THE UNIVERSITY OF NEW SOUTH WALES

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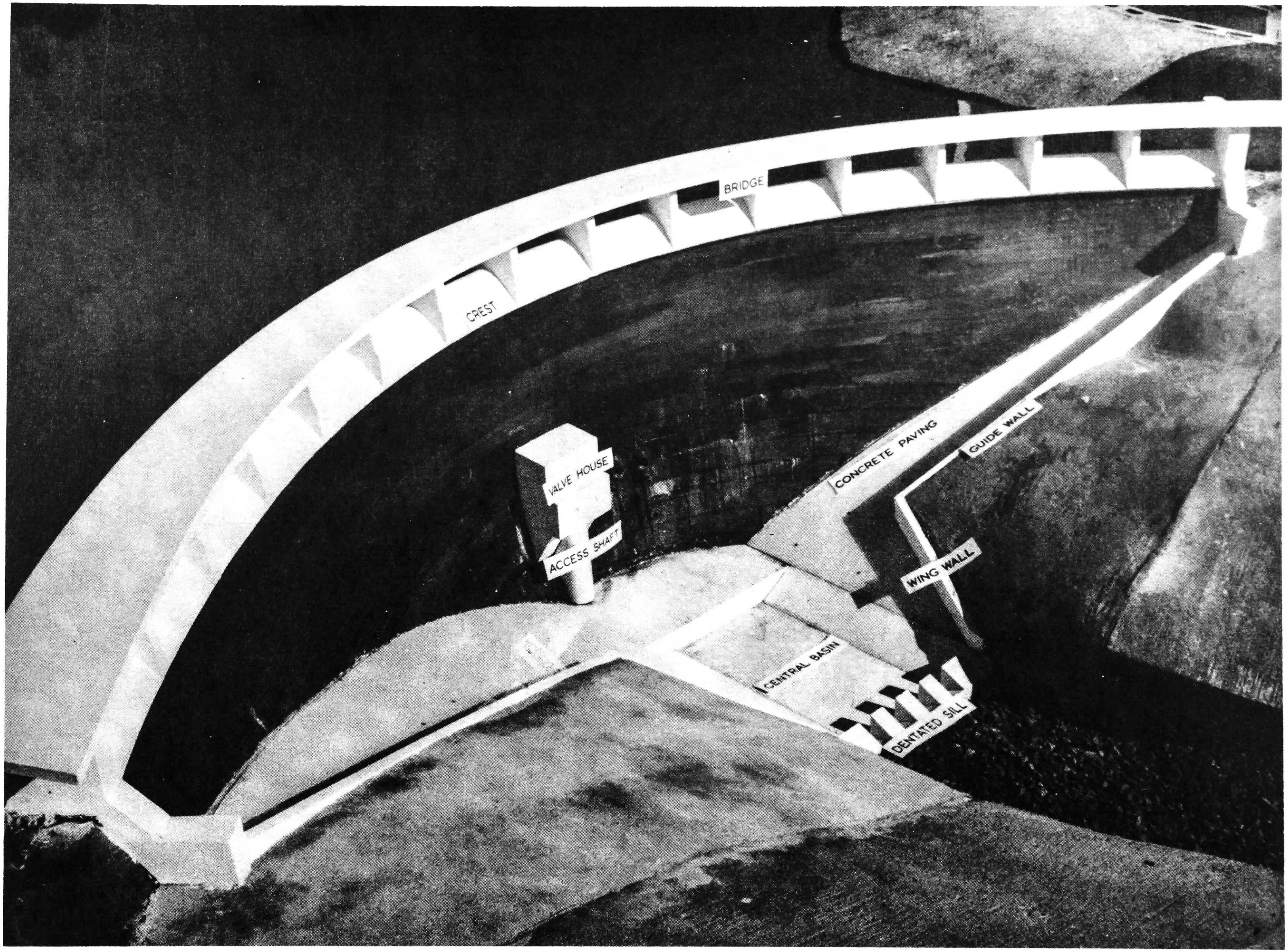
CARCOAR DAM

HYDRAULIC MODEL STUDIES

by

R. C. Nelson, K. K. Lai and I. R. Wood

May, 1968



Carcoar Dam: Model of Final Design.

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Preface

The hydraulic design studies described in this report were carried out for the International Engineering Service Consortium Pty.Ltd., Consultants to the Water Conservation and Irrigation Commission of New South Wales, by arrangement with Unisearch Ltd.

The model studies and associated investigations were carried out by the staff of the Water Research Laboratory under the supervision of Dr. I.R.Wood, Senior Lecturer.

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Summary

The proposed Carcoar Dam is to be constructed for the Irrigation Commission of New South Wales on the Belubula River near Carcoar, N.S.W. Stored water will be used for irrigation. The dam will be a concrete arch dam 140 ft. high with a free overfall crest 600 ft. long, the design discharge being 43,000 c.f.s.

The Water Research Laboratory of the University of New South Wales was requested by International Engineering Services Consortium Pty.Ltd. (Design Consultants to the Irrigation Commission) to execute hydraulic model studies to develop a crest shape and spillway dissipator design. The scope of the studies was bounded by the following requirements of the designers:-

(i) The crest length to be the length of the upper arch.
(ii) The outlet works to be placed on the downstream face of the dam.

This report outlines the procedures of model construction and hydraulic testing used to obtain the following:-

(i)	A satisfactory spillway crest design	Fig. 4.
(ii)	A satisfactory bridge pier shape	Fig. 4.
(iii)	A spillway rating curve	Fig. 12.
(iv)	A satisfactory spillway dissipator design	Fig. 32.
(v)	A satisfactory design for the exterior shape of the valve house and its access structures	Fig. 24.

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Frontispiece: Carcoar Dam: Model of Final Design

1. Introduction

The Carcoar Dam is to be constructed for the Water Conservation and Irrigation Commission of New South Wales on the Belubula River near the township of Carcoar. The stored water will be used for irrigation. The dam will be a thin double curvature arch with a free overflow type crest extending the full length of the parabolic arch. This length is approximately 600 feet. Over the crest of the spillway a roadway will be supported by 14 piers dividing the crest into 15 bays, each about 41 feet wide between pier centre lines. The top of the spillway is at RL. 2780 with the base of the dam at RL. 2640 making a maximum height of 140 feet. Water for irrigation is to be released from valves located in a valve house attached to the lower portion of the downstream wall face. The dam is founded on strong, slightly weathered porphyritic andesite but this is traversed by a shear zone 20 to 30 feet wide running diagonally across the river bed (see Figure 22).

Originally the lower limit of design discharge was determined as 50,000 c.f.s. with no catastrophic damage to occur at discharges up to 75,000 c.f.s. During the existence of this policy two dimensional model studies, described in Section 2, were completed. The design discharges were modified subsequently to 43,000 c.f.s. and 70,000 c.f.s. respectively by International Engineering Service Consortium Pty.Ltd. hereafter referred to as I.E.S.C. On the basis of this policy and at the request of I.E.S.C. sufficient information to compile a preliminary tender design was supplied (see Figure 15). During model testing to settle the final design a further reappraisal was made of the test discharges. A 43,000 c.f.s. flood was in fact the maximum possible discharge. Consequently I.E.S.C. agreed that no further testing should occur for discharges greater than 43,000 c.f.s. and that this flow should remain the design discharge. The final design was developed on this basis.

It was requested that construction and testing of the model proceed with all possible urgency. Therefore, in order to secure the final results as rapidly as possible, it was decided to construct a two dimensional model to investigate:-

(i) the performance of the spillway crest under various discharges;

(ii) the effect of the bridge pier shapes on the performance of the spillway crest;

(iii) the spillway head-discharge relationship;

(iv) the nappe trajectories.

Simultaneous construction of the main three dimensional model was carried out, while the aforementioned two dimensional tests proceeded. This model was first fitted with a sharp edged crest which produced a nappe trajectory similar to that observed on the two dimensional model. This enabled preliminary testing to commence some weeks earlier than would have been the case if the testing had been postponed until the final three dimensional model crest was modelled. Finally, the similarity of the dam to the Cotter Dam (Reference 7) was noted and in order to meet the clients'deadlines for a preliminary design, the initial tests were confined to this type of spillway. Subsequently, more detailed testing confirmed that this type of spillway was a reasonable solution. The details of tests of the three dimensional model are described in Section 3.

2. Two Dimensional Model Tests

The two dimensional tests were carried out by Water Research Laboratory (W.R.L.) staff using facilities made available by the Irrigation Commission of New South Wales in their Hydraulic Laboratories.

2.1 Design of the Two Dimensional Model

From consideration of facilities available and accuracy of the test results required, a model scale of 1 to 50 was chosen.

A length of 117 feet of the crest was modelled. The crest was made straight, instead of having horizontal curvature as in the prototype, thus introducing some flow distortions. The direction and distribution of the approach velocity on the prototype from point to point on the curved crest will vary because the river channel curved, but on the flume model it was uniform and at right angles to the crest. Distortions from varying approach velocity are small and the errors, if any, are smaller than the expected experimental error. Two bridge piers were incorporated on the crest, making a full bay width in the middle of the flume. The crest model was 120 feet (prototype) above the flume floor, which was close to the prototype maximum height of 140 feet.

Water was supplied from a 6-in. main, and the discharge was measured with an orifice meter with flange tappings. The static head behind the crest was measured with a hook gauge. The pressure distribution over the face of the crest was measured with piezometric holes drilled in the crest face and connected to a manometer board. Great care was taken in fitting these piezometric holes to ensure they were both flush and perpendicular to the crest surface.

In determining the pressure distribution over the face of the crest, three prototype discharges, that is, 15,000 c.f.s., 50,000 c.f.s. and 75,000 c.f.s. were used.

The general layout of the model is shown in Fig. 1.

2.2 Shape of the Crest

In the flume test, three crest shapes were used. Details of Crests No.1 and No.2 were supplied by I.E.S.C. while Crest No.3 was a modification of Crest No.2 suggested by the W.R.L.

The downstream face of the first crest (No.1 crest) was a curve described by an equation

$$z = -0.261\ x^{1.75} \quad \text{(in meters)}$$

or

$$z = -0.107\ x^{1.75} \quad \text{(in feet)} \qquad (1)$$

where x = horizontal distance measured positive downstream from the top of the crest,

z = vertical distance measured positive upwards from the top of the crest.

The top of the crest (origin of the co-ordinates) was 6 inches downstream from the vertical axis of the dam. The lip of the crest was 13'6" measured horizontally from this vertical axis. The upstream face of the crest was an arc of 3 feet radius.

For the second crest (no.2 crest), the downstream face was also described by the same equation (Equation 1). The top of the crest (origin of the co-ordinates) was 3 feet and the lip was 17.70 feet. Both measured horizontally from the vertical axis of the dam. The upstream face was made of an arc of 5 feet radius, instead of 3 feet as in the No.1 crest. This change was made by I.E.S.C.

The downstream face of the third crest (No.3 crest) was composed of a curve as described by equation 1, followed by a straight line, having an equation

$$z = -1.207x + 6.21 \quad \text{(in feet)} \qquad (2)$$

This straight line was tangent to the curved spillway surface at 12 feet horizontally from the top of the crest. The use of a straight line for the downstream end of the crest face was an attempt to reduce both the extent and the magnitude of negative pressure near the lip of the crest.

The definition sketch of the three crest shapes is shown respectively in Figs. 2,3 and 4.

When compared with the U.S. Army Engineers Waterway Experimental Station (W.E.S.) standard spillway crest (Ref.2) it was found that the downstream crest profile as described by Equation 1 was close to that of the WES-standard crest when the design head of the latter was 8 feet. It can be seen from this that positive pressure would occur in most parts of the spillway for a head lower than 8 feet and negative pressure would develop for a head higher than 8 feet. The comparison of the shapes of the No.3 crest and the WES-standard crest is shown in Fig. 5.

2.3 Pressure Distributions on Crests

2.31 No.1 Crest

The pressure distribution on the face of the No.1 crest is shown in Fig. 7. At 75,000 c.f.s. the crest face was subject to negative pressure, except on the very upstream face and a region from 8 ft. to 11 ft. measured horizontally downstream from the top. A maximum negative pressure of -2.5 ft. of water was obtained at the top of the crest and a negative pressure of -2.0 ft. of water was obtained near the lip.

At the discharge of 50,000 c.f.s, the negative pressure near the top of the crest was only about -0.4 ft. and it was again about -2.0 ft. near the lip.

At 15,000 c.f.s. the pressure on the face was mostly positive, except near the lip. The pressure measured there was -1.3 ft.

2.32 No.2 Crest

The No.2 Crest differed from the No.1 Crest in that the circular arc that formed the upstream face had a larger radius of 5 ft. instead of 3 ft. and that the length of the crest downstream from the top was 14.7 ft. instead of 13 ft. measured horizontally.

At 75,000 c.f.s. the negative pressure near the top of the crest, compared with the No.1 crest, was slightly reduced, being -1.2 ft. Negative pressure also existed near the lip. However, a region of positive pressure was obtained between 10 ft. and 12.5 ft. measured horizontally downstream from the top of the crest, but the magnitude of this pressure was small, being less than + 0.2 ft.

At 50,000 c.f.s. and below (e.g. 15,000 c.f.s.) positive pressure was obtained at all pressure tappings, but it can be seen that negative pressure existed near the lip in both cases.

The pressure distribution for No.2 Crest is shown in Fig. 8.

2.33 <u>No.3 Crest</u>

In order to reduce the negative pressure near the lip, and to increase both the magnitude and the extent of the positive pressure region upstream of the lip, the No.2 Crest was modified by using a straight line for the downstream face near the lip. This straight line was tangent to the curved crest face at 12 ft. (measured horizontally). With a straight downstream face, instead of a curved one, the level of the lip was raised by about 0.3 ft., but the crest face in this region had a zero curvature.

Fig.9 shows the pressure distribution over the spillway face of the No.3 Crest.

At 75,000 c.f.s. negative pressure still existed near the lip, but it was confined only to a region of about 1 ft. measured horizontally near the lip. Upstream of this region, there was a region of positive pressure covering a length of 6 ft. (measured horizontally), with a maximum pressure of +0.7 ft.

At 50,000 c.f.s. and 15,000 c.f.s., positive pressure was obtained for the whole crest, except in a small region near the lip.

When comparing the pressure distribution of the No.2 Crest with that of the No.3 Crest, as shown respectively in

Figs. 8 and 9, attention must be drawn to the fact that an additional piezometric hole was drilled on the No.3 crest further downstream, and closer to the lip on the crest face, where negative pressure was obtained.

2.4 Effect of Bridge Piers on Spillway Performance

When the spillway is in operation, an atmospheric pressure should be maintained underneath the nappe. Inadequate aeration could result in vibration of the nappe. Aeration will be assured on Carcoar Dam if the bridge piers act as flow splitters so that the nappe is clearly divided between each bay. The effectiveness of the bridge piers as splitters depends on their size and shape. The width of each pier was fixed at 4 feet by the I.E.S.C. with a 2 foot radius upstream.

In the original design (pier shape No.1 on Crest No.1 - Figure 2) the pier was tapered downstream from the crest. The pier terminated 1 foot upstream of the crest lip with a 1 foot radius at its end. With this arrangement the nappe reunited downstream of the pier preventing air penetration underneath the nappe. Nappe vibration occurred (Figure 6) due to resonance with the elastic compression of the enclosed air between the nappe and the dam. A simple means of preventing this undesirable vibration was to split the nappe as shown in Figure 10. This phenomenon occurred only at low flows when the nappe was thin. At high flows, when the inertia of the nappe became appreciable, the vibration was not noticeable.

Several sizes and shapes of the bridge piers were made and used in the tests in an attempt to have the nappe to split clearly both at low and at high flows. The piers that were used are shown in Figs. 2, 3 and 4, together with the crest shapes. Of these piers, the one with square edges at its downstream end and extended to the lip of the crest proved to be the most satisfactory (Pier Shape No.4 on Crest No.3). Figure 11 shows Crest No.3 with Pier Shape No.4 on the left and Pier Shape No.3 on the right. This figure shows that the nappe was split after leaving Pier Shape No.4.

2.5 Head Discharge Relationship

Head-discharge measurements were made for the three crests with different pier shapes. The results are shown in Fig. 12. In Fig. 12 it can be seen that there is no significant difference in the head-discharge relationship among the three crests and the pier shapes used. Since the major difference between pier shape 3 and pier shape 4 is above the water line the results for pier shape 3 can be applied to pier shape 4.

Discharge coefficients were computed for the recommended crest (No.3) using the following equation -

$$C = \frac{Q}{LH^{1.5}} \qquad (3)$$

where

C = discharge coefficient
Q = discharge per bay in c.f.s.
L = clear span between two piers
H = measured head above crest in ft. velocity head being negligible.

The computed discharge coefficients for Crest No.3 are plotted in Fig. 13. A curve was drawn through these points. A rating curve was then computed for a given head with the corresponding discharge coefficient obtained from Fig. 13. The computed rating curve is shown in Fig. 12.

In Fig.12, the abscissa shows the discharge per foot width of the crest in c.f.s. per foot. In addition, the total discharge over the spillway is also shown. In calculating the total discharge, the unit discharge was multiplied by a constant equal to 15 x 37.096. The figure 15 is the number of bays, and 37.096 is the clear span of one bay, in feet.

2.6 Nappe Trajectories

Trajectories of the upper nappe were measured at three discharges, i.e. 15,000 c.f.s., 50,000 c.f.s. and 75,000 c.f.s. on Crest No.3.

The flow surface above and upstream of the crest was measured with a point gauge and the upper nappe downstream of the crest was obtained from photographs. The trajectories of the upper nappe are shown in Fig. 14.

At 75,000 c.f.s., it can be seen that the jet falls at about + 60 ft. measured horizontally from the top of the crest at RL. 2640.

2.7 Discussion of Results

Crest No.3 is considered to be the better design so far as nappe stability over the crest is concerned. A small region of negative pressure exists near the lip but its magnitude is small and is preceded by a large region of positive pressure even at 75,000 c.f.s.

In order to have the nappe split clearly after leaving the crest, the length of the bridge piers should be extended to the lip of the crest with a constant width and have square downstream edges. At the downstream edge the pier shape can be other than square above the level of the upper nappe for the maximum possible discharge. Figure 11 can be used as a reference for this purpose.

The head discharge relationship obtained from the two dimensional model was later compared with that obtained from testing the three dimensional model as described in Section 3 of this report. They agreed to within one per cent which is well within the experimental accuracy.

2.8 Conclusions

As a result of these studies, Crest No.3 and Bridge Pier Shape No.4 (see Figure 4) were adopted for use in the dam design.

3. Three Dimensional Model Studies

These hydraulic studies were to determine an apron and reception basin arrangement which would return over-flowing waters to the river with maximum energy loss and without undue damage to:-

(i) exposed rock surfaces of the valley sides,
(ii) the river bed downstream of the dam,
(iii) the valve house on the downstream face of the dam.

3.1 The Three Dimensional Model

3.11 Model Scale

To minimise the scale effects of those forces which could not be adequately accounted for on the Froudian model (e.g. surface tension and friction) it was desirable for scales to be as large as possible. Considerations of space, water supply and finance available determined 1:50 as the optimum model scale. For this scale thefollowing scale ratios were applicable:-

Velocity scale	=	1:7.07
Time scale	=	1:7.07
Discharge scale	=	1:17, 680
Head scale	=	1:50

Surface tension and friction cause discrepancies between the behaviour of the model and the prototype. This was particularly evident at low model flows by the contraction of the nappe (Figures 21 and 34) a surface tension scale effect. Such contractions will not occur at the corresponding proto-type flows.

3.12 Extent of the Model

It was necessary to model portion of the river upstream and downstream of the dam so that both approach and tailwater conditions could be adequately reproduced.

The only known flood stage-discharge relationship existing near the dam was for a section 3,200 feet downstream of the dam.

This was supplied by the Water Conservation and Irrigation Commission. The curve was based on flood slope, area calculations using observed marks of past floods. Hydraulic computations indicated the existence of supercritical flow at this point, a complication if backwater computations to obtain flood levels at the dam site are required. Such computations were not considered necessary because a sharp, steep sided bend downstream of the dam (see Fig.15) provides a measure of tailwater control. The model therefore was built far enough downstream of this bend, to preserve the effect of the bend.

The model operation demonstrated that, for the range of flows to be tested, supercritical flow existed in the channel immediately downstream of the dam with control forming at the sharp bend.

Sufficient dam storage was constructed to adequately model inlet conditions.

The complete layout of the model is shown in Figure 15.

3.13 <u>Model Construction</u>

The method used for shaping the topography is shown in Figure 16. Masonite templates were nailed to a 4 inch concrete base (8:1 sand cement mix). The boundaries of the model were enclosed by concrete blockwalls (blocks 16" x 8" x 3"). An 8:1 sand, cement mix was used as bulk filling to bring levels to within one inch of the template tops. The surface was then sealed with a 3:1 sand cement mortar about one inch thick. The mortar cap was artificially roughened by brushing with a stiff broom.

A section of the river bed downstream of the dam was cored out and filled with gravel for use as an indicator in scour studies.

A computer was used to compute the dam co-ordinates and the dam wall was modelled in laminated timber using horizontal sections (Fig. 17) and sheathed in fibre glass.

Preliminary studies were done with a sharp edged metal crest (Fig. 18) while the exact crestfor the final tests was being constructed in concrete.

3.14 Flow Measurement

All flows were measured by orifice meter. A 6.30 inch standard orifice plate located in an 8 inch (nominal) diameter pipe with D and D/2 tappings was used.

This orifice meter was capable of accurately measuring flows as low as 10,000 c.f.s. prototype.

3.2 Model Studies to Develop a Preliminary Design

3.21 General

The W.C. and I.C. requirements to have tender drawings available by 31st December 1967 made it advisable to draw as much as possible on knowledge gained from hydraulic model studies of similar types of arch dams.

The proposed Carcoar Dam is similar to the existing Cotter Dam on the Cotter River in the Australian Capital Territory. A similar apron and guide wall arrangement should therefore provide a satisfactory solution. The arrangement is simple and easy to construct. Hence much of the knowledge gained from hydraulic model studies on the Cotter Dam was exploited in designing the spillway dissipator of the Carcoar Dam. The one feature that necessitates variation from a Cotter type solution was the proposal for a valve house on the downstream face of the Carcoar Dam in the central basin area. The Cotter valve house is built downstream of the dissipator area, out of the influence of spillway flows.

