UNSW SYDNEY

Hydraulic studies for Liddell Power Station. Report No. 110, Vol. 3. Hunter River measuring weir. August 1969.

Author:
Bonham, A. J.; Stone, P. B.

Publication details:
Commissioning Body: Electricity Commission of New South Wales
Report No. UNSW Water Research Laboratory Report No. 110, Vol. 3

Publication Date:
1969

DOI:
https://doi.org/10.4225/53/579fd981ff04d

License:
https://creativecommons.org/licenses/by-nc-nd/3.0/au/
Link to license to see what you are allowed to do with this resource.

Downloaded from http://hdl.handle.net/1959.4/36328 in https://unsworks.unsw.edu.au on 2024-04-26

THE UNIVERSITY OF NEW SOUTH WALES

water research laboratory

Manly Vale, N.S.W., Australia

Report No. 110

https://doi.org/10.4225/53/579fd981ff04d

HYDRAULIC STUDIES FOR LIDDELL POWER STATION

VOL. III: HUNTER RIVER MEASURING WEIR

by

A. J. Bonham and P. B. Stone

August, 1969

The University of New South Wales
WATER RESEARCH LABORATORY

HYDRAULIC STUDIES FOR LIDDELL POWER STATION

VOL. III: HUNTER RIVER MEASURING WEIR

THE UNIVERSITY OF NEW SOUTH WALES
WATER RESEARCH
LABORATORY
LIBRARY

by

A. J. Bonham and P. B. Stone

Report No. 110

Final Report to the Electricity Commission of New South Wales

August, 1969.

Preface

The work reported herein is part of a comprehensive series of engineering investigations and hydraulic model studies carried out at the Water Research Laboratory on behalf of the Electricity Commission of New South Wales. The results are reported in a series of volumes, this report being Volume III of the series of seven.

Because of the special conditions affecting the draw-off water from the Hunter River for use at Liddell Power Station, measurement of dry weather flows has many problems not commonly significant in setting up gauging weirs. The solution of this problem has necessitated the comprehensive investigation undertaken.

Throughout the studies, the effective advice and coordination achieved by the Electricity Commission Engineering Staff is gratefully acknowledged and in particular the services of Mr. C. G. Coulter and Mr. N. Lamb.

The detailed supervision of the work at the Laboratory was carried out by Mr. A. H. Bonham under the direction of Mr. P. B. Stone, Supervising Engineer, and Mr. D. N. Foster, Senior Lecturer in Civil Engineering.

R. T. Hattersley,
Associate Professor of Civil Engineering,
Officer-in-Charge.

Summary

This report describes field and laboratory investigations into a gauging structure on the Hunter River at Jerry's Plains. The gauging structure was required to measure river flow downstream of the Jerry's Plains pumping station.

Field surveys were undertaken to establish flow characteristics and sediment type at the site.

Two dimensional model tests were made on a Parshall flume section and a Crump weir section to assess their ability to pass sediment which may be moving along the bed. Some aspects of the structural stability were also investigated using two dimensional models.

The ability of the structures to resist sedimentation and outflanking was studied in a three dimensional model.

The recommended structure was a sloping line crest (Crump) weir.

REPORT 110.

HYDRAULIC STUDIES FOR LIDDELL POWER STATION.

INDEX TO VOLUMES

Table of Contents

Table of Contents (cont'd.)

Table of Contents (cont'd.)

List of Figures.

List of Figures (cont'd.)

List of Photographs

Water Research Laboratory Library

1. Introduction

As part of the Liddell power project a pumping station was designed for the Hunter River at Jerry's Plains. This will supply water to the new lake created to supply Liddell Power Station with cooling water.

The pumping station is of sufficient capacity to draw off the whole of the flow in the river in its low stages. To ensure that riparian requirements are met downstream of Jerry's Plains it is desired to gauge the residual discharge downstream of pumping station.

Because of its alluvial bed the Hunter River lacks suitable natural gauging sites at which flow at low stages could be reliably measured. The Electricity Commission of N. S. W. has therefore decided to instal an artificial gauging structure downstream of the pumping station. The site was chosen some two miles downstream of the pump intake and the Water Research Laboratory was charged with investigating a gauging structure to accurately measure flows up to 400 c. f. s. The problem was to construct a structure suitable for telemetry which will run with a minimum of maintenance and attention, with reliability and accuracy for low flows, notwithstanding the wide range of river flows, the heavy bed load, the risk of river outflanking and the risk of structural collapse due to scour in times of flood.

