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Cofferdams for the Warsak Dam

By

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Synopsis :

Various problems are involved in the design and construction of rockfill and earthen dams. This paper deals with the design criteria and construction methods adoptod on the cofferdams built for the main warsak Dam. While the Upstream and Downstream Cofferdams were required for excavation and concreting of main dam and stilling basin, a third cofferdam was required to protect the power house area from ingress of seepage water and high floods. The objective is to familiarise ourselves with the stability. piping and under seepage problems of cofferdams. The construction of the cofferdams was subject to the limitations (1)of time to stick to construction schedule (2) of strength to stand max-floods and erosion (3) of reducing under-seepage to the minimum to facilitate excavation and concreting (4) and yet be not of a permanent nature, since these were no more required after a year of their construction.

Introduction :

The 35′ dia diversion tunnel constructed on the North bank of the warsak Dam Site was designed to pass a winter discharge of 32000 cusecs with upstream water level 1150′.0 and the cofferdams were required to keep the main dam area dry since the entire river flow was passed into diversion tunnel during winter. The location and axis of the cofferdams were orientated by the position of inlet and exit end of the diversion tunnel and the area covered by main dam and the stilling basin. The river flow conditions in summer were to be such as required the cofferdams to be strong enough to pass and stand the high floods and consequent erosion. The paper deals with the design criteria, and the methods adopted to cope with the various problems such as piping, underseepage and the construction procedure followed to achieve the desired end and stick to the construction schedule which required diversion of the river into the diversion tunnel and completion of cofferdams in a few week. Detailed planning had already been done rega rding the quantities of rock, impervious material and transition materi als and the areas from where these were to be obtained. However, modifications in the specifications and chang of conditions at site required special treatment which are set forth in the paper. This

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will be of interest to all constr uction engineers while a background of the design criteria of this work will be of advantage to those handling similar works in the country.

Design Criteria :

The design of the upstream and downstream and power house coffer. dam envisaged effective measures against piping and check of seepage through its section so that the area between the cofferdams be kept as much dry as possible to facilitate excavatson for the foundation of main dam. In order to prevent piping and keep minimum the seepage flow through the river over burden material and the underlying rock on which the cofferdams were founded, it was considered necessary that means of arresting the foundation seepage flow be explored. The permeability of the river overburden and that of underlying rock was determined to indicate the amount of seepage flow.

Section of the Cofferdams:

When a dam is constructed on other than an impervious foundation, some of the water percolates underneath it and appears on the d/s side. If the velocity or pressure head of this seepage water is sufficient, part of the foundation materials may be carried away. This decreases the resistance to percolation and the resulting increased flow will cause even greater erosion causing piping. Piping or blow out can occur not only in the foundation at d/s of a dam, but also at abutment contacts,and with earth core dams, on the U/s or D/s slope. Failure on u/s slope can occur under rapid draw down condition. An ambankment constructed of heavy clayee material relatively impervious and cohesive may not fail by blow but by sliding due to instability. Upstream protection of dam using paving or rip rap tends to deter blow outs. In the case of Upstream cofferdam at Warsak these features have been achieved, by having an impervious core of clay material and shell of rock fill A & B and gravel. The rock and gravel shells also served as an armour for closing the river and by-passing it into the diversion tunnel. The rockfill forms an effective and stron rip rap protection on the upstream side. Piping failure at abutment contacts and under seepage and arresting of piping at the d/s end will be dealt with later.

The cofferdams are founded on pervious river bed of shingle and sand, underlain by huge boulders and rocks. There could be two suggested designs of dam section.

Out of these two alternatives, it was not possible under the prevailing circumstances to have the impervious key into the overburden, Fig.(1) or the impervious blanket laid as in Fig. (2) above the overburden. The rockfill section and the impervious section as well as the transition section material had to be deposited by the wet method. The rockfill had to be pushed ahead into the running stream by euclids while clay fill had to be deposited in standing water. Details will be found further on.

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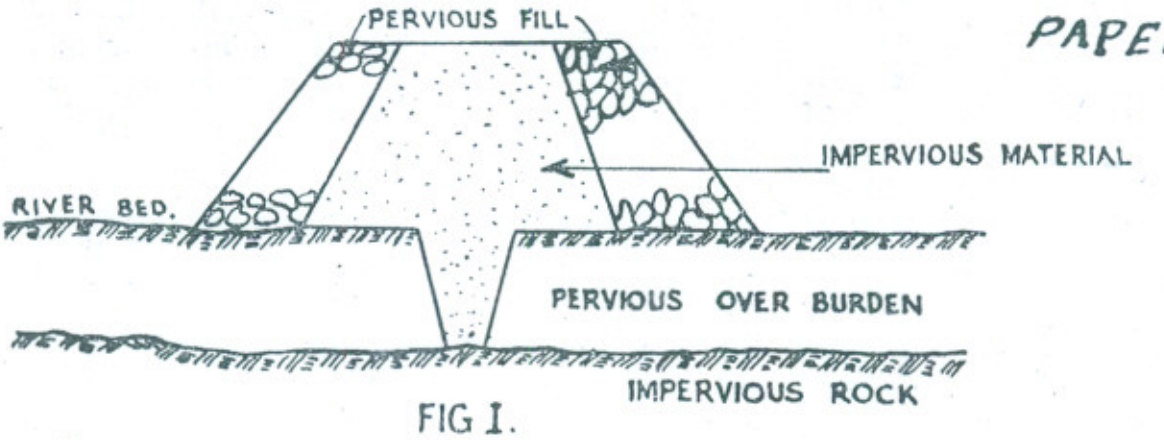


FIG I.

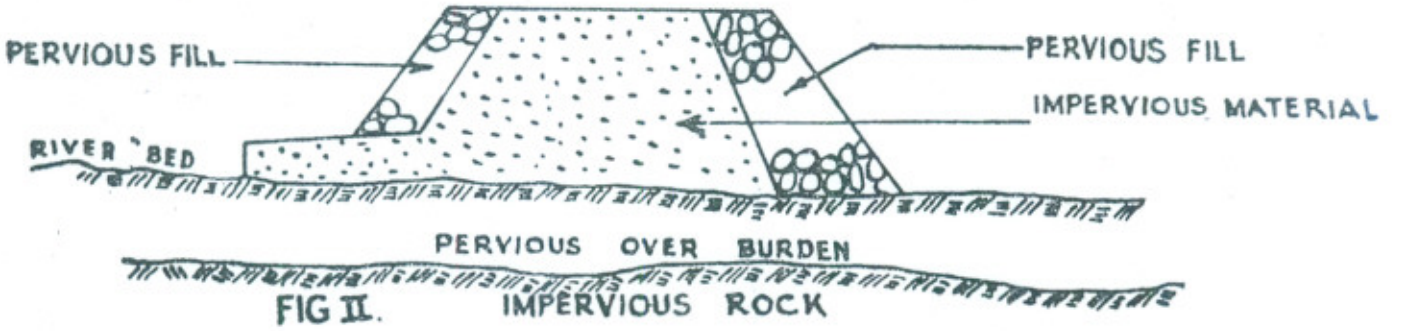


FIG II.

An outer slope of 1 to 2 for rockfill is the usual safe limit and by tests a slope of 1 to 1.4 was considered safe. The width of the core and filter was governed by consideration of head of water, the cofferdam had to stand before overtopping. Plate 1,2 & 3 give the section of cofferdams in different stage of construction.

Downstream Protection Of Upstream Cofferdam:

The original design for downstream protection of upstream cofferdam provided heavy rip rap on the downstream slope (Plate 3).

In order to determine the size of stone for protection of the downstream glacis upto the river bed or below, it was required to know what velocities would be met with. Assuming the sp. weight of rock equai to 165 lb/cft. the dia, in inches was computed from the formula $D = .08\ V^2$.

The theoretical approach in the determination of velocities and stone size was modified in the light of model test. There were of course practical limitation to the size of rock which could be handled with facility.

According to the construction schedules, the excavation of main dam foundation could start only in the end of September 1957 after diverting the river in the diversion tunnel and completing the cofferdams. The time left for concreting of main dam was short so that only the side blocks on North & South abutment of main dam could be concreted in the winter of 1957-58. During the summer season of 1958 the river would overtop the cofferdams

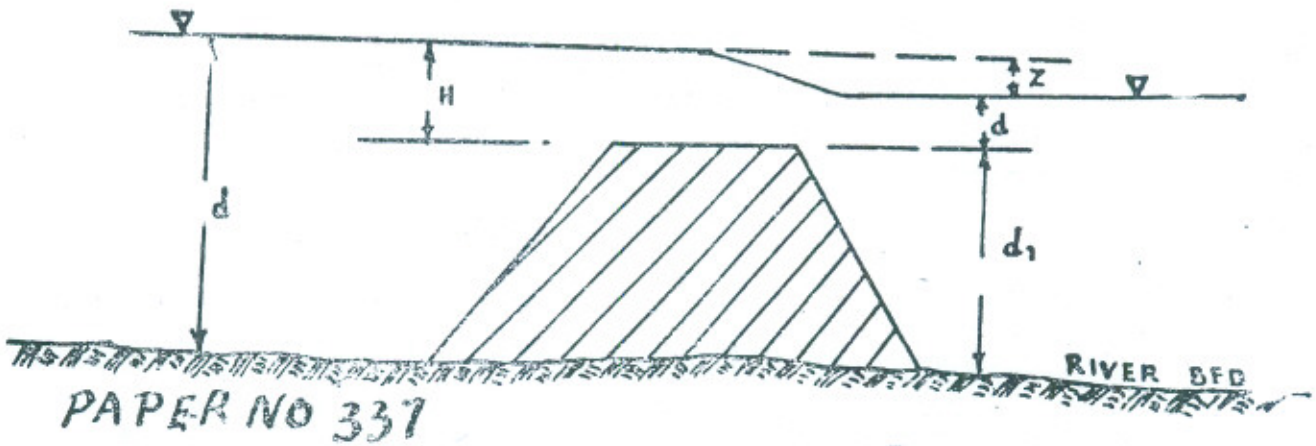
and the central blocks of main dam could be concreted only in the winter of 1958-59 (See plate ¼). The cofferdams had to pass high flood during summer of 1958 and the velocities over the downstream slope of the upstream cofferdam could be reduced depending upon the constriction caused by the side blocks of main dam downstream. But this had its limitation in the capacity of concreting plant and the time available for constriction of the main dam in winter of 1957-58. It was stipulated that blocks 4 to 8 will remain unconcreted dnring winter of 1957-58 and the passage of river through these blocks in the following summer will be sufficiently restricted to cause heading up upstream, thererby reducing the velocities over downstream glacis of upstream cofferdam.

For computing velocities over the upstream cofferdam, discharges of 50,000 ; 100,000 and 150,000 cusecs were considered and the cofferdams treated as broad crested weirs. The losses due to flow through the gaps of central blocks of main dam were calculated assuming conditions identical to the flow around square end.

Following is the available data on which computations were based:

1. Crest of u/s cofferdam El. 1150
2. Tail water rating curve for water elevation.
   D/S of D/S cofferdam.
3. Head water u/s discharge curve for diversion tunnel
4. Longitudinal section of cofferdam and spillways.

The starting point of analysis was the tail water elevation below downstream cofferdam. Since the level u/s of u/s cofferdam depended upon the concrete gap, it became necessary to make an assumption regarding tunnel discharge. For floods under consideration, u/s w. level varied from El. 1160 to El. 1170 and so the tunnel discharge, varied from 35000 to 45,000 cusecs.



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Deducting the tunnel flow from the total discharges of 50,000; 100,000; 150,000; 200,000 cusecs, and the w. level d/s of d/s cofferdam being known, the w. level u/s of d/s cofferdam were computed and tested by trial of arbitrarily selected values of u/s w. level (H. W. L).

$$Q = 3\ 34\ L\ N\ Z^{1\cdot47}\left(1+\cdot56\ \frac{H^2}{d^2}\right) \qquad \text{(Kings formula)}$$

Value of N is obtained from curves = 4·2 in this case.

Flow through concrete gaps :

1. The tail water levels for the case corresponded to the H. W. L. worked out above.

2. Gaps of 100', 150' and 200' were assumed for the above disharges when river width was = 400'

Head losses

(a) Entrance loss
(b) Exit loss
(c) Minor friction loss

$$= K\ \frac{V^2}{2G}$$

Where $K = \frac{\beta^2}{(1-\beta)^2}\left\{\frac{(\alpha-1)^2}{1-\alpha\beta}+\phi\right\}$ and β = Co-eff : of obstruction = Width of Obstruction / Width of river

and $\alpha\beta = 1-\mu(1-\beta)$.

μ = 0·7 for blunt nosed piers.

ϕ = 1 for blunt d/s nose

ϕ = ·5 for sharp d/s nose.

for a gap of 200' head, losses were $= 1\cdot735\frac{V^2}{2g}$

Thus T. W. L of main dam below gap = H. W. L. -- Head loss of main dam. H. W. L. means the water level u/s of the structure.

Having found H. W. L. of main dam, the H. W. L. of u/s cofferdam was determined in the same way as for the d/s cofferdam.

A maximum discharge of 200,00 cusecs is considered for calculations:

Q = 200,000 ———155,000 cusecs
(deducting 45000 cusecs which is passing through the tunnel)

(i) for D/S cofferdam C.L. = 1132.0 H. W. L. = 1157/1158

(ii) For main dam :

From gauge discharge tables :

H. W. L. 1160 .....T.W.L. = 1155·5

H. W. L. 1165..... T. W. L. = 1161·2

∴ H.W.L. 1162 is for T. W. L. = $\frac{1157}{1158}$

(iii) for U/Stream cofferdrm :

T.W.L. = 1160
H.W.L. = 1175.

∴ for T. W. L. of elevation 1162, the H.W.L. is 1175·5′. Width of water way = 426 ft. (Upstream cofferdam) Net discharge over cofferdam = 155,000 cusecs Discharge /ft. = 368 cusecs.

C. L. of u/s cofferdam = 1150·0′

∴ Depth = 1162 – 1150 = 12″ of water

∴ Velocity over the cofferdam = $\frac{368}{12}$ 30 or 31 ft/sec.

The size of stone for downstream protection as determined above was however subjected to a change in the light of revised design for protection of crest and d/s glacis. The original proposal is shown in plate 3, stage III of upstream cofferdam.

The loose rip rap provided on downstream for protection failed under model tests and therefore, stone masonry covering as given in plate 2 was provided. The downstream horizontal apron (wave wall) and the provision of a sill at the lower end of this apron were understood to cause a reverse roller thereby reducing the scouring action and thus contributing to the stability of the cofferdam. This was found to stand a discharge of 200,000 under model tests.

In the final design, 6·0′ masonry protection on the crest El. 1150 and D/s slope upto El. 1114·0′ was adopted as an effective check against blow out, while at the same time it served to pass a maximum flow of 2,00000 cusecs without eroding the crest or the D/S slope. 8″ dia weep holes 8·0′ centre to centre were to be left in the d/s slope of the masonry.

FILTERS :

Protection against internal erosion from clay core into the rock shells of the cofferdam was secured by placing rigidly graded filter leyers in the transition zone between clay core and rockfill shell on the d/s side.

Thus the fine particles of zone 'E' (Plate 2) are prevented from being washed into the rockfill 'A'· Also the impervious material was expected to be ffuid at the time of placement and would not have supported

construction equipment unless adequately drained. This could be drained by dumping river overburden material from the top of the rockfill 'B' so that impervious material were squeezed out and its surface rise above the probable max. elevation of the u/s water surface during winter, El. 1130. This also required more dumping of impervious material E to raise it above water level to be allowed to drain and air dry. Thereafter the surface of zone E was to be trimmed and allowed to air dry before placing the protective filter and rip rap material (Stage III) Section AA.

The filter material for Warsak Cofferdam determined by Consultants was to be such that the ratio of grain size of filter and that of the base material should be :

(I) $$\frac{D_{15}\ \text{Filter}}{D_{15}\ \text{base}} = 4$$

(II) $$\frac{D_{15}\ \text{Filter}}{D_{15}\ \text{base}} = 20$$

D_{15} means 15 percent grain size of the filter and so forth.

(III) $$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{bare}} = 25$$

The sand and gravel from borrow pit area "A" satisfied the requirement for one of the filters and was to be used for the transition material 'D'.

The next filter, *i.e.*, transition material 'C' was gravel from borrow pit 'B'

Under Seepage & Tubewells :

The treatment of under seepage on u/s cofferdam was attempted on theoretical grounds. The quantity of under seepage was computed by flow net studies based on electric analogy methods. The various flow nets were based on a combination of permeabilities of the clay core aporn, overburden, bed rock and the effect of intercepting wells at d/s toe of cofferdam.

K of overburden based on actul field test was found to be 2×10^{-2} ft/minute (9g p.m).

The formula for determining Q = flow per ft. of dam was $= \frac{K \quad H \quad n_2}{n_1}$ Where K = coeff of permeability

H = net head

and n_1 = number of increment of head loss n_2 is the number of flow channels.

It was estimated that the actual capacity of the tubewells for pumping seepage, assuming an impervious aporn would be 280 gallons Hr/ft. Width i. e., 11,200 gallons/hr. or 187 g.p.m. per well with 40′ spacing but not as high as 36,000 gal/hr. or 600 g.p.m. per well with 40 spacing, when K of apron is assumed as high as that of over overburden. If both the rock and apron were impervious the flow of well was estimated to be 108 gal/hr/ft width or 72 g.p.m. per well with 40′ spacing.

Eight tubewells for pumping 25000 g.p.m. were, therefore, to be installed 40′ centre to centre on the d/s apron with the following specification.

22″ dia drilled hole

blind pipe dia=12″.

Screen 12″ I/D and length = 20′+15′. 39 slots were to be perforated in the pipe, slot size being $2'' \times \frac{3}{16}''$ spaced evenly around the pipe and alternate rows staggered. Coarse shrouding material 1/12″ to 4/5″ was to envelope the perforated pipe. The drilled hole for the screen setting was to be 16″ I.D. Brass screen with 5 slots were to be used when overburden was fine.

The pumps to be installed, were WARRICK MODEL Du2, liquid level control, double pole, electrode type to sound alarm on low water level and for shut down of pump on low water level to ensure safety of pump during established operation conditions.

Airline connector for draw down were to be provided. Byron Jackson submersible pumps I5 H.P. 400/3/60 type, 4 stages had been ordered for installation. The pumping unit was to be capable of pumping 25000 g.p.m. to El. 1150 for continuous operation. Discharge control valves at the end of rising mains were to control the discharge from pumps to prevent water level in any well falling below the pump intake.

Bottom of the tubewells was to end in rock and on grouting was to be done with 10, of bottom of tubewells. The grout curtain was to be injected below the bottom of tubewell to El. I100 to cut off all seepage from the rock into the main dam area where excavation was to be done to El. 1050 or above.

Grout Curtain Line along the line of Tubewells to arrest underseepage through rock:

For this purpose, five 22″ dia. percussion exploratory borings were made by Swissboring Corporation in the area of the river covered by the proposed dam site. The borings revealed composition of alluvium consisting of fine sand embeded with shingles and boulders (5″ to 19″ size) forming 40% of the alluvium. There was no continuity of grain size in

the material and gravel was nominal. The permeability was determined by the falling head method and ranged from. ·004 cm/sec. to ·022 cm/sec. Considering an average coefficient of permeability as. ·01 cm/sec. draining the alluvium seepage flow by pnmping from tubewells was feasible, but expected to be ineffective in point of steepness of the draw down curves and time involved. The subgrade condition excluded the possibility of:

(1) A sheet pile cut off due to presence of boulders.

(2) A grout curtain because of the too small size of sand.

(3) A chemical grout curtain for the high cost and uncertain results.

(4) A clay cut off because of the impracticability to dig a trench through alluvium down to the bed rock.

It was therefore decided that the seepage through the overburden material be picked up by tubewells, while a grout curtain be placed at the bottom of tube wells into the rock to seal off all seepage through the rock.

The Swissboring Corp who was given a sub-contract of this work was issued the following instructions.

(*i*) Wells were to be drilled from floating platform during the winter season (1956-57)

(*ii*) 22″ casing was to be used for drilling, telescoping it to 16″ casing whenever the thickness of alluvain made it ncssary.

