THE MANUHERIKIA ROCK FILL DAM.

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The dam was constructed by the Public Works Department to conserve the water in the Manuherikia River for the Omakau Irrigation Scheme (see Mr. Hulse's paper, *Proc. N.Z.Soc.C.E.*, 1935-36, p. 333), and to augment the supply to the Lower Manuherikia Scheme at Chatto Creek. The deficiency in the river flows during the irrigation season varied from nil to an average maximum of 2,700 dayheads and with a peak efficiency of 3,800 dayheads. The dam is built to spillway level 1,840 and has a storage of 4,200 dayheads, covering 315 acres to an average depth of 26 feet.

The location of the site was fairly definitely fixed to the one position where the river after flowing across some eight miles of gravel flat cuts through a gorge in the Home Hills before reaching the open Manuherikia Valley.

This gorge marks the junction of the schist downstream and the greywacke upstream. The farther from this junction of the two rocks the better the quality of the greywacke, and the contours of the country being suitable, the dam was located at the upstream end of the gorge.

The rock is intermediate between a hard blue greywacke and the crushed and shattered rock in the vicinity of the junction with the schist. It is extensively fissured, but the individual blocks of rock between fissures are sound. Fault lines with a strike approximately east and west, and a dip from the vertical of 15 degrees towards north intersect the country at approximately 15 feet centres. These vary in thickness from ¼ inch to an average maximum of 4 inches. One, however, encountered in the diversion tunnel excavation was 4 feet wide. These faults are all filled with a dark blue pug of crushed rock. This is watertight and presented no difficulty other than preventing it from being blown out under water pressure.

These rock conditions are consistent with those to be expected of the greywacke in the vicinity of its junction with schist.

Choice of Type of Dam:

Various factors governed the choice of the type of dam to be built, and in the order of their importance they are:—

(1) Type of foundation.
(2) Ease of future heightening.
(3) Prevailing labour conditions.
(4) Rate of erection.

(1) It was considered that, in spite of the poor class of rock it was possible to build an arch dam on the site. The necessary water-tightness had to be obtained with the cut-off wall whatever type of dam was adopted, and there is no doubt that the rock in the foundations and abutments would have had an ample factor of safety against crushing. It can be seen, however, from the brief geological description of the country given above, that the site is not a desirable one for a rigid concrete dam, and taking into consideration the other factors, the balance was in favour of the rock-fill type.

(2) At some future date a possible extension of the Manuherikia Scheme could bring under irrigation a further 14,000 acres. This would mean heightening the dam 50 feet to conserve the necessary water.

An arch dam is difficult to heighten unless the increased thickness due to the extra height is already in place. In this case it would mean spending extra money on thickening a structure on the chance that it would have to be heightened some time in the future. On the other hand the rock-fill type of dam can be raised at any time with perfect safety by the simple process of widening and raising the bank, providing the slabs have been built for this increased height.

(3) When these facts were under consideration in 1931, the immediate object of the scheme was to provide work to absorb unemployed labour. The expenditure on labour, engaged on the works, for a concrete dam would have been approximately 40% of the total expenditure. For the rock-fill dam it was estimated to be 60% and actually proved to be 56%.

(4) The annual amounts made available necessitated a 4-year programme of building. This would have been too long a period for the building of a concrete dam under an economical system of working. With the rock-fill type, however, it was possible to work on single shift a portion of the time, spreading the construction period over the required 4 years.

After consideration of all the factors mentioned, the decision was made to construct the rock-fill type of dam, but it is pointed out that except where conditions are governed by factors as detailed, this type cannot be considered as an economical competitor of the concrete arch.

General Detail of Dam:

The crest length at level 1853.54 is 510 feet; maximum height at creek bed 110 feet; top width 14ft. 4in.; downstream batter $1\frac{1}{2}$ to 1; upstream batter $1\frac{1}{4}$ to 1 down to level 1770 and $1\frac{1}{2}$ to 1 below this level; the maximum width in the base is approximately 320 feet.

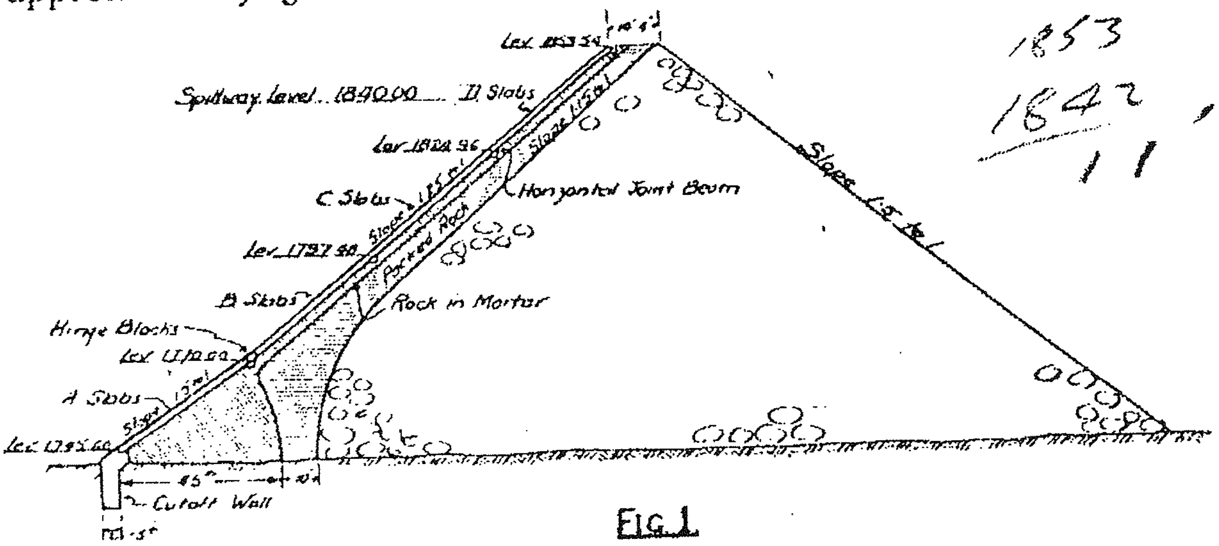


FIG. 1

MAXIMUM SECTION OF DAM

Scale in Feet 0 10 20 30 40 50

The bank is composed of quarried stone, made water-tight on the upstream face by concrete slabs. To provide these concrete slabs with a semi-rigid bearing, a layer of packed stone is built against the bank with a further layer of rock in mortar against this packed stone.

Taken as a whole the dam has a large margin of safety against sliding, but with a compressible rock-fill the shear stress is greatest at the upstream toe and diminishes to zero at the downstream toe.

The horizontal load at the upstream toe tends to compress the rock-fill in a horizontal direction with consequent shearing stresses in the water-tight face at the cut-off wall. It is to reduce this effect that the A slabs are built to a flatter batter and the rock in mortar thickened out to make a rigid prism at the toe.

Some movement of the face must take place due to settlement and compression of the rock-fill. To allow for this the slabs are built in 44 feet squares with flexible copper water-stops at all joints. Along the lines of maximum relative movement, viz., along the top of the A slab and along the juncture of the remainder of the slabs with the cut-off wall, special provision is made to prevent this movement damaging the water-tightness of the structure.

This is done by providing narrow heavily reinforced hinge blocks built along these lines with 1in. diameter reinforcing rods at 15in. centres projecting into the concrete on either side of them. (See Fig. 5).

During construction, the river was diverted through a tunnel. On completion of the works this tunnel was blocked with a concrete plug, and water is now discharged down the river bed by manipulation of valves set in this plug.

The spillway crest is at level 1840, giving a freeboard of 13.54 feet. The spillway is of the circular shaft type discharging into the downstream leg of the diversion tunnel.

General Method of Construction:

An early beginning was made to excavating the diversion tunnel. During its construction, work outside was done on stripping the dam area and quarrying and placing rock to the edge of the river.

When the river was diverted into the tunnel ,a 3-foot reinforced concrete pipe was concreted in a trench excavated in the bottom of the old river-bed.