Paving slopes were adopted such that, for the survey information then available, a minimum of excavation was involved. The two pavings were made plane surfaces. The selection of the adopted paving slopes conformed to natural slopes with little variation possible.

A systematic series of model tests was executed to determine the most satisfactory guide wall height and angle. A wall about 8 feet high making an angle of 50 degrees with the paving produced the best results (Fig.19). Walls of lower height allowed water to splash onto the unprotected valley slopes. The wall angle adopted effectively deflected the water back onto the falling nappe. The deflector walls were most effective when placed about 4 feet downstream of the falling nappe, but a larger spacing was required at the lower end of the apron to accommodate the greater concentration of water arriving from higher up the slope. Walls placed too far behind the nappe increased splash onto the unprotected areas. Since the falling water is farthest from the dam for the maximum discharge, the position of the wall was fixed by the location of the falling nappe for the design discharge of 43,000 c.f.s.

The adopted level of the central basin (RL. 2640) was that which required a minimum of excavation. The adopted basin width of 65 feet approximated the river bed width. The length of the basin (100 feet) and the height of the downstream sill (8 feet) were determined from scour studies.

3.22 <u>Preliminary Scour Studies</u>

Initial tests were done using a sharp edged metal crest. This crest was designed to simulate the nappe trajectory for 43,000 c.f.s. only.

The bed at the dam site is strong, slightly weathered (up to 20 feet) porphyritic andesite traversed obliquely by a shear zone located in a scour prone area immediately downstream of the dam wall. This shear zone is 20 to 30 feet thick, changing progressively to sound rock at about 40 feet either side of the axis of the zone. With such variable prototype bed materials, movable bed scour studies will provide only an indication of scouring tendencies. Nevertheless such indications will be conservative. For the comparison of various alternatives, the movable bed, when subject to the design discharge, had to give a measurable scour hole in a reasonably short time and at the end of the testing time should have reached a stage where further changes in the scour hole were small. These requirements were met by ½ inch to ¾ inch gravel (2 ft. to 3 ft. prototype) and by a testing time of 30 minutes (3½ hours prototype).

The series of tests was commenced with the absolute minimum of basin length (50 ft.) and this was added to as necessary to improve the scour pattern. The first test was conducted using no sill. A large hole developed at the end of the basin. The basin length was gradually increased using various sill heights for each length. The most satisfactory arrangement was a basin length of 100 feet with an 8 foot vertical sill. Wing walls, 8 feet high, joined the end of the guide walls to the end of the basin sill. The scour developed by this arrangement was considered acceptable.

The moulded crest shaped from two dimensional studies was then fitted to the dam so that scour profiles could be observed for a range of discharges namely 20,000, 20,000, 43,000, 60,000 and 70,000 c.f.s. (Fig. 20). These tests showed no serious erosive tendency up to the design discharge (43,000 c.f.s.). No catastrophic scour occurred for flows up to 70,000 c.f.s.

The model operation of the preliminary design is shown in Figure 21 while the plan of the design is given in Figure 22.

3.23 <u>Further Work Required</u>

It was realised that a considerable amount of further work was necessary to:-

(i) refine the design
(ii) investigate the effect of turbulence on the valve house
(iii) investigate possible protection of the valve house.

Nevertheless, International Engineering Service Consortium Pty.Ltd. were satisfied that the paving and basin arrangement was sufficiently refined for use as a tender design (Fig. 22).

3.3 <u>The Valve House</u>

3.31 <u>Valve House Location</u>

When testing commenced (25.11.67) the valve house was attached to the downstream face of the dam in block 6 just to the left of the dam centre line. The centre lines of the pipes were at RL. 2680. On 29th November 1967, I.E.S.C. directed that this location be changed to block 7 just to the right of the dam centre line.

Between 4th and 8th December, some tests were carried out with the valve house raised 15 feet to lift it out of the turbulent water. Subsequently, it was agreed to raise the valve house only 4 feet, placing the centre line of the valves at RL. 2684. The assembly remained in block 7.

3.32 Water Condition at the Valve House of the Preliminary Design

The preliminary design described in Section 3.3 was obtained without consideration of the valve house. Additional model studies demonstrated that, for the valve house of the preliminary design (Figure 23) the valves were inundated by turbulent water at flows near 43,000 c.f.s. Further for all flows, the top of the access gallery directed high velocity water into the right side of the valve house. Figure 21 illustrates these conditions. Table 1 gives a summary of the various items of interest for the valve house.

Table 1: Conditions at Valve House of Preliminary Design

Discharge (c.f.s.)	Measured velocity of water impinging on side of valve house (f.p.s.)	Estimate of force exerted on side of valve house (tons)	Height of water jump at side of valve house (ft.)	General water level at valve house Prototype (ft.)
10,000	-	-	RL.2689	-
20,000	35	30	RL.2690 to RL.2693	RL.2671
30,000	46	50	RL.2692 to RL.2696	RL.2675
43,000	52	130	RL.2696 to RL.2700	RL.2682

In Figure 21 a spillway bay can be seen blocked. This was done for the 43,000 c.f.s. test to obtain better conditions for observation. This has slightly lowered the water level behind the nappe. Under normal model conditions, with all spillway bays operating, the water level would approximate the invert of the outlet pipes. On the prototype, the water level will be higher still, since, on the model, surface tension forces cause the width of the nappe to narrow as the nappe falls. This effect will be negligible on the prototype. The lack of gaps, therefore, will cause the water level at the valve house to be higher on the prototype than on the model. An estimate of the magnitude of this effect was obtained by removing the 5 central piers and comparing the water level with that when the piers were in position. It is estimated that water levels behind the nappe will be 4 feet higher in the prototype than in the model. This correction has already been applied to the levels given in the last column of Table 1.

3.33 Final Valve House Design

A wide range of model testing was executed in an attempt to eliminate the undesirable conditions existing at the valve house of the preliminary design. A summary of this testing is given in Appendix A. It was found that most of the undesirable conditions could be eliminated if the horizontal access gallery was removed. Consequently I.E.S.C. modified the geometry of the structure. The valve house was made smaller and the access gallery replaced by a vertical shaft (see Figure 24). This arrangement when tested on the model operated satisfactorily.

3.4 Modifications Leading to Final Dissipator Design

3.41 The Central Basin

During discussions between the Water Research Laboratory, International Engineering Services Consortium Pty.Ltd. and the Water Conservation and Irrigation Commission of N.S.W., the opinion was expressed that the shear zone near the downstream right corner of the preliminary design basin could have more positive protection. It was agreed that the testing should include -

(i) the basin lengthened to cover this zone and the model tested,
(ii) the basin deepened about 10 feet and the model tested,

(iii) the foregoing designs tested with both dentated sills and straight solid sills.

The most satisfactory model operation was obtained by depressing the lengthened basin to RL.2630 and using a 10 foot Rehbock type dentated sill. Figure 31 gives a comparison of the results. The scour tendencies of the preliminary design were improved and the water levels at the valve house lowered. The water level of RL. 2682 given in Table 1 for the 43,000 c.f.s. flow was reduced to RL. 2678.

A further modification to the basin was made when additional survey information on rock levels was available after stripping operations. The basin was widened from 65 feet at RL.2640 to 84 feet.

3.42 <u>The Sloped Pavings</u>

Additional survey information available after stripping of the abutment areas led to the following changes in the preliminary design.

(i) The left paving remained a plane surface with the same slope but was lowered about 7 feet.

(ii) The right paving remained almost the same as that tested on the preliminary design but with the addition of two 800 feet radius vertical curves. The radii of these curves are large enough so that there is no possibility of cavitation effects.

3.43 <u>The Guide Walls</u>

(i) <u>The Right Guide Wall</u>: During testing of the preliminary design, concern had been expressed at the amount of splash from the right guide wall landing on the downstream valley slopes. The authors are confident that this was caused by a surface tension model effect and would not be as severe on the prototype. Nevertheless it was agreed that the lower end of this guide wall should be raised in height by 2 feet. This eliminated the splash from the model. The 50 degree wall angle of the preliminary design was maintained.

(ii) <u>The Left Guide Wall</u>: The 8 foot guide wall height remained unchanged. However, testing of the final design proved the 50 degree wall angle to be unsatisfactory apparently because the paving was lowered in this area. The guide wall turned the water to sharply causing it to collide with the nappe in such a way that excessive splash landed downstream of the wall. This problem was eliminated by adopting a 60 degree wall angle.

In addition, in order to obtain optimum operating conditions, some minor repositioning of both guide walls was made with respect to their distance from the downstream face of the dam.

3.5 Model Tests on the Final Design

The final design, incorporating all those modifications described in Section 3.5, is shown in Figure 32. Having fixed the spillway geometry, the paving, guide walls, central basin, sills etc. were accurately modelled and scour tests run for discharges of 10,000 c.f.s., 20,000 c.f.s., 30,000 c.f.s. and 43,000 c.f.s. As in the preliminary scour tests ½ inch to ¾ inch gravel was used (2 to 3 feet prototype) with a testing time of 30 minutes (3½ hrs. prototype). The resulting scour patterns are shown in Figure 33 while the model operation is shown in Figures 34, 35, 36 and 37. A full hydrograph corresponding to the design flood over the spillway (43,000 c.f.s. peak) was also passed through the model. The resulting scour pattern was almost identical to that obtained when a steady 43,000 c.f.s. flow of duration 3½ hours (prototype) was used.

It should be noted that no gravel was brought into the basin at any flow thus ensuring that concrete of the prototype will not be damaged by boulders thrown around by the turbulent action in the stilling basin.

The rating curve obtained from the two dimensional model was checked against the three dimensional model. Agreement to within one percent was found. This is within the experimental accuracy.

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2. Chow, Ven Te "Open Channel Hydraulics" McGraw Hill 1959.

3. Hinds, Creager and Justin "Engineering for Dams" John Wiley and Sons 1945.

4. International Engineering Service Consortium Pty.Ltd. and others "Carcoar Dam Study" August 1967: A report to the Irrigation Commission of N.S.W.

5. Mary, M. and Derobert, L. "The Couesque Dam on the River Truyere and the Enchanet Dam on the River Maronne" Fifth International Congress Large Dams, Suppl. to the Review, Travaux No. 247.

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7. Wood, I.R. and Kearsley, B.V. "Upper Cotter Dam Spillway Model Studies" Snowy Mountains Hydro-Electric Authority Fluid Mechanics Report No. S.H.18.

Acknowledgements

The authors wish to thank Associate Professor R.T.Hattersley and the staff of the Water Research Laboratory for the help given them during the course of this work.

The authors appreciate the assistance of the Irrigation Commission for the facilities made available in their hydraulic laboratory to test the two dimensional model and in particular, the co-operation of Mr.D.Watts.

The authors also wish to acknowledge the contributions made, during the course of the study, by the following persons: Messrs. Camilleri, Borde, Gallea, Oberti, Pacquant, Aude of International Engineering Services Consortium Pty.Ltd. and Mr.B.Cantwell of Sinclair and Knight.

Appendix A.

Valve House Protection Studies

A summary is given below of the testing carried out in an attempt to eliminate the undesirable conditions existing at the valve house of the preliminary design. These tests were carried out before I.E.S.C. decided to:-

(i) remove the side access gallery to the valve house,
(ii) reduce the size of the valve house.

These decisions and the subsequent lowering of the central basin removed the necessity of valve house protection measures.

(a) Closing Two Central Crest Bays

The closing of two central crest bays creates a large gap in the nappe through which water could escape freely from behind the nappe. It is estimated that this would lower water levels at the valve house by about 4 feet. The problem of dribble flows impinging on the valve house would also be eliminated.

(b) Paving Deflectors

Various deflector walls were tested for their suitability in deflecting high velocity water jets away from the valve house. The deflectors tested could be classed into two groups:-

(i) plain,
(ii) curved.

(i) Plain Deflectors: Figure 25 illustrates examples of deflectors tested. Their function is to divert from the valve house high velocity water coming down the paving behind the nappe. All the deflectors, with varying degrees of efficiency, reduced turbulence near the valve house and deflected high velocity water from the valve house. Insignificant changes in water level at the valve house were effected. The deflectors had to have a maximum height of at least 18 feet to 20 feet to be satisfactory. The valve house is located on the right side of the central basin and tests showed that only one deflector on the right side was necessary. Also, it is the right side of the valve house which has the access gallery, the source of most water jetting problems. There was sufficient deep water on the left side of the valve house to absorb the energy of high velocity water coming down behind the nappe.

(ii) Curved Deflectors: One of these deflectors placed against the dam wall, projecting outward from the dam wall and down the slope (Figure 26) offered better valve house protection than any of the plain deflectors. It alleviated the problem of high velocity water and reduced turbulence but did not lower the general water level.

Two preliminary deflector designs were obtained (Figure 27), one being capable of diverting water from the valve house for all flows up to 20,000 c.f.s. and the other for diverting flows up to 43,000 c.f.s.

(c) Horizontal Door Protector

A horizontal projection attached to the side of the valve house below the proposed steel door was tested (Figure 28). This was effective in removing high velocity water from the region of the door.

(d) Guide Walls against Dam

Guide walls against the dam wall similar to those downstream of the spillway paving were tested. These failed to alleviate any of the undesirable conditions at the valve house.

(e) Multiple Baffle Blocks

A series of straight baffle blocks down the paving, 4 feet and 8 feet high, were tested using the various positions shown in Figure 29. No satisfactory arrangement of 4 foot blocks could be achieved. They were ineffective in either preventing high velocity water from hitting the valve house or in lowering the water level. Turbulence was slightly reduced.

Eight foot high blocks arranged in position (1) as shown

29(b), were reasonably effective in preventing high velocity water from hitting the valve house. Some reduction in turbulence was achieved but the water level at the valve house was unchanged. Blocks arranged in position (2) tended to force water over the top of the downstream guide walls splashing large amounts of water onto the unprotected valley slopes.

Blocks placed against the guide wall (Fig.29a) also splashed large amounts of water onto the unprotected valley slopes.

(f) Couesque Type Spillway

The Couesque Dam has three "ski jump" type chutes on each sloped paving from which the water is projected into the river basin. Similar arrangements were tested on the Carcoar model (Figure 30). The problem of high velocity water hitting the side of thevalve house was not alleviated. Water levels at the valve house could only be reduced by spilling water onto the unprotected rock slopes and back into the river downstream of the central basin proper. When the jumps were arranged so that all of the water was contained within the paved shute area and central basin the performance was similar to the simpler preliminary design with straight guide walls.

(g) Conclusions Drawn from Tests

From model tests of the preliminary design and of modifications thereto the following conclusions were drawn:-

(i) Water level at the valve house: If no water can be spilt onto the unprotected valley slopes downstream of the paving, no reduction in water level about the valve house of the preliminary design can be achieved.

(ii) Turbulence: Turbulence at the valve house could be reduced, but not eliminated, by using curved deflectors of the type shown in Figures 26 and 27.

(iii) High velocity water hitting the valve house: High velocity water behind the nappe is directed, by the access gallery on to the side of the valve house. The problem resolves itself into two parts -

(a) Were the forces exerted on the side of the valve house considered structurally intolerable?

(b) If these forces were tolerable, does the height to which the water jumps after hitting the side of the valve house create a danger to the proposed side door of the valve house.

The problems of both (a) and (b) can be overcome by the use of a curved deflector. If the forces exerted on the valve house were of little consequence and protection of the side door is primary, the horizontal door protector was a satisfactory solution.

Although no simple solution was found to lower the water level at the valve house, it was obvious from Table 1 that the valves themselves were only subject to inundation at flows near 43,000 c.f.s. The design flow of 43,000 c.f.s. was also the maximum possible discharge. Therefore, at this flow, it was considered that substantial damage to the valve house should be tolerable, short of there being danger to human life. It was noted that authorities, such as the Irrigation Commission, would not allow personnel to enter the valve house during major floods.

If the valve house design accommodates the relatively small forces exerted on its right side and if the possibility of some damage to the side door is acceptable, the valve house and access gallery of the preliminary design was satisfactory for flows up to 30,000 c.f.s. This conclusion was reinforced by the fact that 30,000 c.f.s. has a return period considerably in excess of 1000 years.

The valve house and access gallery of the preliminary design were completely safe for flows up to 10,000 c.f.s. (10,000 c.f.s. has a return period of about 800 years) without the addition of any protective works. At these flows the valve house stood well clear of the basin water level and the water impinging on the side of the valve house failed to reach the level of the proposed steel door.

It was the opinion of the Water Research Laboratory that, on the bases of accepted engineering practice and economy, the valve house and access gallery of the preliminary design were satisfactory. Nevertheless, it was realised that in the final analysis the decisions on design criteria rest with I.E.S.C. If I.E.S.C. required the valve house to be completely safe for flows up to 43,000 c.f.s. then some modification of the tender design arrangement was necessary.

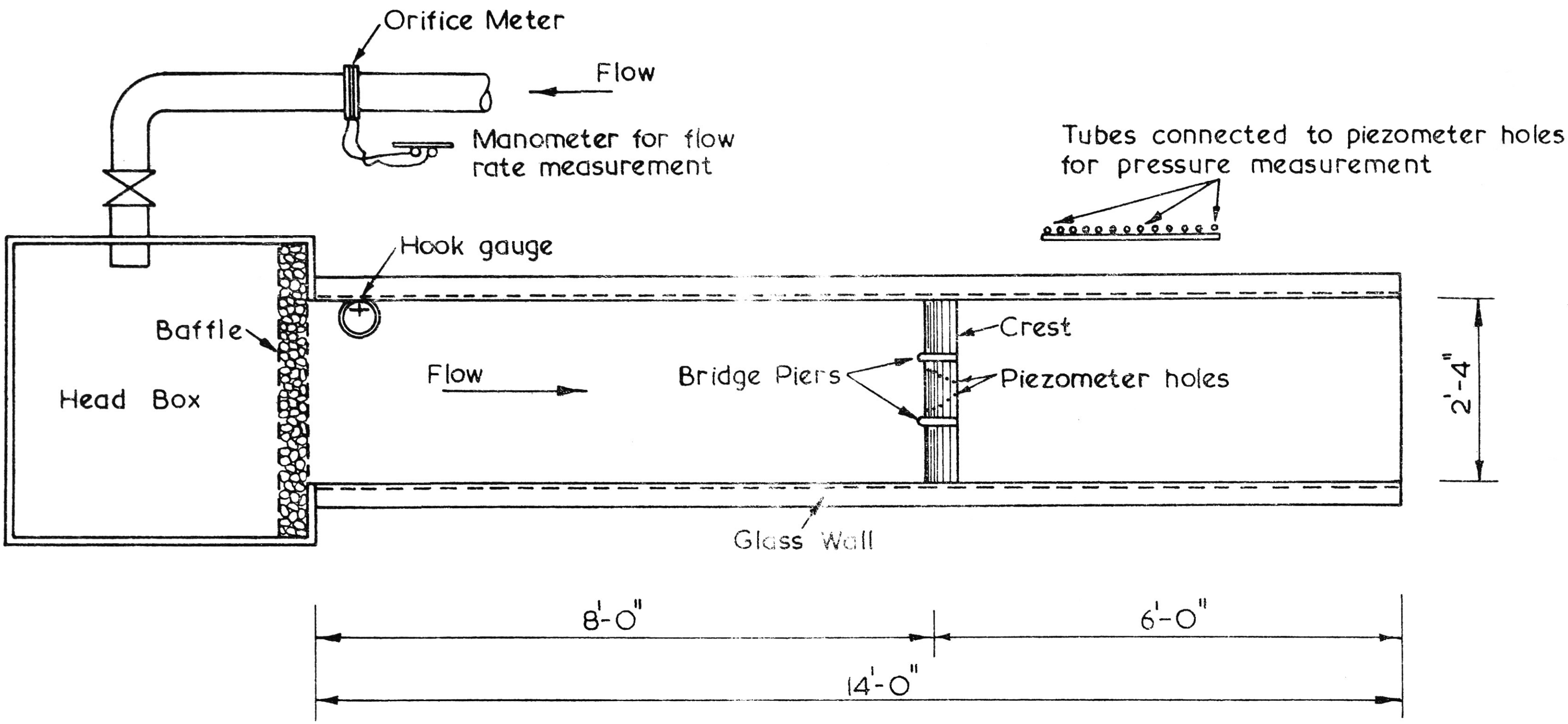

FIGURE 1: GENERAL LAYOUT OF TWO DIMENSIONAL MODEL

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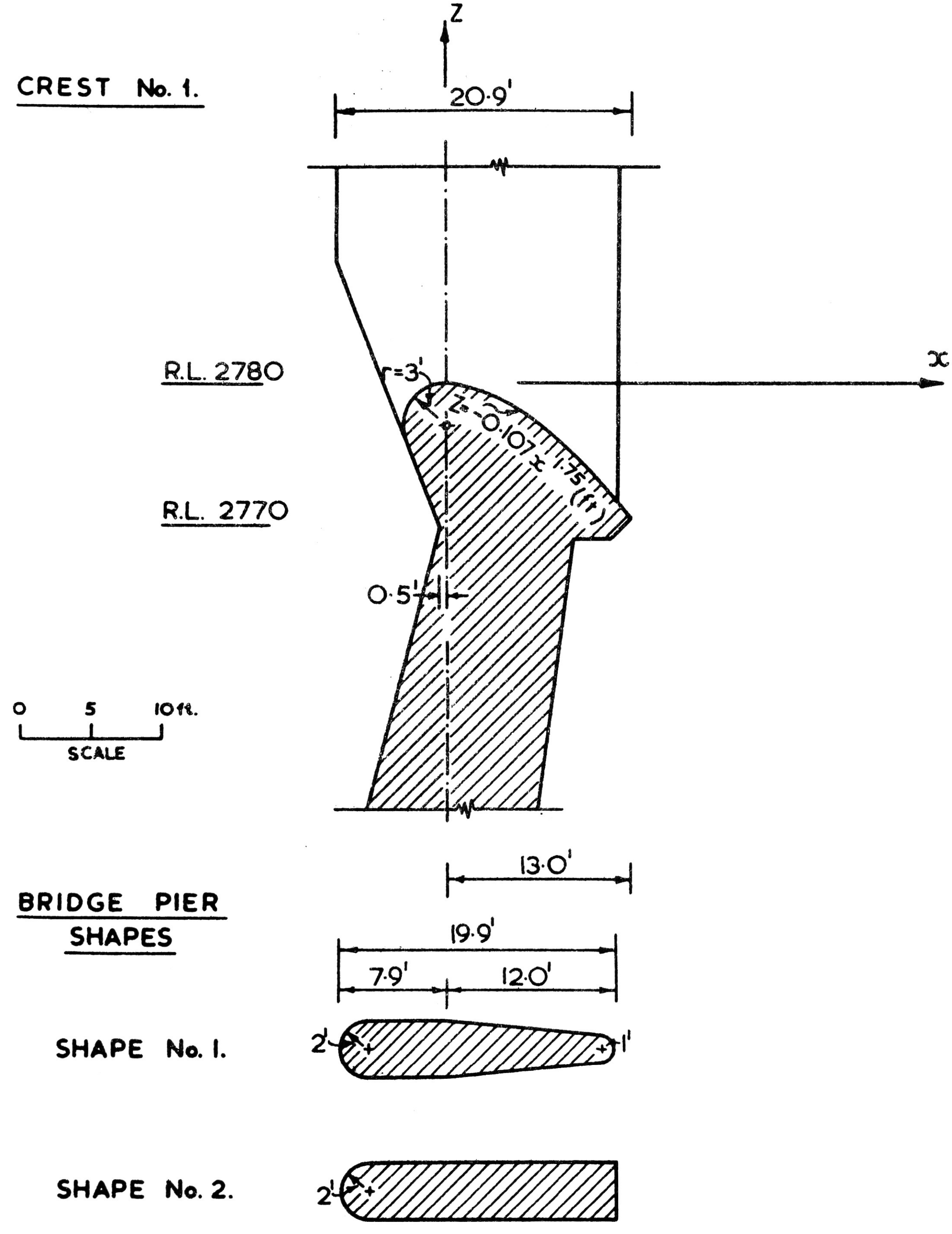


FIGURE 2: DEFINITION SKETCH - CREST No. 1.

