective period of about two years and the shrinkage due to the difference in age of the concrete.

According to Manning (*vide* 'Concrete and Constructional Engineering' May 1935), the relative shrinkage can increase to a maximum of about ·002 per cent. and cause a tension of about 30 lb. at the point, if the interval between casting two lifts of concrete be 7 days. This stress cannot itself be a source of danger, but it may be the final cause of cracks developing if the concrete is already under tension due to other causes.

This shows the necessity of speeding up construction so as to diminish the time interval between lifts of concreting.

It should be realized that reinforcement does nothing to prevent shrinkage from starting. As soon as a crack develops the steel is placed in tension, and as the crack widens the adhesion between the bars and the concrete is destroyed for a certain length on both sides of the crack.

Manning has considered a wall 39 feet long and this incidentally is the total length of each span of the Aqueduct. His conclusion is that if the horizontal steel is more than ·75 per cent. as is the case in the Aqueduct, we should expect to find in the wall after 2 or 3 years a total width of cracks of about 0·125" divided into 8 cracks of ·0156" or 35 cracks of 0·0036" or some arrangement between these two.

It is, therefore, anticipated that the Aqueduct might leak at these cracks after a year or two when it could be painted inside with sodium silicate solution to make it water-tight.

With rapid hardening cement the time shrinkage stresses are naturally much more severe and may cause bad cracks if the time between different layers is large.

Stripping of Shuttering.—The inside shutters for the sides were stripped 24 hours after finishing the concreting. The outside shutters had to be kept in position till the roof was completed. The false work for the roof slab was stripped after one week and a fortnight was allowed for the main bottom shutters.

Surface Treatment and Surface Blemishes.—In order to obtain a good finish, it was suggested by the local representative of the Indian Concrete Association that a retarder, which is in the form of a liquid or jelly, may be brushed over the forms before placing the concrete for retarding the setting of the surface of the cement so that it could be easily brushed off. But this suggestion was not followed as experiments made at site showed that it was possible to obtain good results without the use of a retarder.

The face was, however, made of fine concrete which was applied integrally with the backing of 3/4" concrete. For this purpose iron plate shutters 2'—6" deep provided with lifting hooks were used to separate the facing from the backing. Wooden spacers notched to fit over the top edge of the iron plates were used to retain the plates at the correct distance from the timber panel forms.

The fine concrete at the surface was made 1" thick, which left ample room for the shutter to slide without fouling the reinforcing bars. A layer less than 1 inch thick is difficult to place in this manner and calls for special care and supervision and the ultimate cost due to increased labour charges may exceed the additional cost of a thicker coating.

In using the sliding shutters the level of the fine facing concrete was kept at least 2 to 3 inches above that of the backing so as to prevent the latter penetrating to the face, and it was deposited in 12 inches depth before lifting the shutters.

The plates were carefully washed and scraped after use to ensure a good finish. The consistency to which the facing concrete is to be mixed is of great importace and this should be such that the mix is neither too wet so that the coarse shingle may not penetrate and show on the finished surface, nor should it be too dry as it is sure to cause fractures in the surface layer and may thus cause penetration by the coarse aggregate and may necessitate patching of the surface which must be avoided at all costs.

Although the timber was planed very carefully and only Deodar wood was used, it produced a replica of the boarding on the surface of the finished side walls, which looked unsightly and was removed by brushing and rubbing with carborendum stone.

The only really satisfactory method of avoiding this type of blemish is to line the shuttering with G. I. sheet metal plates, ply-wood or some other special material, but this was considered too expensive and with a little more care in joining and planing the boards, the marks on the surface became invisible from a distance. Grease was used on the sides to allow the shuttering to be stripped without pulling away patches of the green concrete and was found not to leave a stain.

On the bottom a coat of white-washing was put on before laying the reinforcement, because if grease is applied at the bottom, the blacksmiths fixing the steel scrape a good deal of it off with their shoes and by dragging bars over the greased surface, and the bars get soiled.

Other causes of surface blemishes are uneven and improper tamping, air bubbles, weak boarding, and surface cracks caused by a shortage of distributing bars or their incorrect design.

All these defects were guarded against very carefully as such defects in concrete cannot be treated to produce a result so good as originally sound work.

Furthermore, the making good of such defects is both tedious and expensive. It is, therefore, desirable to avoid them entirely by exercising proper care and taking all necessary precautions, as prevention is better than cure.