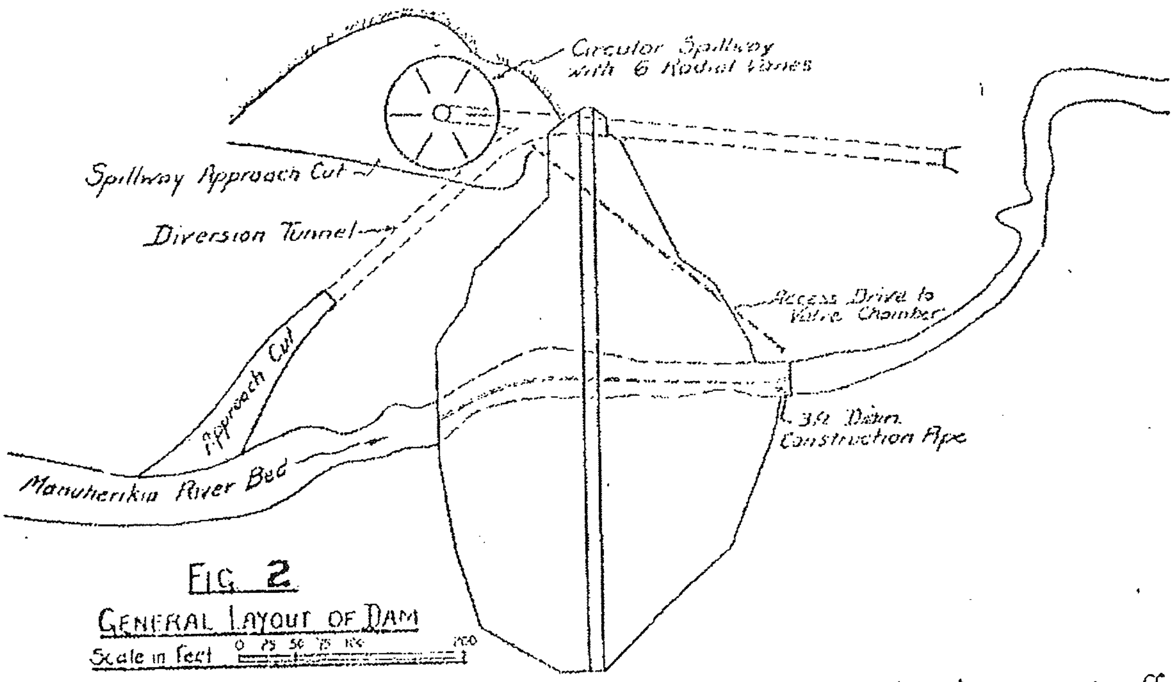


FIG. 2
GENERAL LAYOUT OF DAM

Work then proceeded on completing the rock plant, cut-off wall, concrete slabs, etc.

The excavation of the East Quarry was carried out with the dual purpose of providing rock for the bank and providing a site for the spillway with suitable clearance outside it for waterway.

As soon as this area was excavated, the spillway shaft connecting into the diversion tunnel was completed.

The above operations were completed in time to take advantage of the low-flow period, and the river was diverted through the 3-foot construction pipe under the dam.

The trash racks, concrete plug, valves, etc., were installed in the tunnel and the river again diverted into the tunnel this time through the valves.

The final operation consisted of sealing the upstream end of the construction pipe.

Stripping the Dam Area:

This work was done by hand labour. Suitable stones and spalls were stacked within the dam area on previously cleared ground. The balance was carted outside the dam area and dumped. One-cubic yards trucks on light service track were used on the sides of the valley and drays in the bottom.

The material encountered was mostly silty sands and gravels up to 12 feet deep on the floor of the valley.

The upstream half of the area was thoroughly cleaned down to bed rock, but on the downstream half not so much care was taken and pockets of material were not excavated below the general rock surface. These pockets of material were considered to be sufficiently stable under compression and in no danger of being washed out by a stream of water leaking in from the face.

The costs for 23,095 cubic yards stripping were:—

Labour	2/11.5
Horse-hire	8.5
Explosives	0.4
Stores	2.5
Tool Sharps	1.2
Plant Charges	0.1
	4/0.2

Construction Pipe:

The pipe was set in concrete in a trench excavated in solid rock. A minimum of 12 inches of concrete was placed around the pipe but where the rock was low extra haunching concrete was added. The pipe is 320 feet long and the average grade is 2 per 100. With the available head the capacity of the pipe was 160 cusecs.

Costs for 766 cubic yards trench excavation were:—

	s.	d.
Labour	6	0
Explosives	0	2
Stores	1	1
Sharps	0	7
Plant Charge	2	11
	10	9

Costs for 320 feet of 36in. reinforcing concrete pipe laid in concrete were:—

	s.	d.	
Excavation as above	25	9	per ft.
Pipe	16	5	
Concrete	37	1	
	79	3	

Concrete Cut-off Wall:

This wall serves as a footing for the slabs, and extends vertically downwards into the rock for water-tightness. This water-tightness could be obtained with a wall 2ft. thick, but it would require considerable reinforcing to withstand the heavy shear forces. Moreover, in the type of rock encountered it is practically impossible to excavate such a narrow trench. By the time the necessary depth was obtained, in some places up to 20 feet, difficulty was experienced in limiting the width of the excavation near the top to the required 5 feet.

The edge of rock on the downstream side of the excavation is chamfered off at an angle of 30 degrees to the horizontal to widen out the upper portion of the wall to 8 feet. This obviates a sharp re-entrant angle in the concrete whilst widening out the section where maximum shearing action takes place.

The use of explosives was limited; the excavation being done with paving breakers. The spoil was loaded into tubs and removed by light movable jib cranes operated by air winches. Where practicable, the sound rock was dumped into the dam fill. The trench was excavated along the line of the intersection of the under side of the slab, and the rock surface. It was located with as long straights as possible, and endeavour to make angles coincide with slab joints.

The costs for 2,235 cubic yards excavation were:—

	s.	d.
Labour	9	2
Explosives	0	5
Stores	2	5
Tool Sharps	1	11
Plant Charges	5	7
	19	6

The cut-off wall was concreted in sections 15 to 24 feet long. Construction joints had shear blocks and were made water-tight with copper strips. The concrete was trucked to the cut-off wall on track laid on piers of rock placed in mortar. These piers were later incorporated in the rock in mortar section.

The design strength of the concrete was 3,000 lbs. and 56 blocks tested at 28 days averaged 3,180 lbs. with a maximum divergence of plus and minus 21%.

Costs for 2,069 cubic yards of concrete placed were:—

	s.	d.
Labour—Boxing	3	8
" Concrete	3	7
" Copper	0	7
Stores—Boxing	1	11
" Copper	2	0
" Miscellaneous	2	2
Cement	29	9
Aggregate	5	8
Water	0	6
Chutes and Tracks	1	0
Plant Charges	2	3
	53	1

Grouting Cut-off Wall:

In the bottom of the trench, prior to concreting, 10 ft. holes were drilled at 2 ft. centres up to level 1,770 ft. and at 4 ft. centres from that level to the crest. One-inch diameter water pipes were set into the holes 18 inches and sealed in place. The cut-off wall concrete was then placed leaving the top ends of the pipes projecting above the surface. Grouting was commenced some weeks after concreting, using cement mixed with twice its volume of water.