CE-E-7339

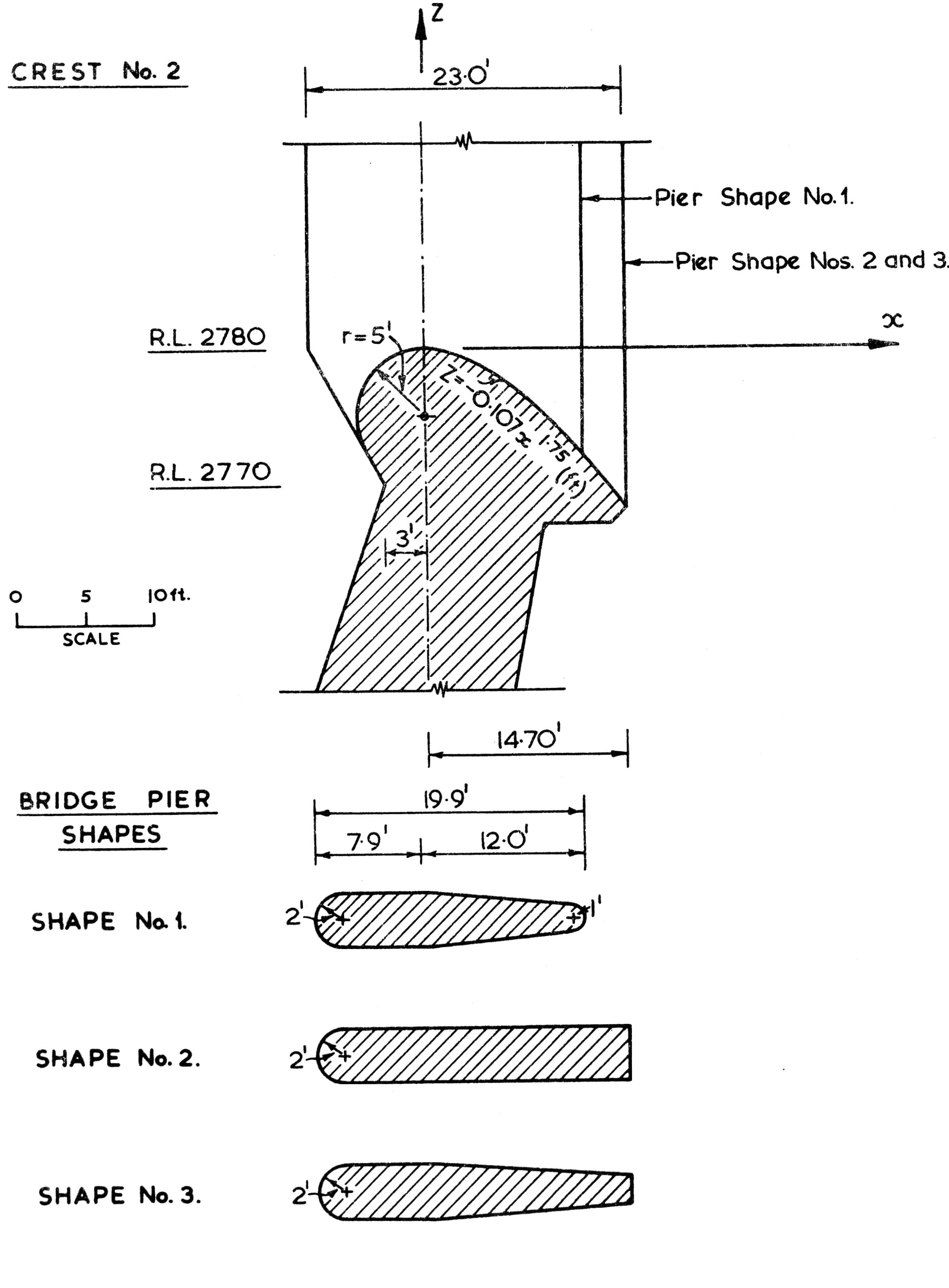


Figure 3: Definition Sketch: Crest No. 2.

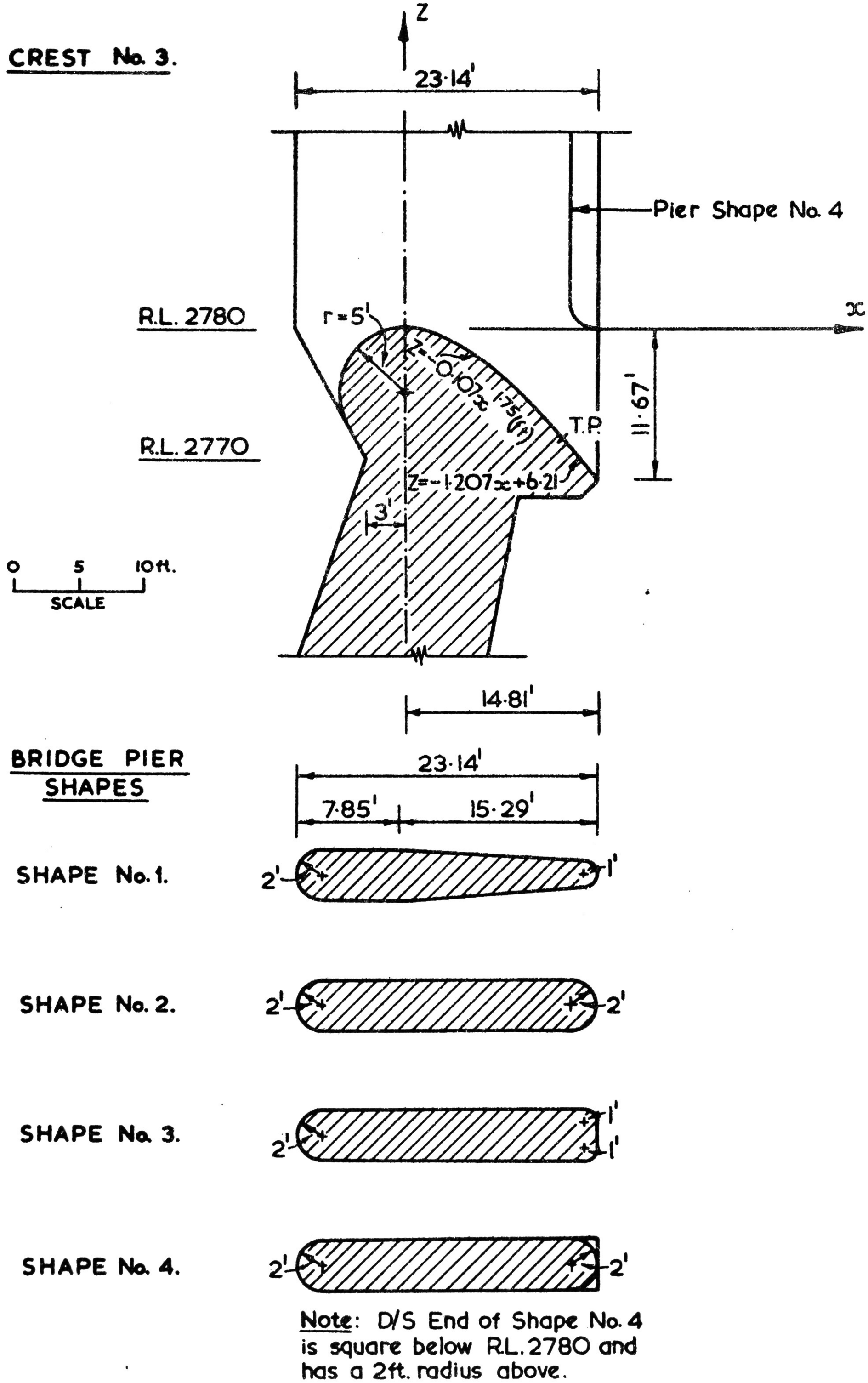


FIGURE 4: DEFINITION SKETCH - CREST No. 3.

CE-E-7341

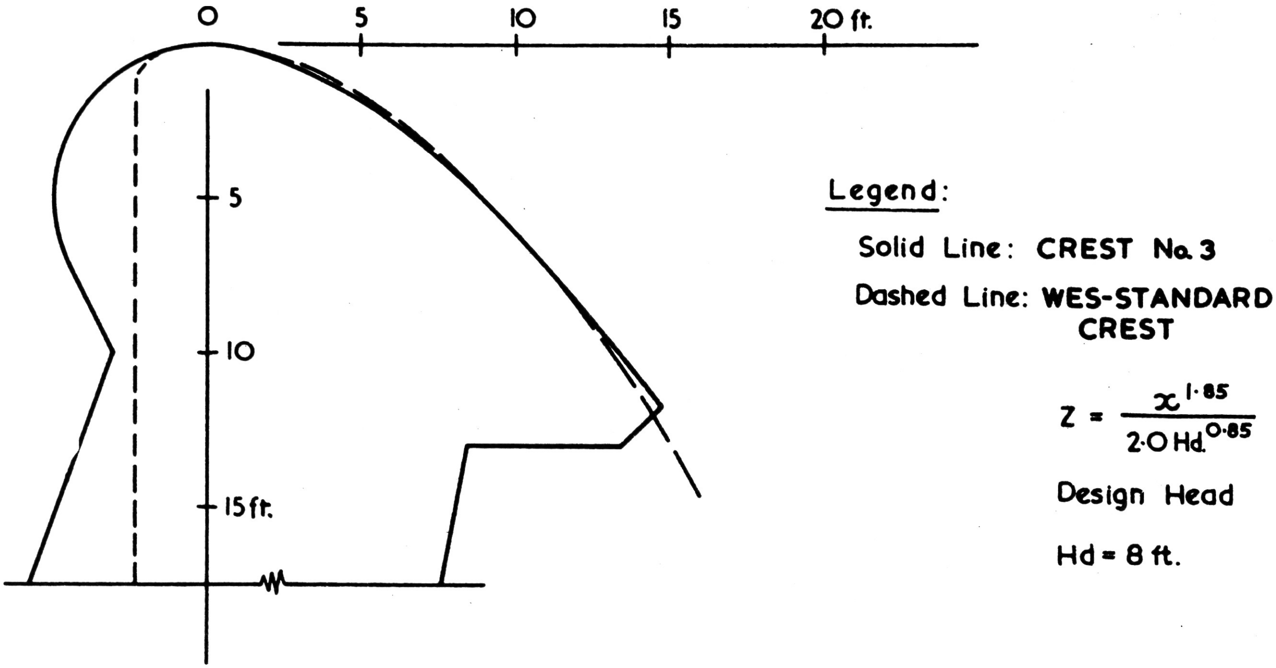


FIGURE 5: COMPARISON OF CREST No. 3 WITH STANDARD CREST OF U.S ARMY ENGINEERS WATERWAYS EXPERIMENTAL STATION

CE-E-7342

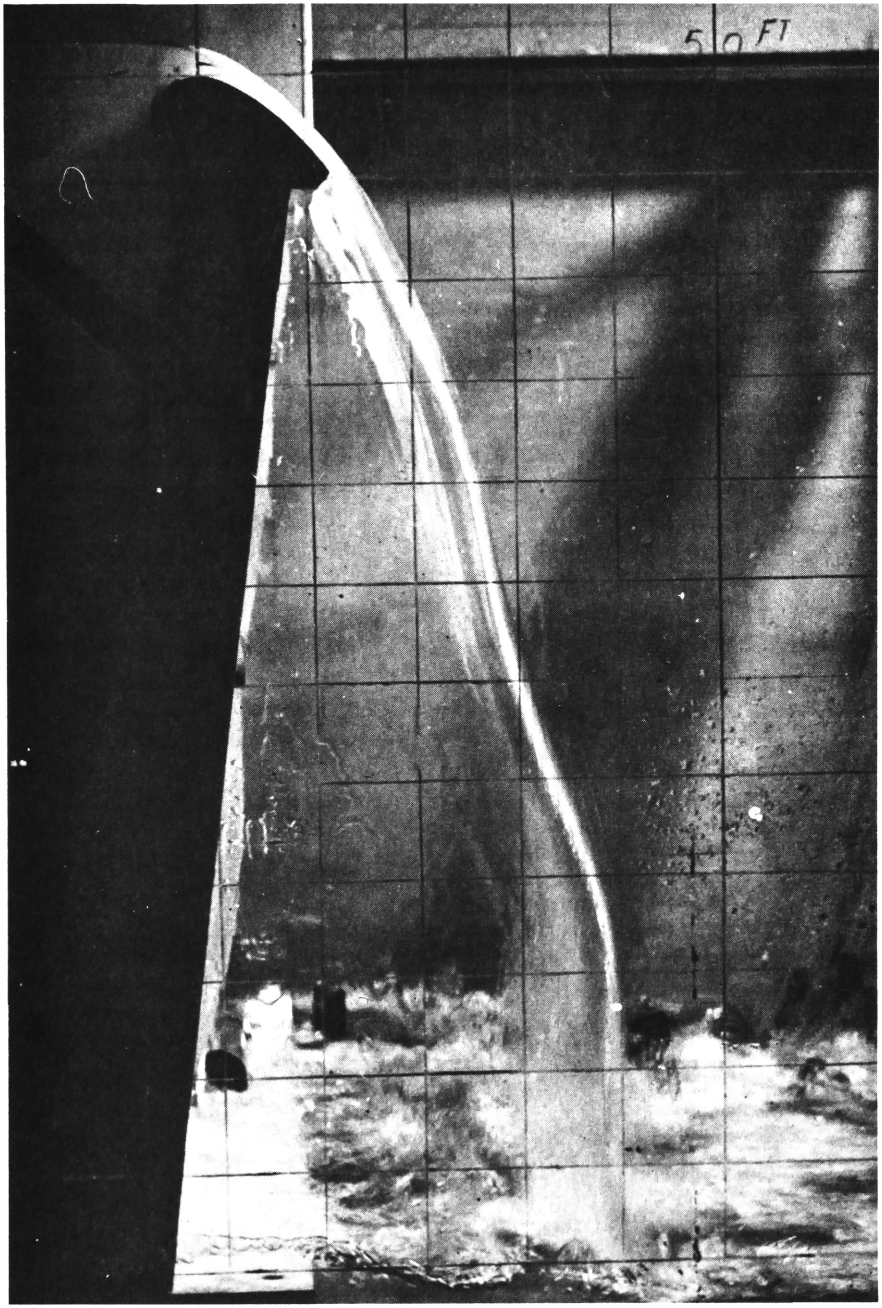


Fig. 6: Vibration of the Nappe.