2. River Morphology

2.1 General

The Hunter River at Jerry's Plains is a mature graded stream. A mature river will be contained in, and may be meandering in, a wide alluvial flood plain located in a valley and side slopes which are still eroding. Sediment load will be intermittent movement, partly derived from the valley sides and partly from young head streams.

The Hunter River contains a wide range of sediment sizes from six inch cobbles, through gravel, sand and silt to clay.

2.2 Morphology of the Hunter River near the Gauge Site

2.21 Locality and Topography

The locality is shown in Figure 1. The site and reaches upstream and downstream were surveyed for the Electricity Commission and additional flood plain contours were obtained from aerial photos by the University of New South Wales Surveying Department of the School of Civil Engineering.

The river and flood plain topography is shown in Figure 2. It can be seen that the river has meandered to the extreme left side of the flood plain at the gauging site. The upstream reach is fairly straight for a mile or so and at the site itself the river is deflected to the right where the flow impinges against a rock outcrop at the foot of the steep bank on the left side of the flood plain. It was proposed to found the recorder tower on this rock.

The straight upstream reach appears to be stable and not currently meandering to left or to right and a fall of several feet occurs at discharges of less than about 800 cusecs which is brought about by a natural shoal in the river which acts as a control at lower discharges. The shoal is at an angle of about 45^{o} to the direction of flow in the upstream reach and produces a cross current towards the left bank at flows less than about 1,000 cusecs. The left bank is undercut and an island shoal has formed but further erosion of the bank is prevented by the underlying rock which rises under the steep left bank. The rock outcrop deflects the downstream flow to the right.

Downstream the steep left bank is well covered with trees and scrub. Upstream there is a row of trees on the line of the left bank. On the right bank vegetation is sparse with signs of slow recovery after the movement of sediment during past flood discharges in the river. (See photographs 1 to 6).

2.22 Flood Levels

Figure 3 shows backwater curves at the gauging site for 169, 353, 530, 1,370, 3,930 and 24,000 cusecs as well as backwater curves for higher discharges (see Photographs 1, 2, 3, 4, 5 and 6).

At discharges exceeding 30,000 cusecs, the stability of the flood plain dictates that the steady state water levels will be substantially parallel to the general valley slope.

Manning's "n" was computed by using observed surface velocities and an average value of 0.026 was obtained for the discharge of 24,000 cusecs, and also for the discharge of 3,930 cusecs.

The backwater curves at the intake site were computed by the Electricity Commission assuming values of Manning's "n" of 0.020 in the channel and 0.040 overbank. The value of n = 0.026 is in reasonable agreement when allowance has been made for the arbitary divisions between channel and overbank at the two sites. The actual backwater curves at the gauging site are consistent with the backwater curves of corresponding computed discharges at the intake site regarding vertical spacing as would be expected from the consistent character of the channel, having made allowance for the sharper bend at the intake site.

2.23 Flood Plains

The flood plain system in the Hunter River at this locality is complex. There are several distinct flood plains which flood progressively as the flow increases (Fig. 2).

The river channel is some 150 to 200 feet wide although at very low flows the stream may meander in the bed of the channel except where ponded by shoals. Above the gauging site the stream is ponded and below the gauging site the stream meanders in the bed. The bank full capacity of the channel is some 1,500 cusecs.

The first flood plain is 200 feet wide with a capacity of some 15,000 cusecs.

The second flood plain varies greatly in width but will contain some 100,000 cusecs at the gauge site.

The third flood plain extends almost to the township of Jerry's Plains and is considerably braided.

Each of the three flood plains shows evidence of sediment movement under flood in recent times. General movement of sediment on the higher flood plains will occur at higher stages and will overshadow movement which may still be in progress on lower flood plains or in the dry weather channel.

2.24 Hydrology

The hydrology of the river has been dealt with extensively in Electricity Commission and Water Research Laboratory reports (Ref. 4).

Flood frequency curves (Fig. 4) show that at Jerry's Plains 20,000 cusecs can be expected annually and 70,000 cusecs can be expected once in 12 or 15 years.