(iii) 16″ whole casing was to be used for 10′ (later on increased to 16′) below rock.

(*iv*) The tubewells were to be developed with air lift.

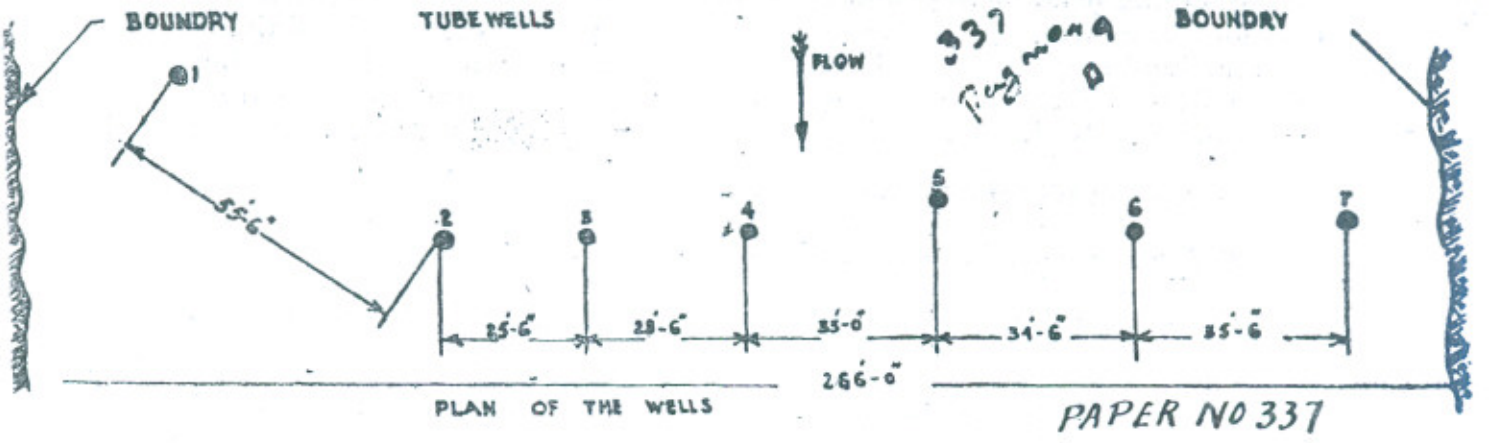
(*v*) The top section of the blind pipes protruding from the river beds was to be covered by a cap, this being done by a diver.

(*vi*) Prior to tem (*v*) the grout curtain was to be completed as explained later on.

(*vii*) Pumping equipment was to be installed after the construction of cofferdams and dewatering of areas within these. This was to be preceeded by the development of tubewells.

The row of the tube wells is situated at approximately 35′ d/s of the toe of the u/s cofferdam spillway and general plan is as shown above. The depth of the tube well number one to seven is 76′, 93′, 73′ 64′, 57′, 52′ and 46′ respectively and the top of all the tube wells is at EL- 1114, which is above the normal river bed level at this site.

Tubewells.



The top of the wells was under water before the diversion of river took place and were properly sealed by means of 10′ long extension pipes by providing reinforced concrete around the casing to protect if from the rolling stone durieg the construction of u/s cofferdam.

In order to cover and protect the top of the tubewells the 22″ casing upto 10′ from river bed was drawn above and a steel reinforcement 10′ long (5 rods 3/4″ dia. bound with 3/8″ bars) was placed around 12″ blind pipes. Annular space between 22″ casing and 12″ pipe was filled by pouring concrets on top of pipes by a special bucket for below-water-concreting. The casing was withdrawn before conrete setting.

Constraction of tubewells.

Dilling through alluvium.

Drilling was started with 22″ casing using percussion rigs in May, 57. The sinking of the casing was done according to the nature of the strata by driving the casing into the soil and bailing or vice versa.

A chisel was used whenever it was necessary to break apart layer of cobbles or, more frequently to chop boulders to small size.

Whenever the size of the boulder was too big to be broken by a chisel, blasting had to be done, by prilling blast holes through the boulders and filling them with strings of forcite.

Drilling through rock.

A hole 16″ dia 16′ long was drilled into the bed rock. An attempt was made to drill this hole with chisel only but due to 'the hardness of the granite rock, the progress was very slow. This was overcome by drilling three holes of 45mm dia. and 16′ long and blasting it to break the rock befor commencing drilling. After the blast the rig was removed and hole was drilled with a 16″ chisel, the cuttings being removed with the bailer.

This 16′ depth of hole was not to be grouted and all grouting commenced further below.

Installation of Straincr.

After completion of drilling grout holes, and their grouting, strainers were lowered in the drill holes. Strainers and blind pipes were to be supplied from Canada but owing to delay, these were supplied locally :—

(*i*) Blind pipe :—

12″ dia. M.S. pipe, 1/4″ thick against 3/8″. specified by Acres.

(*ii*) Strainers :—

12′ dia. Brass strainer, $\frac{1}{4}$″ thick slot size 24/1000″

against 30/1000 specified by Acres.

12″ perforated pipe slot 3/16″×2″.

Slots made by hand.

(*iii*) Filter material :—

$\frac{1}{4}$″ to 1$\frac{1}{2}$″ gravel between slotted pipe and rock.

Finer gravel for Brass strainers.

The starta variation from top to bottom was approximately the same in all the holes and the beds of varying thicknoss were generaly in the following or der :—

(1) *Pebbles upto* 4″.

(2) *Sand shingle cobbles and angular boulder upto I9″ of various rocks-*

(3) Sand. cobbles and rounded and angular boulders upto 13′ of various rocks.

(4) Sand, cobbles and rounded boulders upto 12′.

(5) Coarse sand with angular pieces upto 4′.

(6) Mica clay, golden lustre with coarse sand (1′).

(7) Clay and pieces of granite gneiss upto 3′.

(8) Sand coarse to fine gravel angular piece upto 3″.

(9) Angular piece of various rouk upto 3′ or 4″.

(10) Graite gneiss.

The preliminary investigations of permeability and exploratary drilling were done along the river aixs and missed to supply information about alluvim on the south side river. For this reason, relocation of tubewells near the south bank was necessary.

18″ casing were used in place of 22″ casing to have enough shrouding when reduction of well dia was necessary.

Drilling grout holes,

After the completion of drilling into alluvium and before the lowering of strainer, a Svenska drill rig was inst-alled and 3″ casing was lowered down to the bottom to serve as a stand pipe. The drilling was excuted with 65 mm steel bits for some feet, whereafter 65 mm casings were placed and drilling carried on with 45 mm diamond bits, to the required depth. Water testing and grouting was carried out from the bottom to the top in 10′ stages.

Grouting involved the following operations.

1. Washing the hole with air and water.

2. Water testing the holes at the pressure indicated in the following Para.

3. Grouting, if necessary, with mixes of w/c using the following standard mixes to be progressively thickhened or diluted according to the pressure built up.

1 bag	to	130 liters	w/c	2· 6 : 1	by weight.
,,	100	,,	,,	2 : 1	,,
,,	75	,,	,,	1·5 : 1	,,
,,	100	,,	,,	1 : 1	
,,	80	,,	,,	0·8 : 1	
,,	60	,,	,,	0·6 : I	

112 lbs. being the weight of a bag or 506 kgms.

Additives such as sawdust or fine sand were to be added to the thickest mix in case of too big obsorptions.

Testing the holes to refusal and back pressures by checking that no more grout flowed into the hole applying the max. permissible pressuse with the thinnest mix (1 bag to 130 liters or 2·6 : 1) for 10 minutes, returning the grout to thc sump through a return line, the latter by stopping the pump and closing the lines and checking that there was no flow of grout from the rock into the holes, this being indicated by pressure shown on the manometer.

Grouting curtain (upstream cofferdam).

10 holes had been drilled, but only 7 holes were grouted. Each hole was water tested and grouted, from the bottom to the top, in stage of 10′. Some holes were water tested and grouted in 5′ stages. The pressures for the water test were 48 to 84 p. s. i. whilst the grouting pressure were 2. p. s. i. per foot.

Grouting was executed in eaeh stage, when the water obsorption exceeded 0·04 gals/foot/ minute. According to the specification, solpenetrit was added to the first 20 bags (1% in weight of the mix) in order to have an initial wide range of penetration, thus compensating the big spacing of the holes.

On the total footage of 3 98′, 1240 bags of (cement) were grouted with a min ; absorption of 0·17 bag/ft. and max. absorption of 12 bags/foot.

The average absorption was 3.12 bags/ft.

Grouting was carried out from a grouting station composed of Duplex Pump, mixer and sump, installed on the south bank. The grout and return lines were laid across the river, hung up from a cable fastened to the pontoons floating on river surface.

The grout curtain, originally foreseen to be executed only below the tube wells was later extended with six holes and five holes on the south and north of the row of tube well respectively.

U/S Cofferdam (Extension of the grout eurtain) Sealing of abutments.

The grout curtain under the tube wells was extended towards north andsouth abutment of upstream cofferdam by grouting down to El. 1000 to safegurad against seepage on the flanks. The total length of grouting for these holes was 1307 ft. and it consumed 1951 bags of cement and additive, thus giving an absorption of 1.49 bags/ft. On the south end. only rotary drilling was adopt but on the north side about 20% length of the holes were pressure drilled and remaining 80% rotary drilled.

The original construction schedule anticipated construction of the Coffer Dam by the autumn of 1956, by supposing that the Diversion tunnel would be completed by september, 56. For certain reason, this sehedule however' could not be adhered to. It is to be noted that he Diversion tunnel was oompleted towards the end of August, 57 and it was only after its completion that the construction of the cofferdam could be started. The construction schedule for the Warsak Main Dam was spread over two years and the intention was to complete the North and South sections of the main dam in the winter season, following the completion of diversion tunnel so that during the next flood season, the main dam would restrict the water, velocities over u/s cofferdam would be less, there being a high back water for the sefety of the cofferdam. If adequate quantities of concrete could not be poured in the main dam, the restriction was to be less and the velocities considerably much higher than what the cofferdam would stand.

Coustruction of rock shell (rockfill) section of the cofferdrm had to be done while there was enough flow in the river which had to be

diverted to pass entirely through the diversion tunnel. For this reason the rockfill material on the u/s and d/s of the u/s cofferdam had to be heavy enough to be heavy enough to resist, the strong current of water, specially in the last closing section of the river. The rock fill section had to be capable of passing Euclids and Tractors and so the top width of the cofferdam was kept 18 feet. The side slops being 1:4 to 1 the bottom width was 133 ft and 88 ft. Respectively for the u/s and d/s rockfill section of the u/s cofferdam. The rockfill for the cofferdam was obtained from the excavation which had already been done in the diversion tunnel and surrounding areas.

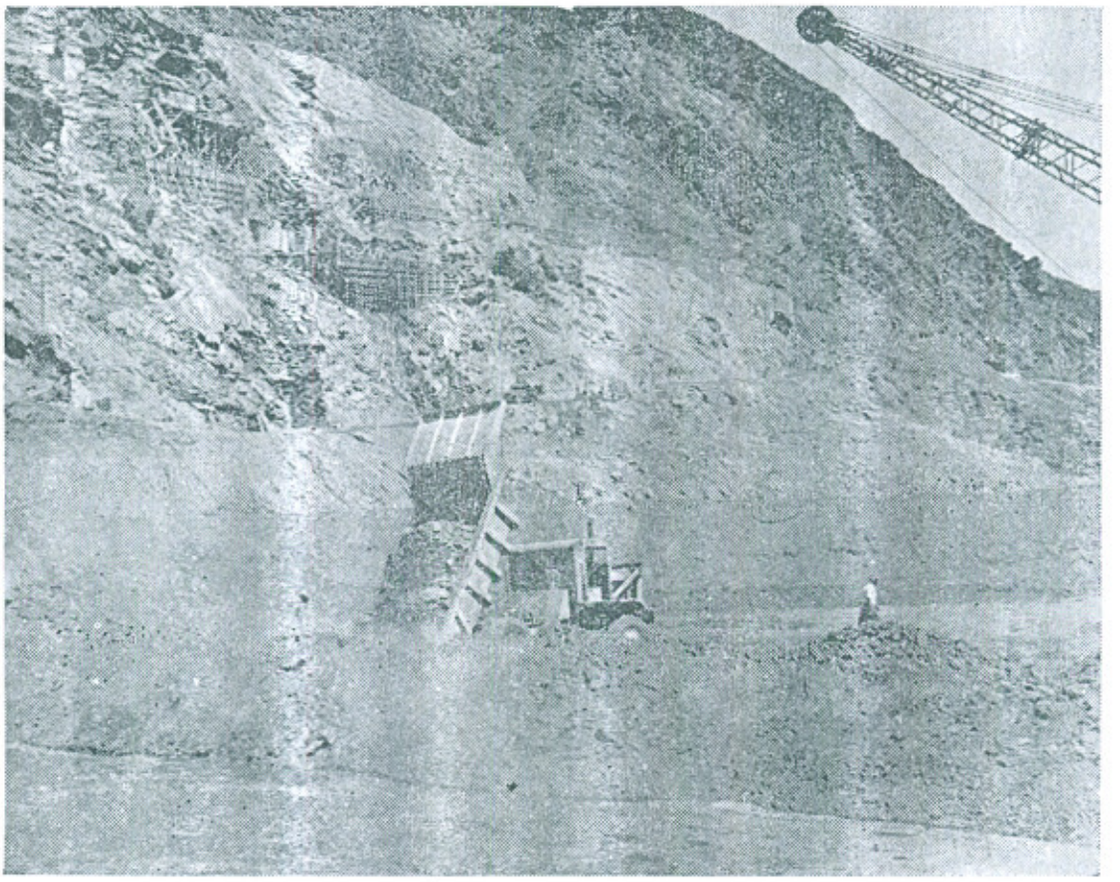
The rockfill A was end tipped from El. 1150 of south bank and was closed first after the diversion tunnel was put in operation in August 57. The rockfill B was end tipped from El. 1144/1150 during construction and the tipping face was kept sufficiently behind that of rockfill sections A. Rockfill A+B had to stand the strong current of water without the impervious fill and were considered sufficiently stable against sliding failure. Sections of Rockfill A was trimmed to El. 1135 when water level fell down. This was followed by dumping of filter layers C & D to an approx slope of 1.71 to 1 from rockfill A. Having dumped the filter material, clay from shaghai pit was brought in Euclids and end tipped from Rockfill A and then rolled to a slope of 7 to 1. This marked end of stage I. River overburden and mostly transition material similar to that of transition material D was transported from aggregate area B, and dumped from Rockfill B, to squeeze out wet clay and drain it later on. More clay was then dumped in the impervious core zone to mark the completion of stage II. Rolling and eompaction of clay was done by tractors, bulldozers and euclids as they moved in and over of the Cofferdam Area.

Development of Tubewells.

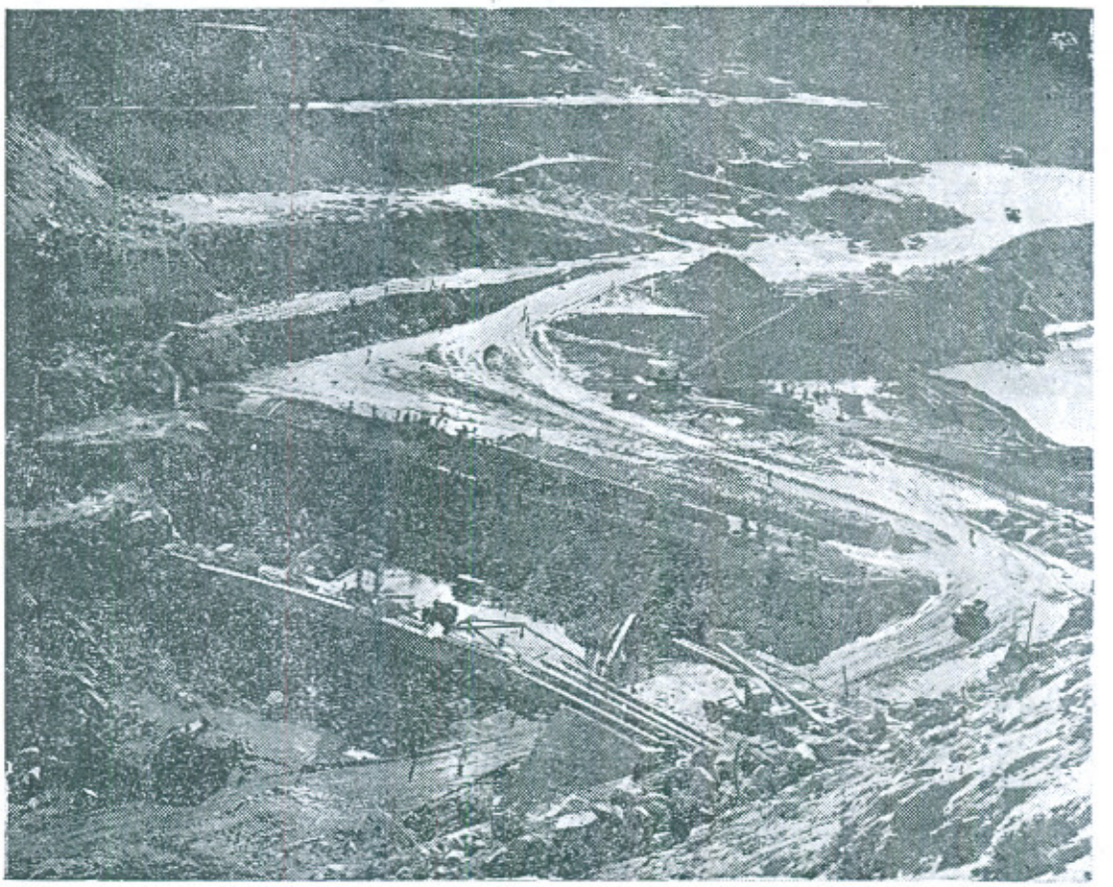
With the completion of stage II of u/s cofferdam the dewatering of main dam area was taken up since by then the d/s cofferdam had also been completed in stage II. After dewatering of the dam area by centrifugal pumps, the caps of tube wells were removed and 2′ pipes added to extend the wells to El. 1114′ of concrete sill.

Air lift consisted of 6″ eductor pipe with $1\frac{1}{2}$″ air pipe placed inside the former with outlet at 1.0′ from 6″ pipe. Compressor capacity was 220 cft/min. A surge block was also used, moving it up and down to remove bridging of sand, but its use was kept restricted. Discharge of two tubewells after developing varied from 23 g.p.m. (CT6A) (Draw down = 23′ – 9″) to 6.7 g.p.m. (draw down = 47′ – 9″ on CT3A).