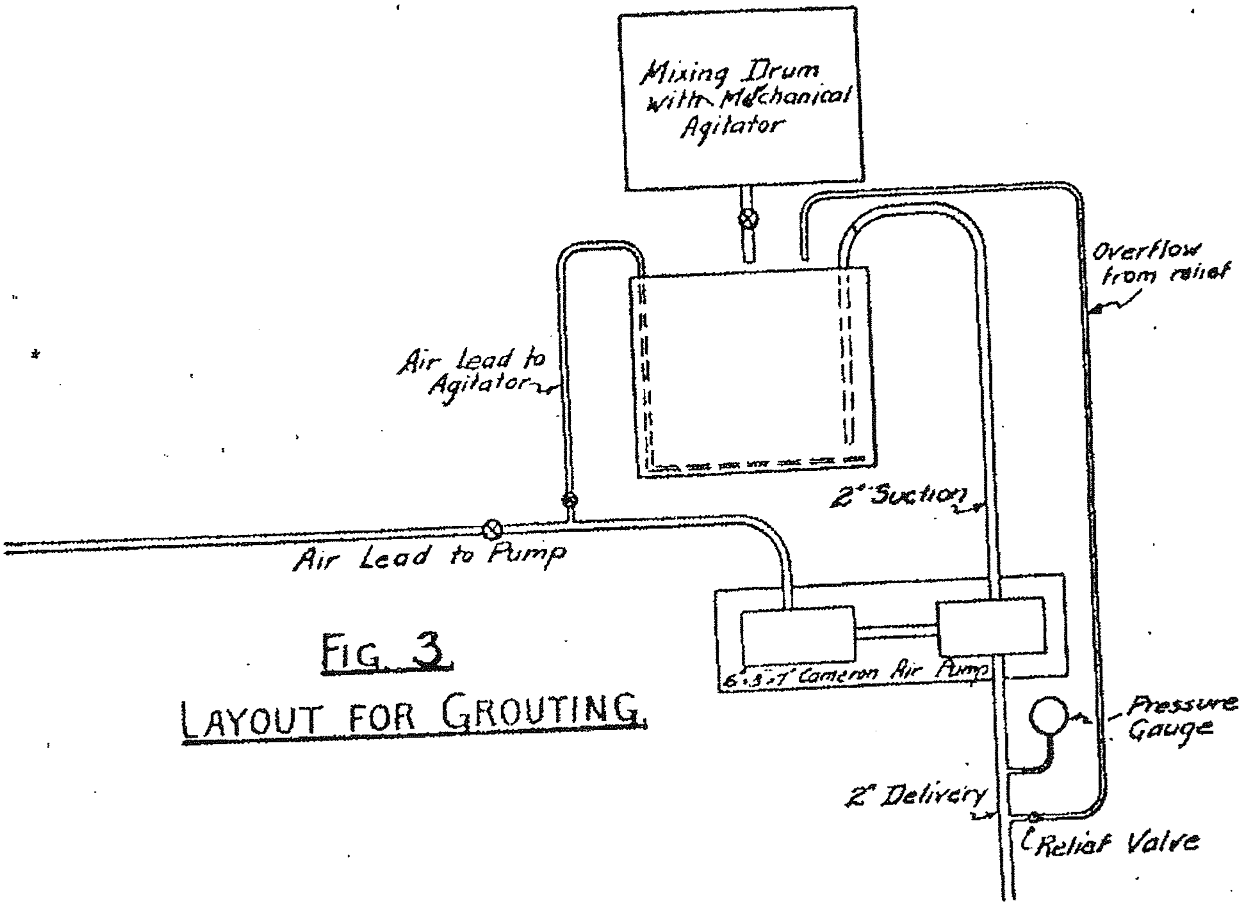


FIG. 3.
LAYOUT FOR GROUTING.

The grout was pumped into the holes by a 6in. x 3in. x 7in. Cameron air pump. On the discharge side a relief valve was provided with over-flow back to the suction tank. This served the purpose of regulating the pressure on the grout pipe and also kept the grout circulating when only a small amount or none at all was being taken by the grout hole. The holes were pumped to refusal at 200 lbs pressure.

The W/C. ratio of 2.0 mentioned above was increased to 3.0 later in the work and it is now considered desirable in this tight, but extensively fractured rock to commence with a water cement ratio of 5.0. This very watery mixture will penetrate where a thicker creamy grout will only block the hole. Once in place the gradually increasing pressure expresses the water out of the grout. If the hole takes much grout it is a matter of judgment if, and when to reduce the water content.

Deducting the quantity to fill holes, 196 holes grouted took 222 bags of cement—an average of 1.13 bags per hole. The 5 worst holes took 87 bags, and the 13 worst holes took 136 bags.

Approximately 10% of the holes when grouted, showed direct communication with adjoining hole or holes. When this arose, water was pumped out of the adjoining pipe and later grout. When the grout was coming at normal consistency the hole was plugged and pumping continued until the hole in question was sealed. The plugged hole was then washed out with a water jet and grouting of it continued.

By drawing up 2½in. water pipes through the cut-off wall concrete as it was placed, 3-inch holes were cored out at 4 ft. centres.

Holes were put down through these 3-inch core holes some 40 ft. into the rock. In sound rock fast drilling was done but in the fractured rock considerable difficulty was experienced. After passing through a fractured section, pieces of rock would fall in at the back of the bit and prevent its removal. These pieces were too big to be flushed out with the cuttings and had to be blown out with an air and water jet after removing the steel.

In several cases the holes had to be abandoned because of lost steel through the above cause.

The only satisfactory method of combating these difficulties was to drill until broken rock was encountered, and then grout the hole and continue drilling through the grouted broken rock.

All this proved expensive; approximately 14/- per poot of hole drilled, and with 300 feet of hole drilled and grouted in different types of rock in the bottom of the dam, no evidence was found that it was necessary for the stability of the structure to continue this work, and it was finally abandoned.

The nature of the rock is such that it is very difficult to stop percolating water. The pug seams of extensively crushed rock along the faces of major rock movements were dealt with by making the cut-off wall deep enough to prevent any blow-out of the fines in the seam. The remainder of the rock is uniformly and extensively fractured, but with the cut-off wall up to 20 feet deep in bad places plus a further 10 feet of grouting, very little leakage can occur.

The only actual cavity of any size exposed at any time in the rock was a hole 48in. x 12in. x 6in. found in the diversion tunnel excavation. It was more than half-filled with a hard chalky material deposited by percolating water. Similarly seams up to ¼ inch wide were occasionally found completely blocked by the same deposit. This deposit was analysed and proved to be 92% carbonate of lime and residue mainly hydrous silicate of aluminium.

Drilling costs of 10 feet grout holes for 1,960 ft. drilled were:—

	s.	d.
Labour	0	10
Stores	1	3
Tool Sharps and Repairs	0	5
Plant Charges	1	3
	3	9

These costs are high for the same reason that the deep drilling was expensive. They were down holes and a lot of trouble was experienced in withdrawing steel.

Cost of grouting per hole for 196 holes were:—

	s.	d.
Labour	5	2
Cement	8	6
Piping	4	8
Stores—Miscellaneous	1	6
Water	0	10
Plant Charges	16	3
	36	11

This is approximately £35 per cubic yard of grout in place.

Leakage:

Several chains below the dam a hard bar of rock, just above the site of the old Falls, was made into a station for measuring the leakage. Unfortunately, this includes the run-off from several acres of land. The best result obtained with a full dam is 2/3 gallons per second, but it is known that this includes drainage water apart from the leakage, and it is estimated that the total leakage from the dam is in the neighbourhood of ½-gallon per second.

This result for some 58,000 square feet of concrete slabs with their 3,300 feet of flexible jointing, and 680 linear feet of cut-off wall is considered very satisfactory, especially in view of the class of country encountered, and it is clear that the expenditure on grouting is completely justified.

Placing the Fill-Rock:

To construct a bank of maximum density it is desirable to place the fill in layers. This could not be closely adopted because of (1) the location of suitable stone for quarrying; (2) the necessity for keeping the river channel clear until the diversion tunnel was completed; (3) the necessity for excavating a spillway site in conjunction with the east quarry operations.

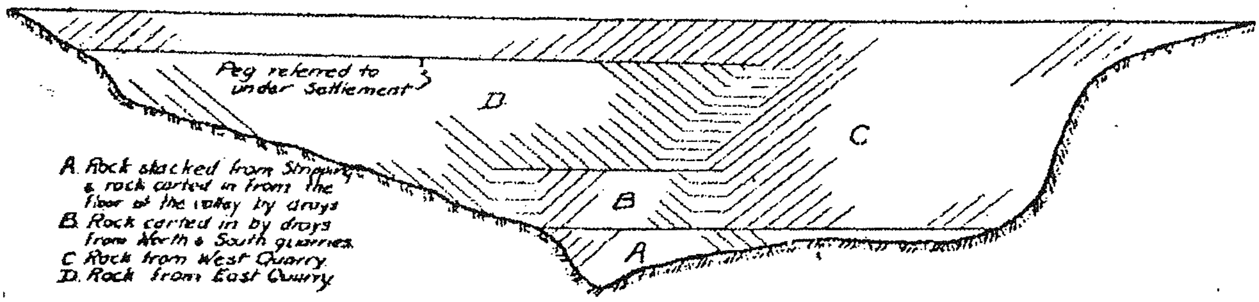


FIG. 4
LONGIT. SECTION ON CREST LINE
SHOWING LAYERING OF FILL
Scale in Feet: 0 20 40 60 80 100

However, as can be seen in Figure 4, certain layering was carried out, and this, in conjunction with the comparatively long time of construction, has resulted in a stable bank.

Selected stone from the tunnel tip and stone stacked during stripping operations was used in the construction of Section A. The material for Section B came from the north, middle, and south quarries. Rock for Section C was obtained from the east quarry, and for Section D from the west quarry.

At no time were two tips allowed to run more closely together than 20 feet. This distance was maintained by hand-packing the stone in level courses between the tips and thus preventing any arching across these tips with consequent loss of density and stability.

The rock weighed 167 to 171.25 lb. per cubic foot, and as tipped from trucks had an angle of repose varying between 37 and 39 degrees.