The flow duration curve (Fig. 5), shows that 200 cusecs may be exceeded 40 pc. of the time, 450 cusecs may be exceeded 20 pc. of the time and 1,500 cusecs may be exceeded 10 pc. of the time.

The general form of the flood hydrographs at Singleton, (Figs. 6, 7 and 8) were utilised in the model testing programme.

2.25 Sediment Movement

A wide range of particle sizes of bed sediment were taken at the intake site (Fig. 9). A bank deposit of sediment deposited at the intake site consisted of medium sand. Samples of the bed and bank sediment at the gauging site have similar gradings.

A uniform deposit of medium fine sand was sampled from Sections 17 and 18 (Fig. 2) near the gauging site in the bed of the river (Fig. 10).

The surface traces of the various sizes of sediment are shown in Fig. 11 as well as the positions of the E.C. boreholes. The borehole logs indicate undisturbed sedimentary rock beneath the river and flood plain at RL. 200 to 210, rising steeply beneath the steep left bank (Figs. 12 and 13). Overlying the rock is sand followed by gravel and the surface material of the river bed and flood plain contains coarse gravel. The high left bank consists of deep gullying topsoil, silt and sand.

Figures 14 and 15 show changes in bed level that have occurred at Sections 9 and 10 near the intake site over a period of 2 years. It can be seen that degradation in the bed may easily occur to the extent of two feet, after a flood or fresh.

Figure 16 is the bed sediment rating curve for the river at Sections 17 and 18 for the bed sediment shown in Figure 10. The modified Einstein procedure was adopted. Very little movement of sediment occurs in the natural stream bed at discharges less than 300 cusecs. However, local scour may still occur near the gauging structure due to the concentration of flow and local increase in hydraulic gradient caused by the structure. General movement of this medium fine sand occurs at about 5,000 cusecs.

The behaviour of the river is interpreted as follows. A wide range of particle sizes of sediment is in intermittent transport in the river channel and flood plains. The sediment may be found with a mixed or segregated particle size composition. The sediment predominantly consists of medium sand, gravel and stones up to 6 inches in diameter. Finer material as found in the left bank may be swept away as a suspended load.

A higher tractive stress is required to shift the larger material than the smaller material. Sand in the 0.2 to 0.3 mm range moves most freely as bed load when and where tractive stresses are high. The high gradient in the river near the gauging site at discharges below 800 cusecs precludes the settlement of the medium fine sand in this location, whereas it is deposited fairly continuously in the bed upstream and downstream. However, photos taken by the W.R.L. from the air show that sand shoals or dunes occur in these low gradient reaches, and the shoal spacing appears to be two or three river widths. The sediment rating curve for the medium fine sand applies for lower discharges but for higher discharges other more complex mechanisms come into play.

Sand, gravel and stones appear to be segregated in the shoals and in the river channel but mixed in the flood plain. The shoal at Section 17 which forms the present control in the river consists of the largest sized stones which have come to rest in an interlocked condition, presumably the stones which failed to become interlocked and the smaller material were swept away due to the steep hydraulic gradient and high tractive stress at that location.

The shoal itself was probably formed during a very large flood and shows every sign of stability. The gauging structure will replace this shoal as the low flow control in the river.

2.26 Pools and Riffles

A field survey was made on 19th April 1968 to the Hunter River to look for evidence of the extreme range of bed movement likely to occur in the reaches immediately upstream and downstream of the gauging site. In order to design a stable and economic structure it is necessary to confirm the probable range of bed aggradation and degradation and to confirm estimates of the lowest probable tailwater levels.

It was observed that cobble and gravel shoals form controls at intervals of 4,000 - 5,000 feet in this part of the river. Sand bed and sand shoals occur between these controls in some places but not in others, (see Fig. 17).

These controls have been termed riffles (Ref. 1.11). Figure 18 shows a plan and profile of a hypothetical channel with pools and riffles. In the Hunter River, one riffle occurs upstream of the gauging site, one at the downstream low level bridge and a third in between.

Probings were made generally in the sand bed of the river and through the sand shoals commencing at the downstream low level bridge and including the reach upstream of the gauging site. A probe was used specially designed to pass smoothly through the sand but with moderate difficulty through underlying gravel in order to determine accurately the sand/ gravel interface.