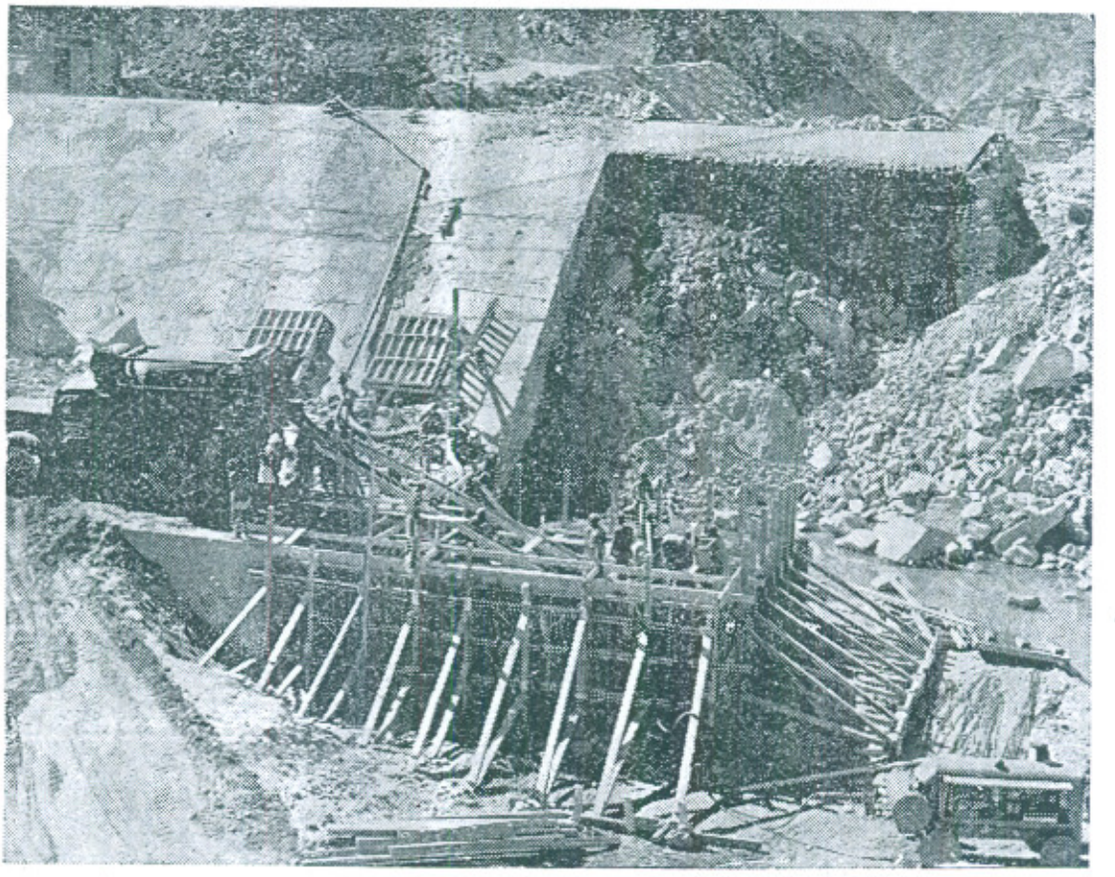
Low discharge of tubewells was due to low permeability confirmed by insignificant seepage of water into excavated area, and this was easily drained by other means. So the development of remaining tubewells was given up and no pumping equipment was installed on any tube wells.



1. Upstream cofferdam impervious fill in progress



2. Upstream cofferdam showing glacis and wave wall



3. Upstream cofferdam glacis concreting

Drainage of main dam area was carried out by collecting seepage water in 2 ponds at different elevation and pumping it by electric and diesel C.F. pumps.

Backfilling of the impervious clay material was completed on the upstream cofferdam during thc lst week of October, 1957. A leak on the north end of the u/s cofferdam was discoverd as dewatering continued an attempt to close the leak by dumping clay and transition material along the u/s side of the cofferdam proved unsuccessful. A 'Lima, dragline was next used to dig a trench along the centre of the clayfill, and it was then discoverd that the leak was beneath the clayfill in an area of heavy rock debris along the northern edge of the river which had been sidecast from the access road to the diversion tunnel intake. During the dredging operations before the clayfill was placed this had been taken to be the river bed. The rock debris were removed and some clay and transition material were dumped into the hole, but the flow of water was too rapid and the clay was washed straight through· Cinders would probably have served the purpose, but these being unobtainable locally, horse manure was tried. While these operations were in progress the river level rose to an elevation 1137.5 on the morning of the 22nd October, 57 and flooded the hole. It seemed that the horse manure had been partially effective as the leakage had been restricted to about half of its original volume. It was then proposed to clean the hole again and try more horse manure, but this·was not completely effective, and the hole was backfilled with clay and a line of sheet piles driven as a cut off 70′ long and about 25′ deep.

After driving the sheet pile, clayfill was rolled along side by tractors. The sheet piles reduced the percolation by about 50%. However, since the tubewells did not function properly and were given up as only a small leakage was taking place through the overburden, therefore, resort was had to pumping from a sump d/s of the cofferdam to catch the leakage through the impervious fill. On 15.3.58. a 10 cusecs capacity pump installed on the u/s face of main dam was discharging only 4 cusecs across the u/s cofferdam.

The u/s cofferdam had been dressed off ready for the masorny surfacing, but on setting out the lines of concrete wave wall, it was found that the north end of rock fill 'A' was 50′ u/s of the correct position. This was rectified by backfilling along the d/s side of rockfill 'A' to attain the correct profile.

A trench along the top of cofferdam about 10′ wide and 6·0′ deep was excavated and spawl concrete was placed in the trench to secure a good key in the cofferdam and for serving as foundation for the 6·0′ thick stone masonry.

Masonry facing on the u/s side of Rockfill 'A' was started by the Warsak Dam project Organization by hand packing stones (size, not less

than 1′×9″×9″) generally the larger dimension being kept verticial, grouted with cement and slurry (1;6) brought on mixer truck from temporary batching plant. Profiles at 40' were laid and layers of stone masonry colcreted as described above. The stones for this ston masonry had been broken to size or collected from blasted debris. These were dumped u/s of the power tunnel inlet or on the right bank of Kabul River d/s of Bailey Bridge well in advance of the work Towards the end of Janaury 58 all the masonry work had been completed by Warsak Dam project Organization except for approx 75′ on the north end, which was being used for an access road, and was later on completed by Angue Robertson Limited.

The construction of reinforced concrete sill El. 1114 and 1118 was started simultaneously with the laying of 6·0′ thick masonry on top of the cofferdam at Elevation 1150 (see photo and drawing not 5). Timber formwork was errected and steel reinforcement laid. This was followed by concreting from buckets lifted to the location by crane or by chuting concrete from mixer trucks in blocks of 33·0′ length leaving a construction joint between each block- The wells after development having been found to yield little seepage flow, their tops were capped and set in the concrete sill.

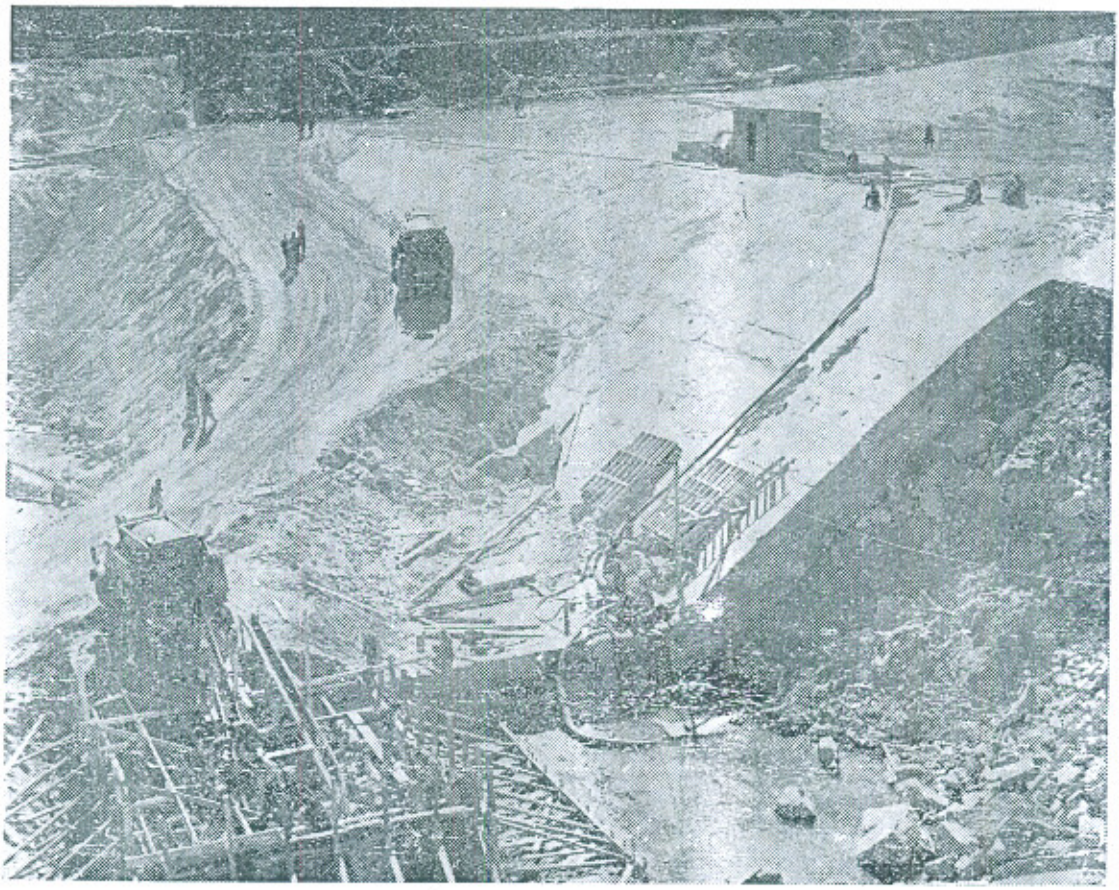
In the middle of March 58, the light overburden d/s of the glacis u/s cofferdam was removed and heavy rip rap and boulders were hand dumped to stand against turbulance and scour.

By the 3rd week of March 58, another precaution taken was to drill 1½″ dia, 2′–6″ c/c relief holes in the 6·0′ masonry blanket on the top and the curved portion of the u/s cofferdam. These holes were only 7·0′ or so deep and were to act as relief pipes against uplift. This uplift was expected on account of the vaccum created by the nappe on the crest d/s glacis and eonsquent differential head. Weep hole pipes had already been provided in the glacis.

Earlier in Deaember 1957' grouting curtain on the north bonk flank of u/s cofferdam was completed by swissboring Corporation and grout curtain on the south bank was completed in November 57, to seal off water seeping through the flanks of the downstream cofferdam simultaneously.

Downstroam Cofferdam.

Plate 6 gives the plan and sections of stage I of the downstream cofferdam. The skew angle of coffer dam, its top elevation and its position downstream of the main dam have been determined keeping in view the excavation area limit of stilling basin, the flood conditions when the u/s and d/s coffer dam would be over topped and the location of exit and of the diervsion tunnel. The top was kept low enough to maintain a low water level during floods in the Stilling Basin area so that the coffer dam of the Power House would not be subjected to a high and consequent greater seepage into the power house area.



4. Upstream cofferdam showing glacis. Concreting of the d/stream wave wall is in progress

The top of the coffer dam protecting the power house was El. 115 (later on raised to 1160) while the top of the d/s cofferdam is E. 1132. Any further rising of d/s cofferdam above El. 1132 would have caused excessive heading up of the flow during the over topping period.

The Warsak Dam Project Organization had been of the view that the d/s & u/s cofferdam were not designed sufficiently strong to stand the high flood of 2,000,00 cusecs. Calculations have been made (appendix I) which show that the hydraulic jump d/s of cofferdams would be formed just at the end of the stone protection when passing flood above 100000 cusecs but nothing was povided to resist the distribance beyond this point. A protection apron 5 times the depth of the stream beyond this point with pacca floor and in addition a flexible apron of a similar length, was suggested d/s ef the u/s and d/s cofferdam (plate 3 A, 38). Alternatively protection on the downstream slope should go right down to the rock as turbulance and hydraulic jump d/s of the cofferdam were estimated to undermine the d/s glacis. It was however understood that the reconstruction of the d/s cofferdam if it failed, would not have taken more than 2—3 weeks. In view of this it was conceded that the d/s cofferdam may be left as designed by Consultants but that the.u/s cofferdam needed more protection.

It is to be noted that the basis for the u/s cofferdam designed was the choice of a limiting floods of 2,00,000 cusecs. This was based on a chance of 1 to 20 keeping in view the 50 years' record for the Kabul River. In other contries the corresponding over topping flood would have been taken at 1 in 8. This will show that actually the coffer dam may not have to stand as high.a flood as 2,00,000 cusecs during the construction period. Also the calculation made by WDPO took into account the discharge per ft width over cofferd am taking cofferdam length projected per-pendicular to the River. This was a canservative approach in calculating the position of the hydraulic jump downstream. The Consultants had carried out model test in which the discharge per ft of actual cofferdam length had been taken into account, and on this basis, they considered their design of the u/s cofferdam safe.

The Consultants were of the view that if the drowing ratio over an embankment were as high as that of fiow over the d/s cofferdam, a prper hydraulic jump would not have occured but rather than an undular type of jump which deflected the high velocity water to the surface with the result that a reverse roller occured under it immediately downstream of the cofferdam. It was therefore anticipated that under such condition the major erosion of the stream bed would occur a cosiderable distanee from the cofferdam and the eroded material would be transported back by the reverse roller to buttress the cofferdam foundations. The model test at Niagra Fills also conflirmed this flow pattern for all discharges from 50,000 to 2,00, 000 cusecs.

The Consultants agreed with the calculation and the protection recommended for the u/s cofferdam by the WDPO, but were of the view that the scour depth could be decreased by the provision of a sill at the end of the

shorter horizontal apron E I. 1114-o. In the view of consultance, this would help in the formation of a reverse roller and the rock fill protection to the excavated slope was sufficient to prevent erosion. The Consultants also thought also thought that the proposal of the W.D. Organisation to continue the masonary topping down the excavated slope to the rock level would not be acceptable since it was essential that the foundation material below the u/s cofferdam should continue level, 120 feet downstream of its centre line. If it did not continue for this distance, the entire stability of the cofferdam would have been endangered.

It was felt that the design of the upstream and downstream cofferdams should be tested on a model of these cofferdams and the irrigation research institute was asked to run model test keeping the above factors in view.

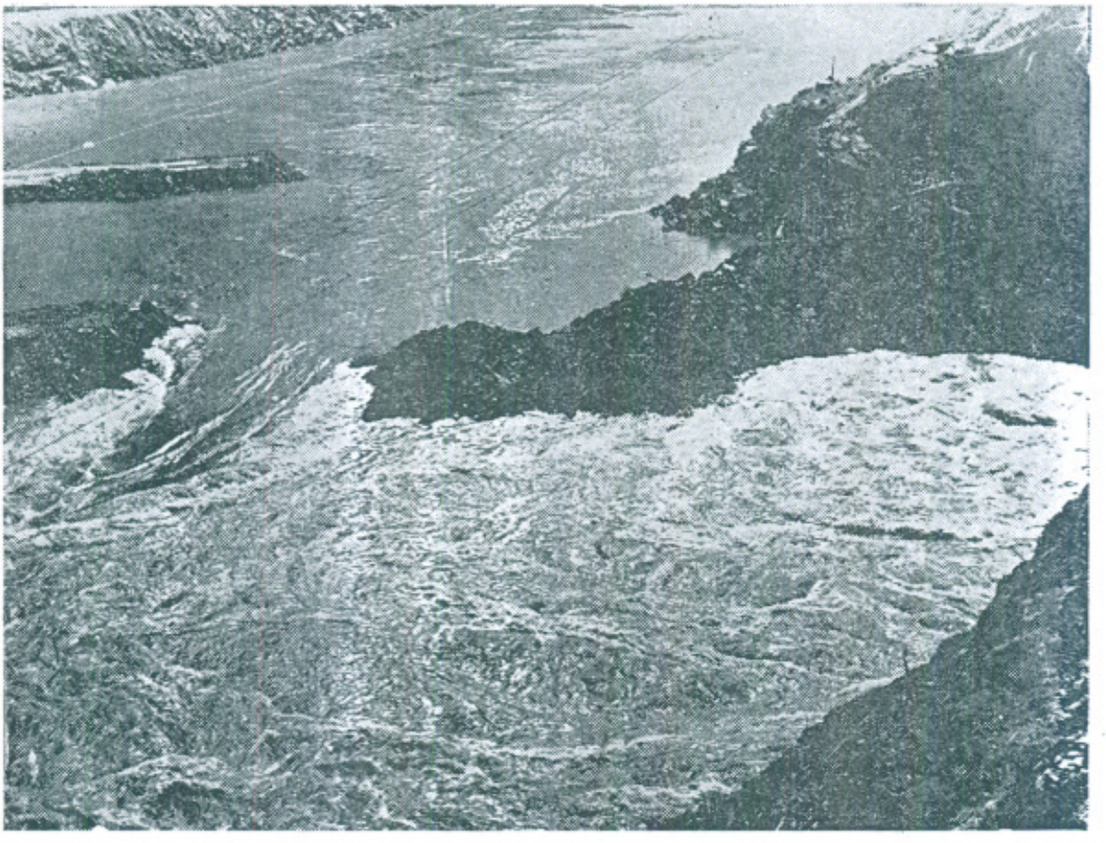
The model tests were run at the Nandi Power field station with a flood discharge of 50000, 100. 000, 150, 000 and 200,000 cusecs.

The model tests conducted by H.G. Acres in their laboratory had the cofferdam placed at right angle to the river flow and the overflow over the D/s cofferdam was hot influenced by the strong jet coming through the exit end of diversion tunnel. These features were improved upon in the model tests conducted at Nandipur by the Irrigation Research Institute Lahore, so that the cofferdams were placed at the designed skew angle and the jet from the diversion tunnel gave true flow conditions and scour effect while testing the cofferdam for a maximum flow of 200,000 cusecs. The results of the model tests conducted by Research Institute, Lahore (The report 190/1140/1958) indicated that the safety of cofferdam was endangered due to heavy scour when discharge above one lakh cusecs passed over the cofferdams, and the rock fill d/s both the u/s & d/s cofferdam disintegrate.

The final construction of the cofferdams having been completed earlier, it was not considered feasible by consultants to introduce any improvement in the design for safety of cofferdams.

Construction.

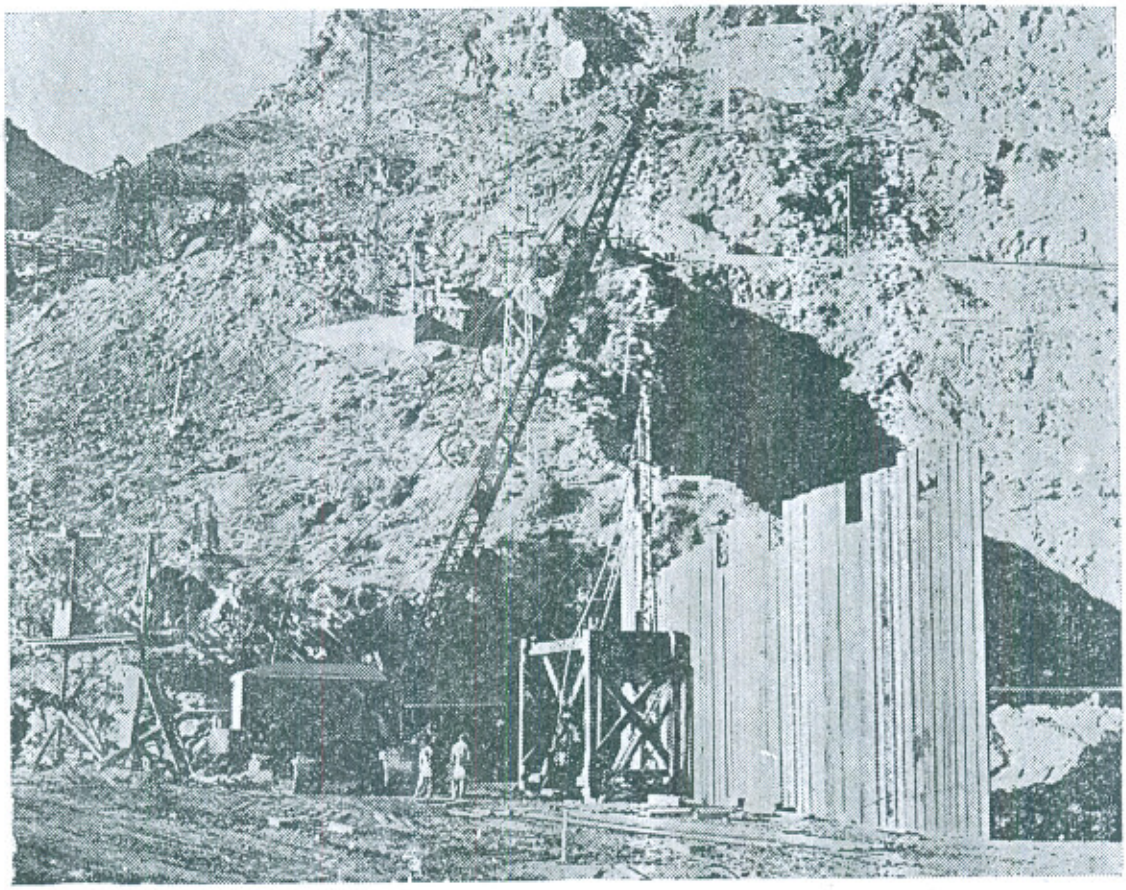
The construction of the Downstream cofferdam is very similar to that of the Upstream cofferdam discussed abovc. The rock-fill 'Q' was end tipped from elevation 1132 and completed after the Diversson tunnel was put into opration. Its advance was kept sufficiently behind rock fill 'P' ensure that rock pieces were not thrown over the area on which seal was to be made by impervious fill There were also transition filter layers of the same design as for the u/s cofferdam. On the u/s cofferddm, only one transition filter was provided between the impervious fill and the d/s rock shell, so that the clay particles did not have an easy passage d/s when the water headed up on the u/s side. In the case of d/s cofferddm, two filter layers were provided between the Impervious fill and the u/s and d/s rock fill to meet the requirement of heading up on both sides of the cofferdam,