The top of the dam was kept downstream approximately 10 feet. This enabled the rock for hand-packing the face to be conveniently placed by side tipping.

The downstream batter of 1½ to 1 being flatter than the angle of repose of the rock, it was necessary to tip sufficient rock outside the required finished batter at the top to make up the deficiency at the bottom.

The bank was then raked down to the correct batter. A rake out of 12in. x 8in. hardwood, 6ft. long, with 1in. diameter steel teeth, was pulled up and down the face with a double drum air winch.

Quarrying Rock:

The rock for the bank was obtained mainly from the east and the west quarries. The east quarry was excavated at level 1,840 providing waterway for the spillway. The west quarry floor level was 1853-54, the crest level of the dam. Any suitable stone available in the floor of the valley was carted in by drays and three small quarries were opened up at lower levels than the main ones, to enable layering to be carried out.

The rock in all the quarries was difficult to drill because of its fissured nature. Drills were sharpened with star bits and wherever possible down holes were avoided because of the liability of the drills to jam with the small fragments which fell down the hole.

It was intended to try some big shots, but the fissured nature of the rock resulted in good shooting and the maximum depth of hole drilled was limited to 12 feet. No mechanical aids were used for loading the stone; very little popping being necessary to reduce it to a suitable size for hand loading.

It was loaded into 2 cubic yard trucks which were hauled to the tip by horses. All the fines were forked over, nothing smaller than 2½-inch stone going into the dam. The balance, 33,305 cubic yards in all, was dumped. No watering of the tip was done; it being considered that the careful removal of all fines was all that was necessary to make a sound and uniform bank.

There was, however, a tendency for the top of the tip to show up any fines that were inadvertently included. It is now considered better practice to reduce the time taken in the removal of the fines and expend it in watering the tips to wash down into the interstices of the bank any fines that may have been included. All stones throughout the bank would then be in close contact and not separated by fines that would slowly crush under load and delay final settlement. Care would have to be taken, however, to keep the inclusion of fines within such limits as would not endanger the free drainage of any water leaking past the face.

Towards the end of the construction some doubt was felt as to the ultimate settlement of the dam. To hasten this on, to enable the location of the slabs to be fixed, a pump was installed and 9 million gallons of water pumped on to the top of the bank over a period of 5 weeks. A close check of the settlement showed no change of rate over this period.

The actual completed size of the bank is 168,000 cubic yards, and the measured loose volume of rock placed in the dam was 175,352. This deficiency of 4·4% can be accounted for by shrinkage in the bank. The sources of the 175,352 cubic yards and the respective quantities of waste, are tabulated below:—

	Rock.	Waste.
East and West Quarries	132,567	31,254
South Quarry	18,947	1,441
North Quarry	3,349	333
Mid Quarry	2,668	277
Carted in by Drays from Floor of Valley	17,626	—
	175,157	33,305

The east and west quarry costs herewith are based on the 132,567 cubic yards loose measure placed in the dam, the actual quantity handled being greater than this by the 31,254 cubic yards of waste.

Labour Drilling Holes	.81d.
Labour Loading Holes	.13d.
Labour Barring Down	.48d.
Labour Filling Trucks	16.13d.
Explosives	2.18d.
Stores—Miscellaneous	1.76d.
Tool Sharps and Repairs	1.66d.
Lead Labour	2.28d.
Lead Stores	.87d.
Horse Hire	.60d.
Truck Repairs	.68d.
Plant Charges	4.22d.
	2/7.80d.

The south quarry rock was the best quality stone placed in the dam. It was carted in by drays 6 chains—3 chains of which was up a 1 in 10 grade. The costs herewith are based on the 18,947 cubic yards loose measure placed in the dam—the actual quantity handled being greater than this by the 1,441 cubic yards of waste.

	c. yd.
Labour Drilling Holes	.83d.
Labour Loading Holes	.16d.
Labour Barring Down	.44d.
Labour Filling Drays	11.51d.
Explosives	2.02d.

Stores—Miscellaneous	1.45d.
Tool Sharps and Repairs	1.83d.
Drivers	3.41d.
Horses and Drays	5.15d.
Tip Labour	1.60d.
Plant Charges	4.11d.
	2/8.51d.

The cost per cubic yard loose measure in place in the dam for rock from the other sources is tabulated below.

Source.	*Quantity.*	*Unit Cost.*
Carted in by Drays from Floor of Valley	17,626	2/4.48d.
North Quarry	3,349	2/7.68d.
Mid Quarry	2,668	2/7.94d.

A final cost of 2/9.9d. is based on the settled quantity of 168,000 cubic yards in the dam and includes cost of watering and raking down the back of the dam to the correct batter, and hand-packing to keep tips from meeting.

Drilling and shooting figures for the main quarries are tabulated below:—

	West Quarry.	*East Quarry.*	*South Quarry.*
Feet Drilled per bit sharpened	1.8	1.9	1.2
Feet Drilled per man hour	5.7	4.8	5.0
Cu. yds. rock in dam per ft. drilled	10.2	9.8	7.9
Cu. yds. rock in dam per lb. gelignite	11.5	11.0	11.5

Placed Rock:

This layer (see Figure 1) varied in thickness from 10 ft. under the B. slabs to 5 ft. on the top.

No stone large enough to warrant the use of mechanical aid in handling was obtained from any of the quarries. The stone had a conchoidal fracture, and was particularly unsuited for close packing. The practice adopted was to bed together a layer of the largest available stone and then chink this layer with smaller stones, the result being a fill of maximum density.

The cost for 13,480 cubic yards, additional to quarrying the stone were:—

Labour	1/7d.
Stores	1d.
Total	1/8d.

Rock in Mortar:

The layer of rock in mortar was placed directly against this layer of placed rock. It is 2ft. 9in. thick except under the A. slab where the back wall goes down vertically to provide a rigid mass under the A. slab. (See Figure 1).

The largest available stones were selected for this work and were laid with their long axis normal to the face of the dam.

The aggregate used for the mortar was the quartz sand used to provide the additional fines in the concrete. A true mix of 1 to 8.7 with W.C. ratio of 2.4 was used with a resultant test of 700 lbs. per square inch.

Holding down pins for the slab shuttering and the slab anchor bolts were bedded in this layer. No shuttering was used except in the construction of the recesses for the slab joint beams. The amount of mortar used was approximately 50% of the total and the resultant mass was firm enough to stand up to the final batter without shuttering. Large stones were kept 3 inches from the outer face which was screeded to give the slabs a smooth bearing.

The costs for 7,130 cubic yards, additional to quarrying the stone, were:—

	s.	d.
Labour	5	6
Cement	8	6
Sand	2	0
Water	0	9
Stores—Various	1	2
Chutes and Tracks	0	8
Plant Charges	1	1
	19	8

Watertight Concrete Face:

This consists of reinforced concrete slabs cast against the smoothly screeded rock in mortar (See Figures 1 and 5). The slabs are 44 feet square and vary in thickness from 24 inches at the bottom of the B. slabs to 15 inches at the top of the D. slabs. The A. slabs are 24 inches thick throughout.

The slabs are reinforced with 1 inch diameter rods laid both ways and in two mats—one at the top of the slab and one at the bottom.

A	slabs rods	are at	18	inch centre
B	,,	,,	12	,,
C	,,	,,	15	,,
D	,,	,,	18	,,

This represents approximately .5% of reinforcing both ways, except for the A. slab; it is .36%.