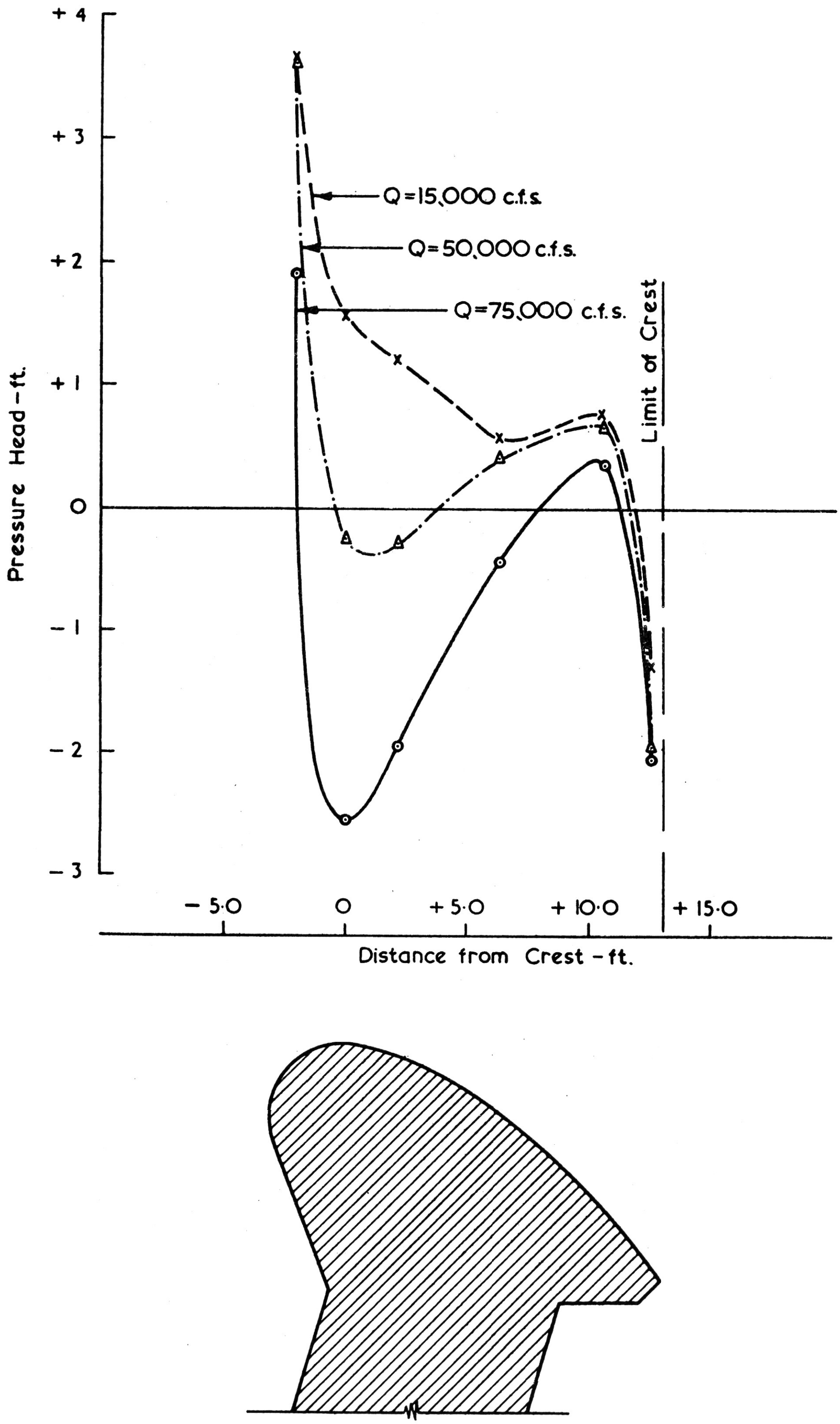


Figure 7: Pressure Distribution: Crest No. 1

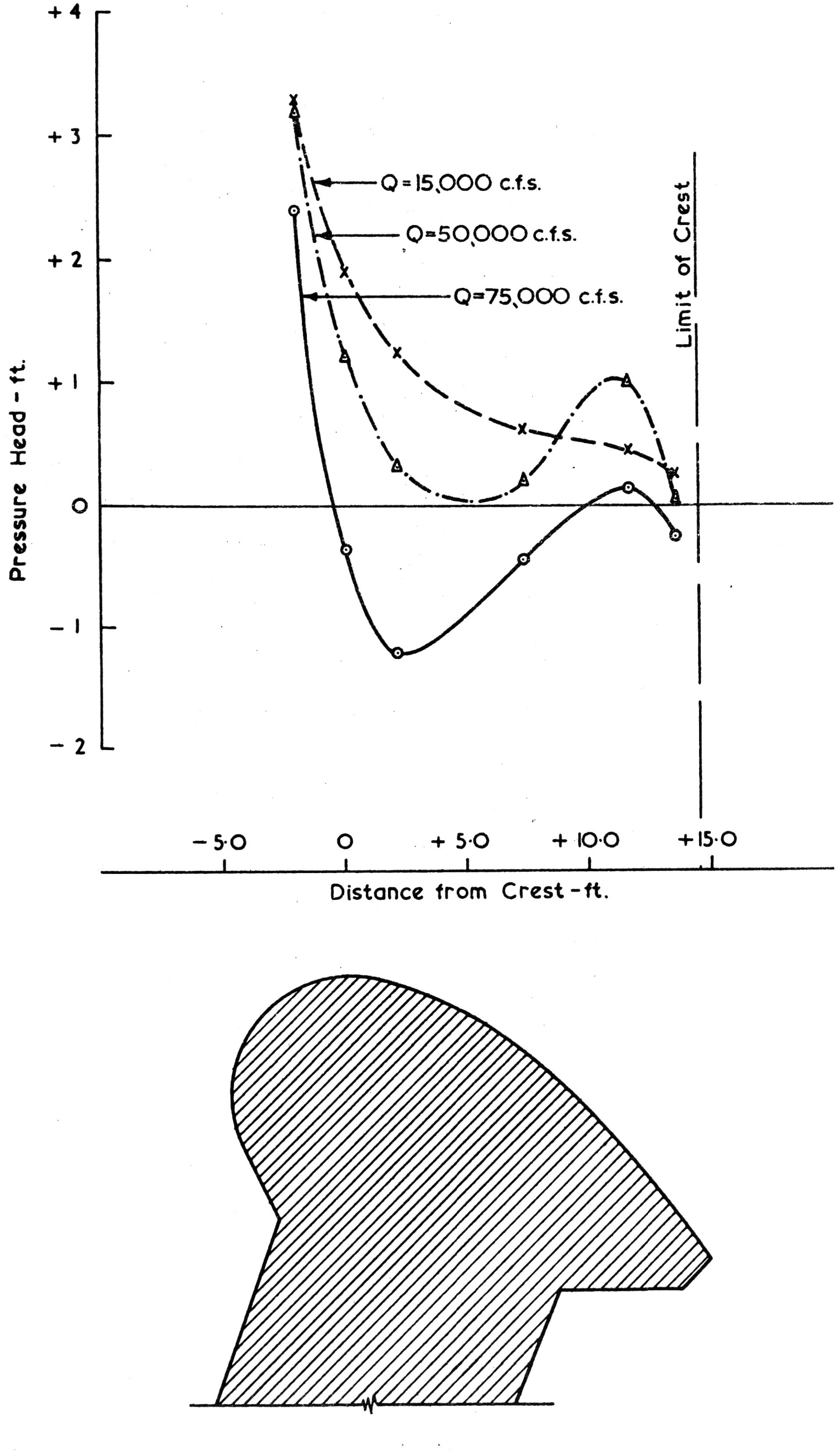
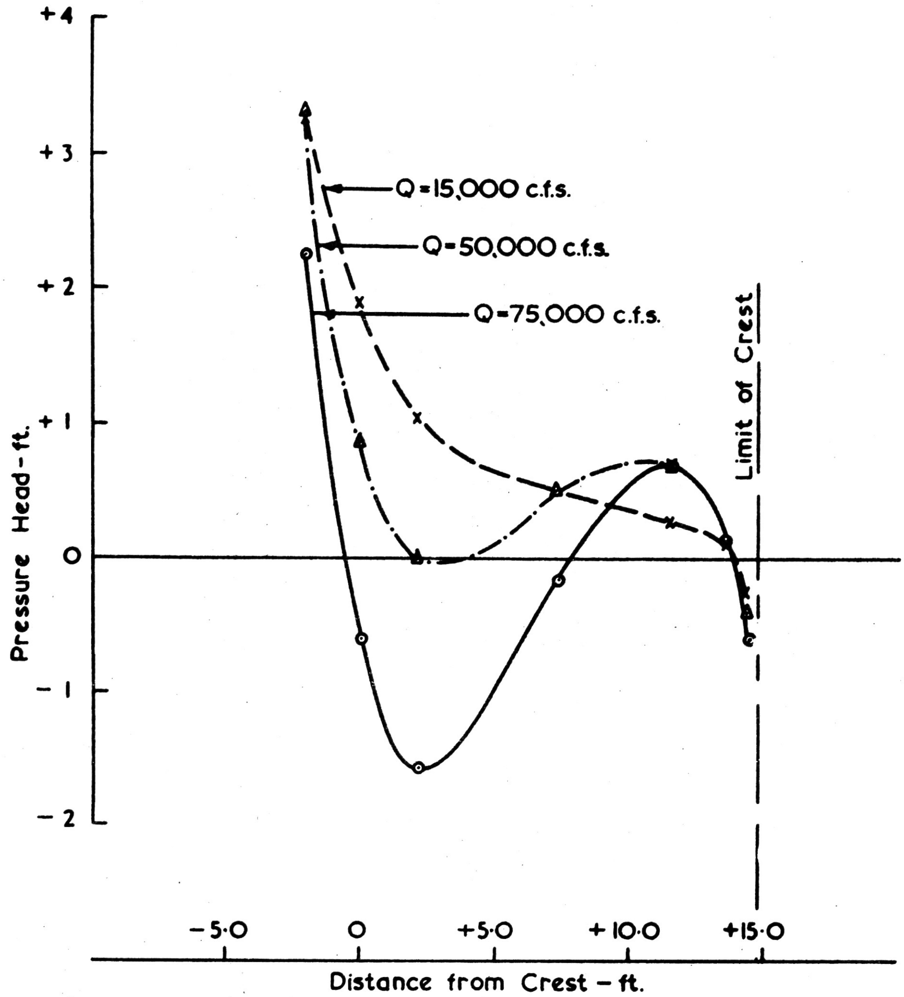


Figure 8: Pressure Distribution: Crest No. 2



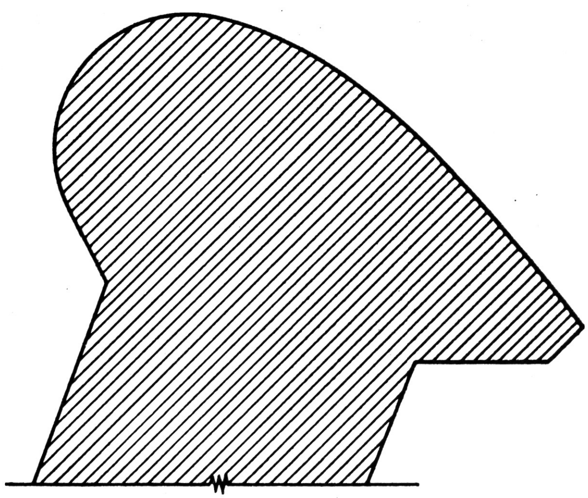


Figure 9: Pressure Distribution: Crest No. 3

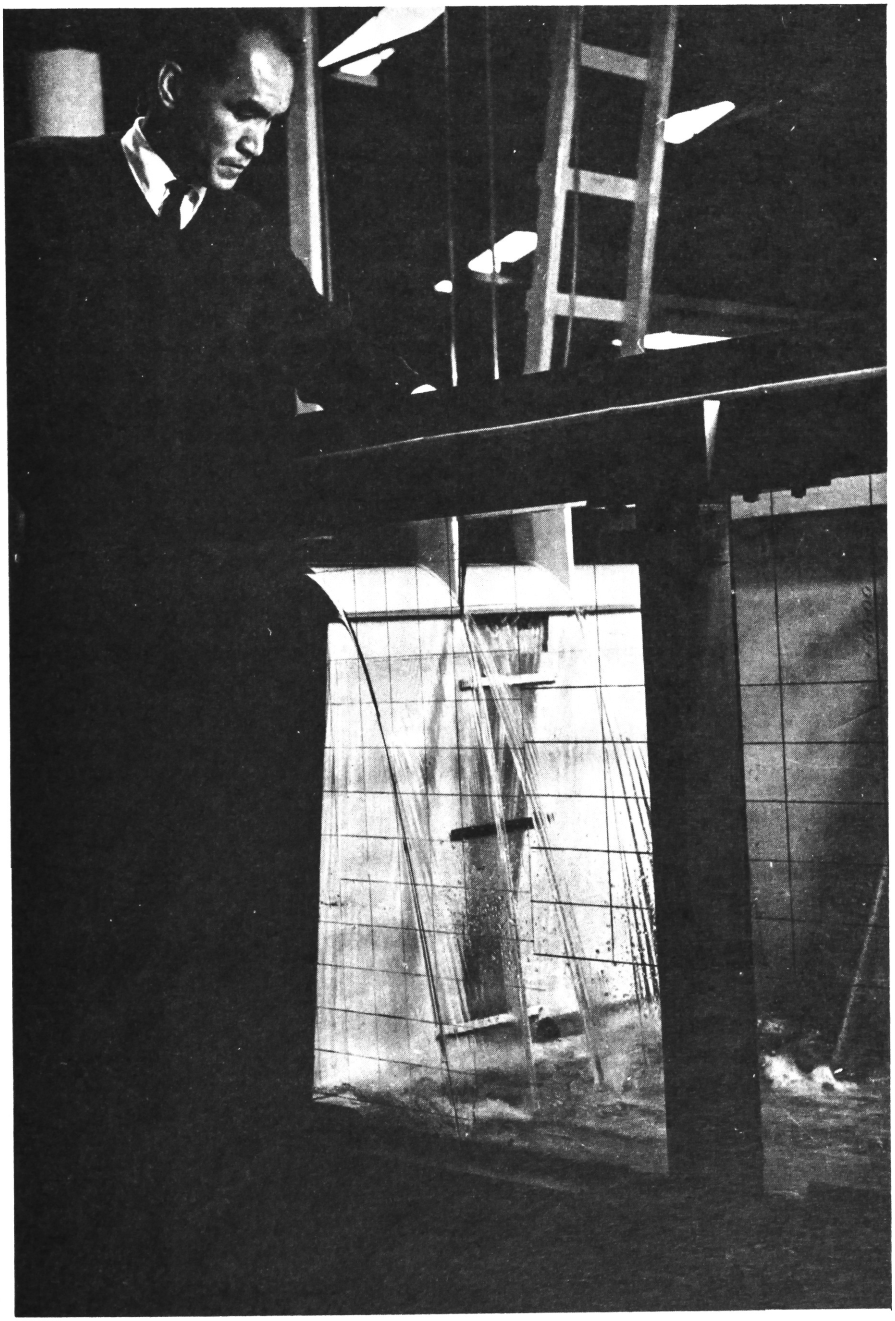


Fig.10: Vibration Prevented by Splitting the Nappe.

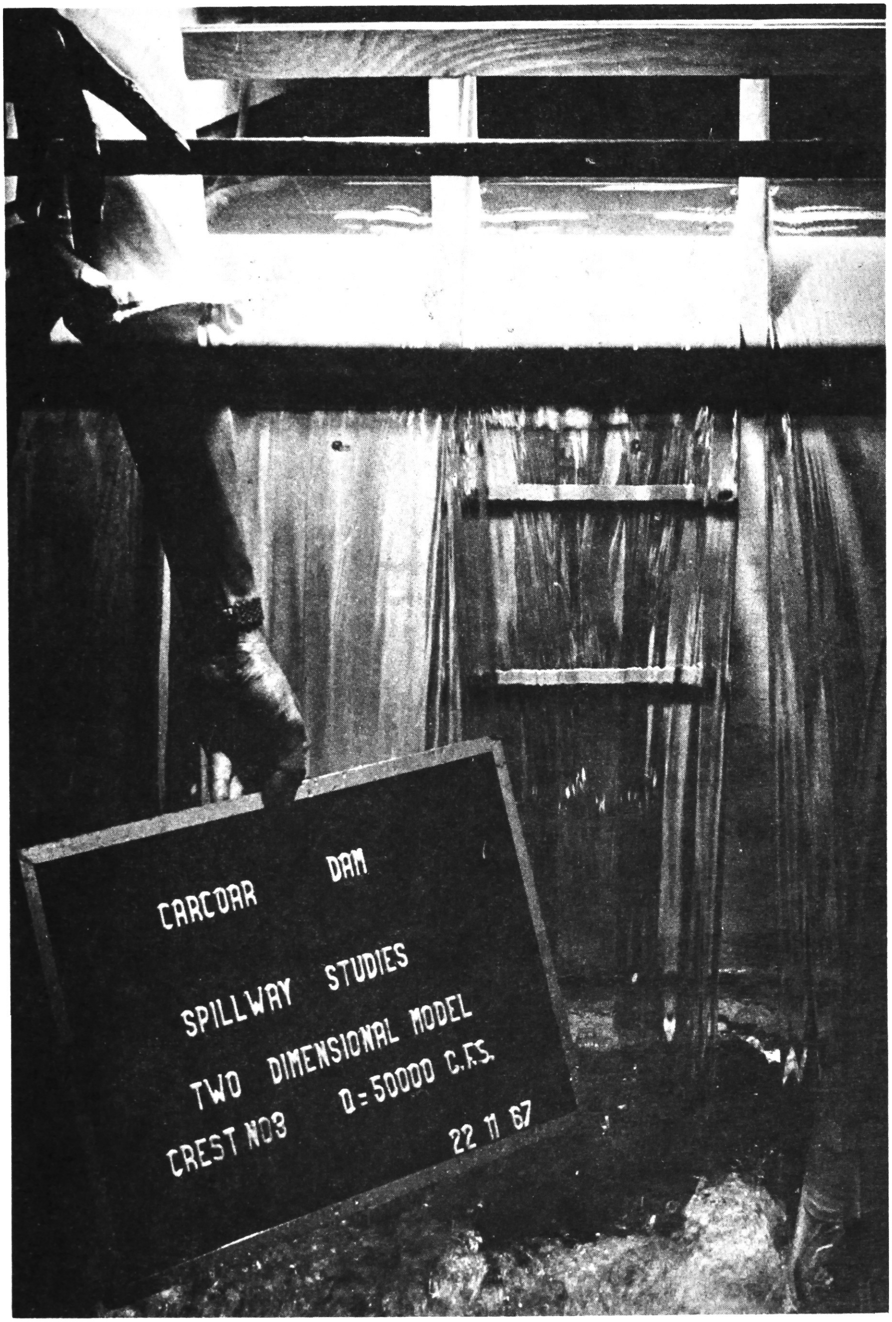


Fig.11: Nappe downstream of Pier Shape No.3 (Left) Pier Shape (4 Right) on Crest 3. Note the split of Nappe on Right.

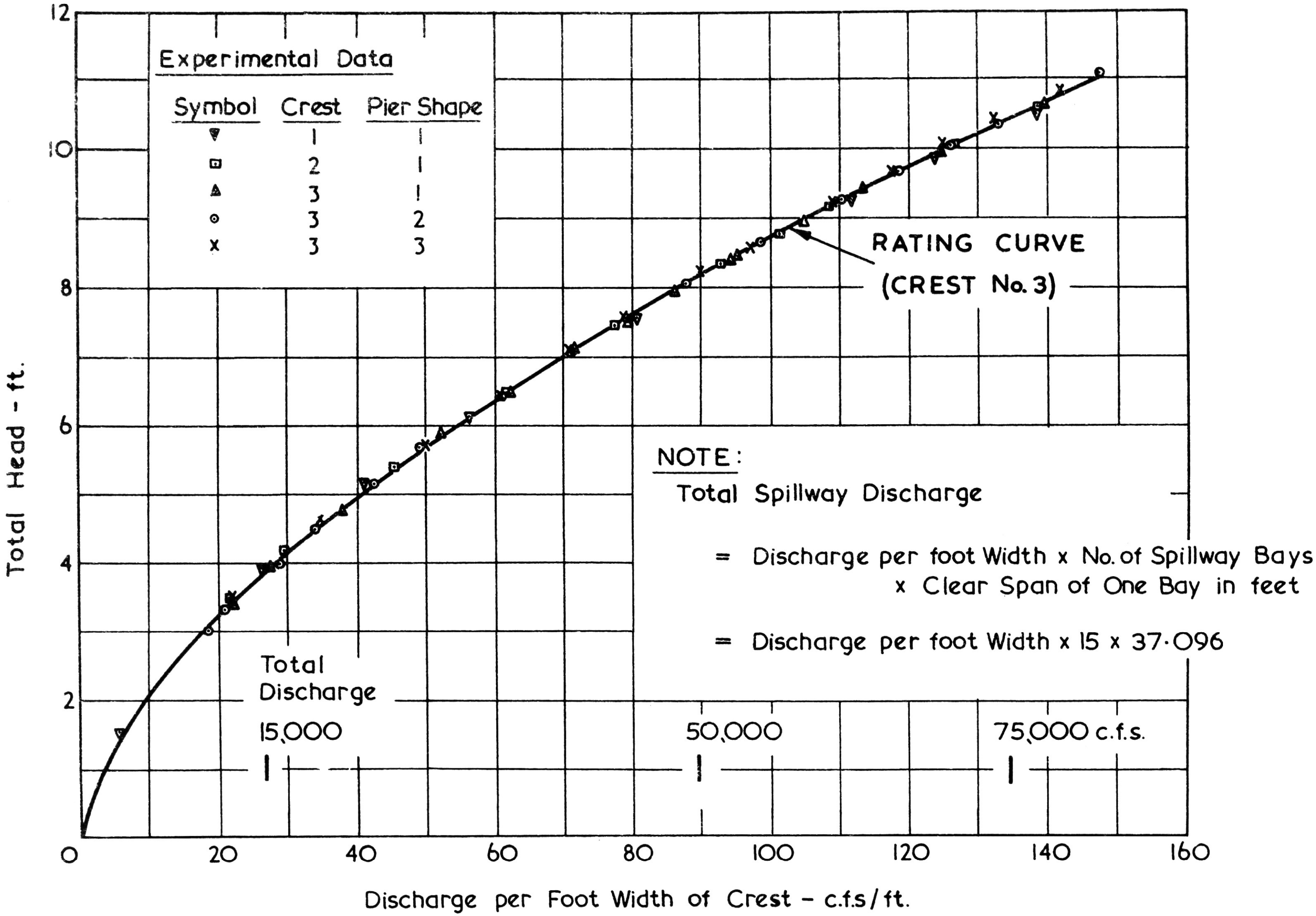


Figure 12; Head-Discharge Relationship.

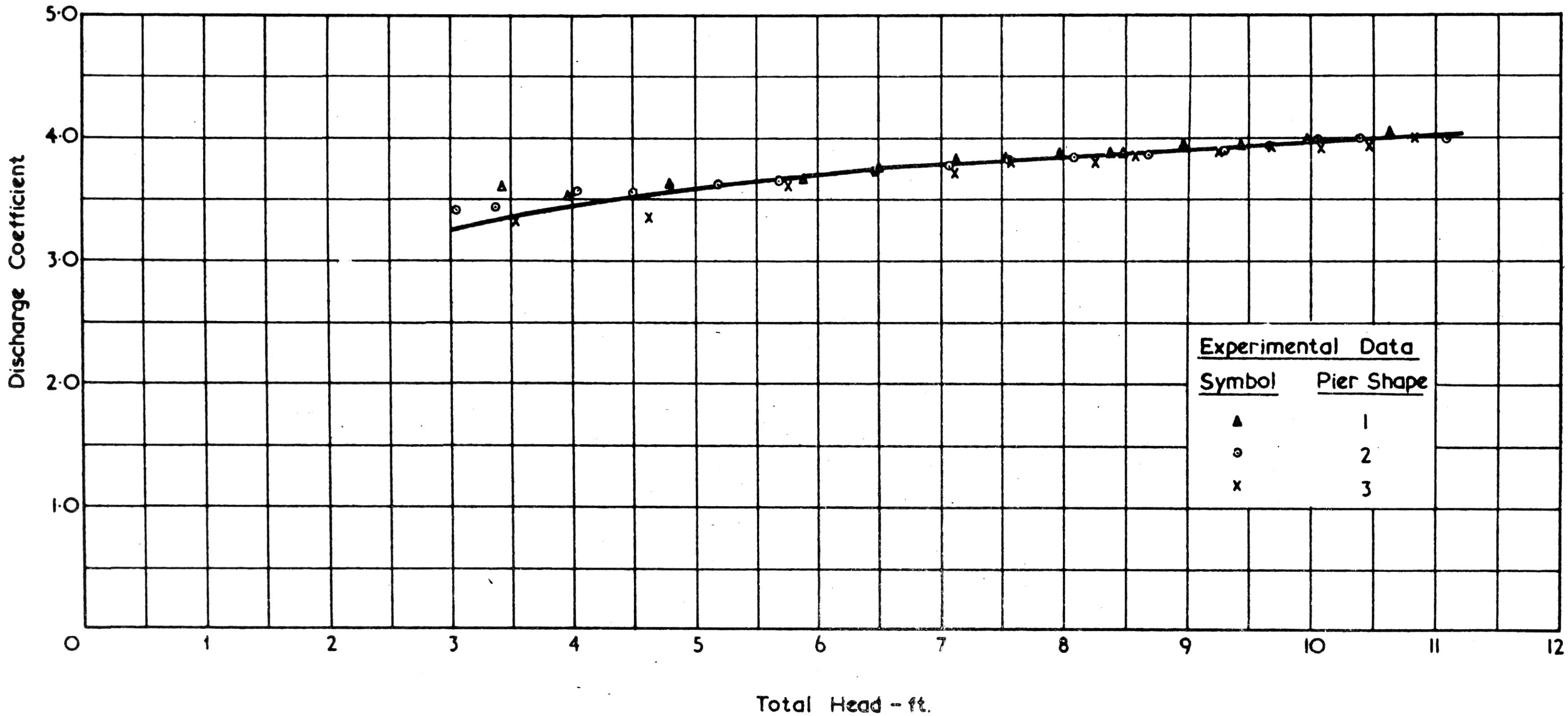


Figure 13: Discharge Coefficient - Crest No. 3

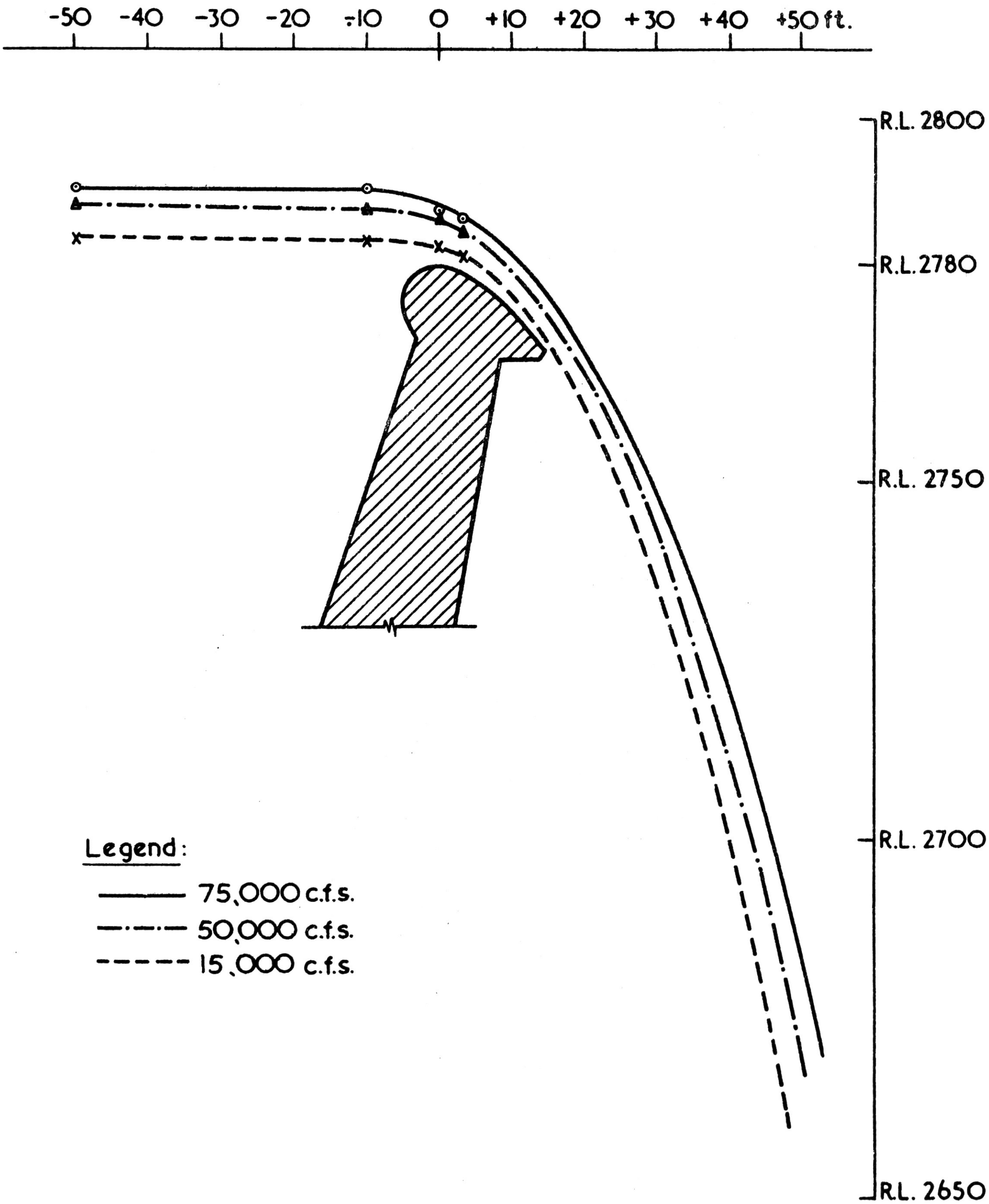


Figure 14: Nappe Trajectories - Crest No. 3.