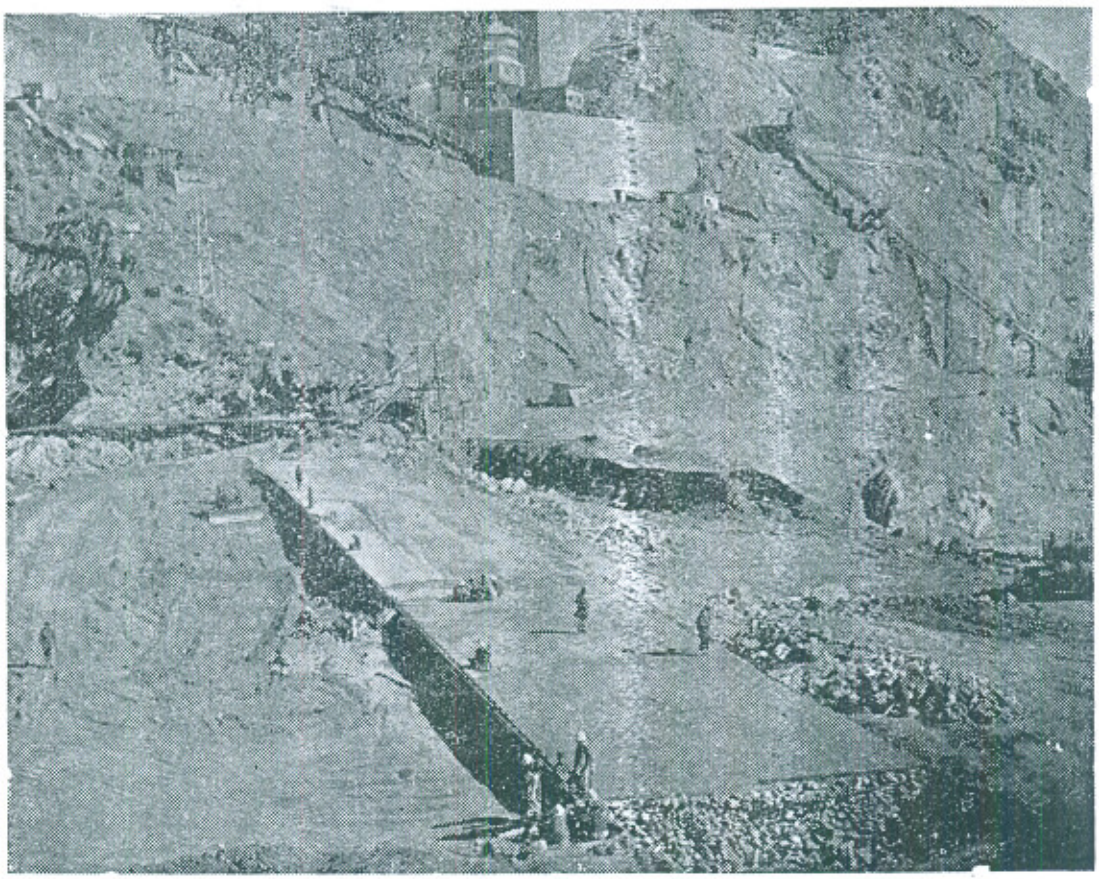
1. Downstream cofferdam showing rockfil



2. Downstream cofferdam dumping of transition material in progress



3. Downstream cofferdam driving of sheet piles



4. Downstream cofferdam showing sheet piles and colcuting



5. Downstream cofferdam. Downstream protection at the end of the cofferdam in the D/tunnel Outlet



6. Downstream cofferdam glacis.

The transition material 'R' and 'S' was gravel taken from borrow-pit area 'B'. Preliminary sieve tests were made of material from borrow pit area B in 7/57 and it was found suitable, grain size being 5 mm to 80 mm whereas the recommendations were 2.00 mm to 152 mm. (coarse marerial) and nothing passed 4 sieve.

The exact grading of this material was specified taking into consideration the nature of the rock fill 'P'. Similarly, the transition material 'T' & 'U' was sand and gravel from the borrow pit area 'A' graded to the limits 14 mm to 35 mm (=100 to 1″). laboratory tests on samples taken from loaded euclids, carrying fine transition material were fitting approx: the specification, (·15 mm to 35.0mm grain size) or 100 to 1″ sievae.

The impervious fill between the rock fills was placed by tipping from rock fill 'Q'. The grain size of this material was specified as/0025 to 2.00 mm and was obtained from Shagai Clay bits. In the centre of the impervious fill were driven two rows of sheet piles 30! apart, jointed at the top by ties and 5′ of grouted rip rap to form a rigid strut. See plate 6 (Stage II). These pile lines were not indicated to reach the rock bottom but were driven 10′ into the overburden unless boulders made it impracticable. These sheets piles were not essentially to act as a vertical cut-off against seepage. They were primarily needed to help in the stability of the cofferdam when there was heavy scour down steam. The possibility of using the single row of sheet piles was not accepted as single row of sheet pile would have bended or collapsed. These piles were not be salvaged when no longer needed.

At the time of driven of the sheet piles a mobils crane with a mobile monkey operated by compressed air was used and precaution was taken by covering the impervious fill with a layer of rock fill to provide good bearing capacity.

The driving of the twin lines of sheet piles was completed by Messrs. Angus Robertson Limited, in the first half of the month of Jan '58. During the last week of Jan '58 backfiiling between the piles with rock and grit was commenced and the surplus material from outside the sheet piling was removed. The grouted stone masonary and concrete capping was completed later on over the entire length of the cofferdam in stages to accommodate the access road. At the d/stream end a trench was excavated to permit the continuation of the capping to the river bed as an added precaution against scour during the flood season.

The sections of the Main Dam in the winter season following the completion of the Diversion Tunnel and that of the Cofferdam were to be completed so that during the next flood season, the main dam restricted the water current thereby helping a high back water for the safety of the u/s Cofferdam. Of 10 main blocks of the main dam the central blocks particularly 4, 5, 6, 7 &8 were not expected to be completed in the winter working season of 1957-58 as the excavation had to be carried down to bed

rock level 1050. On the actual execution of the work it was found that sound rock was available in the centre blocks at a higher elevation and therefore the concreting of side blocks as well as the central blocks had been taken up. This resulted in raising the central blocks to elevations as given in statement A by end of April, 58. Huge boulders and rock had also been encountered down stream of u/s cofferdam opposite bay 1 to3 and 9 & 10. This along with the concreting of the central blocks of the Maindam would provide additional protcion to the Upstream cofferdam.

In the final analysis it may be noted that the question of providing a third sheet pile line of the downstream cofferdam was not necessary in as much as the downetream cofferdam could be allowed to washed away. It was susceptable of quick construction.

Statement of the deepest foundation levels for Block No. 1 to 10.

Block No.	*Elevation*
1.	1095·0
2.	1080·0
3.	1062·5
4.	1062·5
5.	1065·0
6.	1060·5
7.	1060·0
8.	1060·0
9.	1085·0
10.	1087·5

Statment of the ccncrete elevation for Block No. 4 to 8 done uptil April, 1958.

4.	1077·0
5.	1080·0
6.	1075·0
7.	1085·0
8.	1105·0

Power House Cofferdam

Design Criteria :

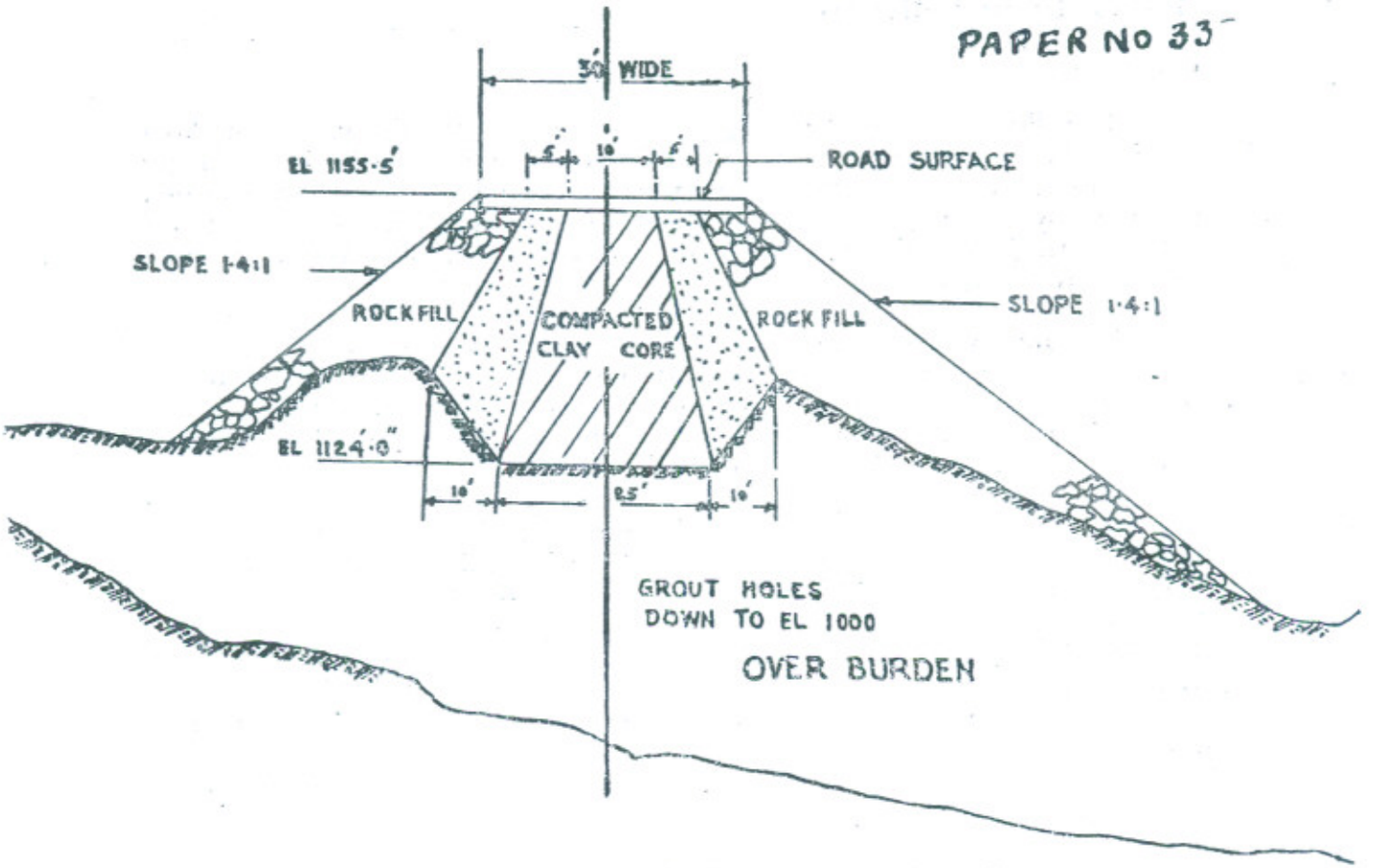
The Power House Cofferdam was to protect the Power House Area from inundation, when the upstream cofferdam would be overtopped during flcod season. It was partially or wholly located on river overburden underlain by pervious rock. The crest of the cofferdam was to be at E.l 1155, length being about 600′, and top width 30′. It was also to serve as an access road.

Keeping in view the economy and minimum interference wiht the flow in the river it was necessary to have the cofferdam as close to the powerhouse as the conditions permitted. On the other hand space had to be allowed for a construction road leading down into the Power House for excavation and carriage of equipment.

The maximum anticipated discharge of the Kabul river during the flood season was 200,000 cusecs. Model studies with this discharge indicated that the probable flcod level for a flow of 200,000 cusecs (without the u/stream and d/stream c/dam across the river) would be at E. 1149. With the Upstream and Downstream cofferdam in position, some additional rise of water was expected. Discharge of such a high magnitude being only a rare possibility and considering eco-nomy in construction and the ease and the readiness with which the crest of the cofferdam could be raised (i. e. by placing sand bags), the top was proposed at EI. 1155, later on raised to EI. 1160.

The section of fhe Power House Cofferdam was strong enough to withstand the hydrostatic pressure of $\frac{70'}{60'}$ of water, being the differeuce of the highest level on the river side and the lowest excavation level in the Power House Area (EI. 1149 EI. 1085).

The Section consisted of a compacted clay core wall, enclosed by transition material to prevent the flowing of the fine particles. The transition material was protected by the rockfill, which served as the outermost shell of the Cofferdam. In the original design the necessary degree of impermeability was obtained by masonary core wall for a certain length of the Cofferdam and by sheet piling and grouting the remaining length. Plate 7 indicates the suggested limit to which the masonary wall could extend. In this reach the rock profile was high and there was relatievly little overburden. Building a low masonary wall on the rock foundation presented no difficulty. Moving Upstream along the masonary wall, the depth of the overburden increased and limiting value was reached when the masonary wall was to be discontinued and a cut off was to be obtained by driving sheet piling. But in the field driving of sheet piles and its non-availability at early satge of construction as well as limited utility of the masonary wall forced the decision in favour of giving up both sheet piling and masonary wall. The designed section of the Cofferdam was as shown in the diagram.



Excavation work for the Power House Cofferdam clay core trench was started in early November, 56 and was nearly completed (towards the end of the month). This was followed by placing the clay core which was completed to EI. 1148′ during the month of December, 56. This work was temporarily suspended to allow excavation of power house area without further hindrance. Later on the Cofferdam was raised to El. 1155′ 1160′. To ensure proper compaction of the clay, it was placed in layers 2·0′ to 3·0′ high, sprinkled with water and compacted by a buldozer. The clay was transported by euclid from shagai area, 7 miles from Power House Site on the Warsak Jamrud Road. The clay core was trapezoidal in section' average base width 25′, top width 10′ height 31′, and had its base at El. 1124·0′.

As the trench for the clay core was excavated in the river overburden, therefore,only a small additional quantity of transition material was required to protect the clay core upto the top, which was dumped from the top of the clay core and compacted thoroughly.

The muck from the Power House excavation was dumped along the sides of the transition material to serve as the outer most rock shell. This rockfill was laid to a slope of 1·4:1.

Grouting :

Opposite of the Power House Cofferdam was the stilling basin area where water was to be headed up by the d/stream Cofferdam during the peri od of Constructi on. The rock and the overburden forming the Power House Cofferdam were leaky and its grouting was considered necessary so that seepage into the powerhouse area duriug high flow was cut off.

The eastern end of the powerhouse cofferdam had it junction with the control building hill and on the westren end joined the d/s end of the north bulkhead of the main dam, where the rock was weatheredd and poor.

These junction points required special grouting treatment to prevent seepage. Five A x size grout holes were laid at 15° interval along the vertical near the control building penetrating to EI 1000, depth of holes being 167′ to 236′ (Plate 7).

On the straight length of the Cofferdam, grout holes 20′ c/c were expected to provide adequate cut off against seepage but the water absorption test on drill holes requaired intermediate holes 10′ c/c and even 5′ centre to centre.

The grout holes were plugged and marked after each phase of drilling, washing and testing so that they could beeasily located, deepened and grouting further if necessary. The grout hole size was AX. diamond drilled, generally from EI. 1155 to E 1000 or above depending on field conditioni.

Water absorption test indicated the permeability of rock, and where the water loss exceeded 10 g.p.m., the upper portion of the drilled hole was grouted to prevent excessive loss of water. The inflow of water was measured by a metre and the outflow by a tank fitted with V-noteh.

Before starting grouting, water under pressure was passed into the drilled holes and the pressure grout mix was designed and controlled. Generally to start with , 1 bag of cement to 130 litres of water (26 gallon) were used. If sufficient pressure was not built up, the grout was thickened. The consistency of grout was increased by reducing the quantity of water gradually from 100 litres to 75 litres and 60 litres to 50 litres per bag of cement. Grouting at control building site was done at 15 to 30 lbs/sq/inch pressure.

After finishing grouting at the stipulated pressure, the water under pressure was passed and its subsequent pressure, checked. This indicated the efficiency of the grouting.

Field observations after the heading up caused by the coffordam indicated that the top of the powerhouse cofferdam should be raised to EI. 1160·0′. This was subsequently done and the river side slope was further strengthened with clay blanket and heavy rip rap in April/May, 58.

Drilling washing and heavy gronting was done by the Swissboring Corporation in accordance with the specifications issued by the consultants Messrs H.G.Acres and Company Limited.

Calculations for Coffer Dams

Data

Total flood discharge	= 2,00,000 cusecs
W.S.L.d/s of the d/s Coffer dam	= 1149
Discharge through the Diversion Tunnel	= 45,000 cusecs.
E.L. of crest of Upstream Cofferdam	= 1150·0
E.L. of crest of Dowastream do	= 1132·0
E.L. of concrete Bulkheads on Upstream face of Dam	= 1155·0
Skew length of u/s Cofferdam	= 400 ft.
Normal length of Cofferdam	= 400 cos 15° = 400 × ·97
	= 388 ft.
Skew length of D/s Coflerdam	= 344 ft.
Normal length of d/s Cofierdam	= 344 cos 40° = 344 × 0·77
	= 265 ft.

Q. = 155,000 cusecs, over the Cofferdams

∴ q per foot width of u/s Cofferdam

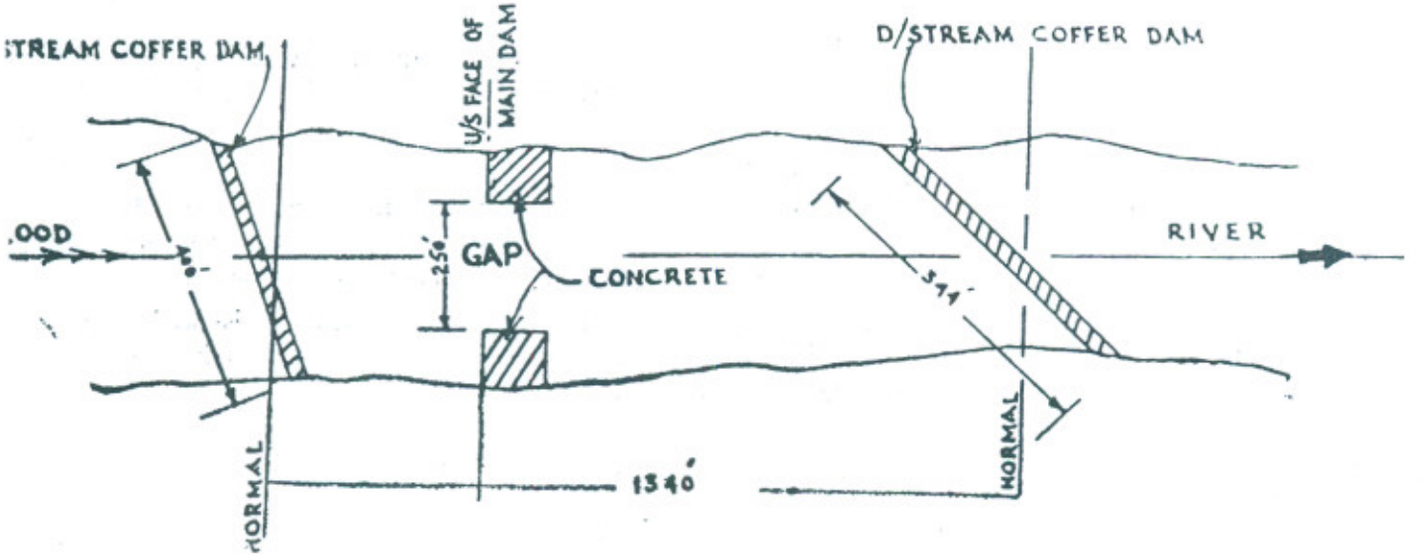
$$= \frac{155000}{388} = 400 \text{ cusec}$$

and q per ft. width of d/s Cofferdam $= \frac{155000}{265} = 585$ cusec

E.L. of Kabul River d/s of D/s Cofferam, with a discharge of 2,00,000 cusecs (155000 over the Cofferdams and 45,000 through the Diversion Tunnel) = 1149.