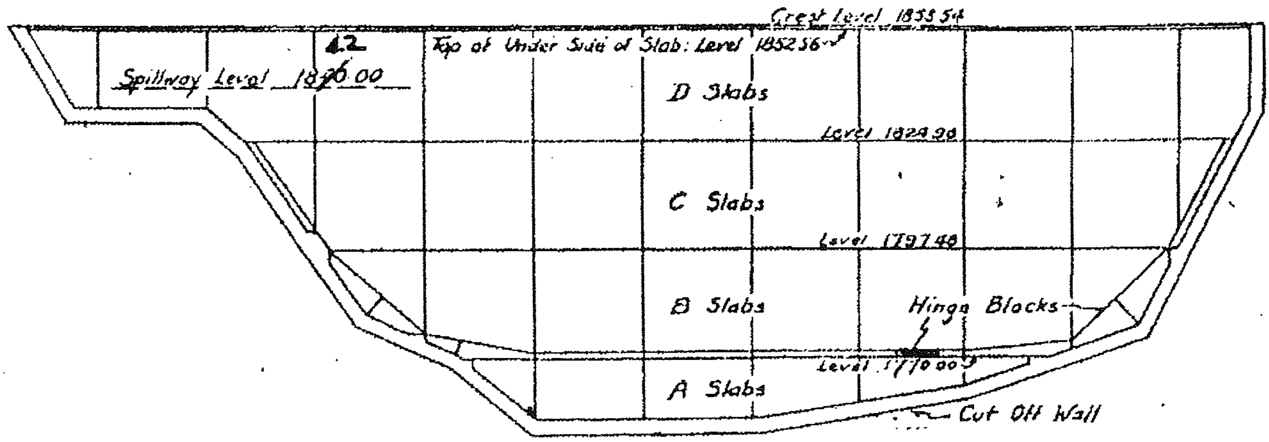


FIG. 5

DEVELOPMENT OF UPSTREAM FACE

Scale in Feet 0 20 40 60 80 100

Underneath the slab joints reinforced concrete beams are cast in recesses left in the rock in mortar. The inclined beams are continuous and are 2 feet thick by 4 feet wide. The horizontal beams are triangular in section with a vertical back 2 feet 9 inches and a horizontal base 3 feet 5 inches. They have a 1 inch gap filled with bitumen at their juncture with the inclined beams. They serve the purpose of providing extra support for the slabs at their edges and in the event of any large movement in the bank they provide a rigid plane to prevent shearing of the copper water stops.

All joints between slabs, hingeblocks and cut off wall are made watertight with copper strips. These are 15 inches wide by 16G. up to the top of the B. slabs, and 12 inches wide by 18G. above that. A corrugation in the centre of the strip 3 inches deep by 1 inch wide takes care of any movement at the joints. All copper joints are rivetted and soldered and at crossings, special junction plates are used.

The A. Slabs are separated from each other and from the cut off wall with a film of bitumen paint. The hinge blocks have a $1\frac{1}{2}$ inch gap at all edges. All other joints between slabs are made with a 1 inch gap. All gaps are filled with 40 penetration bitumen. This was laid in precast slabs below the copper. The gap above the copper stop was filled after the completion of the concreting. The bitumen was poured in hot and kept in position by 6 inches x 3 inches totara strip bolted down with galvanised bolts set in the concrete at 5 ft. 0 inches centre.

The slabs are restrained by double anchor rods 1 inch diameter spaced at 4 feet 0 inches centre parallel to and 11 feet from the edge. Outside the area enclosed by the anchor rods the surfaces of the mortar and the joint beams were coated with emulsified bitumen painted on by hand.

The resultant film of bitumen was dangerously slippery to walk on and would definitely break the bond between the slab and the mortar, and also will act as a lubricant for any movement of the slab. Thus the slab is restrained by its central $\frac{1}{4}$ area and free to move under temperature stresses outside this area with no danger of any downwards creep.

As an aid to watertightness each slab was concreted in a continuous pour. As some doubt was felt as to the load the shuttering would have to take on the sloping face of the dam, a small experimental slab was made. The shuttering for this slab had a loose panel and the load on this panel was observed for a rate of concreting comparable to that expected to be obtained when concreting the dam. From these results the shuttering was designed and proved satisfactory. It consisted of shutters 2 feet 9 inches long by 2 feet 0 inches deep. These were held in position by 6 inches x 3 inches joists at 2 feet 9 inches centres and by walings at 4 feet 0 inch centre. One double No. 8 wire tie was used for each 4 shutters.

Concrete was delivered from a temporary track set above each row of slabs at a convenient height. Four lines of chutes were used for each bay. They were rectangular in section resting on top of the shuttering and could be swung through the arc necessary for uniform placing. Shutters were never placed more than one row ahead of the concrete. The men thus had free access for placing purposes. No mechanical tamping was used. The mix adopted was approximately 4 bags to the cubic yard in place with a water cement ratio of .92, and sufficient fines to give a very workable mix. No difficulty was experienced in placing this concrete by hand.

73 concrete blocks were crushed at 28 days with an average test of 2890 lbs. and with maximum divergence from this average of plus 33% and minus 20%. The design strength was 3,000 lbs. To test the porosity of the mix, red ink was applied to one end of a test block under a pressure of 100 lbs. per square inch for 400 hours. When sawn in two the maximum distance the ink had penetrated was $\frac{5}{8}$ inch.

When stripped the surface was spaded over, wires were cut out to a depth of 1 inch and the holes plastered over. The surface was then painted over with the following mixture:—Cement, 14 vols; sand, 14 vols; silicate of soda, 1 vol. This formed a hard watertight skin on the face improving the watertightness of the slab and also assisting in curing.

Costs for 4130 cubic yards slab concrete were—

	s.	d.
Labour boxing	7	6
,, concrete	5	4
,, copper	1	0
,, steel	3	5
,, bitumen		10
,, surfacing	1	2
Stores boxing	3	7
,, copper	4	7
,, steel	15	0
,, bitumen	1	6
,, various	2	2
Cement	30	6
Aggregate	6	3
Water		11
Chutes and tracks		6
Plant Charges	5	0
	89	3

Above steel costs amount to £14 per ton in place for 266 tons.

Settlement and Drift:

These are the indeterminate factors in the construction of the rock fill dam that have to be taken into consideration in the design of the watertight face. The rocks in the dam can be considered to have mainly point and edge contact with one another. It is the crushing of these points and edges until the area is developed sufficiently to reduce the stresses that is the main cause of settlement, with its accompanying inward drift of the faces. As mentioned previously, various methods were adopted to reduce settlement, but undoubtedly the most important was the long period of construction.

In addition to the inherent settlement of the bank itself, a further movement must take place under water load. It is to take care of any remaining settlement in the bank plus this settlement due to the water load that the watertight face is made flexible.

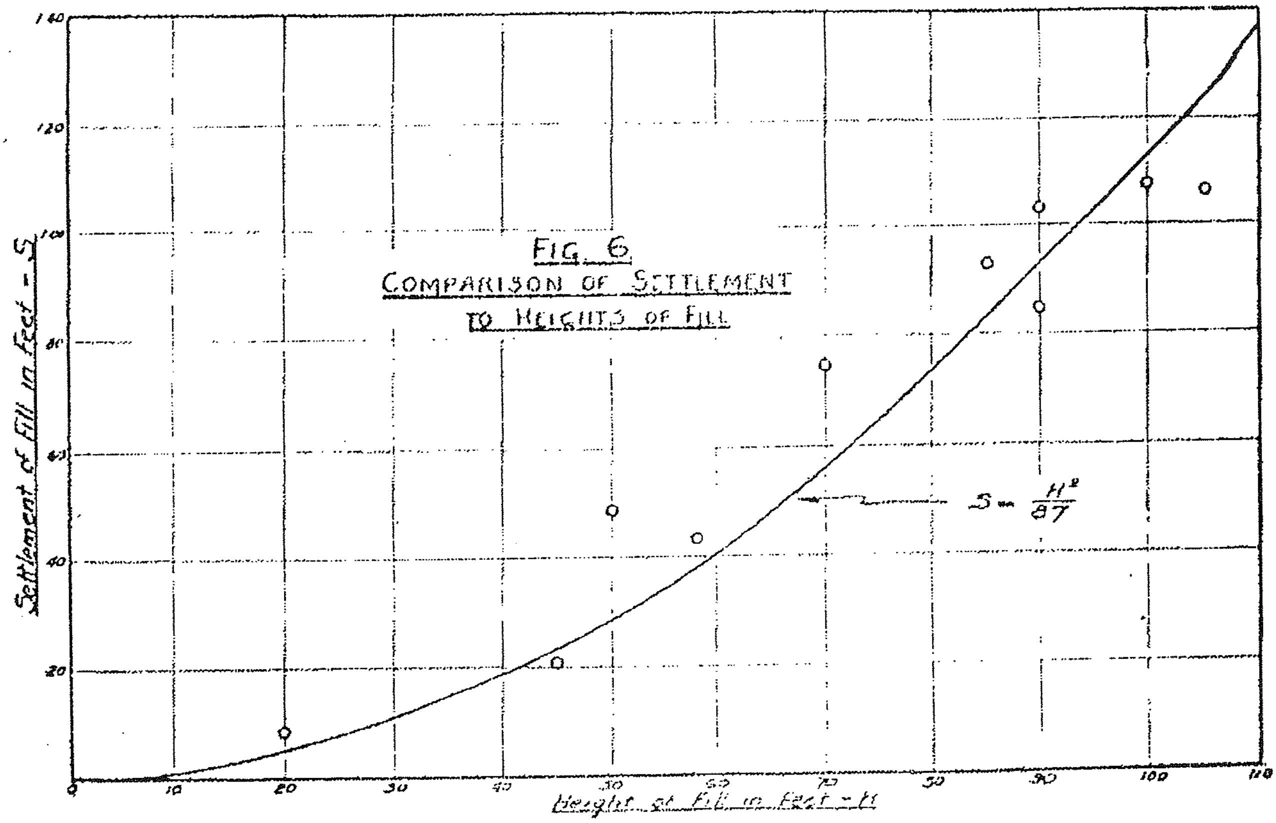


FIG. 6
COMPARISON OF SETTLEMENT TO HEIGHTS OF FILL

The settlement of the bank appeared to be proportional to the square of the depth of fill. This statement is based on figures taken both before and after filling. In Figure 6 the settlement as measured on top of the dam over the period of April 1935 to October 1936 (see Figure 8) is plotted against depth of fill. This is compared to curve $S = \frac{H^2}{87}$ where S = settlement and H = depth of fill. The comparison is reasonable considering the varying methods of placing as shown in Figure 4.