Where sand shoals occur, the depth of sand may exceed 5 feet below the low flow energy grade line in the pool between riffles, but generally the depth of sand did not exceed 2 feet. Sand shoals also form controls in the river between riffles at low flows. Some parts of the pools contained sand and some parts contained no sand. No sand occurred near the riffles. Figure 18 shows the profile diagramatically of a zone of sand bed. Figure 17 is a plan showing the principal areas of bed sand and sand shoals which remain after the recent flood of 24,000 cusecs in mid January.

Figures 19 and 20 show sections 9 and 18 respectively in the pools downstream and upstream of the gauging site. The sand bed levels surveyed on 10th August 1967 are shown as well as the sand bed levels surveyed on 10th April 1968 and the underlying gravel beds located by probing at the same time. These sections were chosen as typical of conditions at the present time in the portions of these two pools where a continuous sand bed was present but sand shoals were absent. It can be seen that the bed has been lowered some 12 inches by the January 1966 fresh and some 6 to 12 inches of sand remains over the gravel. Downstream of the gauging site, at Section 9, the lowering of the sand level has apparently resulted in an almost corresponding lowering of the stage discharge curve at low flows.

2.27 Possible Changes in the Location of Bed Sand

The field trip of April 1968, the survey of August 1967, the aerial photographs of August 1965 and the aerial photographs of April 1956 show that the locations of the sand shoals differ markedly. Changes in the locations of gravel riffles are small and much less obvious although it should be noted that the small gravel island at the gauging site riffle was just a very small shoal in 1956. However, the distribution of sand in the pools between riffles changed significantly in January 1966 at the time of the 24,000 cusec fresh. Some 200-400 feet downstream of the gauging site deep scour holes now exist down to approximately RL. 207 especially downstream of the zone of very turbulent river flow caused by the rock outcrops. It is not known if the material in these scour holes was sand or gravel prior to the January fresh, but it was most probably only sand. However, these holes are well away from the proposed gauging structure.

2.28 Discussion

It would seem that the sand bed may move by some slug mechanism between gravel riffles in addition to the more usual methods of bed movement involving dunes and ripples. The reason for the sand slugs may be an intermittent sand feed from the Hunter River tributaries due to the varying sediment characteristics of these streams combined with random areal rainfall distribution.

It would therefore seem that from time to time the bed of the river may be starved of sand at a particular location and the bed may scour down to the level of the gravel. The sand shoals and sand bed give secondary control to the pool energy grade or slope between riffles. The maximum lowering of the tailwater level at the gauging site would tend to approach the maximum lowering of the bed likely to occur, which would sensibly be the scouring out of all of the sand down to the gravel in the downstream pool.

2.29 Conclusion

Provision must be made for the effect on the tailwater curve of the complete absence of sand in the downstream pool since the location of sand in the river can change significantly after a flood of 24,000 cusecs with a return period of less than two years. The range of future conditions likely to occur in the pools upstream and downstream of the gauging site is as follows:-

(a) The river bed starved of sand, without a continuous sand bed and without sand shoals.

(b) A continuous sand bed but without sand shoals.

(c) A continuous sand bed and continuous trains of sand shoals.

Since the reach downstream of the gauging site contained an estimated average of two feet of sand during 1967, and having regard to the scour holes downstream of the gauging site, it is concluded that the structure should be designed to withstand a lowering of tailwater by at least two feet below the tailwater stage discharge relationship observed during 1967. The minimum probable low flow tailwater level is therefore + 212.

2.3 Design Requirements

2.31 Design Considerations

Regarding toe scour, it is considered that design allowance should be made for degradation of two feet in the channel bed downstream of the gauging site.

For flood discharges, the river channel and flood plain show promise of stability near the gauging site, at least up to 70,000 cusecs. This is because the river is not actively meandering in the vicinity of the gauging site, since the upstream reach is fairly straight and erosion of the outer bend at the gauging site is restricted by the rock level which rises under the steep left bank.

The structure must be designed to preserve the present river stability, especially on the right bank where the end of the structure must achieve a stable connection to the alluvial flood plain.