Expansion Joint.—The necessity of providing expansion joints is fully dealt with in Paper No. 181 of this Congrss and need not be repeated here. It may, however, be mentioned that the provision of expansion joints is the only practical remedy against unsightly cracks and water leaks.

A section through the expansion joint constructed between adjacent spans of the Aqueduct is shown in Fig. 2, Plate VIII. It consists of 2 pieces of copper flashing 12" wide and 1/16" thick bent in the manner shown and placed 9" apart with the space between filled with bitumastic consisting of grade 115 Socony Asphalt mixed with sand in the ratio of 40 per cent. and 60 per cent. respectively. A square hole 3"×3" was left in the middle as shown for pouring the bitumastic.

The copper flashing on the outside was carried down to the bottom and along the bottom slab as shown in section, Plate VIII, Fig. 1, and continued up the other side wall, while those on the inside were just bent

round leaving a gap of about 6 feet along the top of the bottom slab for pouring in the bitumastic. The Asphalt was heated to a temperature of 340° before mixing it with sand and the copper was thoroughly cleaned with kerosene oil before pouring in the bitumastic.

Provision has also been made for the movement of the Aqueduct as in the absence of such provision the abutments and piers are liable to crack and become tilted. For this, horizontal joints, entirely separating the bridge structure from the abutments and piers, have been provided.

These consist of four thin rectangular plates of metal 4 feet×2 feet the two outer plates being of steel, 1/4" thick and the two inner plates of copper 1/8" thick. The beams rest entirely on these plates as shown in Fig. 1 and Fig. 3, Plate VIII.

The Aqueduct has been subjected to the full load of 7 feet of water for which it was designed and no crack has appeared so far.

The expansion joints have also safely withstood the pressure caused by a 7 feet head of water.

Pressure Pipes.

A series of pressure pipes were erected in the work in consultation with R. B. A. N. Khosla, The total number of pipes put in is 68 and iheir position is shown both in plan and elevation on Plate IX.

The pipes and strainers are of the same type as erected at Panjnad and described in Paper No. 162 of the Punjab Engineering Congress.

The subsoil consists of a strata of fine sand over medium sand.

All observations were taken with the bell sounder. The distribution of pressures as obtained in the Punjab Irrigation Research Laboratory ts shown in Fig. 2, Plate IX, and Fig. 3 shows the percentage effective pressures at various points. The laboratory percentages are plotted in firm line while those calculated from field observations are shown dotted.

As mentioned before, the channel downstream of the fall has not yet developed fully and therefore there is normally no fall in the water surface at the work, but a few observations were made in the early stages when the downstream water level was kept low and water was headed up above the Fall.

The date of observations and the actual difference in water levels upstream and downstream has been noted on each graph.

The first observation was made on 18th November, 1935 with a head of 7.2 feet across the crest.

The percentage effective pressures throughout were much higher than those obtained in the laboratory. This appears to be due largely to the high spring level outside, which was about 5 feet higher than the water surface upstream of the Fall.

The second observation plotted in the diagram was made on 22nd November, 1935, with a head of 9·0 feet.

Some silt had deposited on the upstream side by then and its effect is noticeable at pipe 1.

The third observation plotted was made on 17th December, 1935, with a head of only 4·1. The silt blanket above the Fall was over 2 feet thick and the effect at pipe 1 is very marked.

However, the percentage effective pressure in pipe 14, *i.e.*, the pressure at exit was rather high. At the time of observation, the water level upstream of the Fall was at R.L. 674·9 and that downstream at R. L. 670·8, while the spring level was at R.L. 674·0. These figures lead one to think that the high spring level outside directly effected the pressure in pipe 14, but the data so far available is too meagre to arrive at any definite conclusions, but the Deg Head Fall is one of the few works in the Province, where pressure pipes have been inserted liberally, and the data that will become available, after the channel downstream develops to designed bed levels, will be very valuable indeed.

Development of the Deg Diversion Channel.