The downstream drift up to the time of filling was approximately .8 of the settlement, i.e., the movement of any peg was approximately normal to the face.

During construction, observations were made on various pegs to check the movement of the bank. Records of a typical peg, the location of which is shown in Figure 4, are given below. The peg was in the middle of the East quarry bank which was approximately 35 feet wide.

Date	*Level*	*Transverse Drift*	*Drift from Abutment.*
13/4/34	1838.48	.00	.00 ft.
29/9/34	1837.46	.35 ft (up stream)	1.56 ,,
20/11/34	1837.40	.39 ,,	1.78 ,,
12/2/35	1837.33	.20 ,,	1.85 ,,

It is interesting to note that to 20/11/34, side tipping upstream was being done for the layer of hand packed rock, and there was a transverse drift of the bank in this direction. This direction of drift was reversed when side tipping downstream was done for widening out the bank. The West quarry tip at level 1853.54 met this East quarry tip early in November, 1934, and for three months advanced towards this peg in question, and yet the longitudinal drift away from the abutment continued at an undiminished rate.

From these and similar figures obtained during construction of the bank an attempt was made to build the upstream face with sufficient camber to enable it to remain convex when final settlement had taken place. The settlement will therefore cause slab joints to close rather than to open, and also the final position of the slabs will be more sightly than if concave.

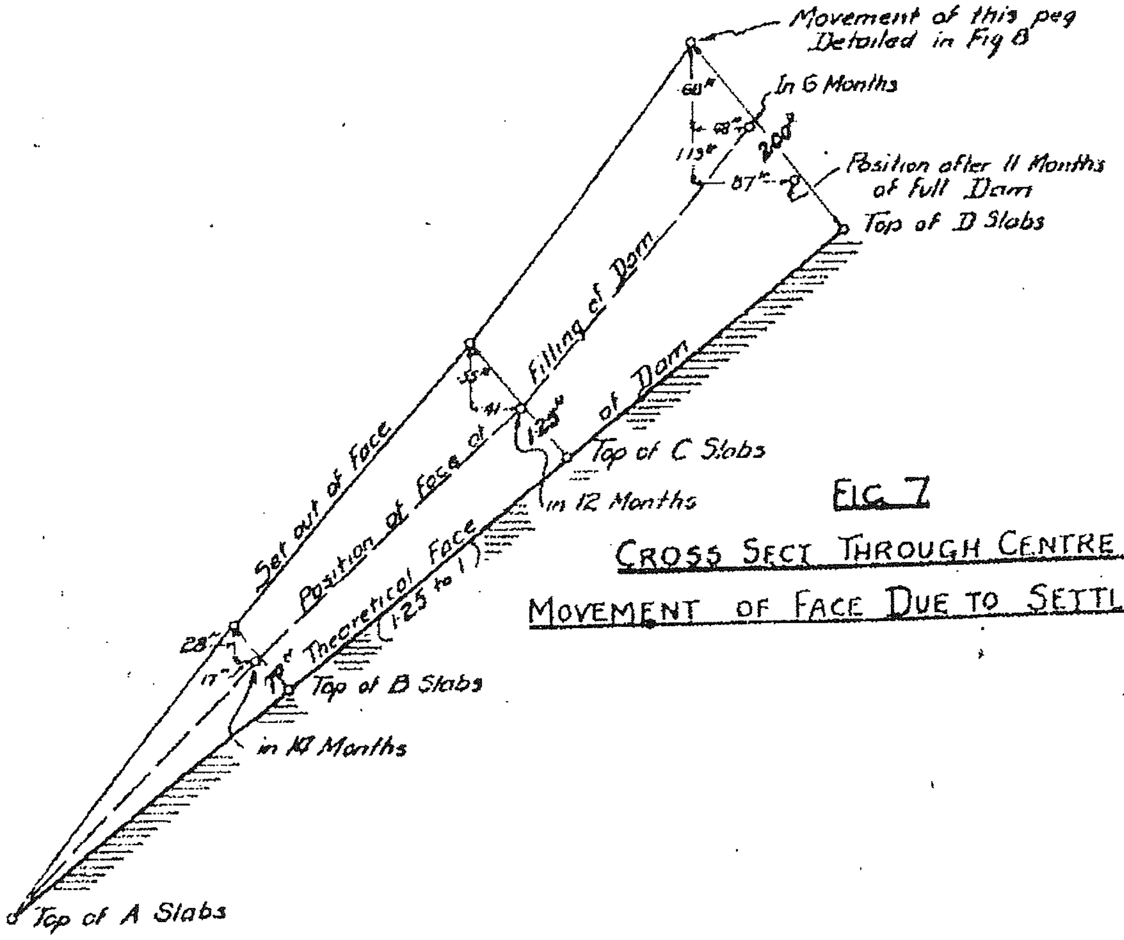


FIG 7

CROSS SECT THROUGH CENTRE OF DAM

MOVEMENT OF FACE DUE TO SETTLEMENT

Figure 7 shows the offsets square to the face given to the centre of the face; viz., .70 ft., 1.25 ft. and 2.00 ft. at the tops of the B., C. and D. slabs respectively. It is hoped that this camber will be sufficient to prevent any concavity of the face, and, as can be seen from Figures 7 and 8, it appears reasonable to expect this to be so. Pegs have been bedded in the concrete face at each slab joint at the top of the A. slabs, the bottom of the B. slabs, and the tops of the B., C. and D. slabs. These were observed for settlement, downstream drift, and longitudinal drift.