The structure must inhibit any tendency that the river may have to outflanking of the structure, which might occur by the following sequence of events. Since the structure must raise the control level in the river, the upstream reach might aggrade with medium fine sand so that at higher discharges a steeper hydraulic gradient might occur on the flood plain at the end of the structure, producing locally an increased sediment movement beyond the end of the structure leading to scour. Such scour, if deep enough, could result in diversion of the river about the end of the structure. The river channel would then outflank the structure with the structure acting as a groyne on the left bank. The structure must be extended and raised sufficiently on the right bank to inhibit this possible tendency to outflanking of the structure by the river.

2.32 Summary of Design Requirements

1. The structure must preserve the stability of the river channel and flood plain, and inhibit any tendency to outflanking of the structure by the river.

2. The structure must be stable in itself despite downstream degradation in the river channel.

3. Any tapping pipes must not block up with bed sediment.

4. The structure should not attract the accumulation of sediment and drown itself out. All sediment in transport approaching the structure should be conveyed across the structure without seriously affecting the rating of the structure. That is to say, the structure should operate with the minimum of maintenance dredging and clearing.

3. Two Dimensional Tests - First Series

3.1 Introduction

This series was designed to evaluate the more important characteristics of a variety of structures, namely;

(i) the scour characteristics of the piled cutoff wall proposed for use with a Parshall flume (Figures 21, 22);

(ii) the scour characteristics of a sloping concrete apron alternative to (i) (Figure 24);

(iii) the sediment passing capacity of a typical section of a wide Parshall flume;

(iv) the sediment passing capacity of a typical section of a wide Crump weir.

3.2 Equipment

The design requirements for the gauging structure called for model investigations at a large scale because of problems associated with scaling the wide range of sediment which is in intermittent transport in the Hunter River.

A 6 inch wide flume was built (Fig. 23) having a slotted tailgate and a sand feed head box. Flows of up to 6 c.f.s. per foot width were attained.

3.3 Test Procedure

Terminal scour tests were carried out using medium fine sand. Fine sand was not used because of its tendency to ripple. In addition the tests were concluded using a particle size to represent the stone to be found in the bed of the river. The material used for this was 1/4 inch blue metal mixed with coarse sand.

The procedure adopted was to obtain terminal scour with medium fine sand and then to add the blue metal mix. It was observed that the blue metal would gradually settle into the sand until the final profile of the scour hole was substantially the same as with sand only.

Test programs included long periods of steady state discharges for flows up to an equivalent of 10,000 c.f.s. prototype as well as hydrographs of the approximate form shown in Figures 6, 7 and 8.

3.4 Results of Tests

3.41 Piled Weir

Two dimensional tests were run for a proposed weir of sheet piling adjacent to the Parshall flume. The weir crest level was taken as RL. +220 suitable for a 20 ft. wide Parshall flume with a crest level at RL. +217.

In the range below 15,000 c.f.s. the tailwater stage discharge relationship used in testing was 2 feet lower than that applying in the river at the present time to allow for possible degradation of the stream bed. The worst terminal scour occurred at 10,000 c.f.s. when the tailwater level was at RL. +220. Hydrographs were run to represent typical Hunter River floods.

Figure 22 shows the worst case of terminal scour. The model scale was 1 in 20.

Figure 25 shows the armouring required to protect the sheet piling. No factor of safety was applied and the stone apron was only just stable. The model scale was 1 in 10.

3.42 Concrete Sloping Apron Alternative

As an alternative to the sheet piled cutoff a sloping apron shown in Figure 24 was tested. The sloping sill reduced the strength of the bottom jet at high flows deflected the jet upwards such that at all discharges a back eddy was produced resulting in a protective slope of sediment against the toe of the structure. The model scale for these tests was 1 in 10.

By adopting a top level for the sill of RL. +213 the sill will always be submerged by at least one foot of water and no appreciable scour will occur at the toe.

3.43 Parshall Flume

The classical Parshall flume produces critical flow partly by side contractions and partly by changes in level of the floor slab. The Hunter River structure (Figure 21) is so wide that the effects of side contractions are small and the structure acts much like a broad crested weir.

A two dimensional section of the Parshall flume was tested with a sediment laden flow. The pressure tapping was level with the invert and drew in a mixture of sand and water to the float well. In the piezometer system used in the model the concentration of sand drawn in was 2.8 times the average concentration but the piezometer tube did not block on a rising flood. However, when standing waves were present in the upstream section of the model the piezometer tube blocked rapidly.