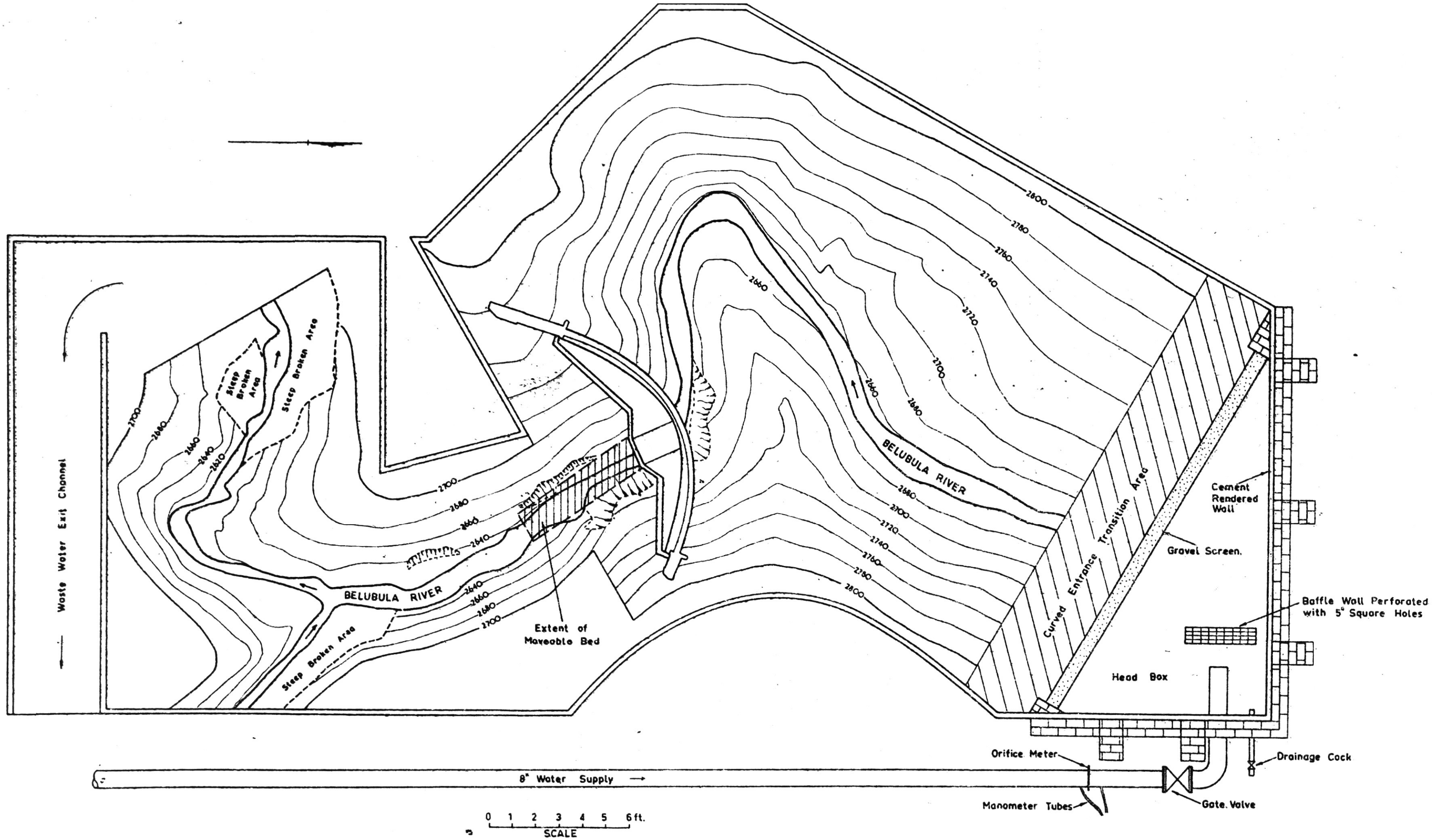
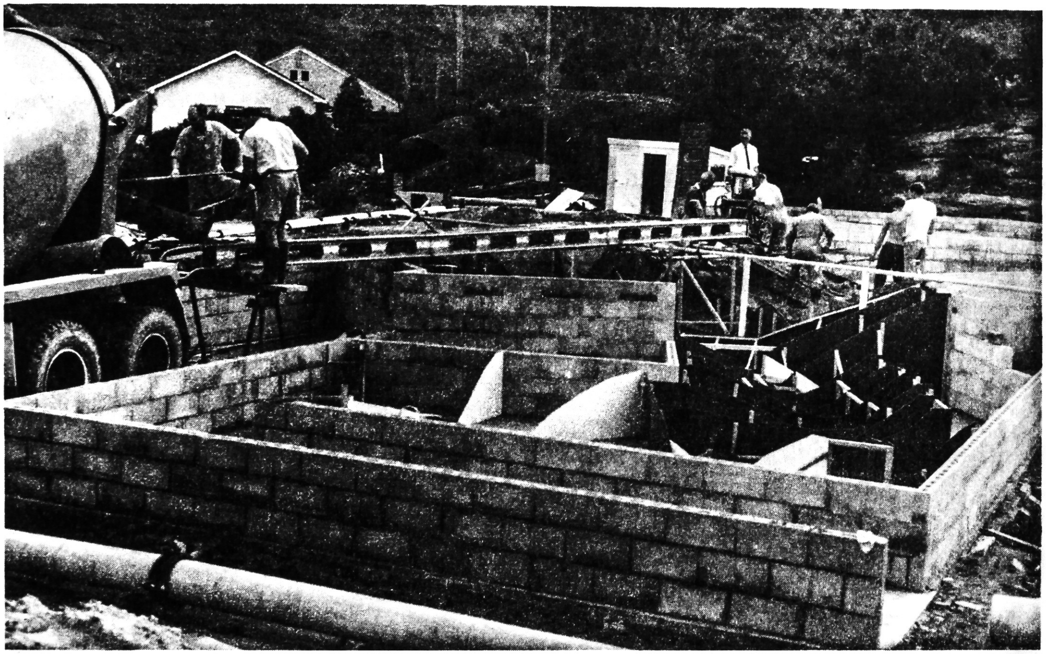
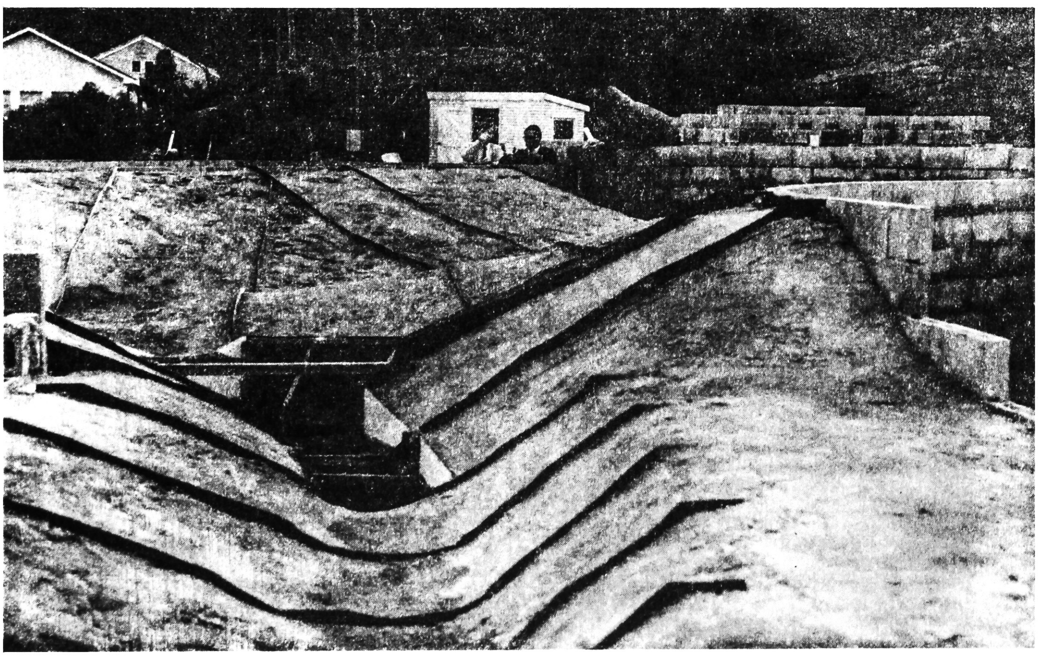


Figure 15: General Arrangement of 1:50 Spillway Model.



(a) Templates Positioned Ready for Bulk Filling.



(b) Bulk Filling Completed. Model Ready for Placing of Mortar Cap.

Figure 16: Model Construction: Land Form.

Figure 17: Model Construction: Dam Wall.

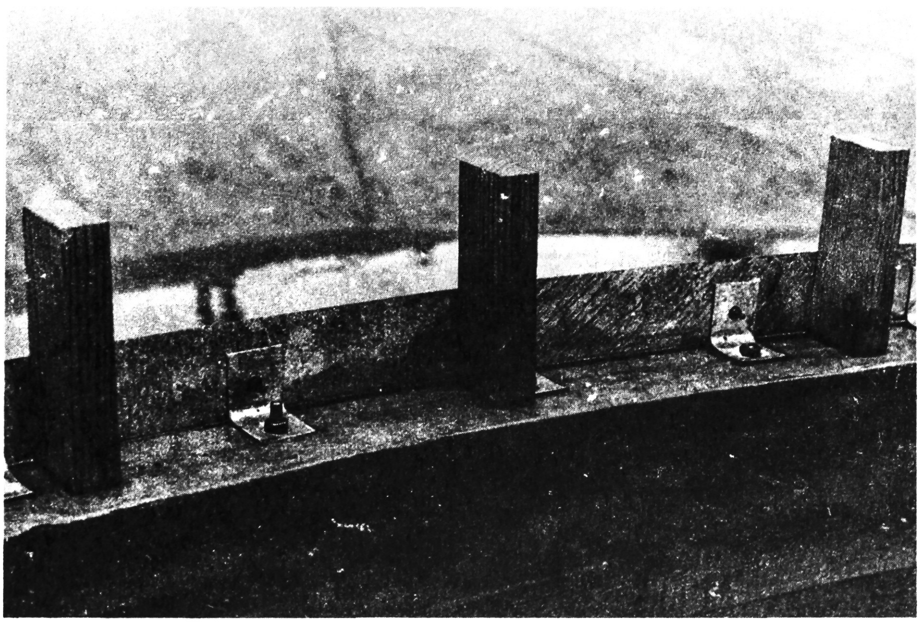


Figure 18: Sharp Edged Metal Crest.

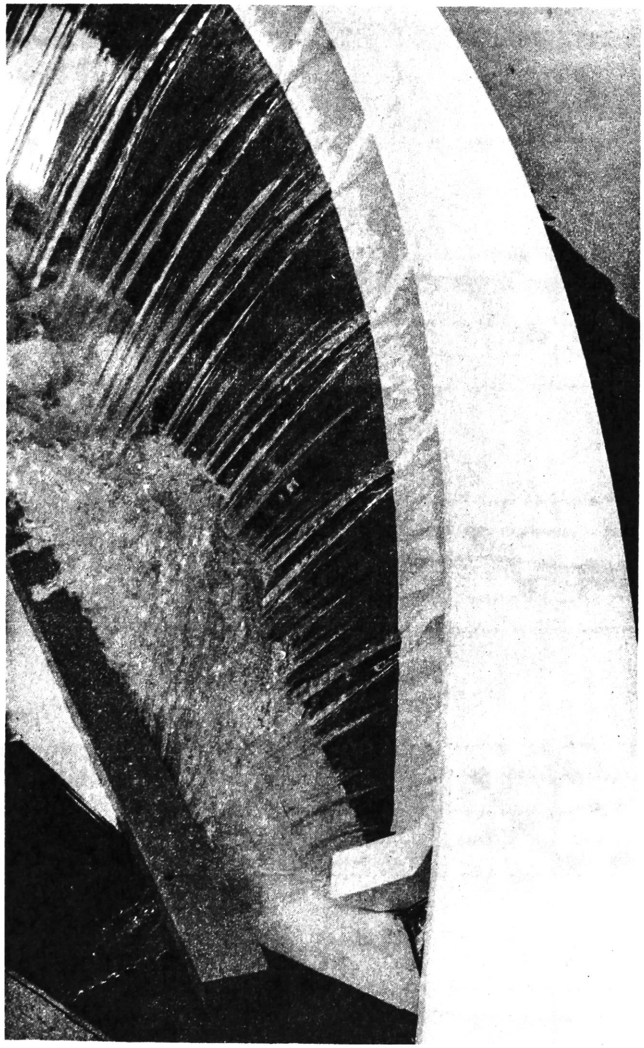


Figure 19: Guide Wall Operation: 8 Foot High with 50 Degree Angle.

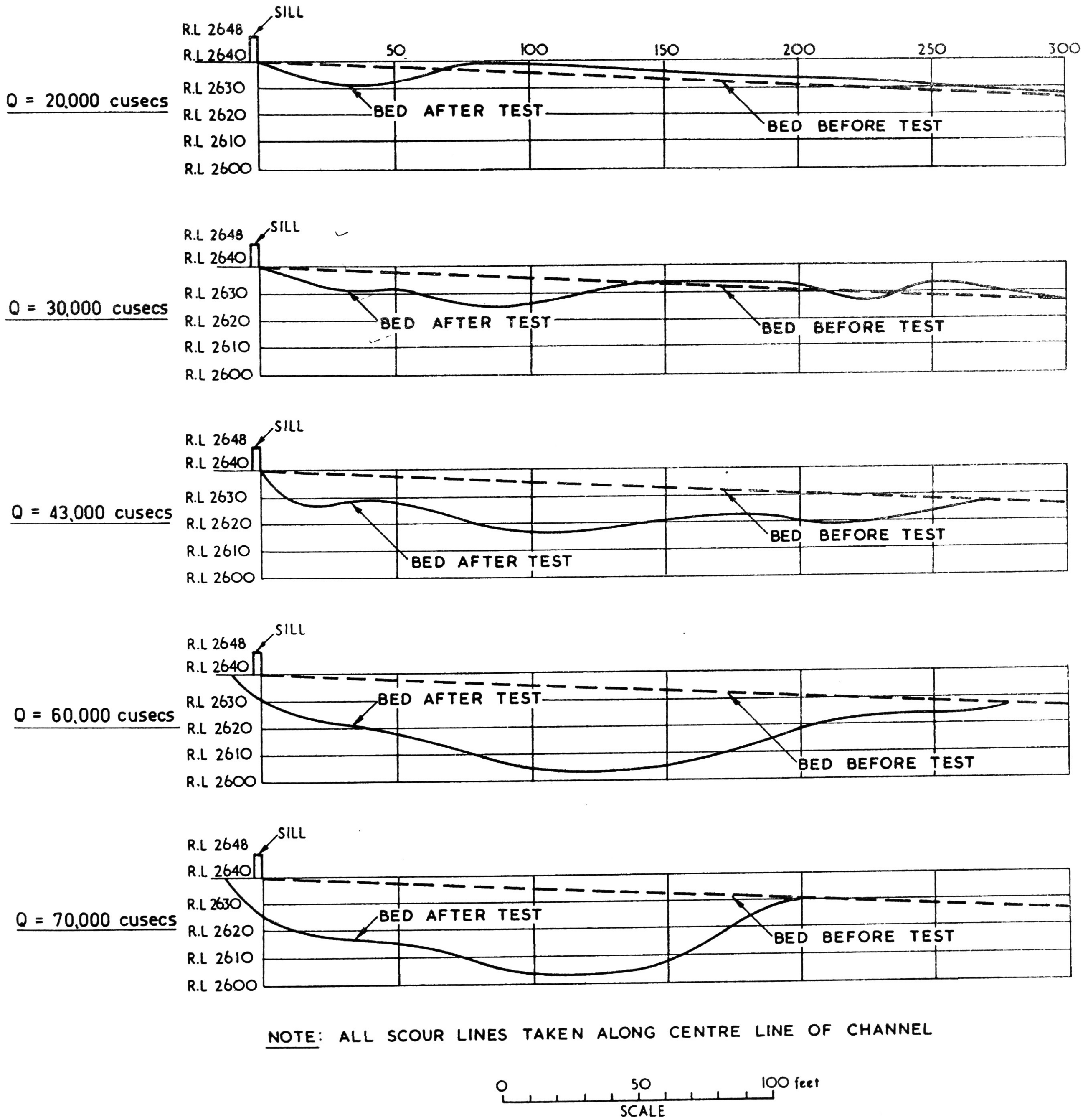
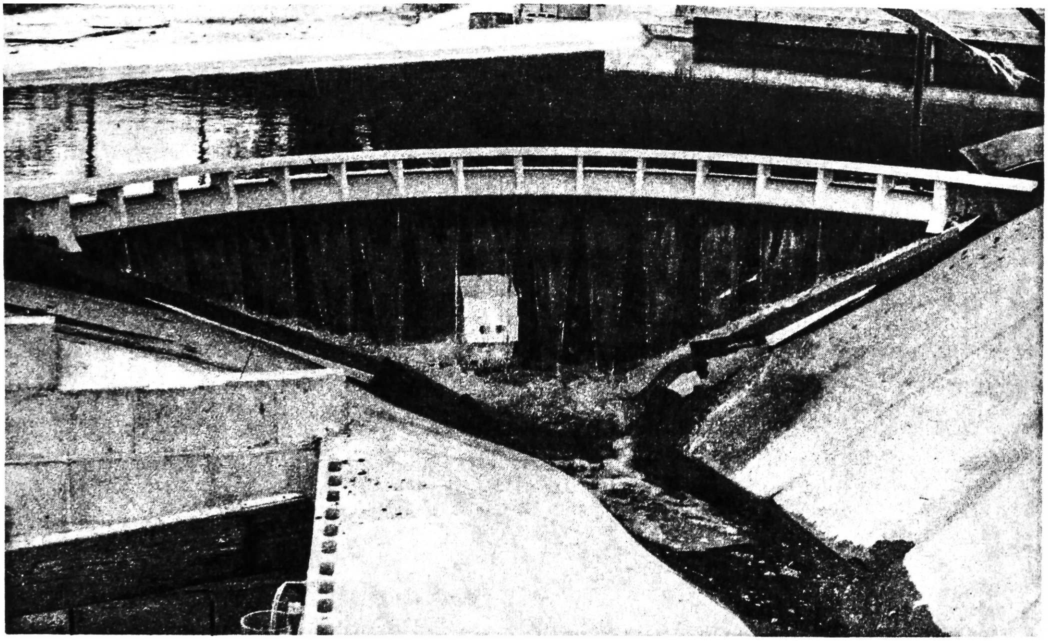
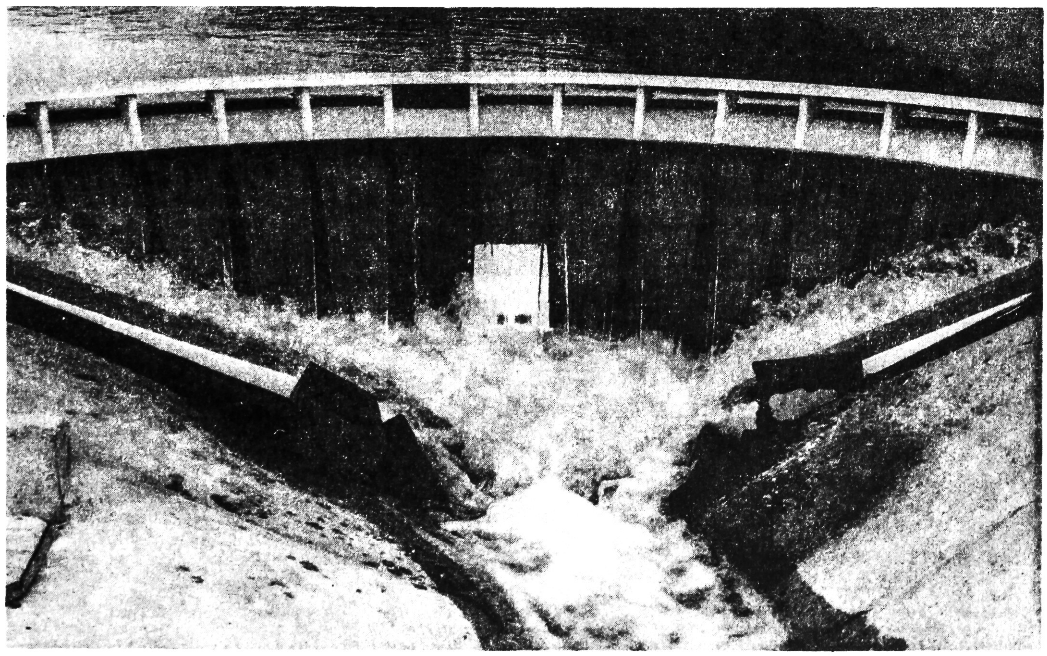


Figure 20: Preliminary Design - Scour Profiles.