One more feature of some scientific importance remains to be mentioned about the Deg Diversion Scheme. From the canal to the old existing deep channel of the Deg Nala, a distance of about 20,000 feet was in high ground, (as shown in the attached L. Section Plate X,) and a connecting link between the canal and the existing deep channel was to be made. The excavation of this connecting link in the ordinary way by means of manual labour was considered; but for reasons of economy it was decided to let the canal dig out a channel for itself. From the canal to the old existing deep channel a cunette to carry about 20 cusecs was artificially made and water was let in through this cunette into the deep channel in February, 1936. From the edge of the cunette to the deep channel, there was a drop of about 10 feet. Water in falling over this edge cut away the bed and the sides of the cunette, and this process rapidly progressed backwards towards the canal. In this process of retrogression of levels, the cunette was progressively enlarged both in the bed and the sides; so that the cross section of the successive stages of the formation of the new channel were like so many rings one larger than the other as shown in the accompanying cross section, Plate XI. The one main fall at the edge of the cunette in the process of retrogression of levels was split up into several small falls and rapids at short intervals throughout the length of the new connectin

channel; the situation of these falls and rapids, and their durations before their disappearance into one steep sloped continuous channel apparently depended on the comparative strengths of the soils in different reaches of the new channel, and the resistances which the different soils offered to the erosive action of water, as indicated by Mr. R. K. Khanna in his pamphlet entitled "Silt theory of flow of water." In this simple and inexpensive way a channel to carry over 3,500 cusecs has already formed at the time of writing, (October, 1936) and it is expected that within the next few months the channel will enlarge to carry the full required capacity of 6,000 cusecs and thus a work which would have cost Rs. $3\frac{1}{2}$ lacs to excavate will have scoured out at 1/10th the cost. The formation of the connecting channel has taken such a long time only because it was not possible to head up water in the canal high enough to pass sufficient discharges through the newly made cunette. Throughout the formation of the new channel, the tendency to form a wide and shallow channel has been prominently noticeable in the upper reaches. This tendency which was plainly attributable to the extremely steep slopes of flow of water, was countered by means of artificial spurs built on the sides of the channel consisting of sal ballis sunk with a water jet 8 feet below the bed. In this way the action of water was primarily directed to scouring out the bed of the channel. With the side action of water thus controlled, the channel has become progressively deeper and its slope of flow has become progressively flatter beginning from its lower end and it may be taken for granted that in a short time the channel will have acquired the full dimensions and the requisite slope of flow as demanded by the considerations of volume of flowing water and the average strength of the soils through which the channel flows.

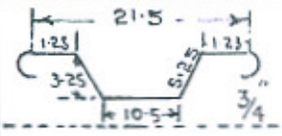
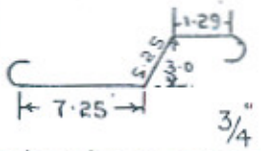
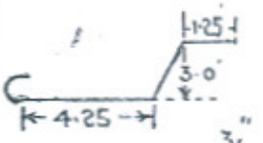
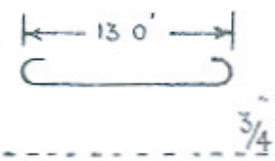
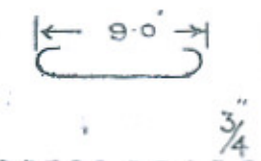
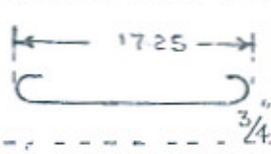
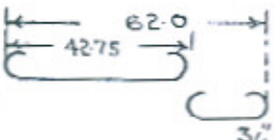
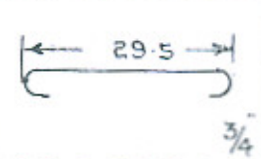
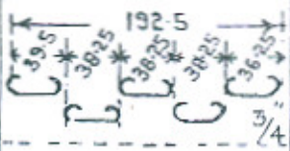
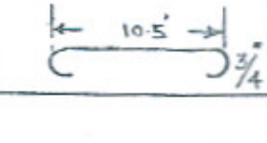
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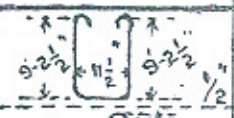
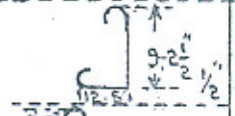
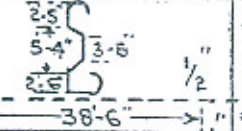
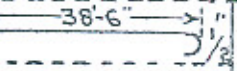
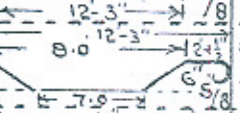
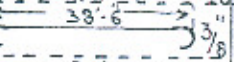
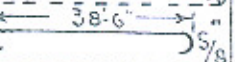
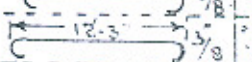
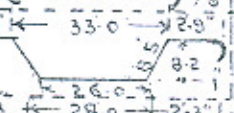
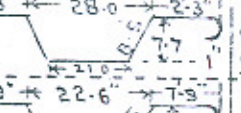
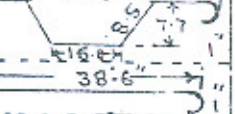
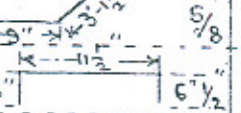
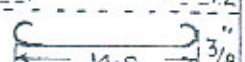
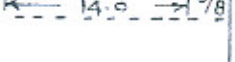
Acknowledgment is also due to the Concrete Publications, Ltd., for permission to reproduce some of the graphs and tables from the 'Concrete and Constructional Engineer.'