PLAN OF THE RIVER AT D/SITE SHOWING COFFER DAM

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Calculations for W.S.L. of D/S Cofferdam

Downstream Cofferdam

$(H + ha)^{3/2}$ $\quad q = 585 = 3{\cdot}1\ (h+ha)^{3/2}$

$= 585/3{\cdot}1 = 189.$

$h+ha = 189^{2/3} = 32{\cdot}9$

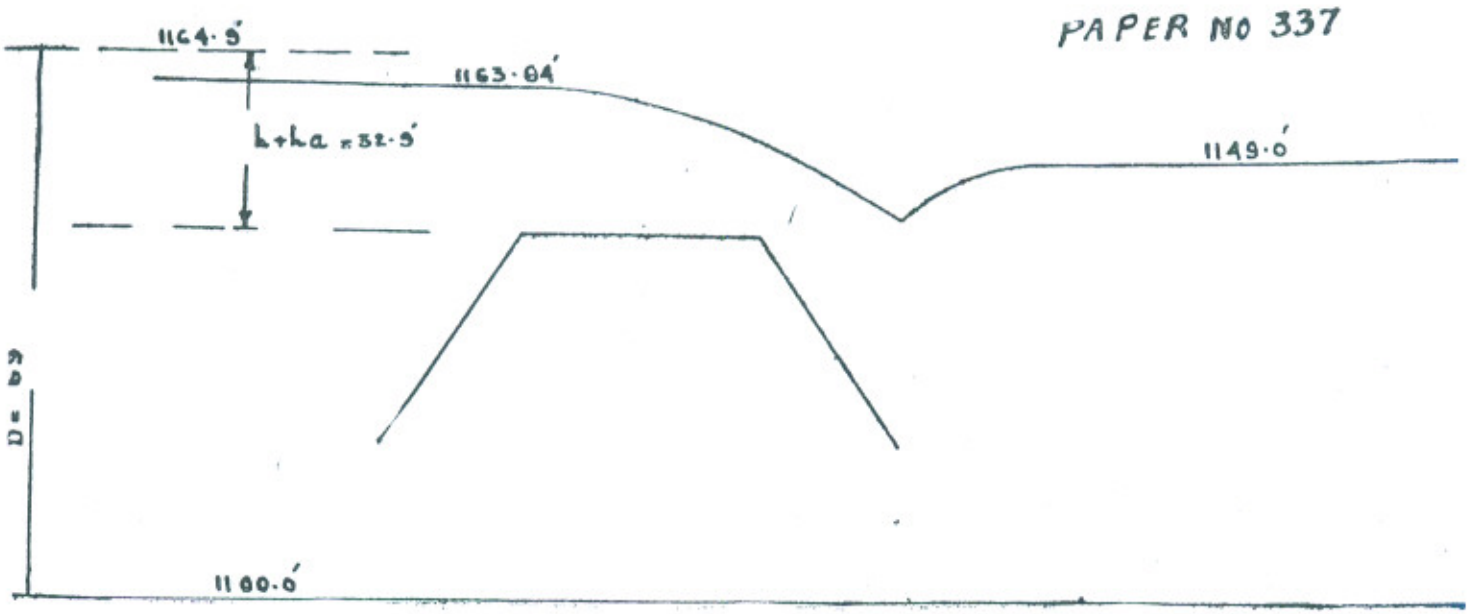
D = Depth Upstream $= 64{\cdot}9$

$V_a = \frac{q}{D} = \frac{585}{65} = 9$ ft./sec.

$h_a = V_a^{2/2g} = \frac{9 \times 9}{64{\cdot}4} = 1{\cdot}26$

∴ E.L. of Water Surface Upstream $= 1164{\cdot}9 - 1{\cdot}26 = 1163{\cdot}64$

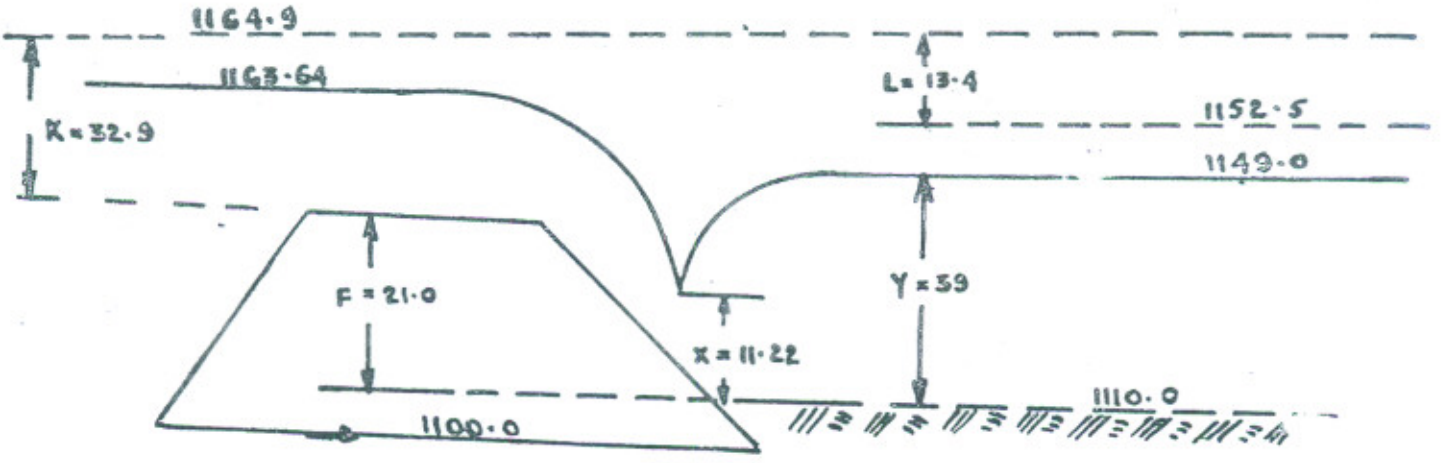
Drowning Ratio $= \frac{1149 - 1132}{1163{\cdot}64 - 1132} = \frac{17}{31{\cdot}64} = 0{\cdot}55$ OK



Position of Standing Wave

Calculations for D/S Cofferdam

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Assume river bed E.L. 1110

q = 585 cusecs

$C = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} = \left(\frac{585^2}{32{\cdot}2}\right)^{\frac{1}{3}} = 22$

$Va = \frac{585}{39} = 15$ ft/sec.

$h_a = \frac{Va}{2g} = \frac{15 \times 15}{64{\cdot}4} = 3{\cdot}5$

D/S T.E.L. $= h + h_a = 1149 + 3{\cdot}5 + 1152{\cdot}5.$

L = Difference of total energy lines u/s & d/s.
= 1164·9—1152·5 = 12·4

$\frac{L}{C} = \frac{12{\cdot}4}{22} = 0{\cdot}564$

$\frac{X}{C} = 0{\cdot}51$ $\qquad X = 0{\cdot}51 \times 22 = 11{\cdot}22$ ft.

$\frac{X}{C} = 1{\cdot}75$ $\qquad Y = 1{\cdot}75 \times 22 = 38{\cdot}5$ say 39 ft.

$\frac{K+F}{C} = 2{\cdot}44$ $\qquad K + F = 2{\cdot}44 \times 22 = 53{\cdot}6$

$\therefore \quad F = 53{\cdot}6 - 32{\cdot}9 = 20{\cdot}7$ say 21′.

Calculations for the position of Hydraulic jump with the D/S Cofferdam washed away & 250′ wide Gap at the Dam Site

Tail Water Level = 1149

Assume water level 1150 u/s of 250′ gap.

Drowning ratio $= \frac{49}{50} = 0{\cdot}98$

$\therefore$ C = 1·9 In the weir formula.

Depth below T.E.L. D/S $= h + h_a = 50 + \left(\frac{15500}{250 \times 50}\right)^2 \frac{1}{64{\cdot}4}$

$= 50 + 2{\cdot}4 = 52{\cdot}4$

$\therefore Q = 1{\cdot}9 \times 2{\cdot}50 \times (52{\cdot}4)^{\frac{3}{2}}$

$= 1{,}79{,}500$ cusecs.

For the purposes of hydraulic jump condition without the gap are more serious. Hence W.L. D/S of U/S Cofferdam may be taken as 1149.

L = 1175·5—1151·4 = 24·1

K = 1175·5—1150 = 25·5

C = Critical depth $= \left(\frac{q}{g}\right)^{\frac{1}{3}} = \left(\frac{400 \times 400}{32{\cdot}2}\right)^{\frac{1}{3}} = 17{\cdot}0'$

$\frac{L}{C} = \frac{84{\cdot}1}{17} = 1{\cdot}42$

$\therefore \quad \frac{X}{C} = 0{\cdot}41$; $\quad \therefore X = 0{\cdot}41 + 17 = 6{\cdot}97$ say 7′

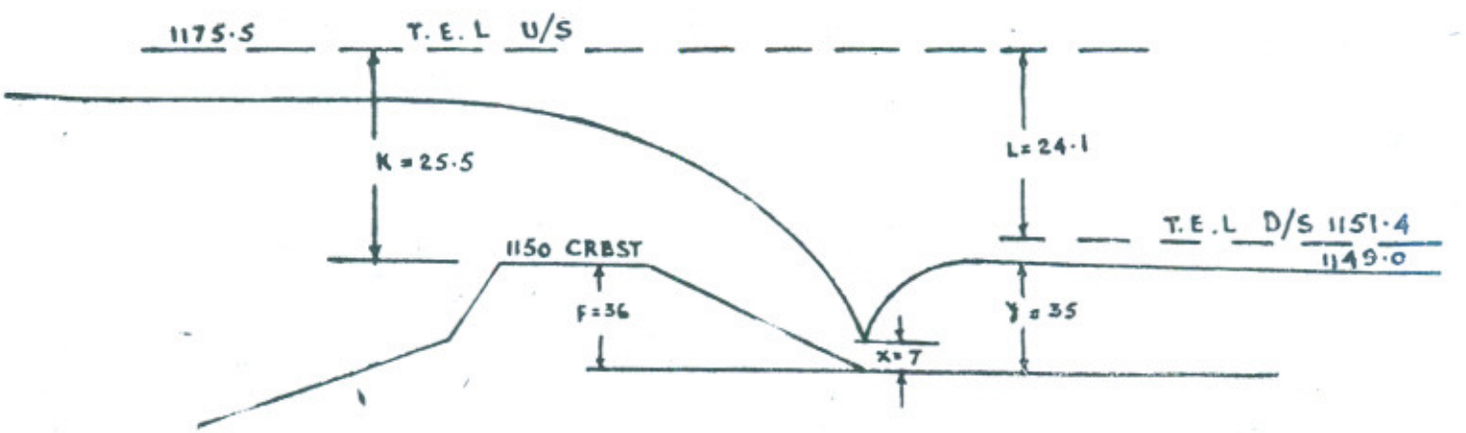
$\frac{Y}{C} = 2{\cdot}05$; $\quad \therefore Y = 2{\cdot}05 \times 17 = 34{\cdot}8$ say 35′

$\frac{K+F}{C} = 3{\cdot}6$; $\quad \therefore F = 3{\cdot}6 \times 17 = 25{\cdot}5 = 36{\cdot}0$

$\therefore$ R.L. of base line = 1150—36 = 1114

The position of the Hydraulic Jump is shown in Fig. I.

PAPER No 337



Calculations for Scour Depths by Lacey's Formula

D/S Cofferdam

Total Q =2,00,000 cusecs

Q through tunnel = 45,000 cusecs

Q over Cofferdam =1,55,000 cusecs

Normal length of D/S =265′

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

for (*i*) f=1 and (*ii*) f=2

$\therefore\ q=\frac{155000}{265}$ =585

$R=0{\cdot}9+\left(\frac{585^2}{1}\right)^{\frac{1}{3}}$

=70 ; For f=2, R=53′

Maximum Scour Depth may be taken as 1.5 R in this case.

∴ Maximum scour Depth with f=2, is 53 x1.5

=79.5′ say 80′

∴ R.L. bottom of Scour pit =1149—80=1069

U/S Cofferdam

f=1 R=48·8′=49

f=1.5 R=43

f=2 R=39

Maximum Scour Depth with f=2, is 1.5×39

=×58.5 say 59′

R.L. of bottom of Scour Pit=1149—59=1090

Calculation for Scour Depth Below the Upstream and Downstream Coffer dam by Born's Formula

D/S Coffer dam

Q = 155000 cusecs

WI = Water surface width = 265′

Cr = 2.5

Scour depth $R=2{\cdot}25\left(\frac{Q}{WICr}\right)^{3/5}$

$=2{\cdot}25\left(\frac{155000}{265\times2.5}\right)^{3/5}=2{\cdot}25\ (234)^{3/5}$

$=2{\cdot}25\times26{\cdot}4=60'$

Maximum Scour 33% more =80′

R.L. of water surface D/S of the downstream Coffer dam = 1149·0

∴ DL of Maximum Scour depth = 1149 — 80 = 1069

Upstream Cofferdam

Q = Discharge =155000 cusecs

W1 = Water surface of Kabul River just downstream of the upstream Cofferdam = 388′

Cr = 2·5 (from Brown's formula)

$$\text{Scour depth } R = 2{\cdot}25 \left(\frac{q}{WCr}\right)^{3/5} = 2{\cdot}25 \left(160\right)^{3/5}$$

$$= 2{\cdot}25 \times 21 = 47{\cdot}25 \text{ say } 48,$$

To allow for construction etc. maximum scour will be 33% more =64 ft.

Elevation of water surface d/s of upstream Cofferdam

Assuming D/S cofferdam washed away:

Water surface d/s in this case = El. 1149·0

∴ EL of scour depth =1149 — 64 = 1095

Summarising the maximum scour depth are:

	Lacey's	*Bromn's*
D/S Cofferdam	80	80
U/S Cofferdam	59	64

The calculations of the scour depth for the d/s cofferdam show that the river bed will scour upto on El. of 1069 i. e. 41′ below the average river bed (El.1110).

There is no provision in the present design to arrest this scouring action. Also no protection is provided against the scouring section due to the turbulance created by the hydraulic jump, the position which is shown in Fig. 2.

'The practice in this country based on the experience of design of weirs and barranges on sandy soils, is to provide a masonry floor upto a length nearly to five times the depth of flow and an equivalent length of the flexible apron consisting of big size concrete blocks.

say 4′×4″×3″ deep or
5″ × 5″ × 3″ deep concrete

and about 3 feet to 5 feet deep length of loose stone. This flexible apron settles as the scour progresses and it ultimately lines the scour pit and prevents damage to the main structure.

If the d/s cofferdam is allowed to be washed away during the next floods, utmost attention must be paid to safeguard the u/s cofferdam.

The u/s cofferdam be fully safeguarded as per proposals shown in Fig. 1, *i. e.* (*i*) by taking the masonry protection right upto the rock foundation, or (*ii*) by providing the necessary length of masoury floor for the hydraulic jump and also an equal length of flexible apron.

Maximum Scour 33% more =80′

R.L. of water surface D/S of the downstream Coffer dam = 1149·0

∴ DL of Maximum Scour depth = 1149 — 80 = 1069

Upstream Cofferdam

Q = Discharge =155000 cusecs

W1 = Water surface of Kabul River just downstream of the upstream Cofferdam = 388′

Cr = 2·5 (from Brown's formula)

$$\text{Scour depth } R = 2{\cdot}25 \left(\frac{q}{WCr}\right)^{3/5} = 2{\cdot}25 \left(160\right)^{3/5}$$

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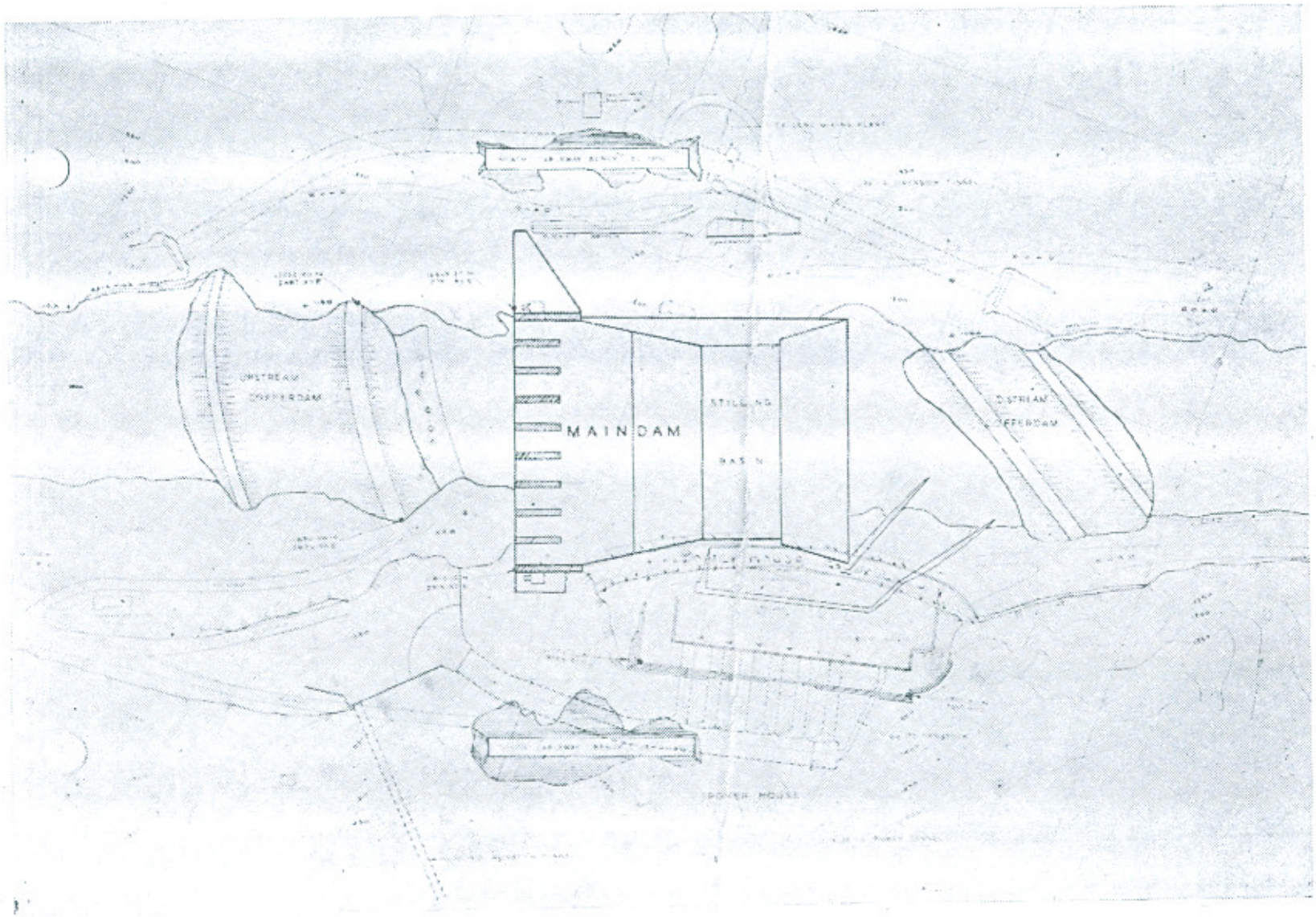
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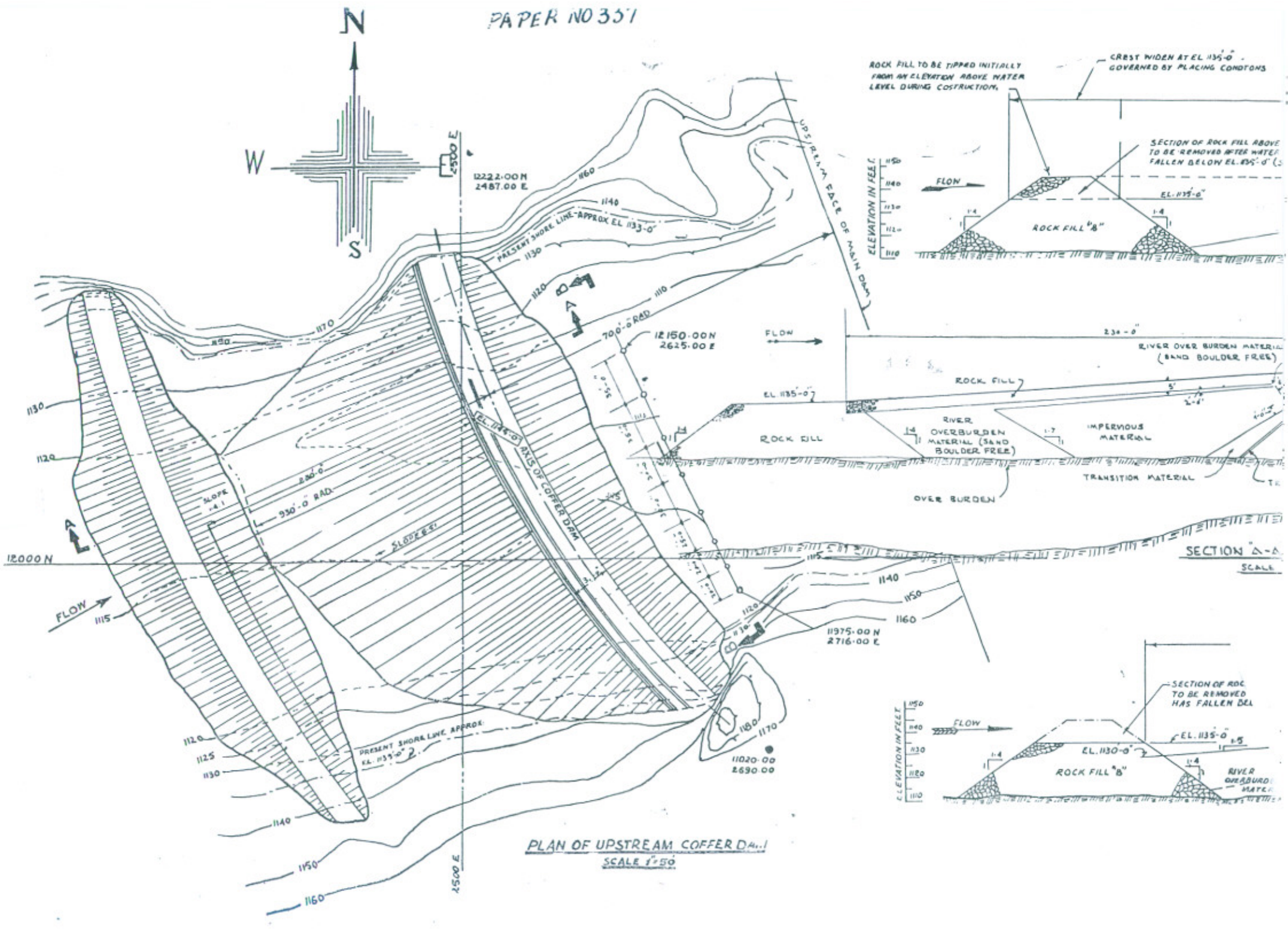
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Construction Schedule