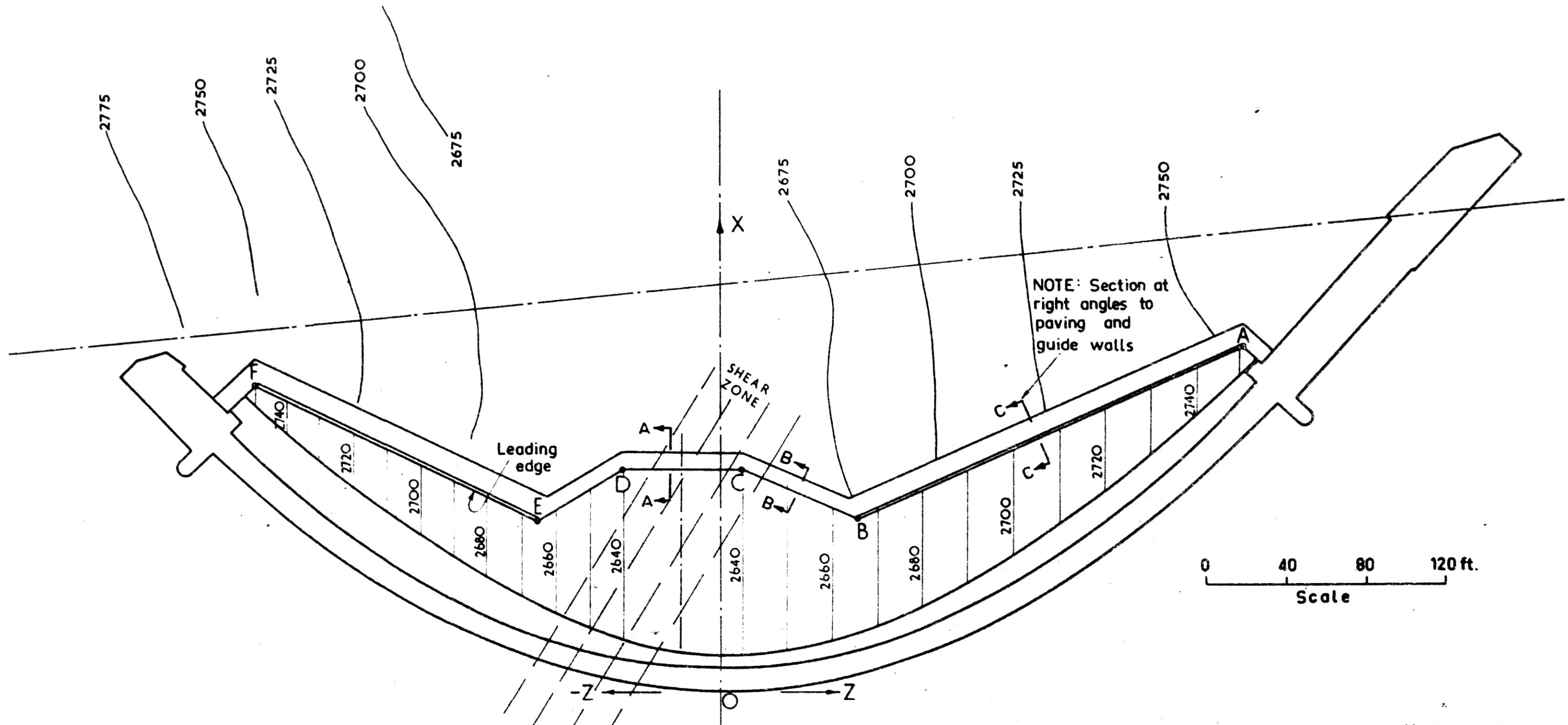


(a) 10,000 c.f.s.



(b) 43,000 c.f.s.

Figure 21: Preliminary Design: Model Operation.

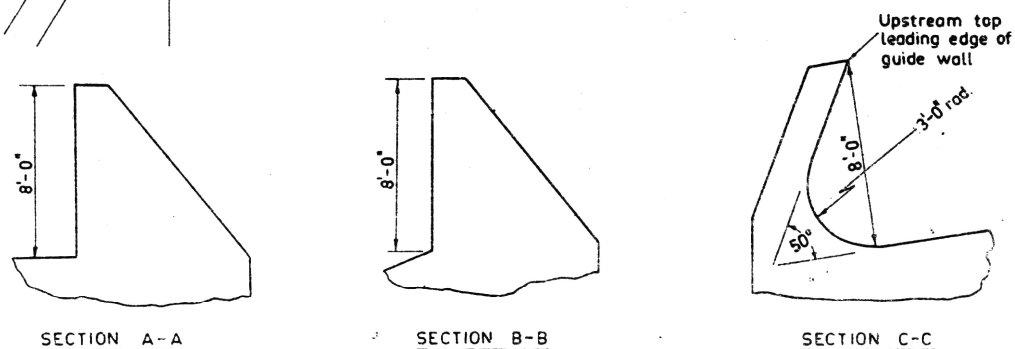


Scaled co-ordinates

POINT	X	Y	Z
A	172·0	2751·01	262·5
B	87·5	2664·89	70·0
C	112·5	2640·00	15·0
D	112·5	2640·00	- 50·0
E	87·5	2664·81	- 91·3
F	155·0	2749·95	-233·0
O	0	2770·00	0

NOTE: Subject to possible alteration

All Points Measured to Top Surface of Apron Vertically under Upstream Top Leading Edge of Guide Wall



Sections not to Scale

Figure 22: Preliminary Spillway Arrangement.

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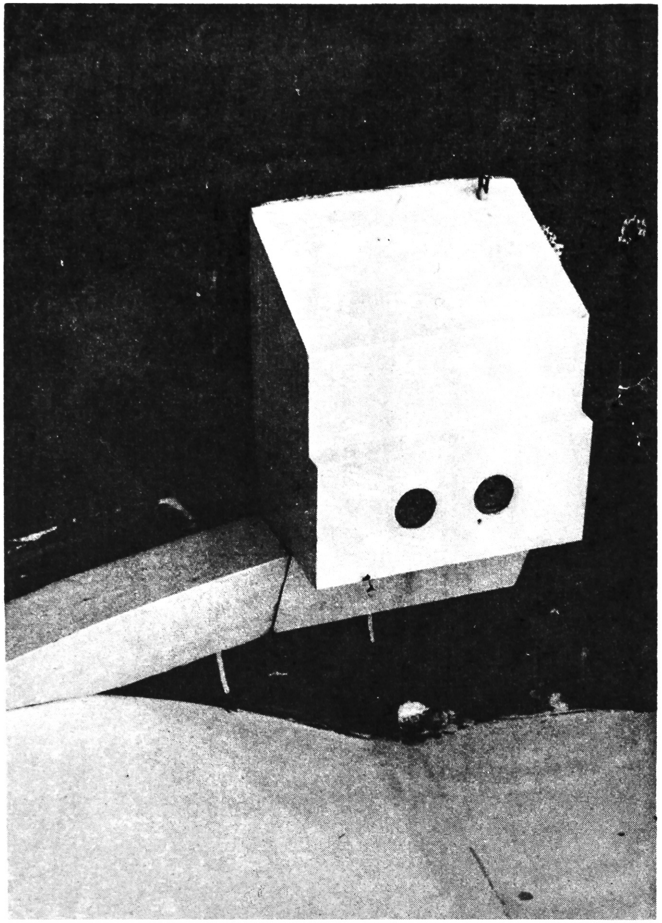


Fig. 23. Valve House

Preliminary Design

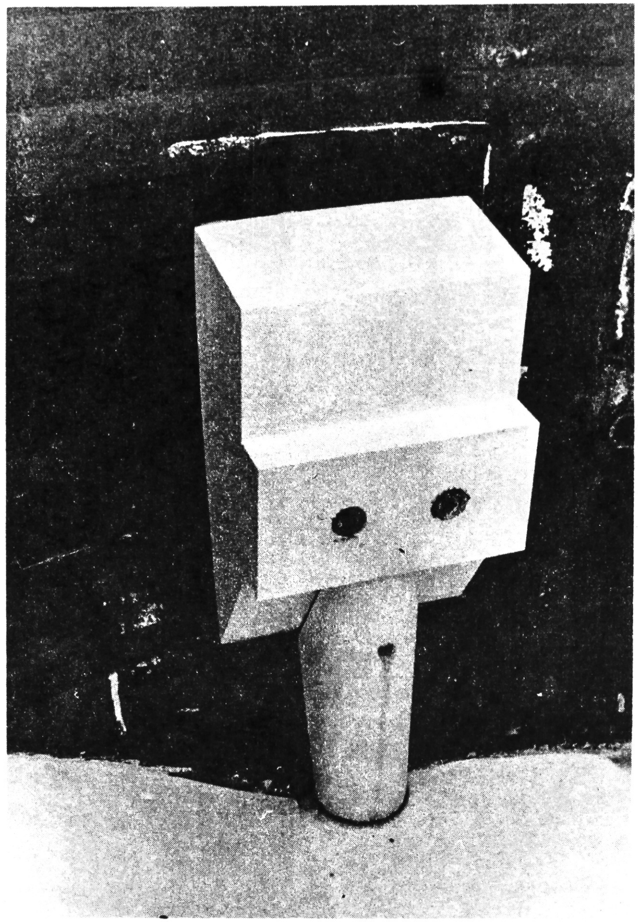
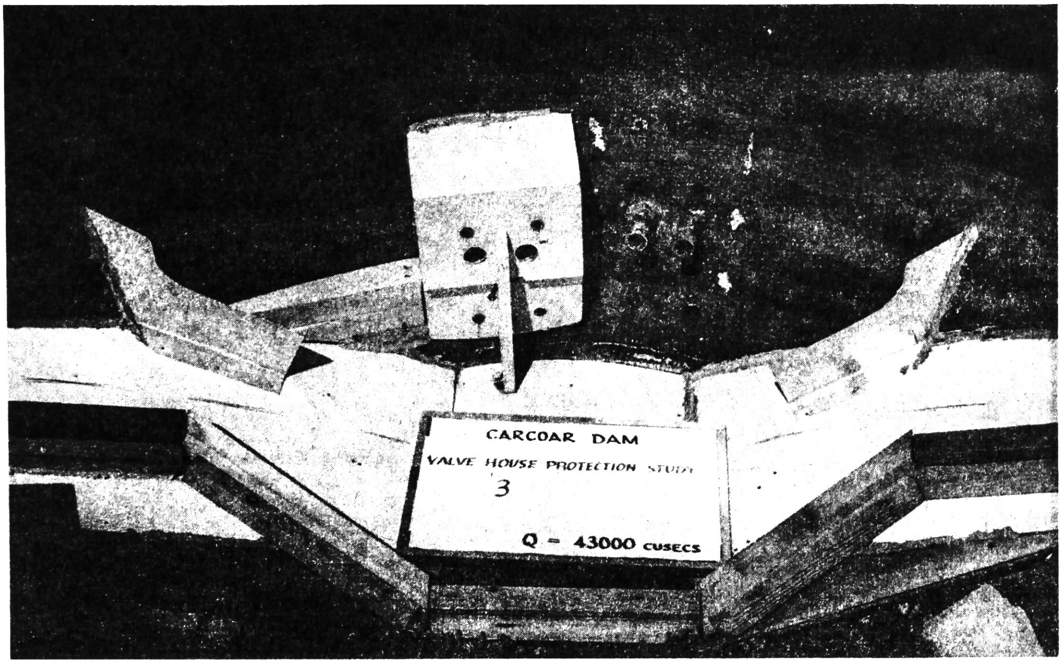
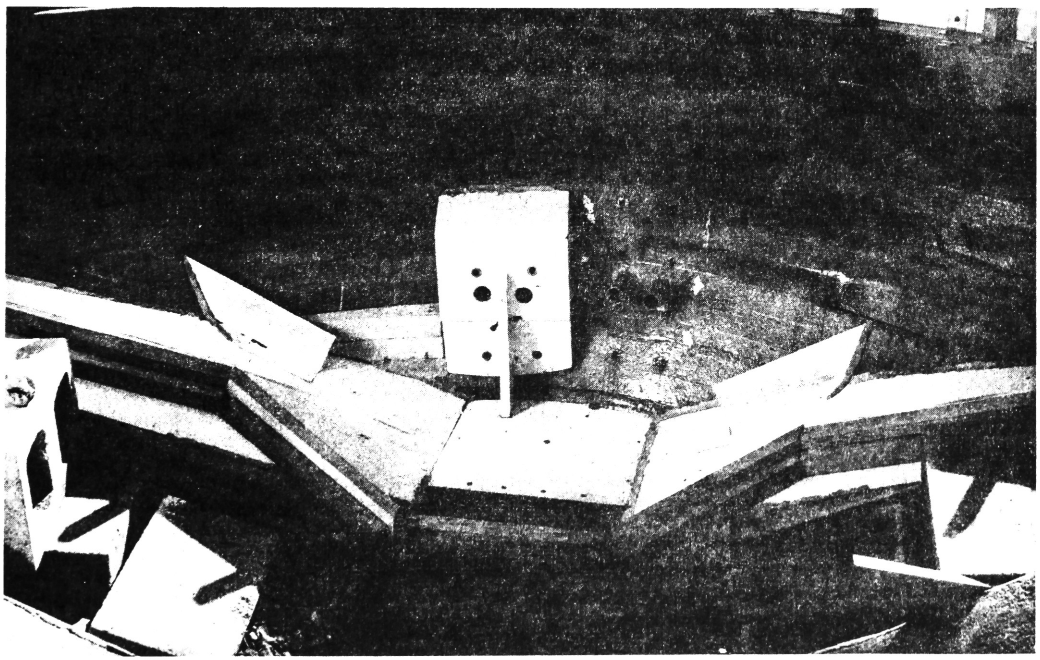


Fig. 24.Valve House

Final Design



(a) Maximum Height 21 Feet Reducing to 13 Feet.



(b) 13 Feet High

Figure 25: Examples of Plain Deflectors tested.

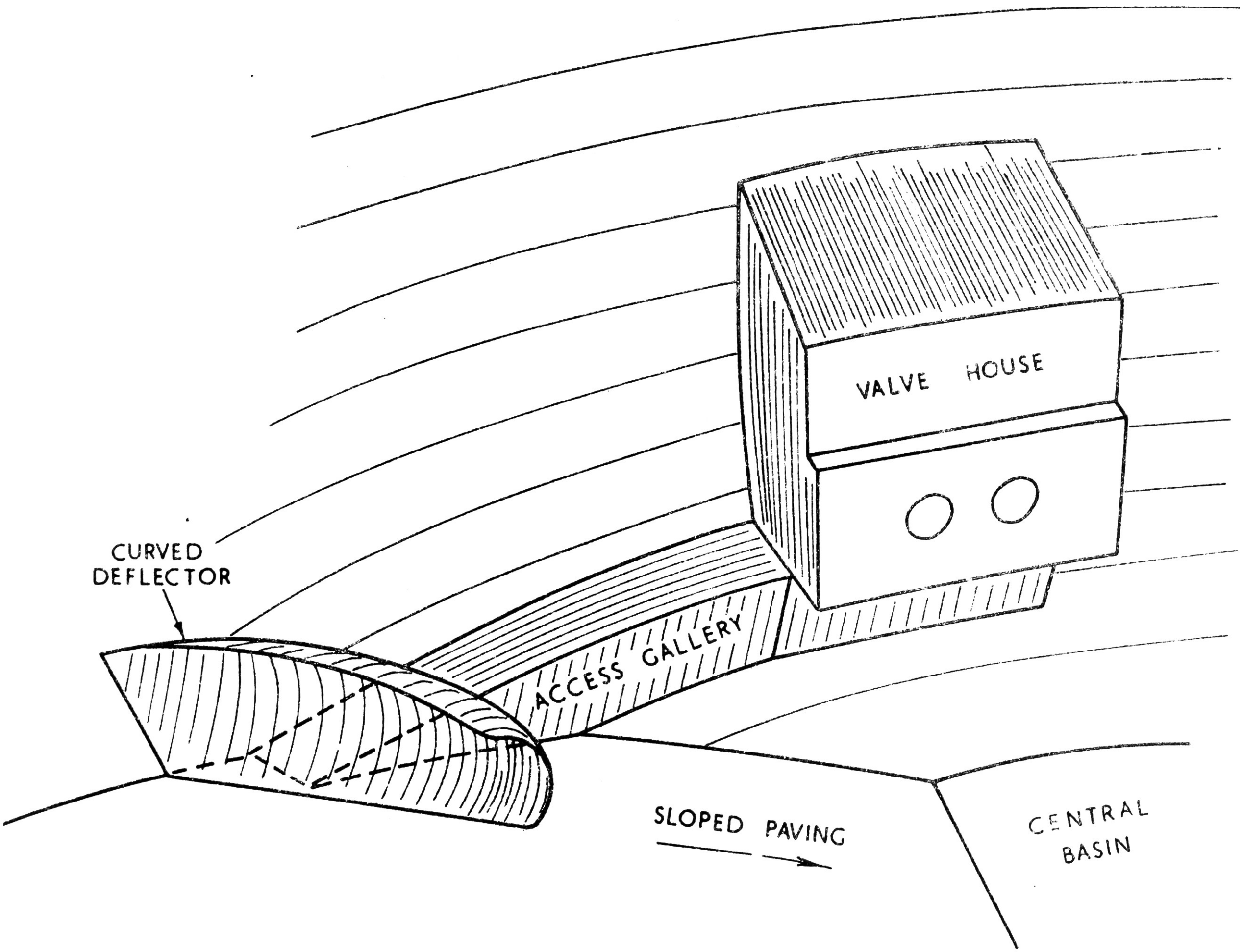


Figure 26: Curved Deflector.

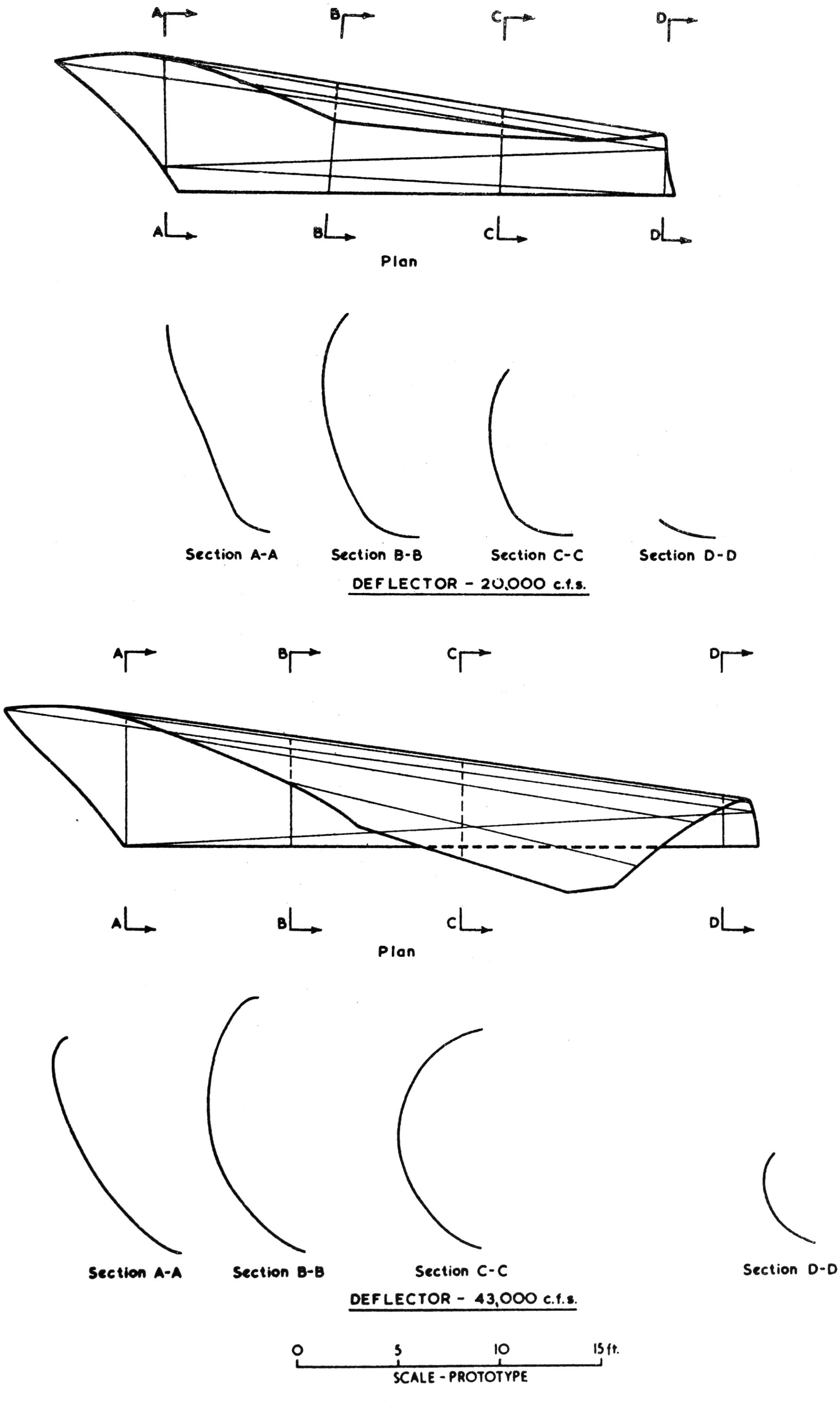


Figure 27: Preliminary Deflector Designs.