SCHEDULE OF BARS IN FLOOR SLAB & WINGS

TYPE	SHAPE	LENGTH OF BAR NEEDED	NUMBER NEEDED	POSITION IN THE STRUCTURE
1		24·5	544	MAIN SLAB AND UPSTREAM WALL TOP GRID CROSS REINFORCEMENTS CONTRAFLEXIONS UNDER PIER
2		15·0	136	MAIN SLAB AND UPSTREAM WALL TOP GRID CROSS REINFORCEMENTS CONTRAFLEXIONS AT ABUTMENTS AND UP TO SIDE WALLS (ALTERNATE WITH NO 3 AND 5)
3		11·75	136	MAIN SLAB AND UPSTREAM WALL TOP GRID CROSS REINFORCEMENTS CONTRAFLEXIONS AT ABUTMENTS AND UP TO SIDE WALLS (ALTERNATE WITH NO 2 AND NO. 5)
4		14·25	272	MAIN SLAB AND UPSTREAM WALL BOTTOM GRID (CROSS REINFORCEMENTS) EXTRA REINFORCEMENT UNDER PIERS BELOW THE TOP BARS (ONE IN EVERY THREE) WHICH DO NOT CONTRAFLECT)
5		10·25	136	MAIN SLAB AND UPSTREAM WALL BOTTOM GRID (CROSS REINFORCEMENT) EXTRA REINFORCEMENT UNDER ABUTMENTS & UP TO SIDE WALLS BELOW THE TOP BARS (ONE IN EVERY THREE) WHICH DO NOT CONTRAFLECT.
6		18·5	340	MAIN SLAB AND UPSTREAM WALL TOP GRID CROSS REINFORCEMENTS CENTRE OF SPANS WHERE BARS UNDER ADJACENT PIERS ARE CONTRAFLEXED
7		66·5	272	MAIN SLAB AND UPSTREAM WALL. TOP GRID CROSS REINFORCEMENTS THROUGH TWO SPANS AND UNDER ONE PIER WHERE THE BARS (ONE IN EVERY THREE) ARE NOT CONTRAFLEXED
8		30·75	136	MAIN SLAB AND UPSTREAM WALL TOP GRID CROSS REINFORCEMENTS UNDER ABUTMENTS UP TO SIDE WALLS WHERE THE BARS (ONE IN EVERY THREE) ARE NOT CONTRAFLEXED
9		20·5	12	MAIN SLAB DOWNSTREAM END TOP GRID CROSS REINFORCEMENTS BARS RIGHT ALONG THE SLAB DOWNSTREAM OF PIERS, ARE NOT CONTRAFLEXED
10		11·75	160	SIDE WALLS. TOP GRID STRAIGHT BARS CARRIED OVER PILE LINE