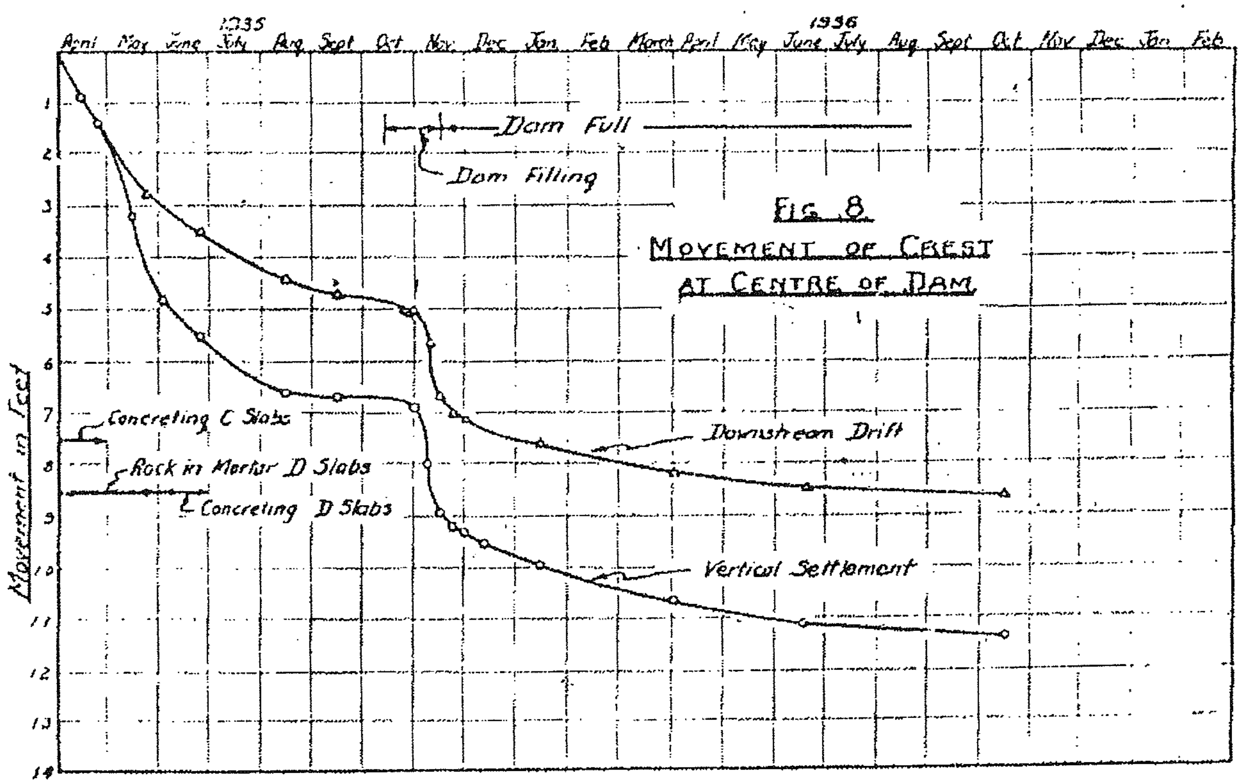


Figure eight shows the movement of the centre slab at the crest of the dam. For the four month period, June to October, 1936, the settlement amounted to .03 feet and the downstream drift to .03 feet. The face at this point is still .43 feet above the theoretical crest and .38 feet upstream of it, so it appears that the camber allowed at the top has been sufficient. Unfortunately, it was not possible to dewater the dam during the winter of 1936 and no observations of pegs other than those on the crest have been possible since filling the dam in November, 1935. Observations of the leakage from the dam shows no increase, so any movement of the face that may have taken place below water level does not appear to have ruptured any of the flexible joints.

Diversion Tunnel:

With the rock fill type of dam it is essential that it be not overtopped either during or after completion of construction. Records showed that a flood of 3000 cusecs could be expected in any month of the year and that there was a fair probability of a flood exceeding 5000 cusecs during the 4 years' construction period. The diversion tunnel was designed with a area of 227 square feet on a down grade of 1 in 100, thus providing for a flow of 5000 cusecs with very little heading up. The inlet portal was extensively bell-mouthed to cut down entry losses. During the construction period one flood of 4900 cusecs was experienced, and several about the 3000 cusecs mark.

The invert at the inlet portal is at level 1739, which involved 35 feet of cut. Leaving a dumpling between the river and the tunnel this cut was taken out on a 1 in 4 grade. At the outlet portal the invert was 13 feet above water level. The country was overhanging dangerously and it was necessary to concrete the portal and a length of barrel before going underground.

The tunnel was constructed by the top heading and bench method. The English system of timbering was adopted, using 12 inch x 12 inch hardwood sills and 7 bars. (See Figure 9 for tunnel section.) The spoil was trucked out by horses in 2 cubic yard trucks from the outlet end and by 1 cubic yard trucks at the inlet end. At the inlet end the trucks were winched up the 1 in 4 grade from the portal.

The 12 inch lining of concrete was done in 14 feet lengths and kept close up to the excavation. All timber was removed from behind the concrete. In certain lengths where the weight of the country was too great to be taken by the green concrete and the formwork, the timber bars were replaced by 53 lb. rails which were concreted in. Immediately after stripping formwork the surface was gone over with an air grinder and every effort made to leave it as smooth as possible.

The concrete was handled on a separate line laid on transoms set across the centre of the tunnel. This track offered very little resistance to high flows and was left in and used as a working platform for grouting the tunnel and excavating the spillway shaft after the river had been diverted.

The design strength of the concrete was 3000 lbs. 105 blocks gave an average test of 2940 when tested at 28 days with a maximum divergence of plus 33% and minus 25%.

The costs for 500 feet of barrel, neglecting portals and grouting were:—

Excavation:

	s.	d.
Labour excavation	99	10
„ timbering	11	4
„ lead	7	6
Stores explosives	19	5
„ timber	13	1
„ miscellaneous	27	8
Truck repairs	1	4
Tool sharps, etc.	13	6
Horse Hire	3	6
Plant Charge	61	10
	259	0

Concreting:

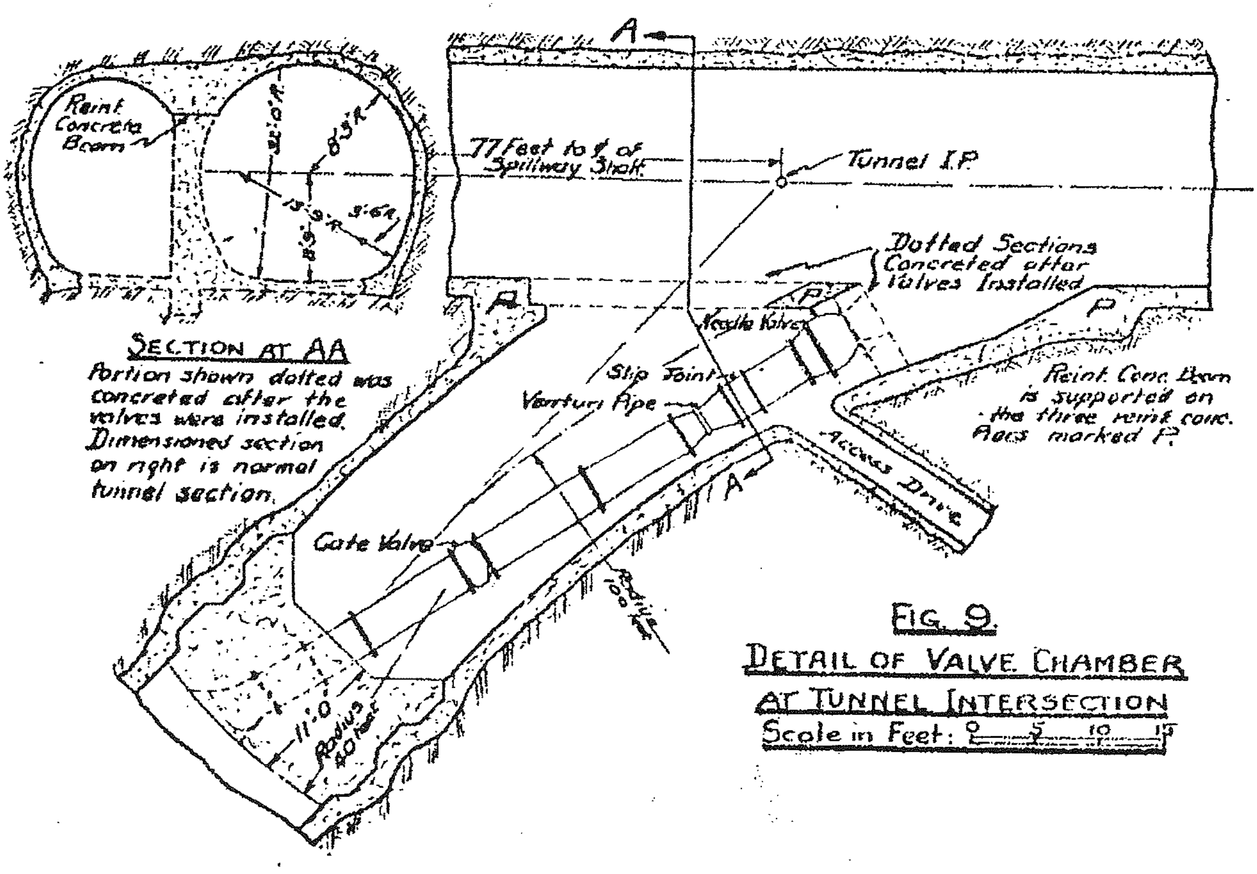
	s.	d.
Labour boxing	8	5
„ concrete	21	7
„ surfacing	5	0
Stores boxing	12	6
„ miscellaneous	15	7
Cement	76	2
Aggregate	13	4
Water	4	6
Chutes and track	5	4
Plant Charges	14	5
	176	10

Total cost —£21/15/10 per foot.

Nominal	excavation	10.7	cubic yards per ft.	=24/2	per c. yd.
Actual	„	11.5	„	=22/6	„
Nominal	concrete	2.3	„	=76/11	„
Actual	„	3.1	„	=57/1	„

Junction Section:

Making portion of the diversion tunnel serve also as a spillway tunnel involved the construction of a junction in which the streamlined characteristic of each were not materially interfered with.



SECTION AT AA

Portion shown dotted was concreted after the valves were installed. Dimensioned section on right is normal tunnel section.