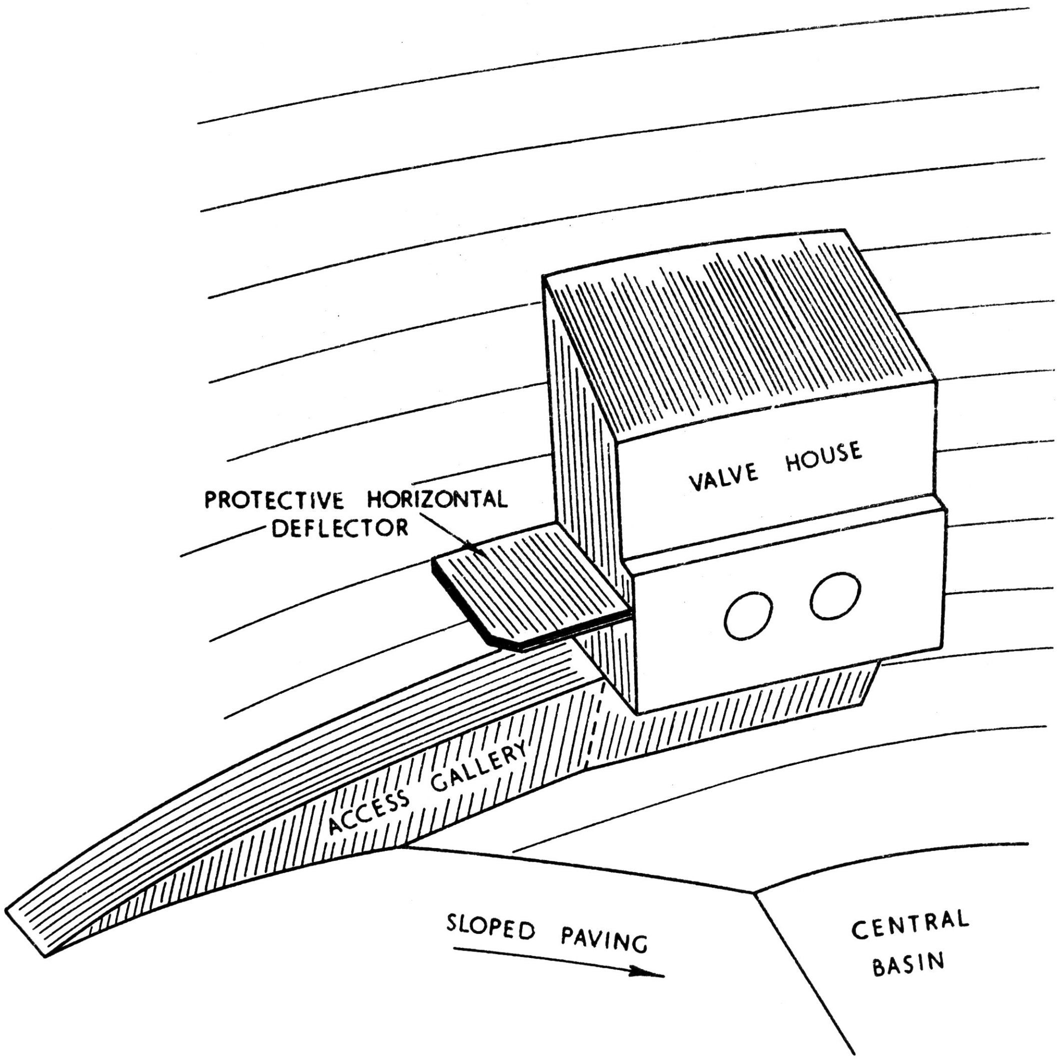


Figure 28: Horizontal Door Protector.

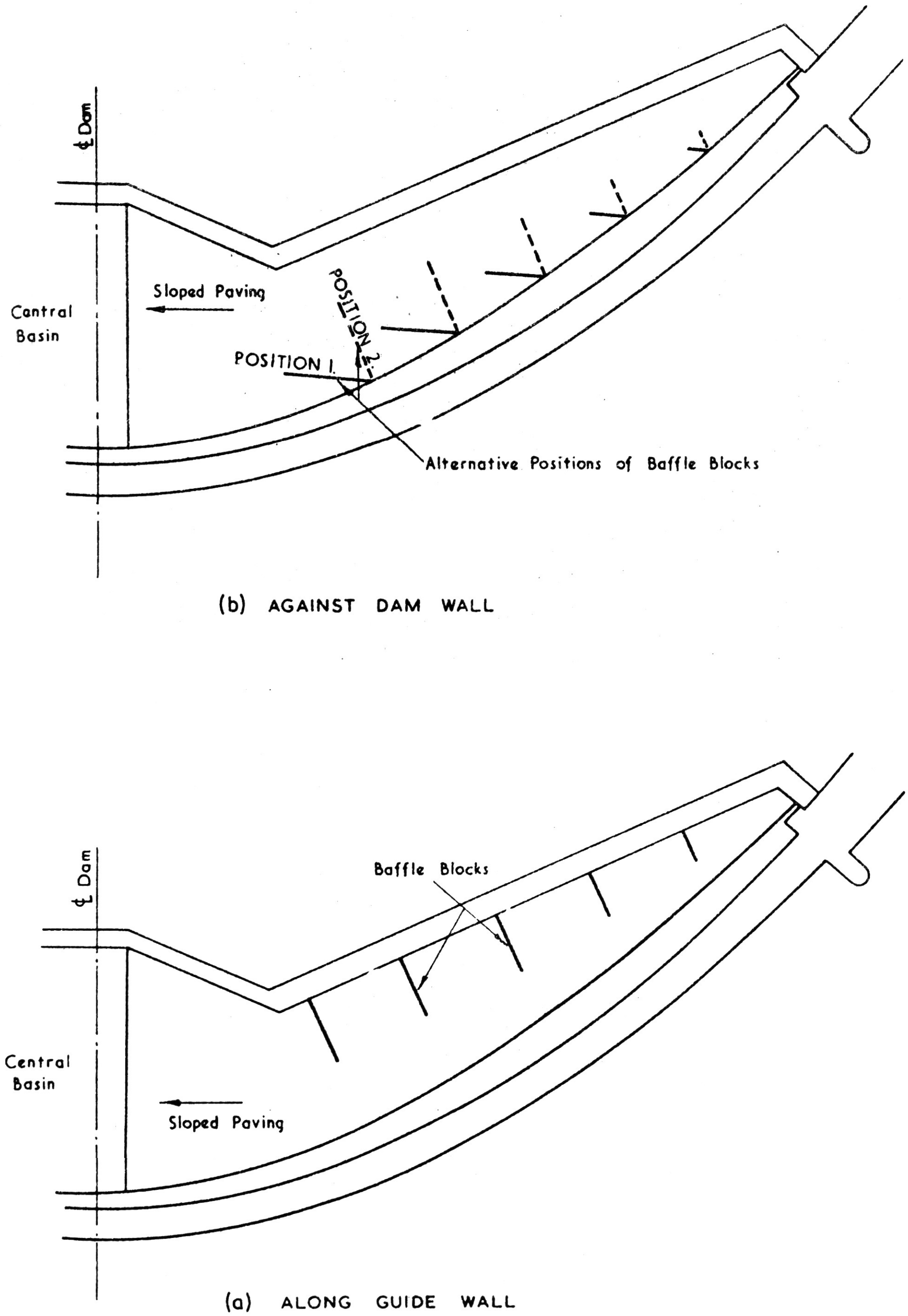


Figure 29: Multiple Baffle Blocks.

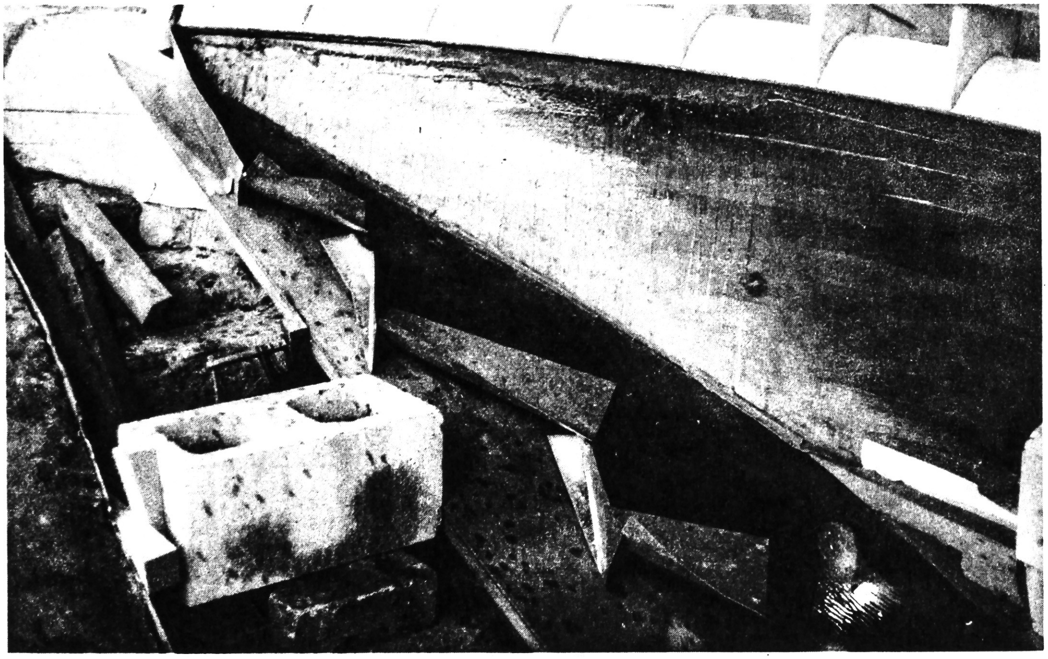


Figure 30: One Example of Couesque
Type Spillway Tested.

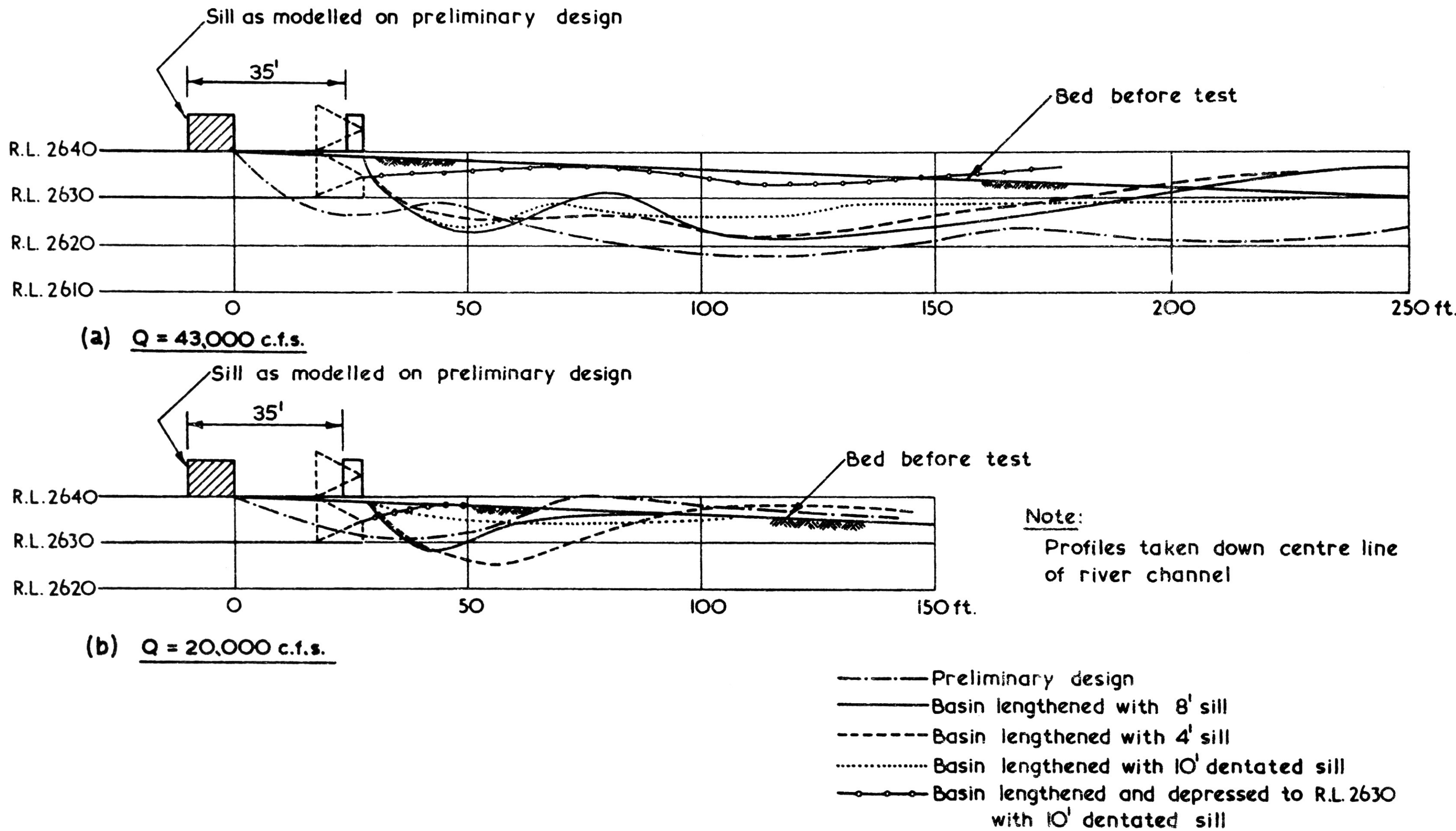


FIGURE 31: LENGTHENED BASIN OF PRELIMINARY DESIGN - SCOUR PROFILES

CE-E-7349

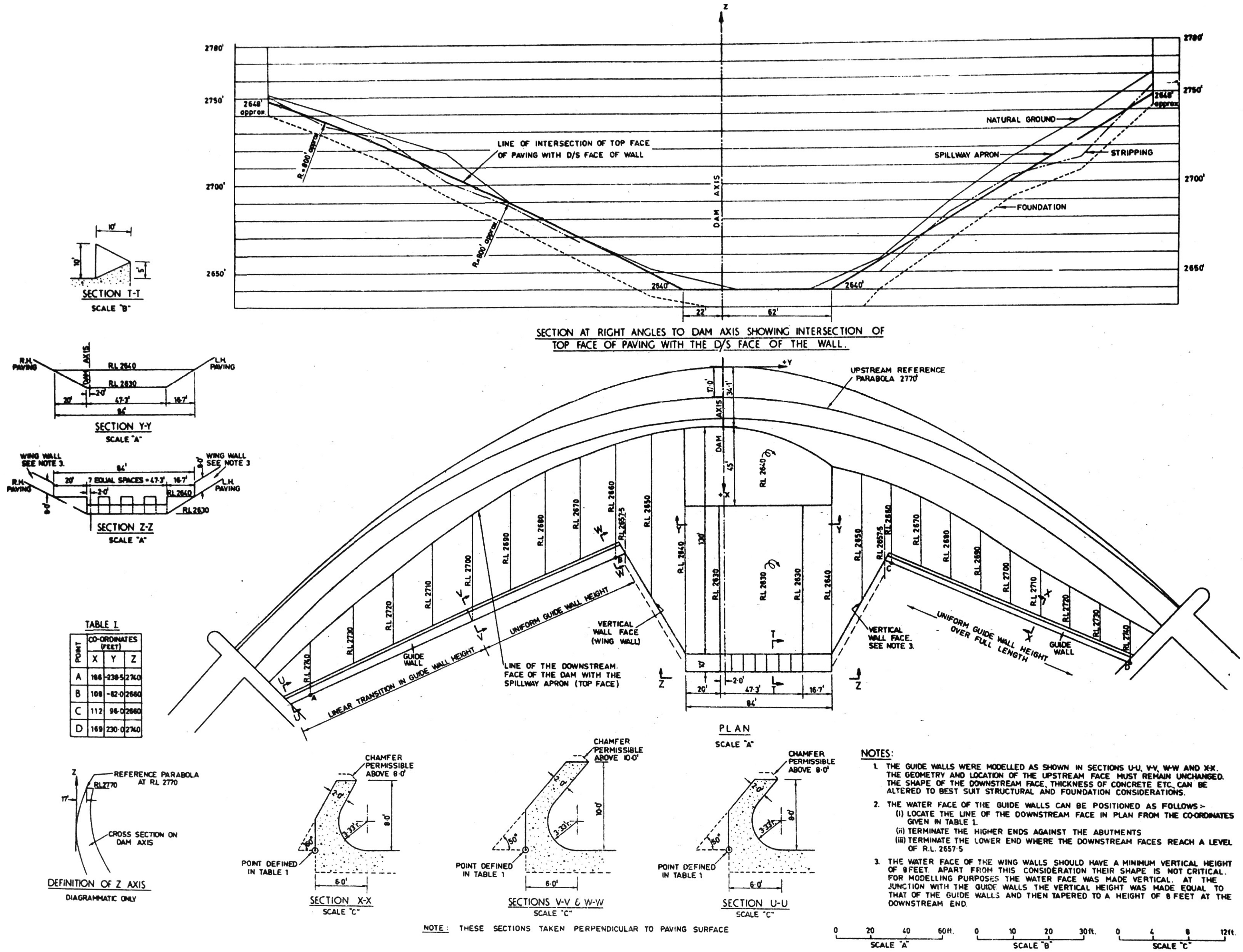


TABLE 1.

POINT	CO-ORDINATES (FEET) X	Y	Z
A	186	-238·5	2740
B	108	-62·0	2660
C	112	96·0	2660
D	169	230·0	2740

NOTES:

1. THE GUIDE WALLS WERE MODELLED AS SHOWN IN SECTIONS U-U, V-V, W-W AND X-X. THE GEOMETRY AND LOCATION OF THE UPSTREAM FACE MUST REMAIN UNCHANGED. THE SHAPE OF THE DOWNSTREAM FACE, THICKNESS OF CONCRETE ETC. CAN BE ALTERED TO BEST SUIT STRUCTURAL AND FOUNDATION CONSIDERATIONS.
2. THE WATER FACE OF THE GUIDE WALLS CAN BE POSITIONED AS FOLLOWS:-
   (i) LOCATE THE LINE OF THE DOWNSTREAM FACE IN PLAN FROM THE CO-ORDINATES GIVEN IN TABLE 1.
   (ii) TERMINATE THE HIGHER ENDS AGAINST THE ABUTMENTS
   (iii) TERMINATE THE LOWER END WHERE THE DOWNSTREAM FACES REACH A LEVEL OF R.L. 2657·5
3. THE WATER FACE OF THE WING WALLS SHOULD HAVE A MINIMUM VERTICAL HEIGHT OF 8 FEET. APART FROM THIS CONSIDERATION THEIR SHAPE IS NOT CRITICAL. FOR MODELLING PURPOSES THE WATER FACE WAS MADE VERTICAL. AT THE JUNCTION WITH THE GUIDE WALLS THE VERTICAL HEIGHT WAS MADE EQUAL TO THAT OF THE GUIDE WALLS AND THEN TAPERED TO A HEIGHT OF 8 FEET AT THE DOWNSTREAM END.

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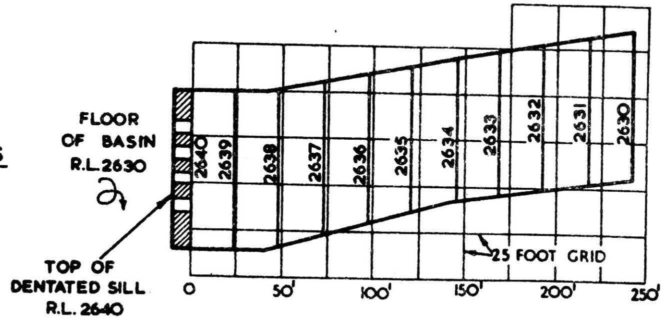
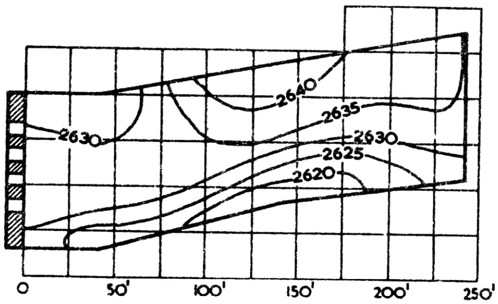
Figure 32: Final Design for Geometry of Spillway Dissipator

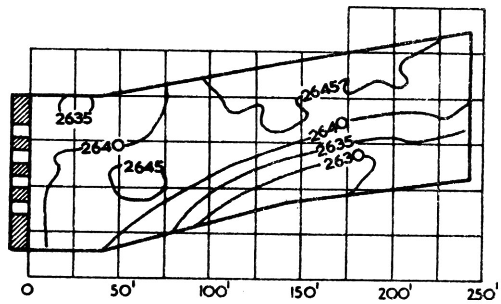
(a.) BED BEFORE TESTS



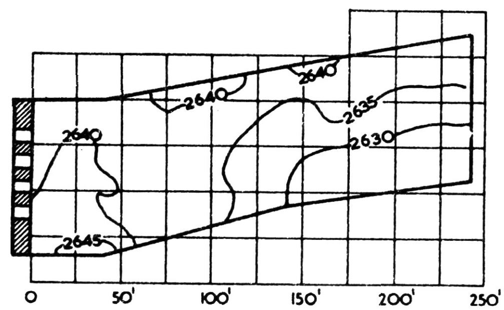
(b.) 43,000 c.f.s.



(c.) 30,000 c.f.s.



(d.) 20,000 c.f.s.



(e.) 10,000 c.f.s NEGLIGIBLE SCOUR

FIGURE 33: FINAL DESIGN SCOUR PATTERNS

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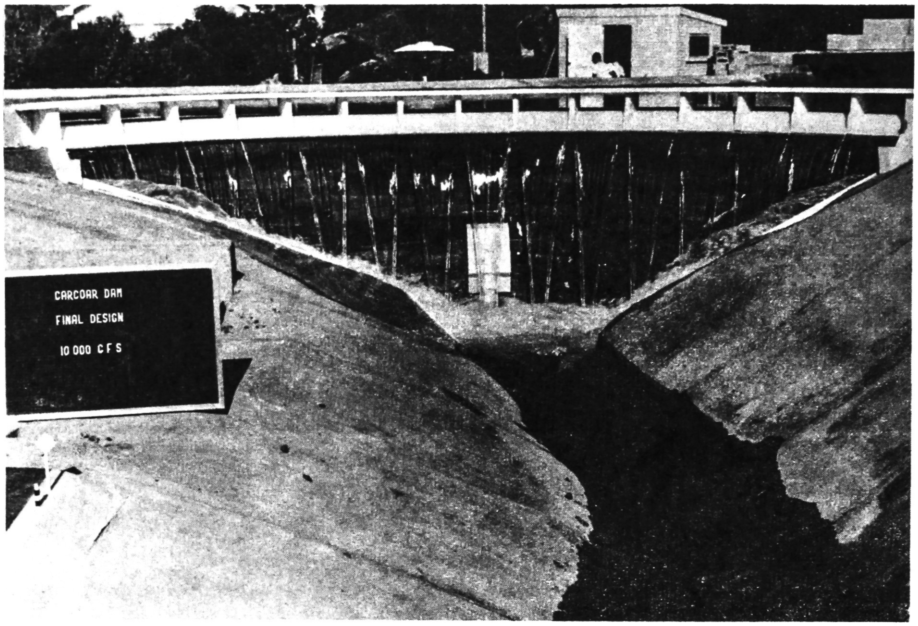
Figure 34: Final Design - Model Operation
10,000 c.f.s.