November 1955 to October 1956	=	Exploratory Boring of River bed to determine the nature and permeability of overburden material.
December 1956	=	Power House Cofferdam
November 1956 to May, 1957	=	Drilling of tubewells through overburden of river and then through rock.
= do =	=	Drilling holes for grouting in rock below the tubewell holes.
= do =	=	Grouting of rock and completion of grout curtain at toe of the proposed upstream cofferdam.
= do =	=	Installation of screens and blind pipes in the tubewells and concrete capping of their tops.
August 1957	=	Diversion of River into the diversion tunnel.
September & October 1957	=	Stage I of Upstream Cofferdam :— Dumping of rockfill A & B, of transition material C & D and impervious material E
= do =	=	Stage I of Downstream Cofferdam :— Rockfill P & Q and tipped from El. 1132 transition material R & S dumped on rockfill P & Q respectivcly first and covered with transition material T & U. Impervious fill V dumped in between the transition material sections from rockfill Q
November 1957 to February 1958	=	Stage II of Upstream Cofferdam :— Trimming rockfill A to elevation 1143 and rockfill B to El. 1135 and dumping river overburden material from rockfill B to squeeze up impervious fill E and trim it to a slope of 1s 6·6 from El. 1143 rockfill A.
= do =	=	Development of tubewells and groutiong of North and South abutment of Upstream Cofferdam.
January 1958 to March 1958	=	Stage II of Downstream Cefferdam :— Driving of two rows of sheet-piles 30′ apart joining these at top by ties and grouted rip rap 5·0′ to cover the top of the sheet pile El. 1032, to be laid at the crest as well as at the downstream slope.
January 1958 to February 1958	=	Stage III of Upsream Cofferdam ;— Stone masonry on the top of upstream cofferdam, glacis and wave wall El. 1114.


MAIN DAM
STILLING
BASIN
POWER HOUSE

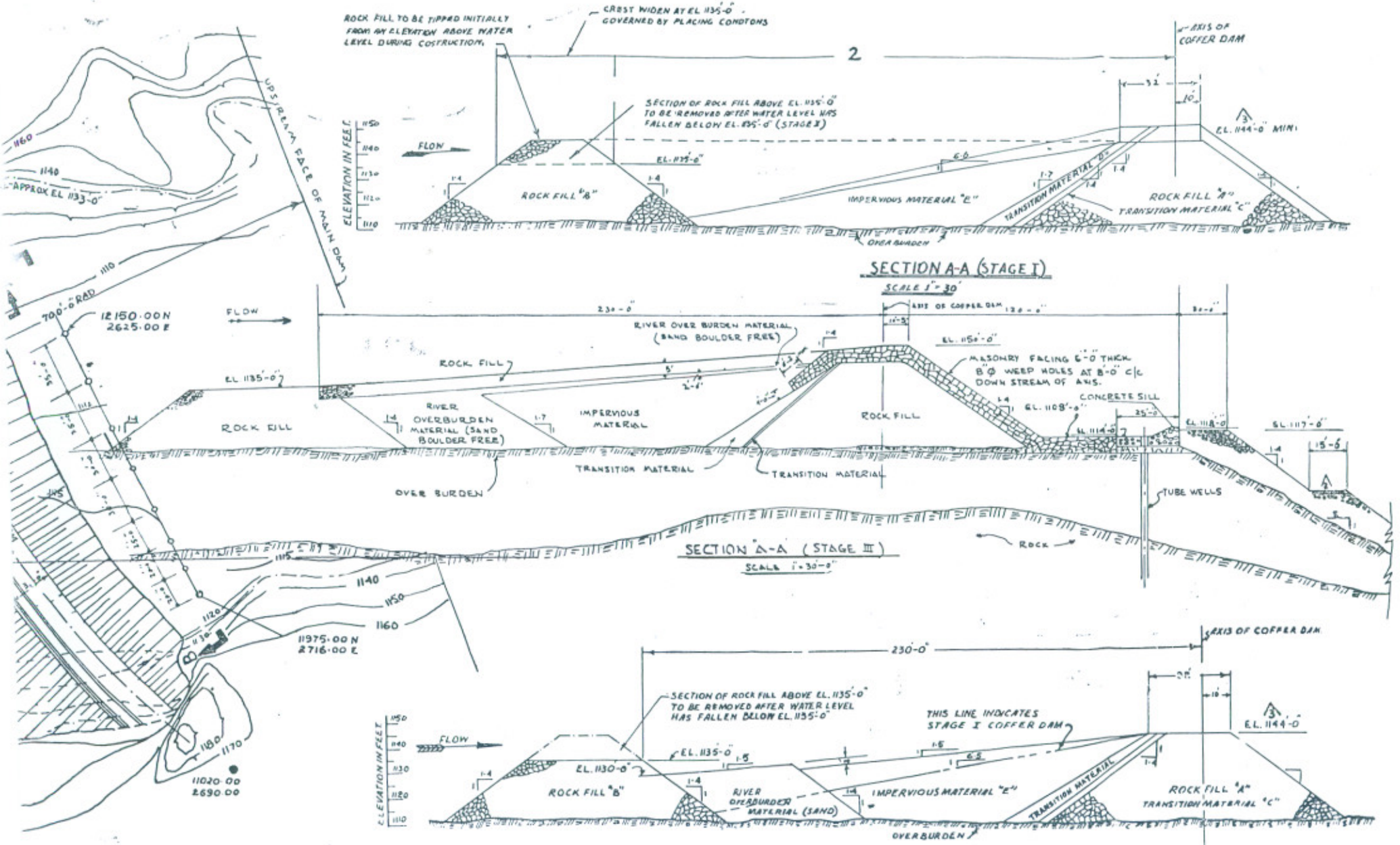
PAPER NO 351



PLAN OF UPSTREAM COFFER DAM
SCALE 1"=50'

NO 337

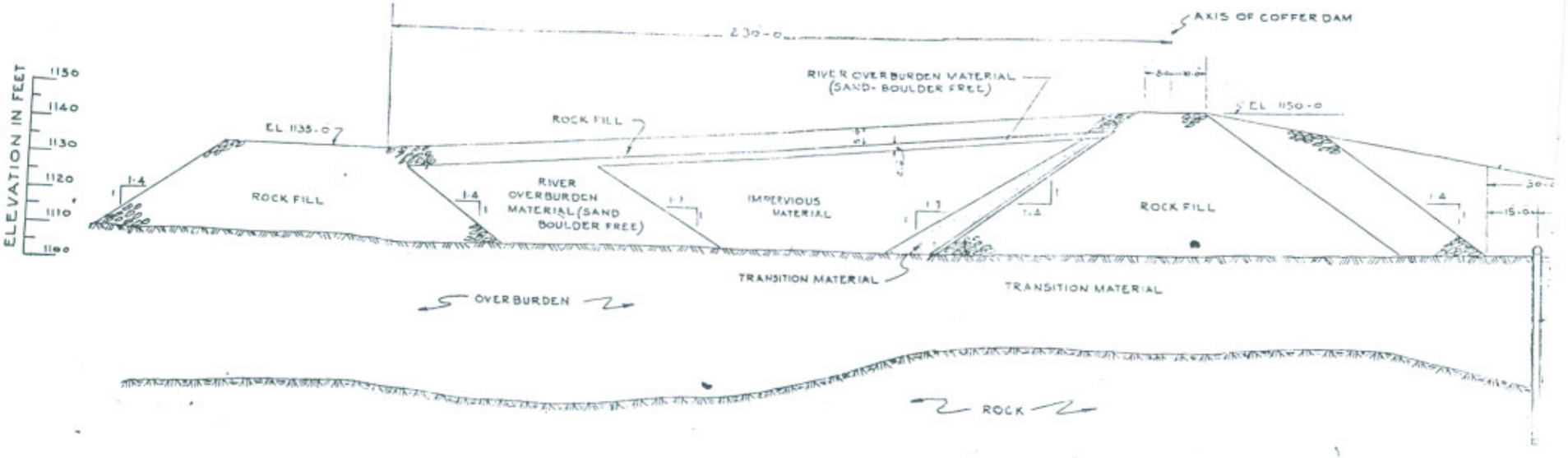
PLATE NO. 2



E UPSTREAM COFFER DAM

SCALE 1"=50'

PAPER NO 337

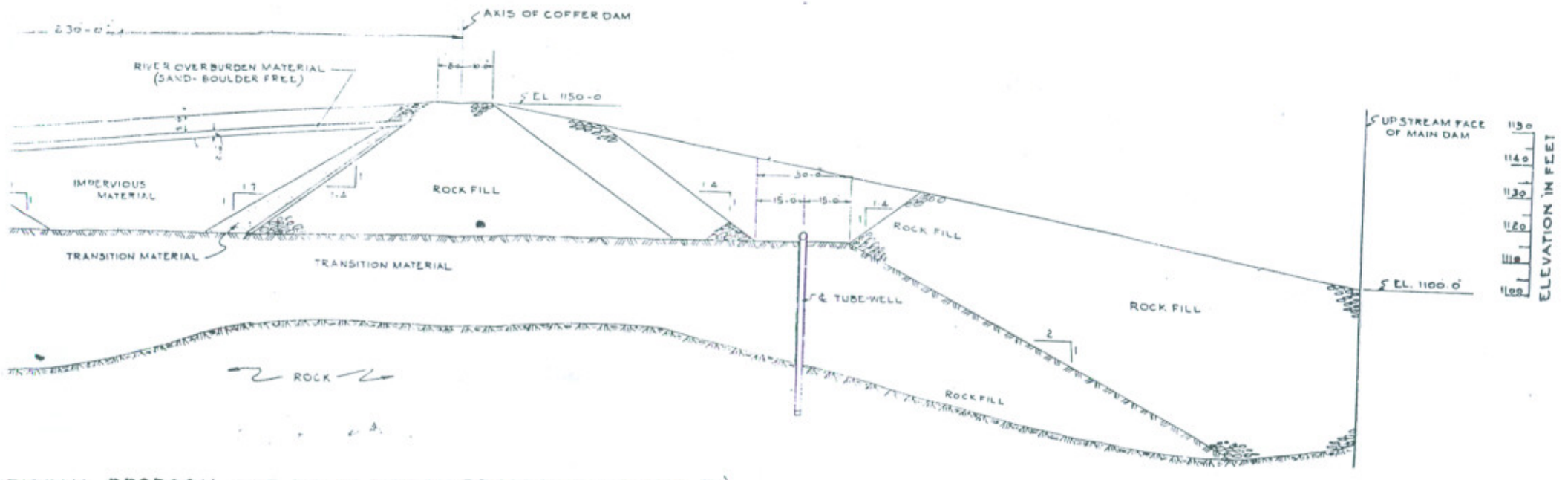


(ORIGINAL PROPOSAL FOR DOWN STREAM PROTECTION STAGE III)

SECTION OF UP STREAM COFFER DAM

SCALE:- 1" = 6'

PAPER NO 337

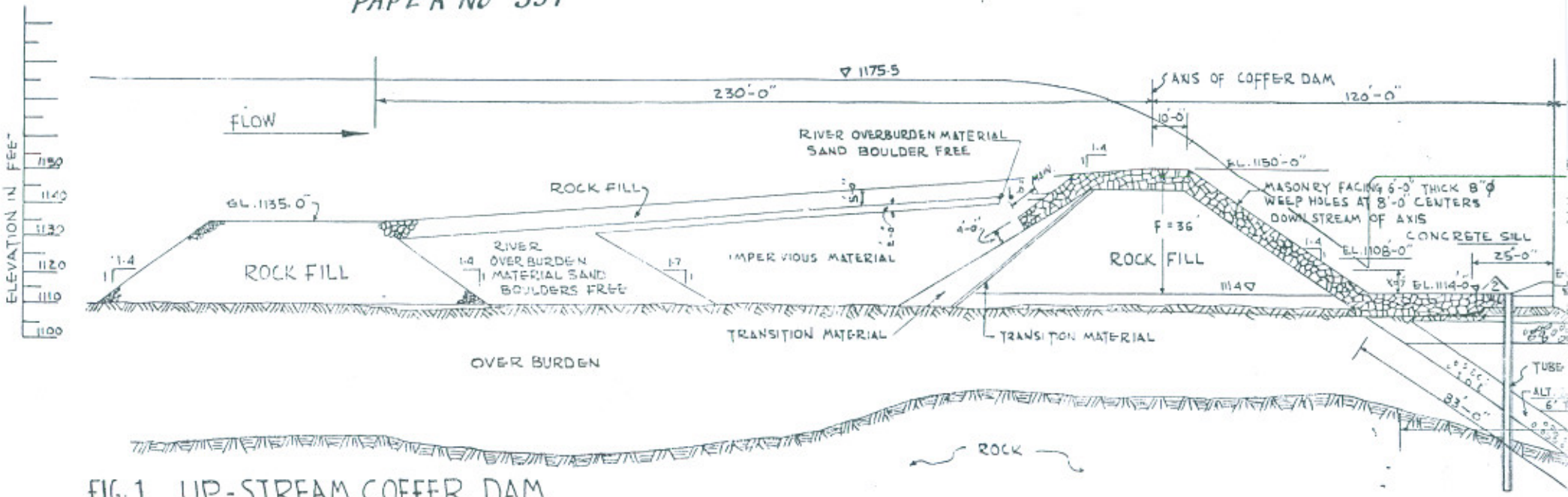


ORIGINAL PROPOSAL FOR DOWN STREAM PROTECTION STAGE III)

SECTION OF UP STREAM COFFER DAM

SCALE:- 1" = 6'

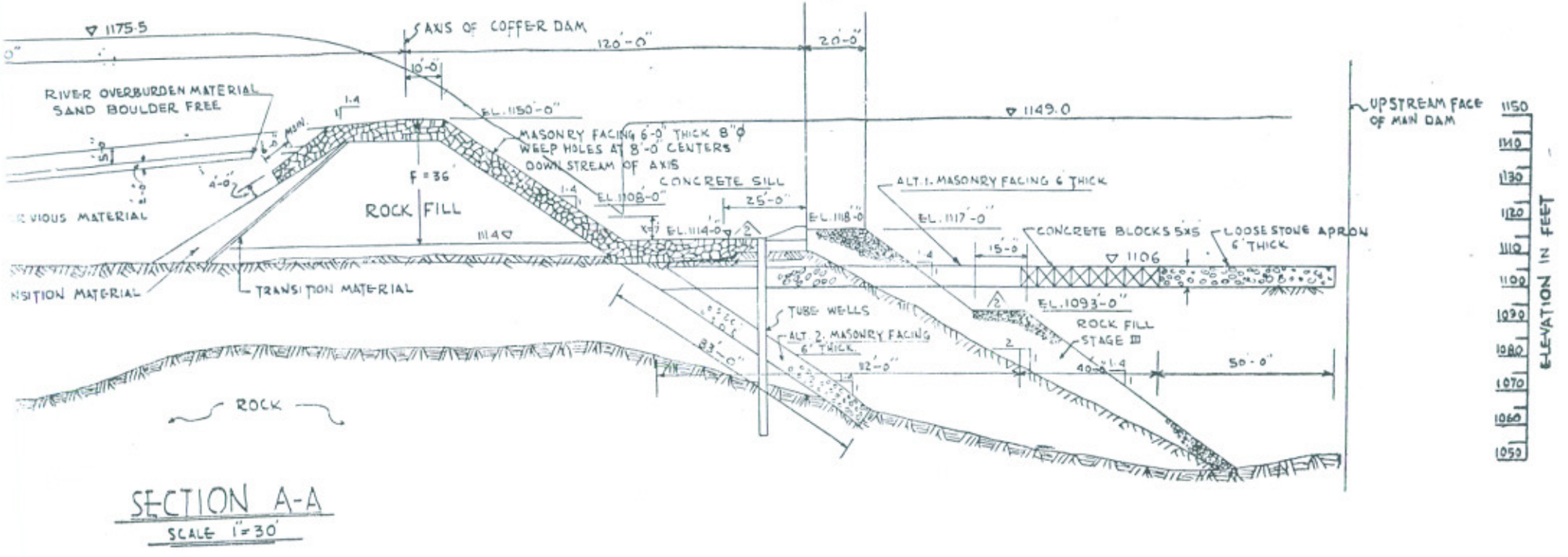
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FIG; 1 UP-STREAM COFFER DAM
SHOWING PROPOSED PROTECTION

SECTION A-A
SCALE 1"=30'

PLATE No.3A



SECTION A-A

SCALE 1"=30'