TYPE	SHAPE	LENGTH OF BAR NEEDED	NUMBER NEEDED	POSITION IN THE STRUCTURE
11	6 25 — 3/4"	17·5	278	SIDE WALLS TOP GRID (BENT DOWN) CROSS REINFORCEMENTS CONTRAFLEXED BARS ONLY TAKEN UPTO PILE LINE
12	12·5 — 3/4"	13·75	132	SIDE WALLS TOP GRID (BENT DOWN) CROSS REINFORCEMENTS CONTRAFLEXED BARS ONLY TAKEN THROUGH PILE LINE
13	89 0 — 5/8"	89	203	MAIN SLAB AND SIDE WALLS. TOP GRID LONGITUDINAL BARS @ 12" CENTRE TO CENTRE
14	86 0 — 5/8"	86	171	MAIN SLAB AND SIDE WALLS BOTTOM GRID LONGITUDINAL BARS @ 14½" CENTRE TO CENTRE
15	28·5 — 5/8"	29·5	154	MAIN SLAB AND UPSTREAM WALLS BOTTOM GRID CROSS BARS UNDER SPANS BETWEEN TWO ADJACENT CONTRAFLEXIONS OF 3/4" BARS.
16	24·5 — 5/8"	25 5	65	MAIN SLAB AND UPSTREAM WALLS BOTTOM GRID CROSS BARS UNDER END SPANS BETWEEN EXTRA 3/4" BOTTOM AND CONTRAFLEXIONS UNDER PIER
17	31·25 — 5/8"	32·25	226	MAIN SLAB AND UPSTREAM WALLS CROSS BARS UNDER CENTRAL 3 SPANS BETWEEN EXTRA 3/4" BOTTOM BAR AND CONTRAFLEXIONS UNDER PIERS
18	186 5 — 5/8"	196·5	5	MAIN SLAB DOWNSTREAM END BOTTOM GRID CROSS REINFORCEMENTS BARS DOWNSTREAM OF PIER ENDS CARRIED RIGHT THROUGH ALONG THE BOTTOM.
19	8·0 — 5/8"	9	203	UPSTREAM WALL TOP GRID TOP BARS THROUGH WALL @ 12" CENTRE TO CENTRE
20	11·0 — 5/8"	12·0	171	UPSTREAM WALL BOTTOM GRID BARS THROUGH PILES @ 14½" CENTRE TO CENTRE
21	4·25 4·25 1·75 1·75 — 5/8"	11·5	234	UPSTREAM AND SIDE WALLS PILE CONNECTING BARS EVERY ALTERNATE PILE
22	2·0			UPSTREAM AND SIDE WALLS JOINT BARS

SCHEDULE OF BARS IN AQUEDUCT

TYPE	SHAPE	LENGTH OF BAR NEEDED	NUMBER NEEDED	POSITION IN THE STRUCTURE
1		20	220	STIRUPS IN SIDE WALLS
2		11-3"	170	STRAIGHT BARS IN SIDE WALLS
3		12-3"	290	BENT STIRUPS IN SIDE WALLS
4		39-6"	90	BOTTOM GRID LONGITUDINAL BARS
5		13-2"	530	CROSS BARS STRAIGHT REINFORCEMENT IN BOTTOM AND TOP SLABS
6		13-5"	530	BENT UP BARS IN BOTTOM AND TOP SLABS
7		39-6"	200	HORIZONTAL BARS IN SIDE WALLS
8		39-6"	120	LONGITUDINAL BARS IN UPPER SLAB AND LOWER SLAB
9		11-0"	190	TEMPERATURE BARS 12" APART UPPER LOWER
10		13-3"	190	TEMPERATURE BARS 12" APART UPPER LOWER
11		18-20 24-30	20	BENT UP BARS IN SIDE WALLS LONGITUDINAL
12		18-20 24-30	20	BENT UP BARS IN SIDE WALLS LONGITUDINAL
13		18-20 24-30	20	BENT UP BARS IN SIDE WALLS LONGITUDINAL
14		18-56 16-4 24-60	60	LONGITUDINAL BARS IN SIDE WALLS
15		15	580	STRUTS IN CORNICES
16		12	950	SPACES
17				STRAIGHT BARS 8" APART

APPENDIX II.

ANALYSIS OF RATES.

A. Working 15 H. P. Portable Steam Engine with Centrifugal Pump.

24 hours working—3 shifts of 8 hours each.

	Rs.	a.	p.
Labour :—			
3 drivers at 1/4 each	3	12	0
3 fireman at -/11/- each	2	1	0
4 coolies for supply of steam coal and water at -/7/- each	1	12	0
Total	7	9	0
Stores :—			
Steam coal 1½ tons at 19/1 per ton	28	10	0
Cylinder oil 2 gallons at 2/4 - per gallon ..	4	8	0
Mobil oil ¼ gallon at 2/4/- ,, ...	0	9	0
Cotton waste 1 lb. at -/2/- per lb.	0	2	0
Grease 1 lb. at -/6/- per lb.	0	6	0
Kerosene oil for lamps 1 bottle at -/4 - each ..	0	4	0
Rosin ½ lb. at -/2/- per lb.	0	1	0
Spares—belts, etc.	5	0	0
Total ..	39	8	0
Grand Total ..	47	1	0
Say Rs. 47/- ..			

B. *Working 12 H. P. Portable Steam Engine with one cubic yard concrete mixer.*

12 hours working.