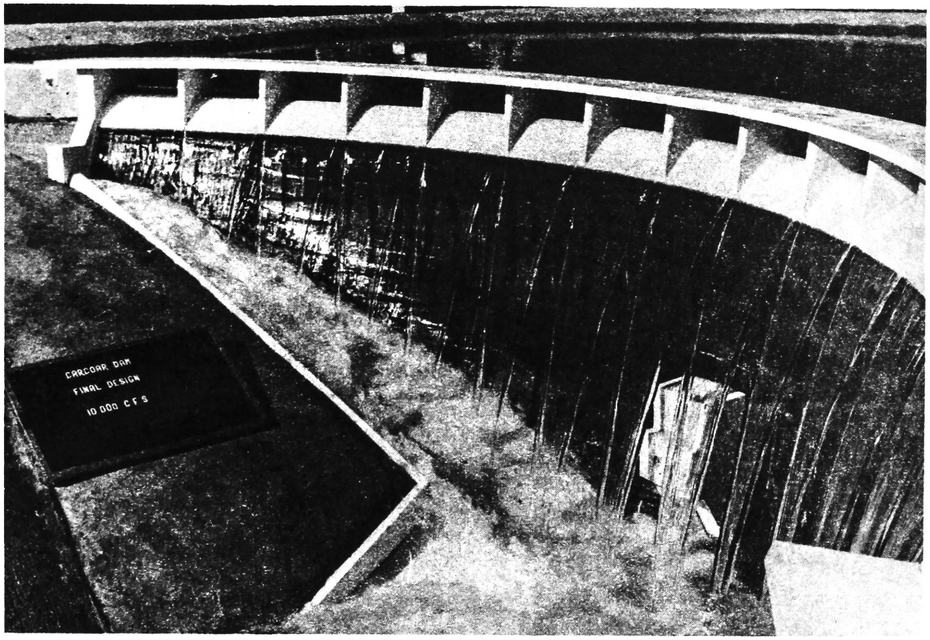


(a)

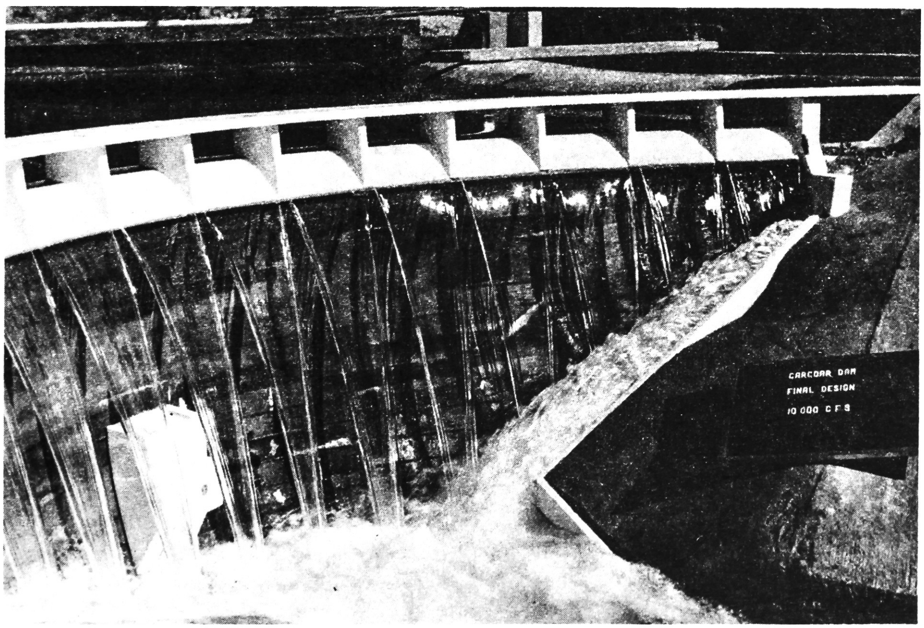
(a) General View from Downstream

(b) Right Hand Paving

(c) Left Hand Paving

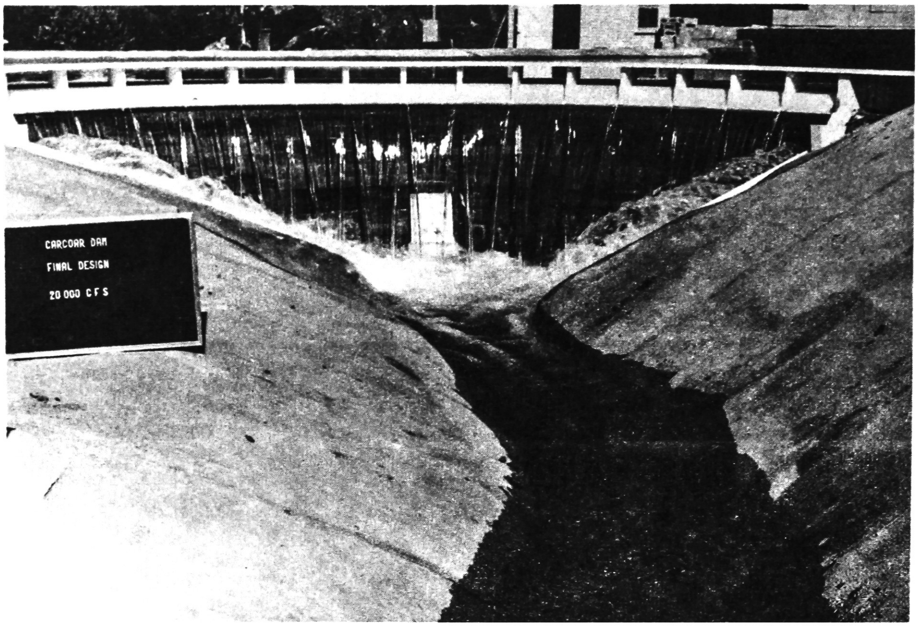

CARCOAR DAM
FINAL DESIGN
10 000 C F S

(b)


CARCOAR DAM
FINAL DESIGN
10 000 C F S

(c)

Figure 35: Final Design - Model Operation
20,000 c.f.s.

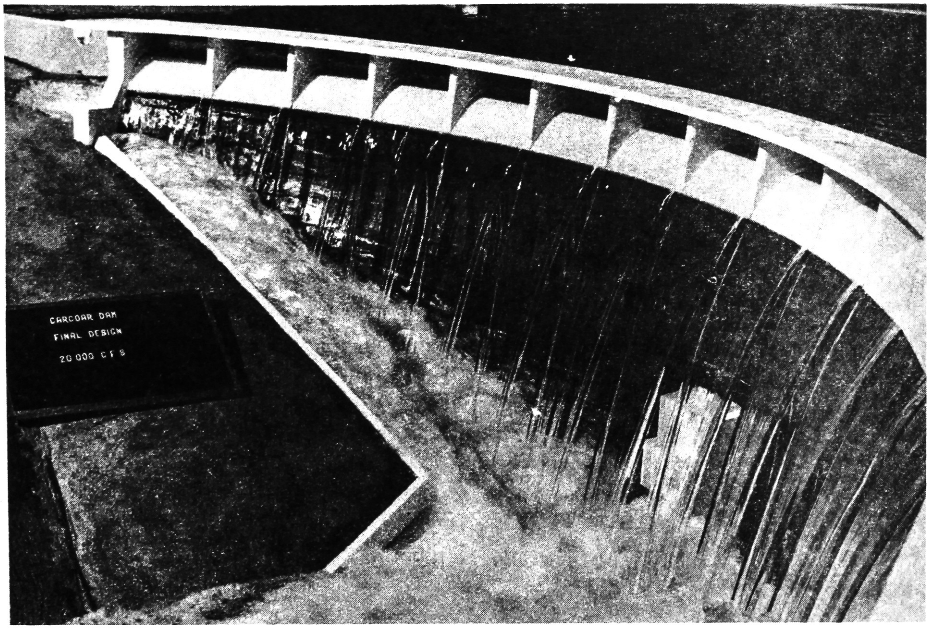


(a)

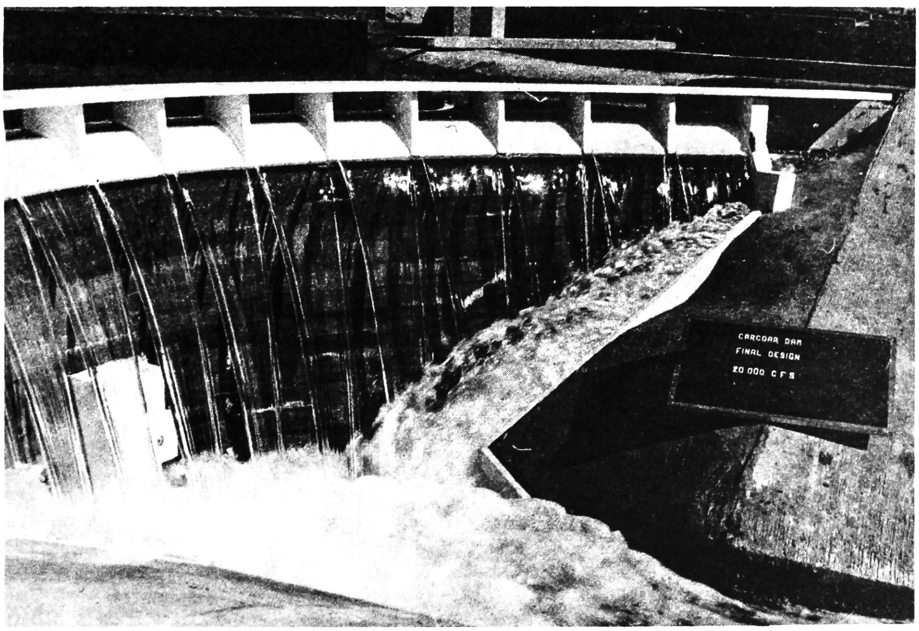
(a) General View from Downstream

(b) Right Hand Paving

(c) Left Hand Paving

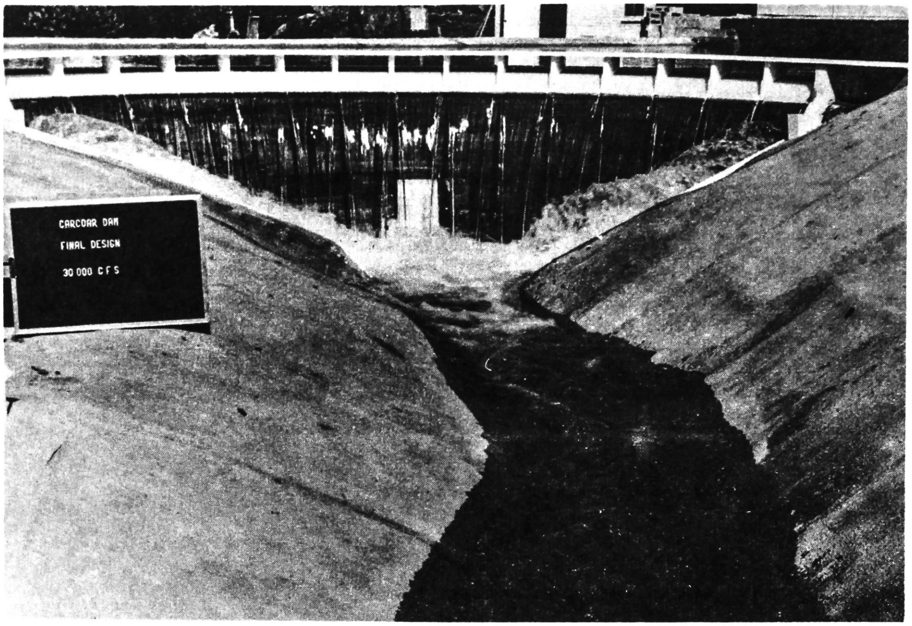

CARCOAR DAM
FINAL DESIGN
20 000 C F S

(b)


CARCOAR DAM
FINAL DESIGN
20 000 C F S

(c)

Figure 36: Final Design - Model Operation
30,000 c.f.s.

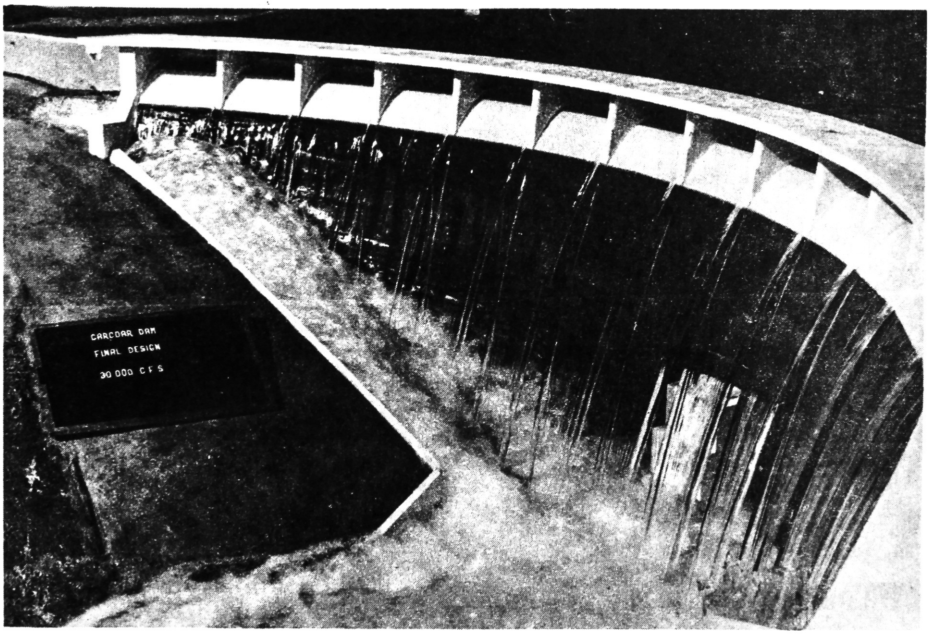


(a)

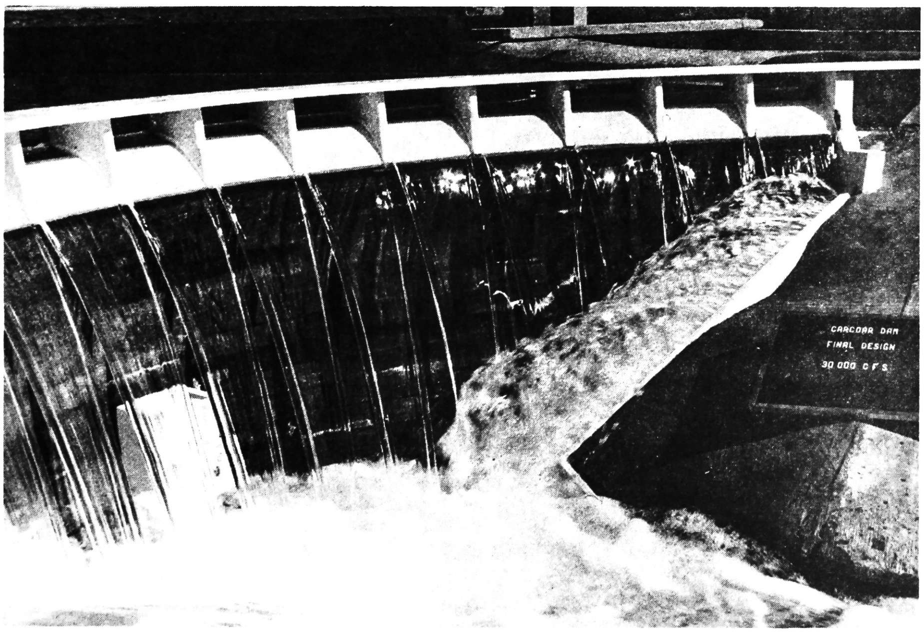
(a) General View from Downstream

(b) Right Hand Paving

(c) Left Hand Paving

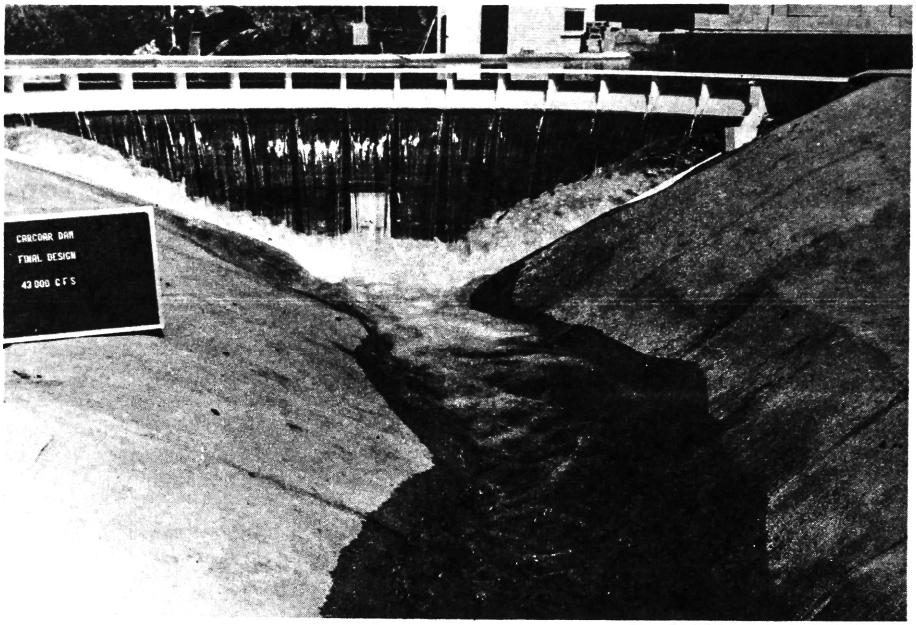

CARCOAR DAM
FINAL DESIGN
30 000 C F S

(b)


CARCOAR DAM
FINAL DESIGN
30 000 C F S

(c)

Figure 37: Final Design - Model Operation
43,000 c.f.s.

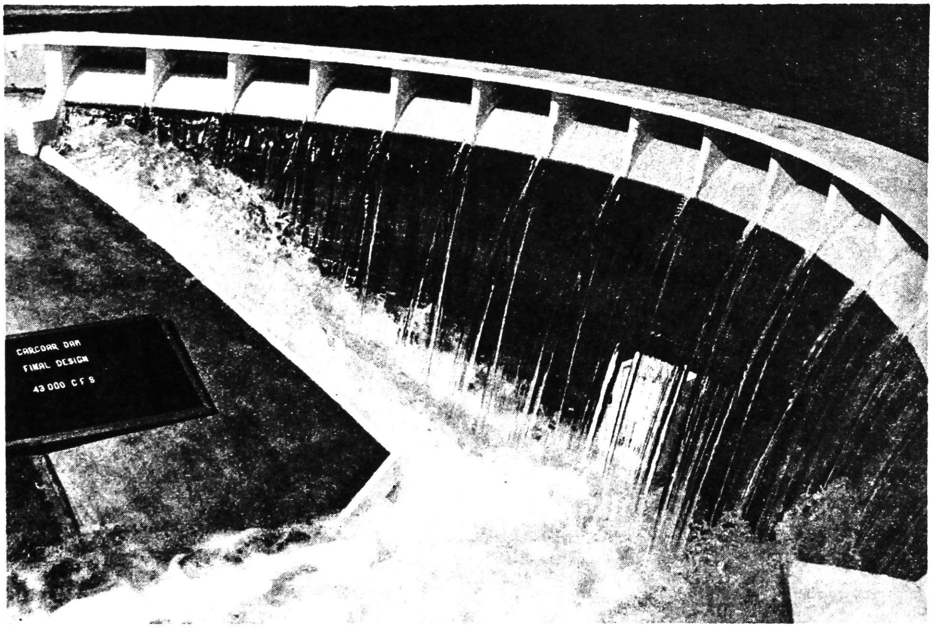


(a)

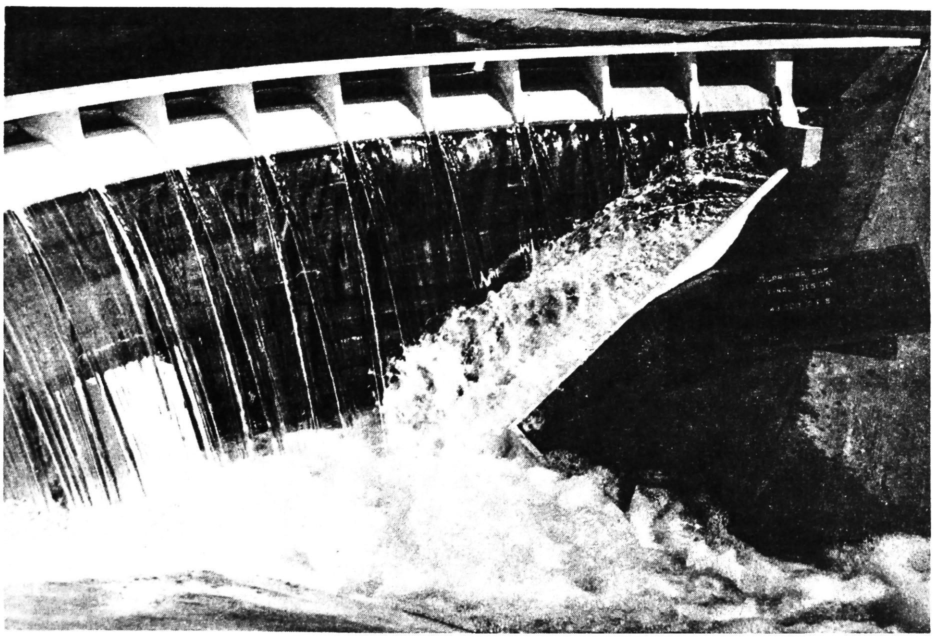
(a) General View from Downstream

(b) Right Hand Paving

(c) Left Hand Paving



(b)



(c)