Bad scour downstream of the structure indicated that the structure should be founded on rock.

3.44 Crump Weir

The Crump weir is as shown in cross section in Figure 26. The approach apron serves to establish steady uniform flow conditions at the section where the pressure tapping is located.

A two dimensional section of the Crump weir was tested in the sediment flume.

The side wall tapping was inserted with the soffit of the pipe level with the weir crest and a negligible amount of sediment accumulated at this point during testing. The crest tapping, which can be used as an indication of whether the weir is drowned, was rapidly blocked by sand. No useful purpose would be served by installing such a tapping in the Hunter River weir.

All sediment sizes passed freely over the weir, although, at high rates of sediment feed a small wedge was deposited at the foot of the upstream 1 in 2 slope. The rates of feed used to produce these conditions were in excess of those normally found in the prototype.

3.5 Comparison of Parshall Flume and Crump Weir

Both structures are such that two dimensional preliminary tests gave reasonable indications of their potential.

3.51 Pressure Tappings

The Parshall flume has a broad crest in which the piezometric tapping is necessarily positioned. The tapping is prone to blockage by sediment especially when there are waves upstream. The Crump weir piezometric tapping is located clear of any bed sediment.

3.52 Sediment Accumulation

At high flood discharges both structures can be expected to accumulate bed sediment but this will clear away during the recession as the structures ceased to be drowned out. Under these conditions the tests showed that both were able to pass any sediment load carried by the Hunter River.

3.53 Parshall Flume Reverse Apron

The Parshall flume incorporates a reverse slope downstream apron originally devised for use in a lined conduit. The apron appears to serve no useful purpose in a mobile bed.

4. Two Dimensional Tests - Second Series

4.1 Introduction

The first series of tests showed the Crump weir to be most promising of the types tested. This series was designed for a more complete evaluation of its performance. The requirements of the structure were:-

(i) the entire length of the slope line crest weir should act as a measuring crest over the required range of flows;

(ii) the downstream toe should be structurally stable for all discharges and for any probable downstream bed degradation preferably without recourse to sheet piling;

(iii) the crest should be set at a suitable level in relation to the sediment level so that the structure will remain modular (not drowned out) over the requisite range of flows;

(iv) if the structure were buried during a flood it should clear itself of sediment during the recession;

(v) the coefficient of discharge should remain constant in the modular range unaffected by bed sediment load for the range of sediment sizes found in the river.

4.2 Test Procedures

Testing was carried out in the 6 inch wide flume shown in Figure 23. This flume had a maximum discharge of 3 c.f.s. which permitted testing of a full size model of the crest section in the modular range of discharges. Figure 23 shows the model installed in the flume. The slope downstream of the crest was 1 in 5 and upstream removable sections permitted testing of slopes of 1 in $1\frac{1}{2}$, 1 in 2 and 1 in 3.

Flows greater than 0.55 c.f.s. were measured with a Dall Tube and flows less than 0.55 c.f.s. were metered by a 2 inch orifice plate in a 3 inch line. Volumetric calibration gave the relationship $Q = 1.14h^{\frac{1}{2}}$ for the Dall tube and $Q = .12h^{\frac{1}{2}}$ for the orifice meter.

Previous tests have demonstrated that the Crump weir is capable of passing sand sizes in the modular range of discharges and so the sediment sizes used in this series of tests were 3/8, 3/4 and $1\frac{1}{2}$ inch gravel and mixtures of these sizes. The width of the flume set the practical upper limit at $1\frac{1}{2}$ inches.

4.3 Preliminary Tests

4.31 General

Preliminary tests were carried out using 3/8 inch gravel and a discharge of approximately 5.9 c.f.s. per foot to study the bed forms and coefficients of discharge with this size of sediment. Table 1 is a summary of the results obtained and Figure 27 shows some typical profiles.

4.32 Discussion of Results

The approach channel to the weir was long enough to establish uniform flow conditions over a length of 10 feet and so could be representative of conditions in the field.

Test 1 with no sediment and a low apron gave a coefficient of discharge of 3.55. At this discharge the tailwater level was approximately 1.1 feet above the weir crest for which the theoretical discharge coefficient is 3.48 (Ref. 3.6) showing agreement to within 2 per cent. Subsequent tests showed the changes in discharge coefficient attributable to changes in apron or sediment levels.