	Rs.	a.	p.
Labour :—			
1½ driver at 1/4 each	1	14	0
1½ fireman at -/11/- each	1	1	0
2 coolies for supply of steam coal and water at -/7/-each	0	14	0
Total ..	3	13	0
Stores :—			
Steam coal 13 cwt. at 19/1 per ton	12	6	0
Cylinder oil 1½ gallon at 2/4/- per gallon ..	3	6	0
Mobil oil ½ gallon at 2/4/- ,, ...	1	2	0
Cotton waste 1 lb. at -/2/- per lb.	0	2	0
Grease 1½ lb. at -/6/- per lb.	0	9	0
Rosin 1 lb. at -/2/- per lb.	0	2	0
Kerosene oil for lamps, etc., one bottle at -/4/- each	0	4	0
Spares	5	0	0
Total ..	22	15	0
Grand Total ..	26	12	0
Say 27/-			

C. *Working one Donkey Pump with boiler for 24 hours of 3 shifts of 8 hours each.*

Labour :—

	Rs.	a.	p.
3 drivers at 1/4/- each	3	12	0
3 firemen at -/11/- each	2	1	0
4 coolies for supply of steam coal and water at -/7/- each	1	12	0
Total ..	7	9	0

Stores :—			Rs.	a.	p.
Steam coal 2 tons at 19/1 per ton	..	..	38	2	0
Cylinder oil 1¼ gallon at 2/4/- per gal.	..	..	2	13	0
Mobil oil ¼ gallon at 2/4/-	..	..	0	9	0
Cotton waste ½ lb. at -/2/- per lb.	..	..	0	1	0
Kerosene oil 1 bottle at -/4/-	..	..	0	4	0
Spares	..	..	0	10	0
	Total	..	42	7	0
	Grand Total	..	50	0	0

D. Working 15 H. P. Ruston Crude Oil Engine for 24 hours, 3 shifts of 8 hours each.

Labour :—					
3 drivers at 1/- each	..	..	3	0	0
3 oilmen at -/8/- each	..	..	1	8	0
	Total	..	4	8	0
Stores :—					
Crude oil 24 gallons at -/7/- per gal.	..	..	10	8	0
Kerosene oil 1 bottle at -/4/-	..	..	0	4	0
Cotton waste 1 lb. at -/2/- per lb.	..	..	0	2	0
Mobil oil 1¼ gallons at 2/4/- per gal.	..	..	2	13	0
Rosin ¼ lb. at -/2/- per lb.	..	..	0	1	0
Spares belts, etc.	..	..	1	12	0
	Total	..	15	8	0
	Grand Total	..	20	0	0

E. Working 8 H. P. Petter Engine for 24 hours of 3 shifts of 8 hours each.

Labour :—

3 drivers at 1/- each	3	0	0
3 oilmen at -/8/- each	1	8	0
Total ..	4	8	0

Stores :—

12 gallons crude oil at -/7/- per gal.	5	4	0
1½ gallons mobil oil at 2/4/- „	3	6	0
¾ gallon kerosene oil for lamps and blow lamps at 1/- per gallon	0	12	0
1½ lb. cotton waste at -/2/- per lb.	0	3	0
Spares, etc.	0	7	0
Total ..	10	0	0
Grand Total ..	14	8	0

APPENDIX III.

Cost of Driving Steel Sheet Piles.

Particulars.	Ransome D. Type Uniform Section.
1. No. of piles driven 20′ piles	206
15′ piles	385
2. Total length of pile line	701·3
3. ,, area ,, ...	11,750 S.ft.
4. No. of working shifts of 9 hours	26
5. Average progress per shift	450 S.ft.
6. Departmental labour—	Rs. a. p.
(1) Carriage of pile driver from railway station to site distance 8 miles	36 0 0
(2) Erection of machine	120 0 0
(3) Mechanical staff	160 14 0
1/2 Fitter foreman at 2/- a day 1/-	
One Fitter at 1/8 ,, 1/8	
One Driver at 1/4 ,, 1/4	
One Fireman at -/11/- ,, -/11/-	
4 Coolies for supply of steam coal and water at -/7/- each 1/12/-	
6/3/-	
26 shifts=160/14/-	
(4) Dismantling	72 0 0
7. Contractor's labour—	
(1) Driving piles	554 11 0
(2) Turning machine and track	168 10 0
(3) Running machine light	47 8 0
Total expenditure ..	1,159 11 0
Rate per s. ft.	0 1 7