FIG. 9.

DETAIL OF VALVE CHAMBER AT TUNNEL INTERSECTION

Scale in Feet: 0 5 10 15

As can be seen from Figure 9 at the junction of the two tunnels their respective arch loads were taken by a massive reinforced concrete beam running parallel to the spillway tunnel. This beam is 42 feet long and has for a central support a pier designed to offer little resistance to the diversion flow and to be incorporated in the concrete when sealing off the diversion tunnel.

Advantage was taken of the Christmas shut down to put in a small prospect drive. The country proved typical of the majority of that passed through; hard rock, but extensively fissured. There were however, no major fault planes and it was possible to open up all the diversion side of the junction with no undue load on the timbering. This side was then concreted together with the beam, and work proceeded on the Spillway tunnel. Side struts were kept against the beam until the load was balanced by the spillway arch.

The Excavation Costs for 1160 Cubic Yards excavated were:—

	s.	d.
Labour excavation	10	5
,, timbering	2	1
,, lead	1	3
Stores explosives	1	7
,, timber		9
,, miscellaneous	2	6
Tool sharpening, etc.	1	11
Horse Hire		7
Plant Charges	9	7
	30	8

The Concrete Costs for 350 Cubic Yards in place, were:—

	s.	d.
Labour boxing	8	5
,, concrete	12	8
,, Reinf. steel	3	1
,, surfacing	1	0
Stores Boxing	5	9
,, Reinf. steel	14	8
,, miscellaneous	4	7
Cement	27	4
Aggregate	6	5
Water	1	0
Chutes and track	1	0
Plant Charges	5	2
	91	1

The above steel costs amount to £24/0/0 per ton for 13 ton in place. The total length of the junction is 62 feet and the cost is £54/6/4 per foot.

Transition Section:

From the junction towards the spillway shaft the tunnel changes from the horseshoe section of the diversion tunnel to the 17 ft. diameter of the spillway shaft. It is reinforced throughout—approximately 260 lbs. of steel per lineal foot, and the lining is progressively thickened to 24 inches. This section, 40 feet long, costing £43/0/0 per foot, was completed far enough up the spillway curve to be clear of the diversion water.

Spillway Shaft:

After the river was diverted into the tunnel it was possible, by using the overhead concrete track, to gain access for the continuation of the shaft excavation. A rise 9 feet x 5 feet was then put up the centre line of the shaft. This had a 4 feet square timbered muck pass fitted with a chute at the bottom discharging into trucks on the overhead track.

Excavation costs for 70 feet of rise were:—

	s.	d.
Labour	20	8
Stores explosives	5	8
„ timber	9	2
„ miscellaneous	7	3
Tool sharpening and repairs	6	8
Plant Charges	25	7
	75	0

When the East quarry operations were far enough advanced the rise was broken into from the top and the balance of the spillway shaft excavation completed using the muck pass in the rise for the disposal of spoil. The excavation was done in approximately 10 foot lengths to fit in with lengths of reinforcing steel and lengths were concreted as the work went down.

The spillway itself (see Figure 10) is 102 feet diameter on top at level 1840 and is shaped in to 17 ft. diameter at level 1776. It turns through 90 degrees at 30 ft. radius at the bottom, into the transition section which changes from the 17 ft. diameter to the horseshoe section of the tunnel in 40 feet.

A temporary bridge was constructed across the top of the spillway and concrete trucked on to here was lowered down to the work. At the crest the spillway concrete is lightly reinforced and is 12 inches thick. The reinforcing is progressively increased to ¾ inch rods placed circumferentially at 12 inches centre and longitudinally at 24 inches centres at the bottom, and the concrete to a thickness of 24 inches.

Above level 1825 the design strength of the concrete was 2500 lbs. and 27 blocks tested at 28 days gave an average of 2600 lbs. with a maximum divergence of plus and minus 13%.

Below level 1825 the design strength of the concrete was 3000 lbs. and 15 blocks gave an average of 3410 lbs. at 28 days with a maximum divergence of plus 9% and minus 12%. For this section of the work the best available sand and selected stone from the quarry was used for the concrete.

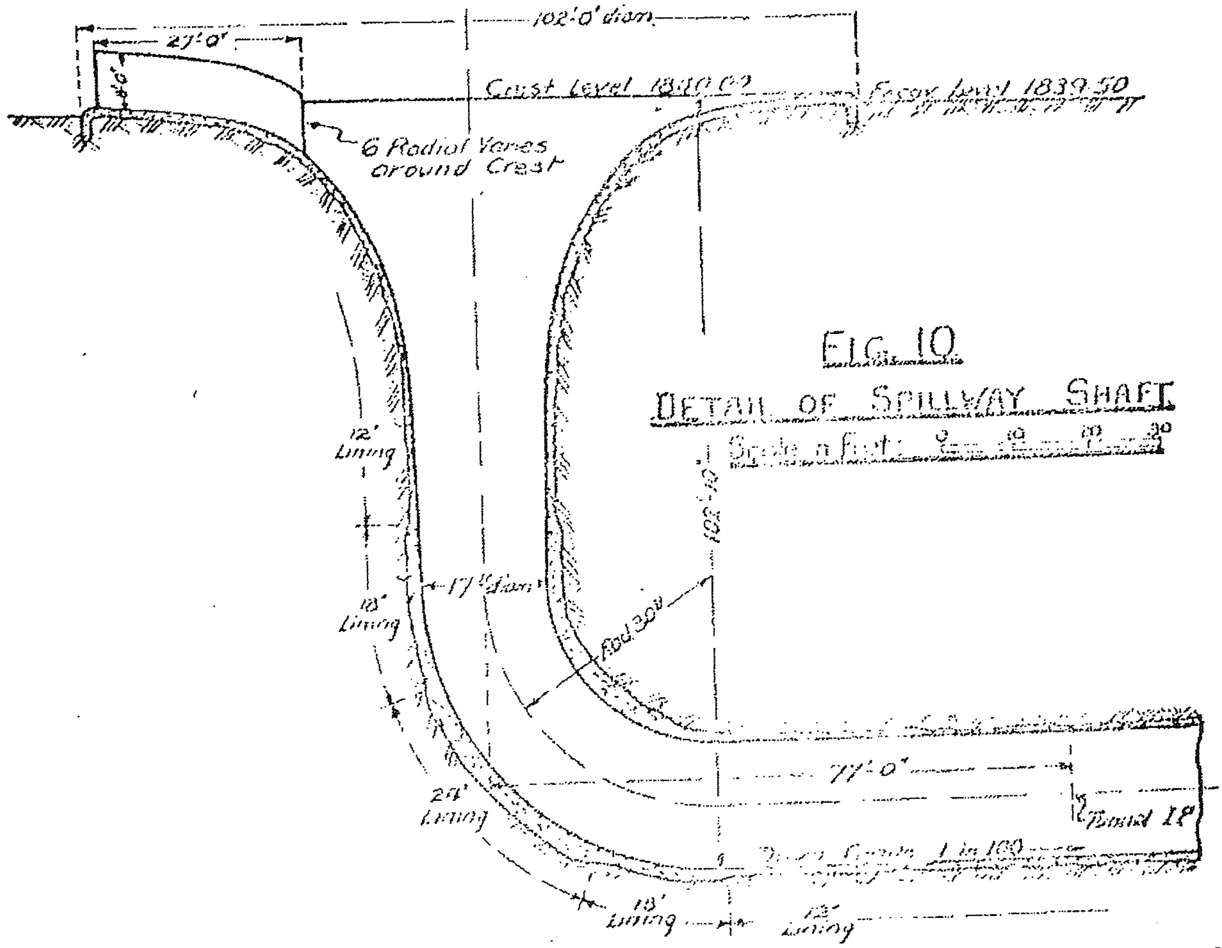


FIG. 10
DETAIL OF SPILLWAY SHAFT

The costs for 2450 cubic yards of shaft excavation were:—

	s.	d.
Cost of rise	2	2
Labour	5	2
Explosives		3
Stores, miscellaneous		11
Tool sharpening, etc.,	1	6
Plant charges	3	5
	13	5

The costs for 875 cubic yards of shaft concrete in place were:—

	s.	d.
Labour boxing	13	11
,, concrete	8	8
,, steel	1	4
,, surfacing	3	2
Stores boxing	10	0
,, steel	5	9
,, miscellaneous	3	10
Cement	29	4
Aggregate	9	6
Water	2	1
Chutes and track	2	7
Plant charges	7	2