It was found that the upstream sediment level was unaffected by the apron level where the apron was located below the experimentally determined bed level. Similarly, the provision of a transition curve between the apron and upstream slope did not materially affect sediment movement.

In general, it can be stated that with an upstream slope of 1 in 2 an error in the coefficient of discharge of 3 to 4 per cent can be expected if there is a copious supply of 3/8 inch sediment in movement. However, below 600 c.f.s. the material in movement will be sand so that only after freshes in the river, if at all, will maintenance clearing of the upstream apron be necessary.

It was noticed that with high upstream Froude Numbers surface waves were set up by bed irregularities and a further series of tests was planned to investigate the effect of these waves on the measured head.

4.4 Piezometer Comparison Tests

4.41 Objectives

The objective of these tests was to select the most reliable piezometer system for use with the Crump weir. Ideally such a system should yield constant or predictable coefficients of discharge under variable conditions of sediment size and sediment level experienced in the Hunter River. It should also be insensitive to effects of standing

Table 1: Tests with Crump Weir and 3/8" Gravel (see Figure 27)

Test No.	Discharge per ft. width q.	Total Head ft. H_1	Bed level below crest ft. P	Experimental Coeff. C_d	Pc. Change in C_d	Velocity at tapping, U(f. p. s.)	Velocity in Upstream Channel (f. p. s.)	Remarks
1	5.60	1.35	0.745	3.55*	0	2.82	0.125	No sediment, low apron 1:2 upstream slope.
2	4.05	1.46	0.525	3.295	7.1	2.49	0.119	Slight movement of sediment, 1:2 upstream slope.
3	5.27	1.33	0.53	3.435	3.2	2.99	0.157	Sediment movement 1:2 upstream slope.
4	5.53	1.36	0.540	3.45	2.8	3.09	0.166	General sediment movement 1:2 upstream slope.
5	5.84	1.43	0.565	3.42	3.6	3.16	0.168	General sediment movement 1:2 upstream slope.
6	5.88	1.43	0.570	3.44	3.0	3.16	0.168	Movement of sediment, 1:2 upstream slope.
7	5.88	1.43	0.575	3.44	3.0	3.16	0.168	Slight movement, 1:2 upstream slope.
8	5.86	1.43	0.58	3.43	3.3	3.16	0.168	Slight movement 1:3 upstream slope.
9	5.82	1.44	0.58	3.38	4.7	3.14	0.164	Slight movement 1:1½ upstream slope.
10	5.85	1.43	0.58	3.42	3.5	3.16	0.151	Slight movement, raised apron 1:2 upstream slope.
11	5.84	1.45	0.45	3.34	5.8	3.39	0.207	Slight movement over raised apron 1:2 upstream slope.
12	5.90	1.44	0.575	3.425	3.4	3.18	0.170	Curved transition. Slight movement 1:2 upstream slope.
13	5.85	1.43	0.570	3.42	3.6	3.15	0.151	Curved transition. Slight movement 1:2 upstream slope.

*Theoretical value C_d = 3.48

waves upstream of the crest.

4.42 Types Tested

Four types of piezometer tappings were tested.

4.421 Crest Side Tapping

In the preliminary tests it was observed that there were no surface waves when the Froude number was greater than 1. For this reason a 1/8 inch tapping was tested located 1 inch above crest level in the side wall.

4.422 Manifold Tapping

A manifold tested consisted of six 3/8 inch holes spaced at 9 inch centres on a level with the crest. This seemed a practical method of averaging the pressure variations caused by standing waves.

4.423 Standard Crump Tapping

This 3/8 inch tapping was located in the side wall on a level with the crest and 57 inches upstream from the crest.

4.424 Ground Water Pressure Head

A tapping point was located in the porous bed material below the standard Crump tapping. This was an attempt to obtain mean water level when standing waves were present.

4.43 Sediments Used

Three sediment sizes were used in the tests. They were:-

(i) Nepean River sand which approximates to Kramer No. III sand (Figure 28, Ref. 2.2, 2.3).

(ii) 3/8 inch gravel with a specific gravity of 2.44 and an average weight of .001 lb.

(iii) 3/4 inch gravel with a specific gravity of 2.56 and an average weight of .007 lb.