Particulars.				Ransome D. Type Uniform Section.
Materials				744 2 0
Steam coal 1¼ tons at 19/1 ton	23	13	0	
Cylinder oil 2 gallons at 2/4	4	8	0	
Cotton waste 1 lb. at -/2/- lb.	0	2	0	
Grease 1/2 lb. at -/6/- lb.	0	3	0	
	28	10	0	
26 shifts 744/2/-				
Material per s.ft.				0 1 0
Total cost of driving per s.ft.				0 2 7
Cost of Piles, per s. ft.				
Cost per s.ft. F.O.R. Kiamari ..	1	14	1	
Railway freight to Chichoki Mallian per s.ft. ..	0	8	3	
Carriage from Chichoki Mallian Station to site distance 8 miles ..	0	0	8	
Total ..	2	7	0	2 7 0
Total cost per sq. ft. ..				2 9 7

APPENDIX IV.

ANALYSIS OF RATES.

Cement Concrete 1 : 2 : 4.

Materials—

		Rs.	a.	p.
23 c.ft. cement at Deg Head at 2/- c.ft.	..	46	0	0
46 c.ft. screened sand at -/12/- per cent. c.ft.	..	0	6	0
92 c.ft. Pathankot shingle at 29/- per cent.	..	26	11	0
		73	1	0

Labour—

Paid to contractor per 100 c.ft.	.. ..	4	4	0
,, extra for basket work per 100 c.ft.	..	0	8	0
,, extra for wriggling per 100 c.ft.	..	0	8	0
Per 100 c.ft.	..	5	4	0
Working expenses of mixer per 100 c.ft. as per details given below		0	12	4
Total Labour	..	6	0	4
Grand Total	..	79	1	4
Say 80/- per 100 c.ft.				
Portable steam engine per day		27	0	0
Mixer—Operator		0	14	0
Coolie for water		0	7	0
		28	5	0
Progress per day		3664 c.ft.		
Hence cost per 100 c.ft.		0	12	4

APPENDIX V.

ANALYSIS OF RATES.

Aqueduct Shuttering.

Materials—

	Rs.	a.	p.
Deodar sleepers 453	2,100	0	0
Railway freight for sleepers and unloading ..	85	0	0
Chil *ballas* 100 No. with carriage from Lahore ..	132	8	0
Carriage of 453 sleepers from Chichoki Mallian to site	100	0	0
Wire nails and other petty stores	100	0	0
Total ..	2,517	8	0
Cost debitable to this work=25 per cent. of 2,518/-=	630	0	0

Labour :—

Carpenters and coolies	2,316	4	0
Masonry pillars under Aqueduct	370	0	0
Total ..	3,316	4	0

Work done .. .. 9,281 cft.

$$\text{Rate per c.ft.} = \frac{3316}{9281} = -/5/9$$

Note.—Shuttering was made for 2 complete spans and for a third bottom. If it were made for only one span and one bottom, the cost would have gone down to less than -/4/- per c. ft. of concrete, but it would have taken longer to complete the Aqueduct.

APPENDIX VI.

ANALYSIS OF RATES.

Expansion Joints.

(a) *Copper flashing joint 1/16" thick.*
One joint of Aqueduct.

	Rs.	a.	p.
Materials :—			
Copper plates 4′×12″×1/16″=14 sheets= ..	70	0	0
Railway freight from Karachi do. ..	5	9	0
Carriage from Chichoki Mallian Railway Station to site of 14 sheets=	0	5	0
Copper rivets	2	12	0
Stores for soldering	1	8	0
Total ..	80	2	0
Labour :—			
Cutting and bending edges 14 sheets at -/4/- each ..	3	8	0
Soldering 14 sheets at -/4/- each	3	8	0
Making 'U' shape projection in all sheets—14 sheets at -/4/-	3	8	0
Bending sheets 4 No. at -/4/- each	1	0	0
Erection of copper flashing at site—one job ..	5	0	0
Total ..	16	8	0
Grand Total	96	10	0
Say	97	0	0

Total length of copper flashing=32+24=56 R. ft.

Cost per ft. run=1/11/9.