PAPER NO 337

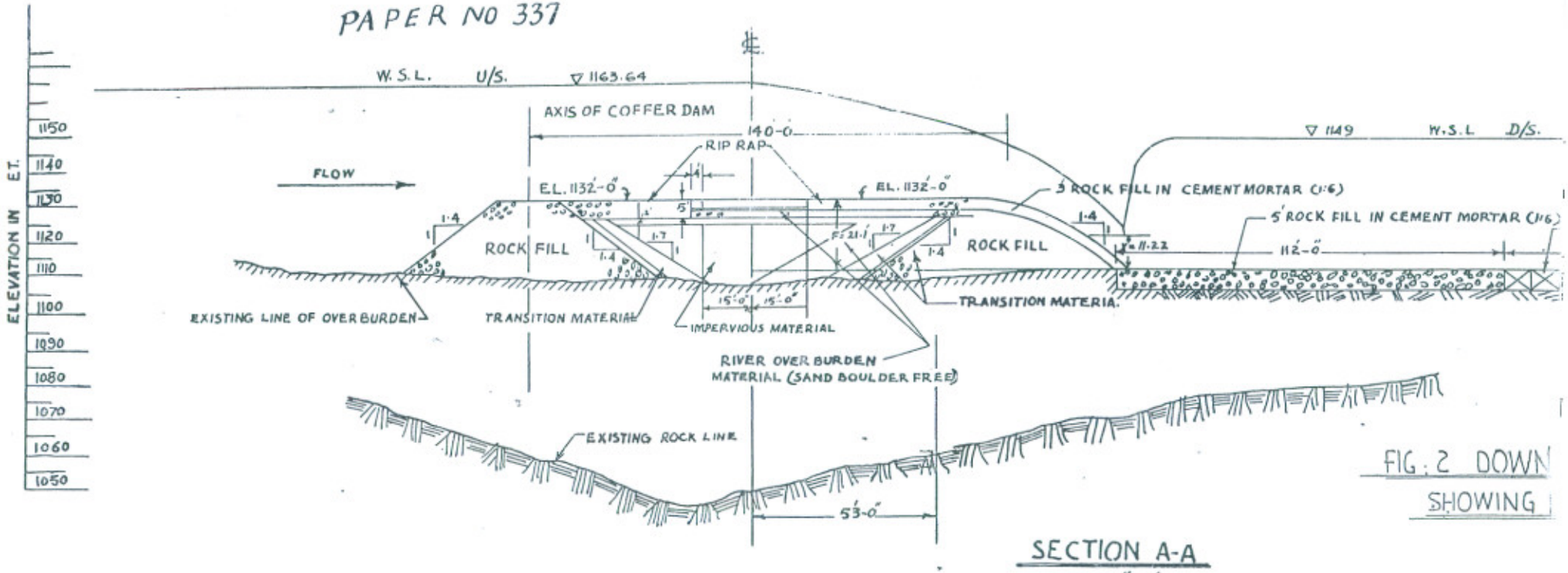


FIG. 2 DOWN
SHOWING

SECTION A-A
SCALE 1"=30'

NO 337

PLATE No 38

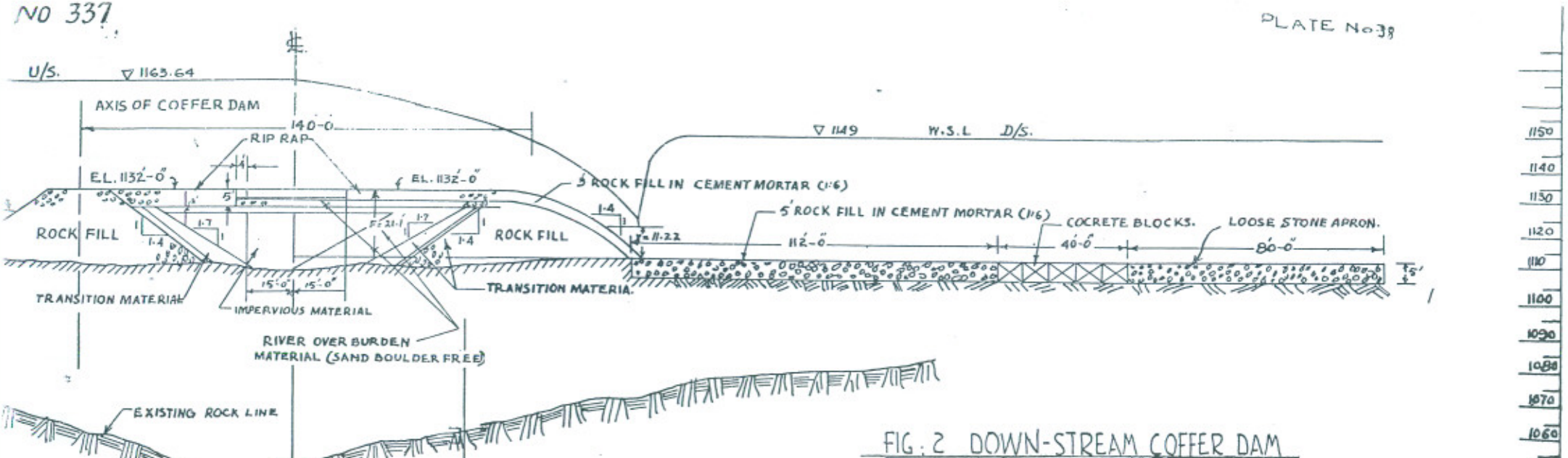
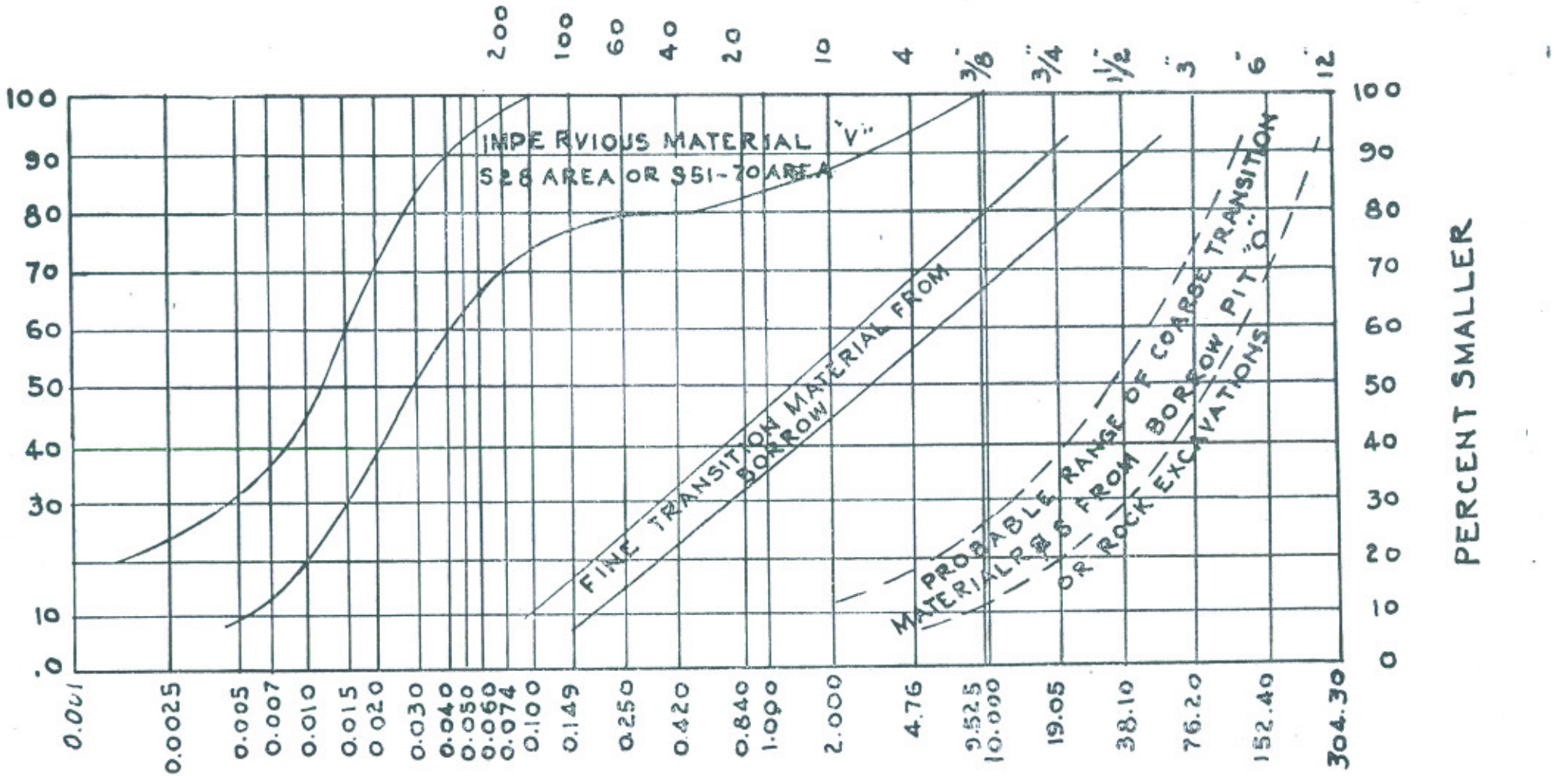


FIG. 2 DOWN-STREAM COFFER DAM
SHOWING PROPOSED PROTECTION

SECTION A-A
SCALE 1"=30'

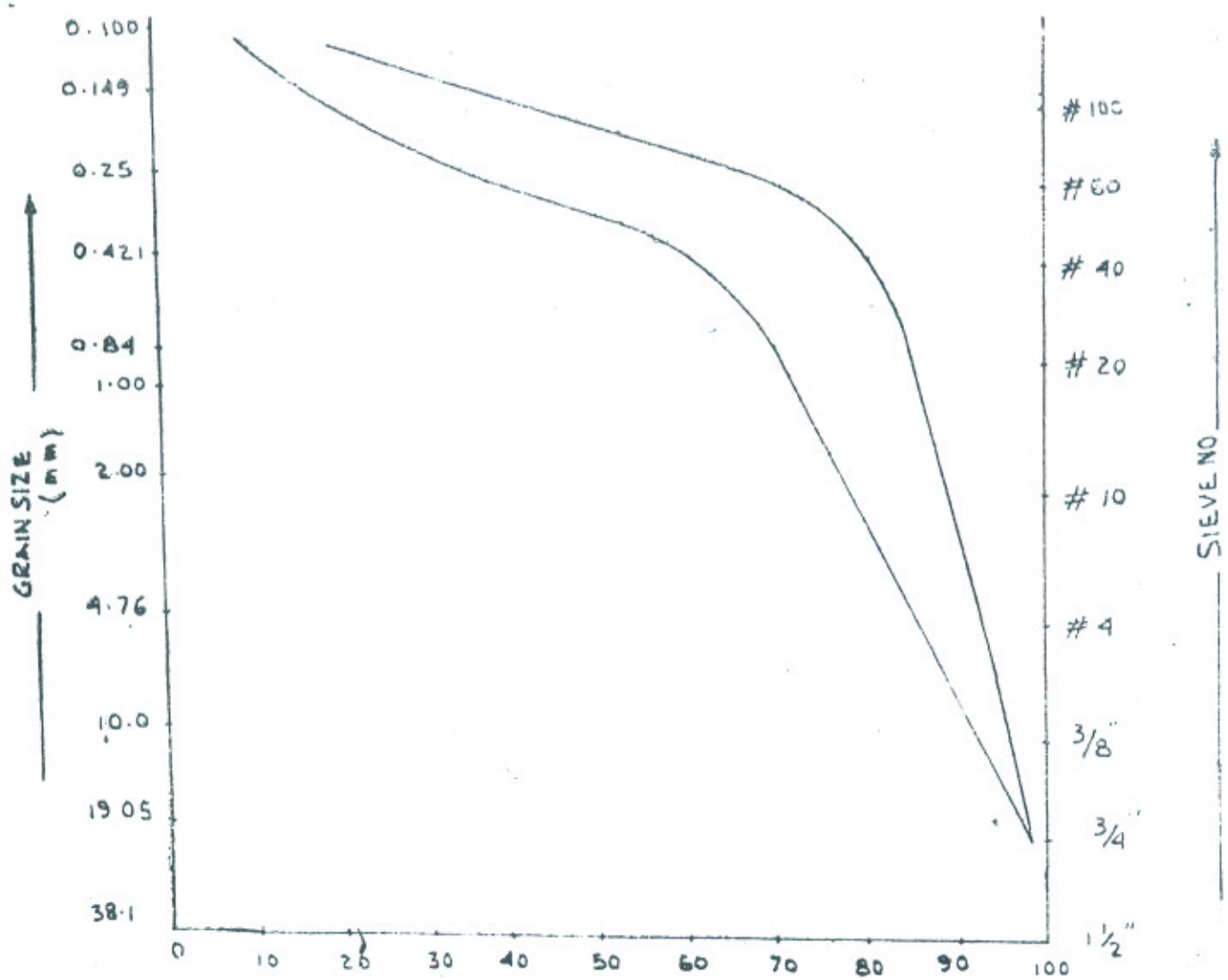


GRAIN SIZE IN MILLIMETERS

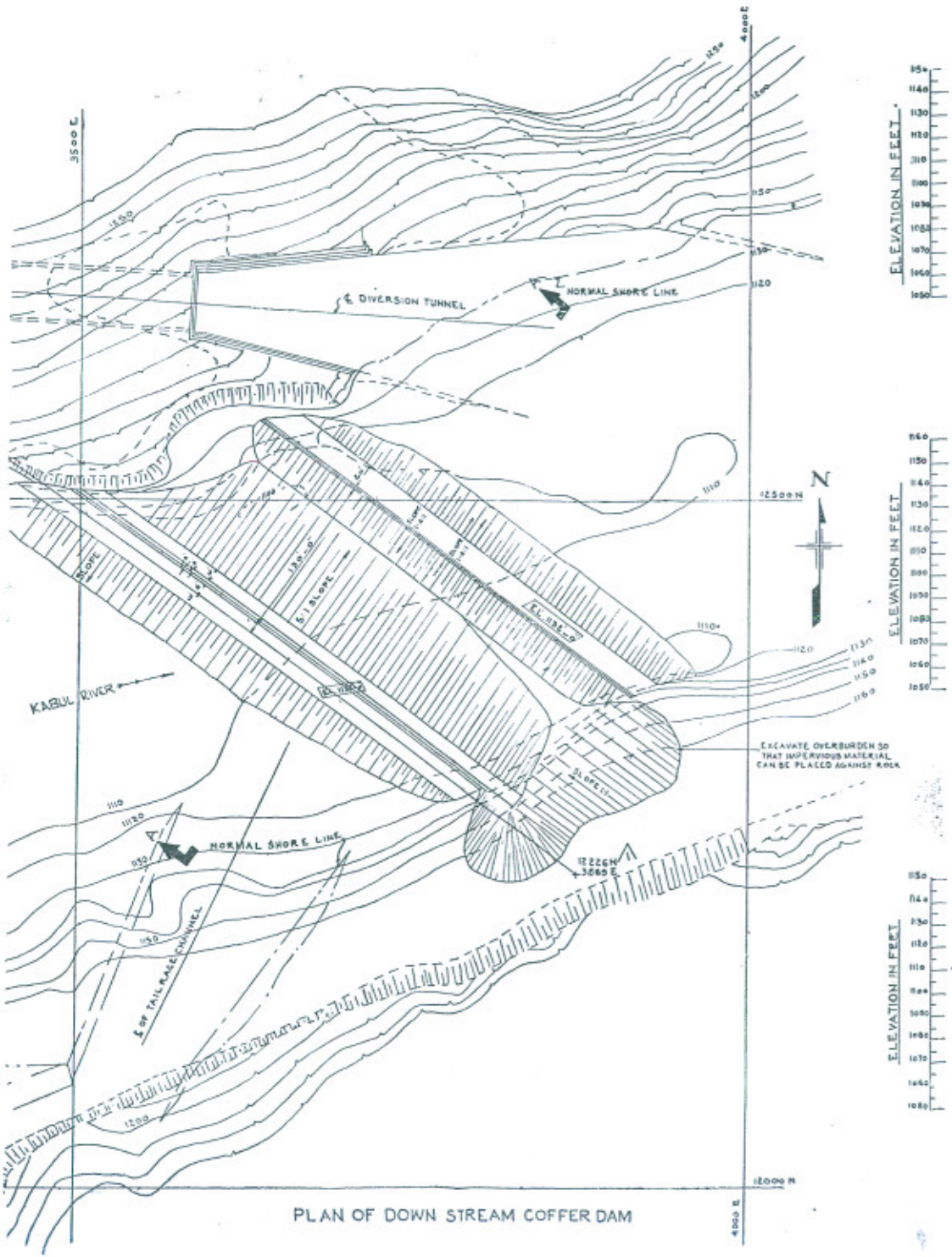
IMPERVIOUS APRON AND TRANSITION - GRAIN SIZE DISTRIBUTION

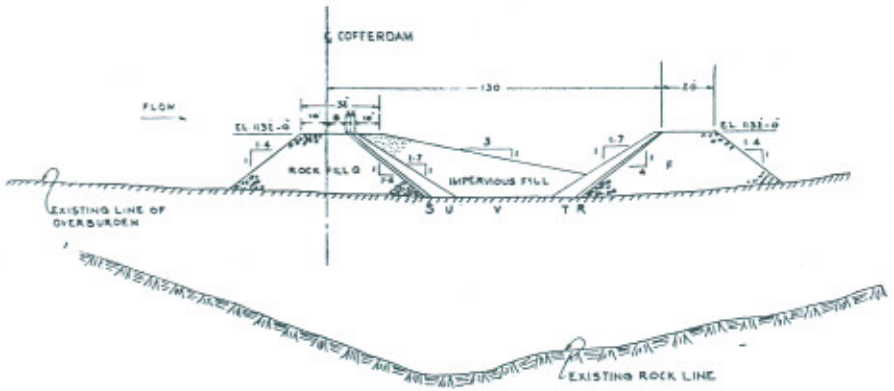
PLATE NO 5

PAPER NO 337

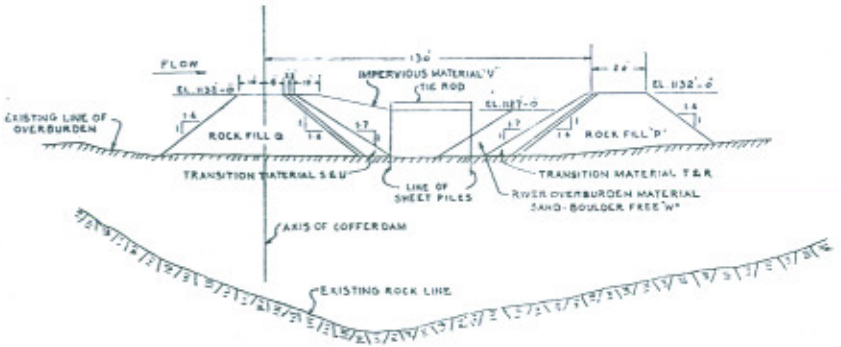


PARTICLE SIZE DISTRIBUTION CURVE OF RIVER BED MATERIAL

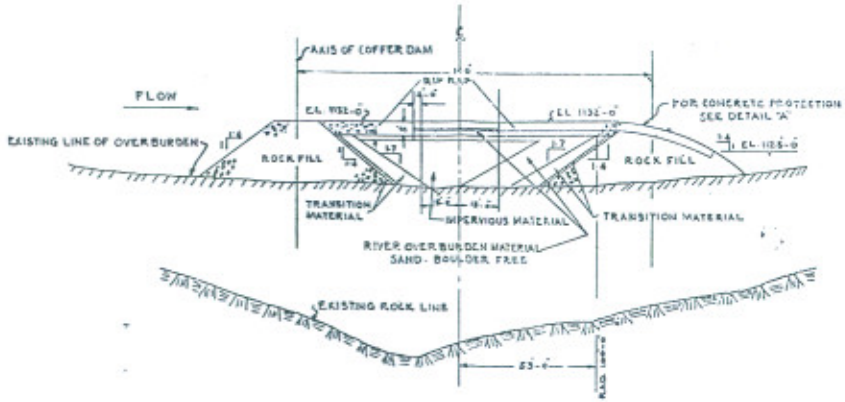




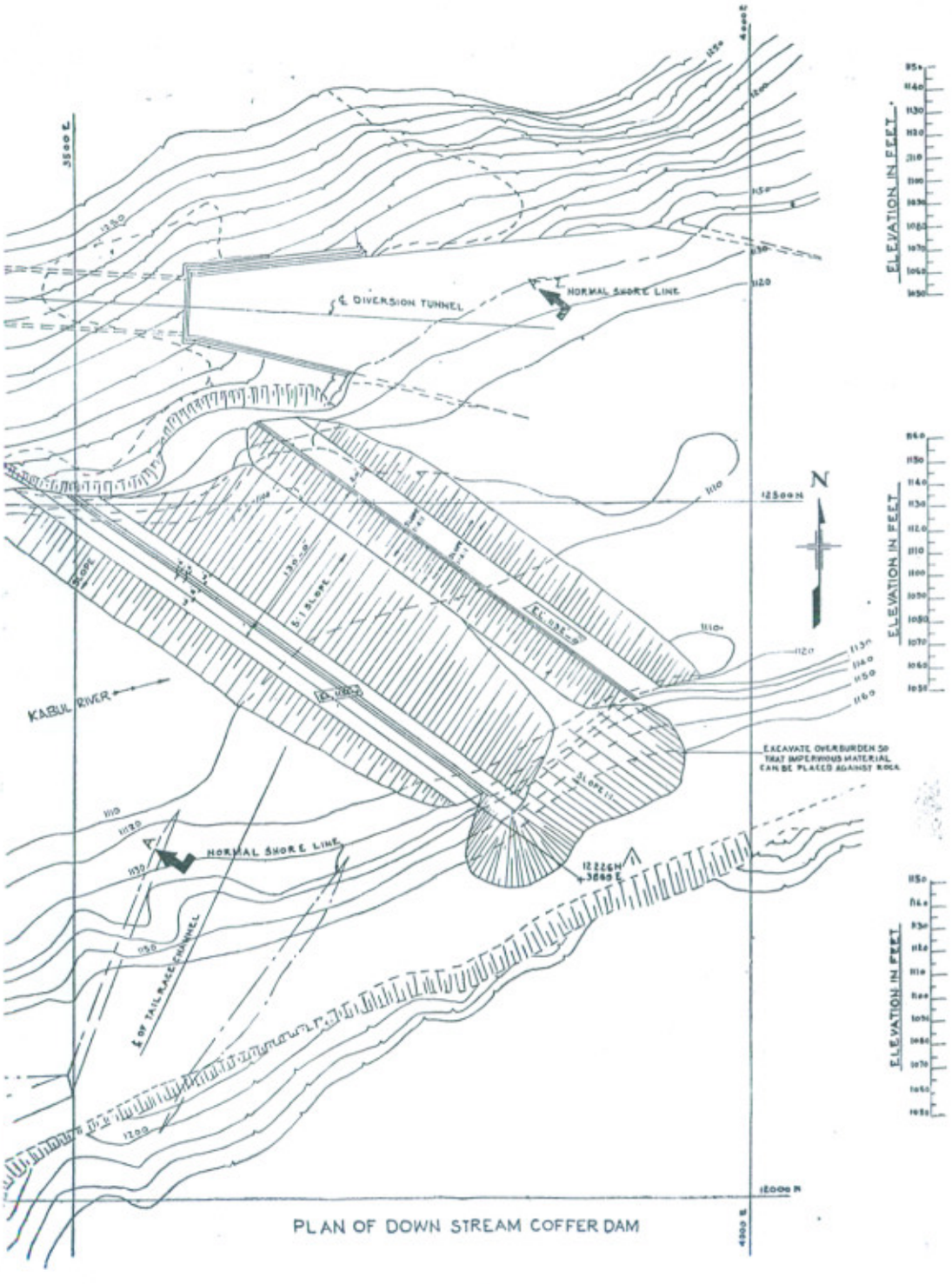
SECTION A - A
SCALE 1" = 30'
(STAGE I)



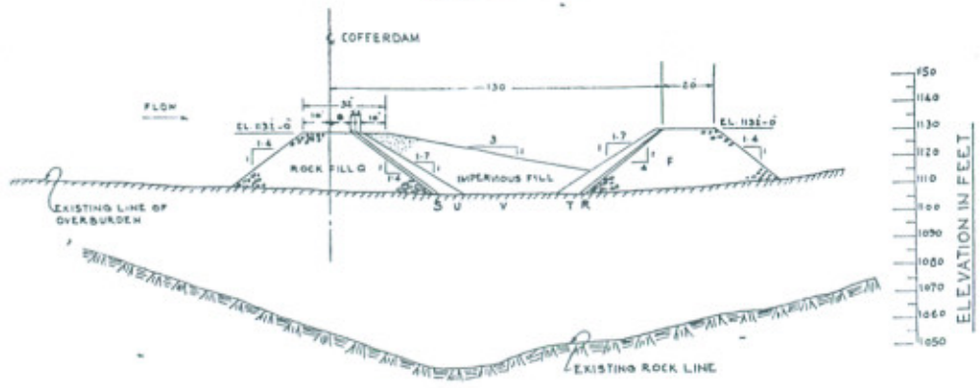
SECTION A - A
SCALE 1" = 30'
(STAGE II)



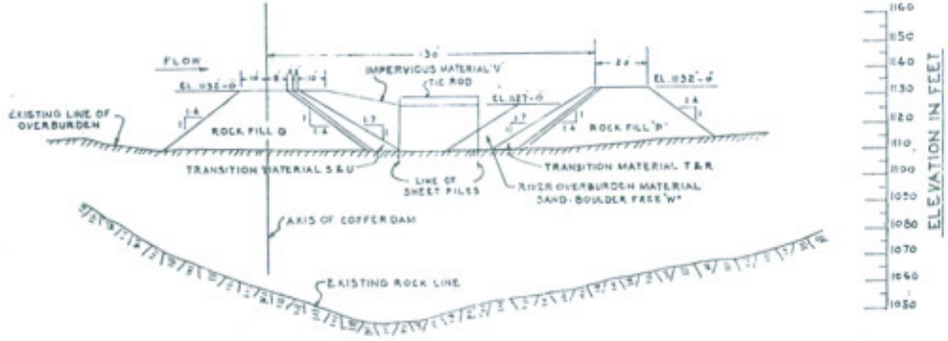
SECTION A - A
SCALE 1" = 30'
(STAGE III)



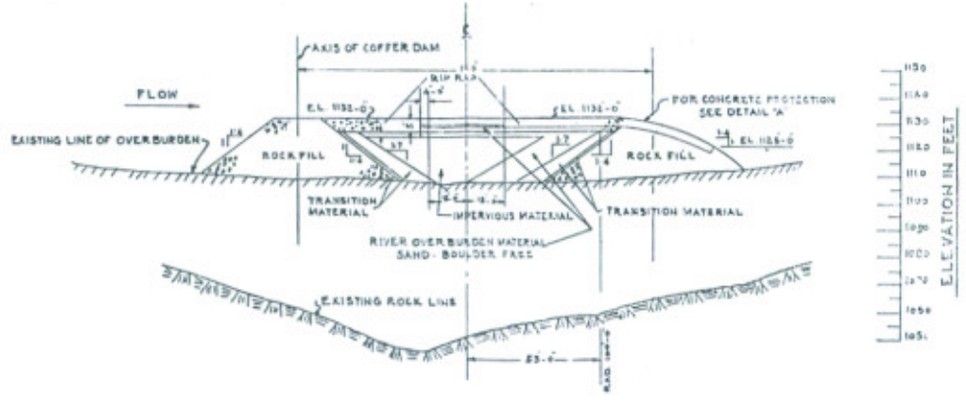
PLAN OF DOWN STREAM COFFER DAM



SECTION A-A
SCALE 1"=30'
(STAGE I)

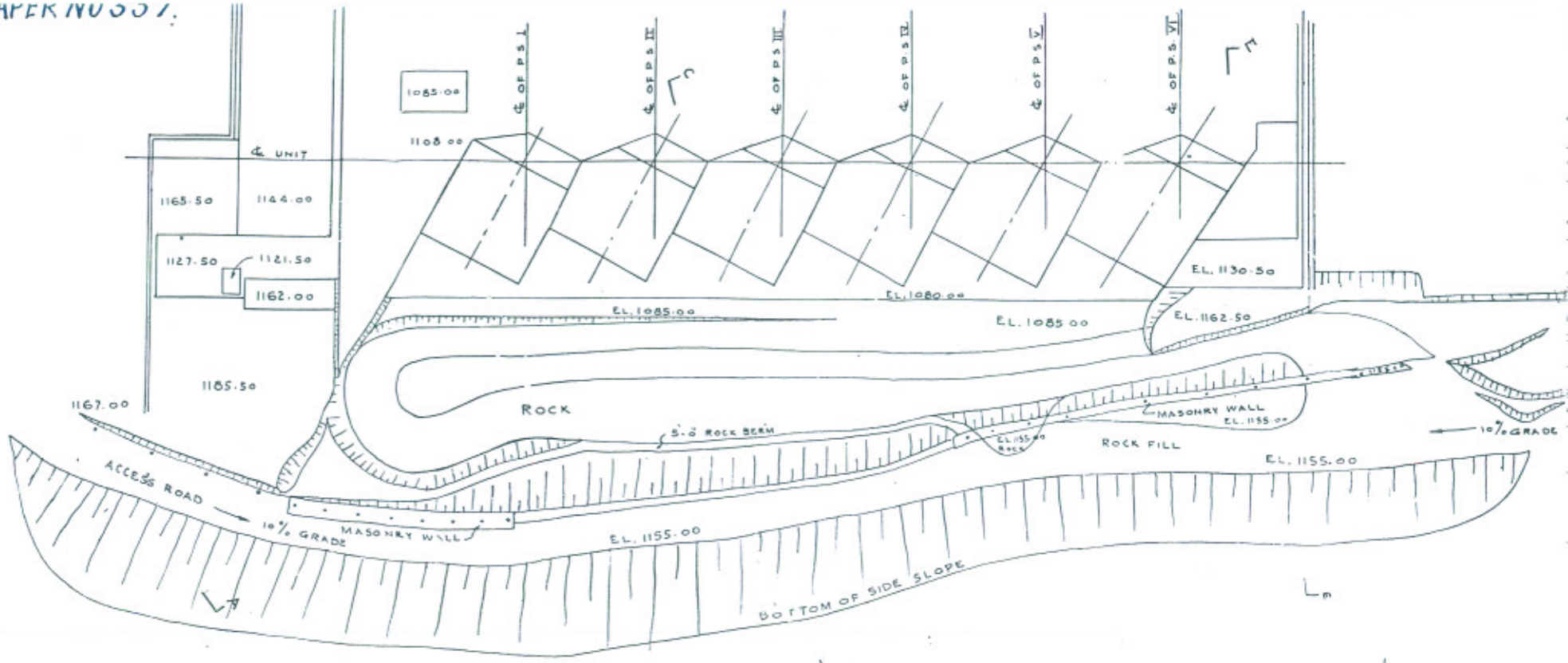


SECTION A-A
SCALE 1"=30'
(STAGE II)



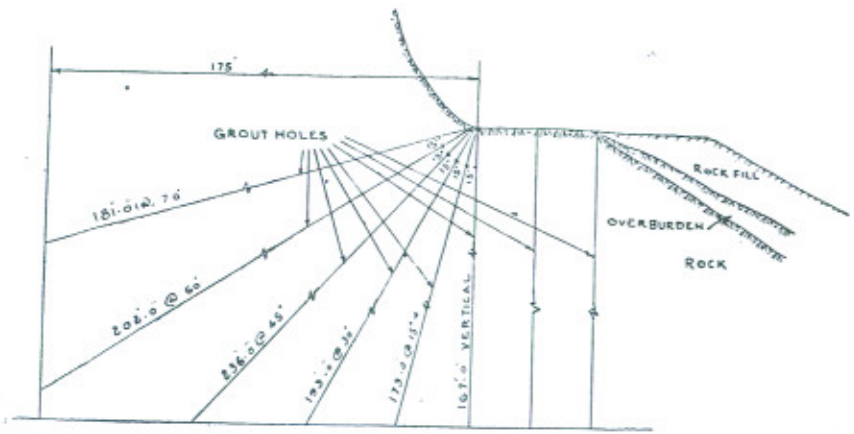
SECTION A-A
SCALE 1"=30'
(STAGE III)

PAPER NO 337.



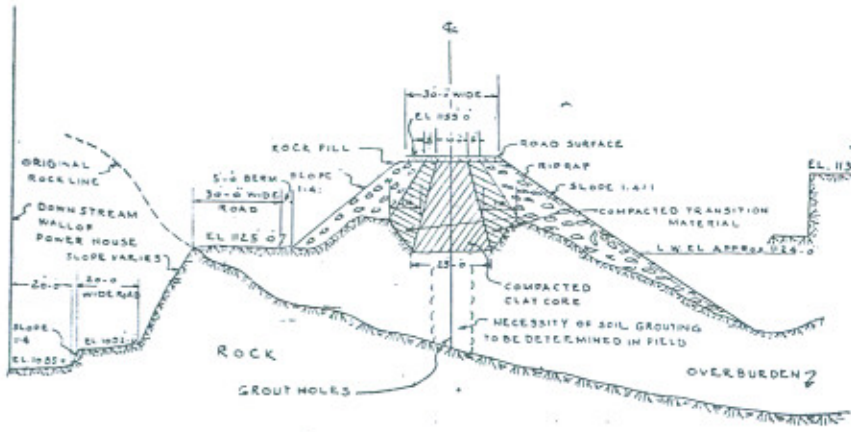
POWER HOUSE COFFER DAM

SCALE:- 1" = 40'



SECTION A.A

SCALE:- 1" = 40'



SECTION C-C

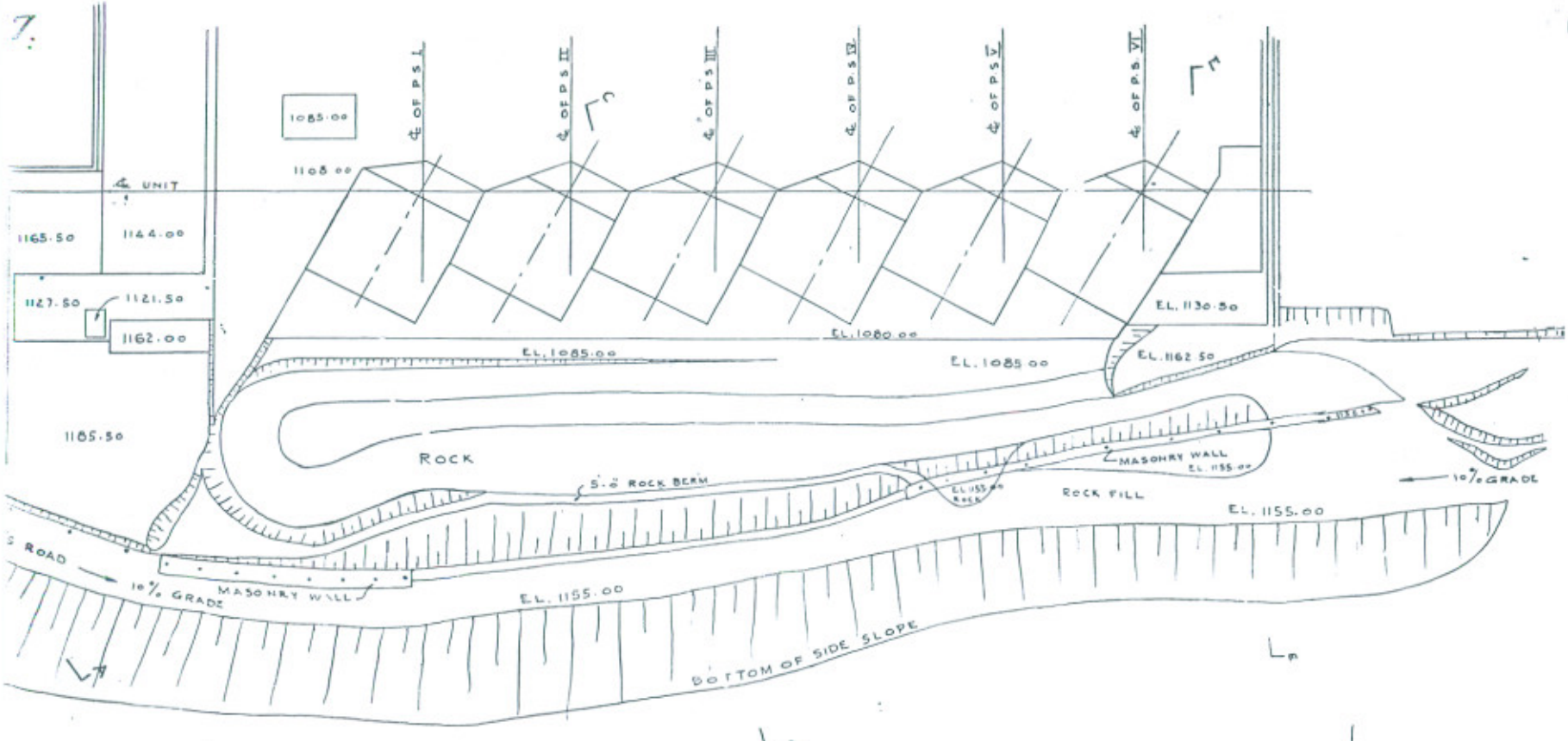
SCALE:- 1" = 40'



SECTION E-

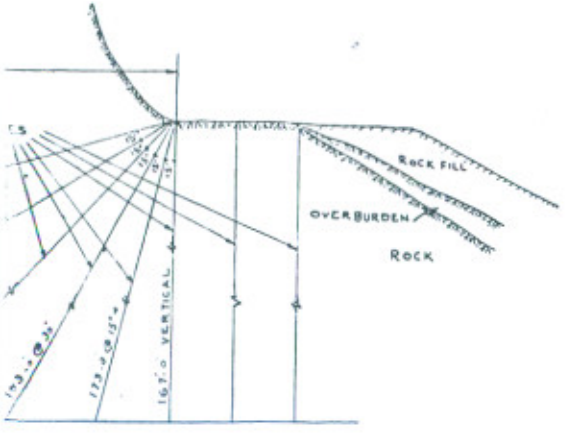
SCALE: 1" = 40'

PLATE No; 7

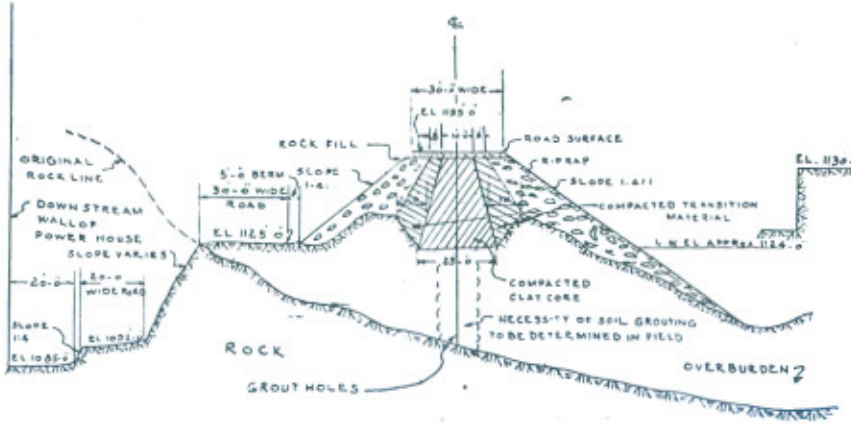


POWER HOUSE COFFER DAM

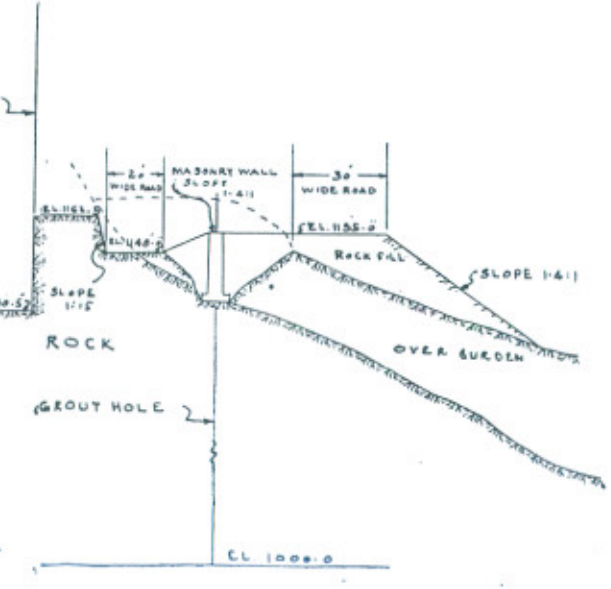
SCALE:— 1" = 40'



ION A.A



SECTION C C



SECTION E E