	Rs.	a.	p.
(b) Filling Socony Asphaltum in joints. Per joint.			
Materials.			
Socony Asphaltum 225 lb.	13	2	0
Railway freight from Karachi to Chichoki Mallian	4	12	0
Carriage from Chichoki Mallian to site ..	0	6	0
Total ..	18	4	0
Labour:—			
Heating, cleaning joints	2	10	0
Fuelwood and kerosene oil	1	0	0
Total ..	3	10	0
Grand Total	21	14	0
Say	22	0	0

Length of joint =30 r. ft.

Cost per r. ft. = -/11/9

PART INDEX PLAN OF SHEIKHUPURA DIVISION

SCALE = 1/2" = 1 MILE.

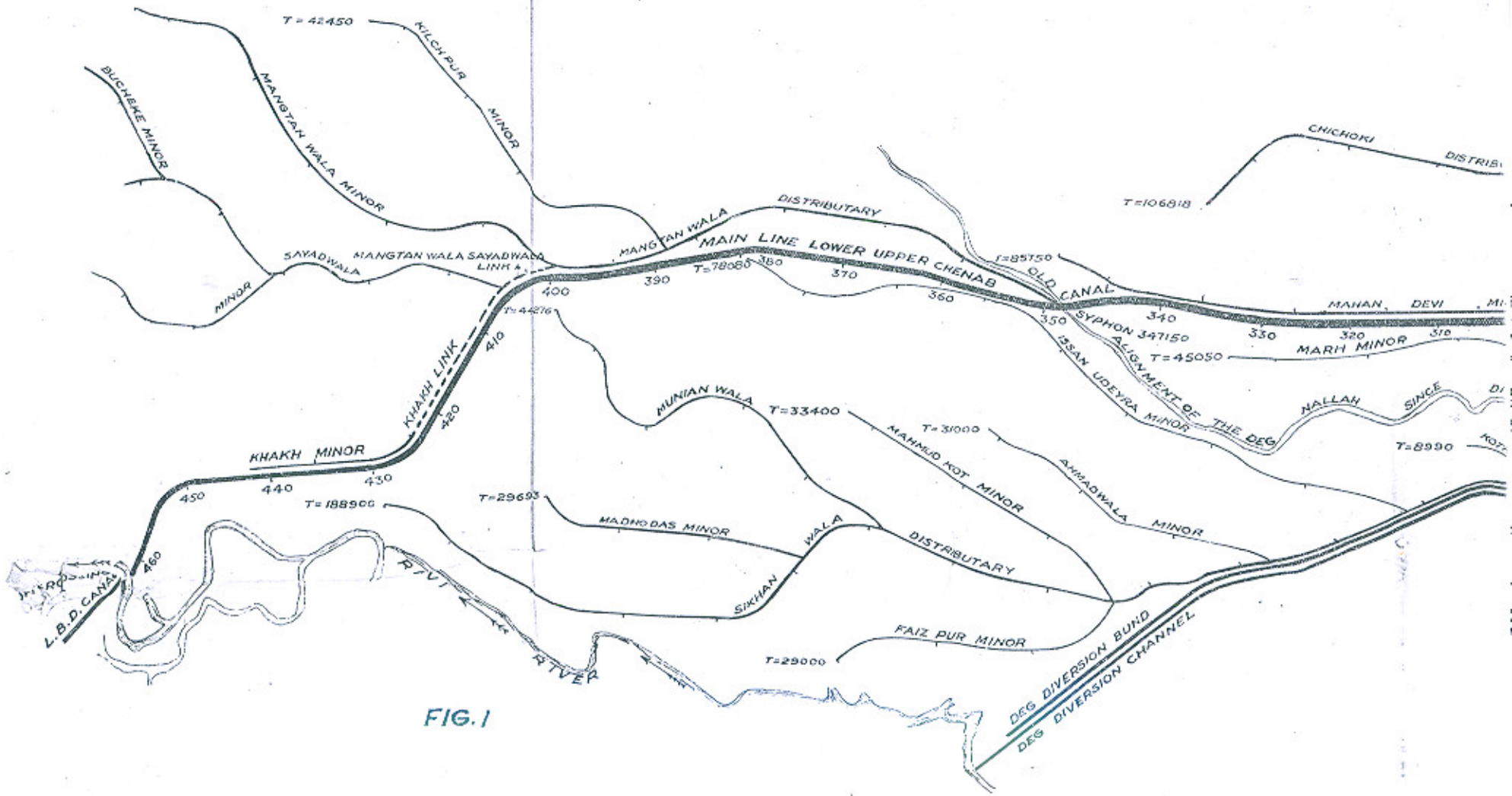


FIG. 1