4.44 Discharge Reduction Factor

Figure 28 shows the discharge reduction factor for drowned conditions.

In this figure the ratio of discharge to modular discharge for the same head is expressed in terms of the ratio of the tailwater level to the upstream head

$$\frac{Q}{Q_{MOD}} = f\left(\frac{H_2}{H_1}\right)$$

4.45 Apron Levels

Three upstream apron levels were tested being 0.750, 0.516 and 0.150 ft. below crest level.

4.46 Test Results

4.461 Upstream Standing Waves

Some 120 successful tests were completed with various sediment sizes and apron levels. In 34 of these there were standing waves upstream from the crest.

Figure 30 shows the wave lengths of the upstream standing waves over the experimental range of upstream depths of flow and upstream average velocities (V). One, two or more waves were formed and characteristically the first or second wave upstream from the crest was of a maximum amplitude which in several tests exceeded 0.2 feet. A standing wave system in the test flume could be eliminated by careful grading of the bed and consequent prevention of any perturbations arising from an uneven moving bed. However, in the natural stream, perturbations will often occur due to bed irregularities notwithstanding any horizontal upstream apron construction provided at the weir.

The wave length and location of significant standing waves is the main factor in the location and proportions to be adopted for a manifold piezometric array. At discharges of 3 c.f.s. the first wave crest occurred at some 2 to 3 feet upstream of the weir crest and at 6 c.f.s. some 3 to 4 feet. Figure 30 shows that the wave length has no simple relationship with either upstream depth of flow, upstream velocity of flow, Froude number or discharge.

Table 2 is a comparison of the standard Crump tapping and the manifold tapping in tests where there were upstream standing waves. The comparison was based on the variation of tapping head in the flume. The mean deviation for the Crump tapping was 0.86 per cent and its maximum deviation was 3.59 per cent. Using the manifold tapping resulted in a mean deviation of 0.52 per cent. Under these conditions the manifold tapping was more effective than the standard Crump tapping. It was particularly effective where its length coincided with the prevailing stable wave length or a multiple of it.

4.462 Coefficient of Discharge

The purpose of the results contained in this section is to demonstrate the effect of small changes in bed levels, bed roughness and shape on the coefficient of discharge.

Figure 31 shows discharge coefficients obtained from the sidewall tapping above the crest. The coefficient of discharge appears to be a function of P, H_1/P, and H_1 and H_2 (see Reference 3.7). The change in slope of the characteristics in Figure 31 at total heads of greater than 1.5 to 2.2 feet is due to drowning.

Figure 32 is a plot of results obtained for the manifold tapping. The mean value is 3.55 and the scatter is contained within a band of ± 4 per cent.

Figure 33 is the result of tests on the standard Crump tapping and Figure 34 is the result of measurement of piezometric head in a porous bed below the Crump tapping. Both show a variation of ± 4 per cent.

4.47 Conclusions

Coefficients of discharge based on the weir crest tapping are independent of upstream waves but are markedly influenced by upstream apron or bed levels. The other methods gave similar accuracies over the full range of tests but where there were upstream waves the manifold tapping was obviously superior.

For these reasons, it is recommended that a manifold tapping plate as shown in Figure 35 be used in the Hunter River measuring weir. For comparison Figure 36 shows a standard Crump tapping plate.

5. Instrumentation

A preliminary examination was made of pressure measuring and recording systems to assess their accuracy and reliability. It was decided that a conventional float well would be the most suitable for the gauging structure.

5.1 Mechanically driven with Float Well

Suitable equipment available compatible with telemetering systems is the Stevens Memomark Encoder mechanically driven by a water level sensing float.

It is usual to provide a 4 foot diameter float well for a deep well installation, complete with step irons for access purposes.

In order to minimise diurnal temperature effects on the float line the tower should be kept as short as possible.

Figure 2 shows that the highest and largest flood plain has a general level less than RL. 250 at Jerry's Plains. A discharge of 200,000 cusecs with a frequency of one in a hundred years would have a stage level between RL. 245 and RL. 250 at Jerry's Plains. An instrument room floor level in the tower of RL. 250 would be adequate giving a bench level in the instrument room of RL. 253. This would ensure that the float cable would not exceed 40 feet. Instruments which could be seriously damaged by flood water should be placed above bench level.

5.2 Resolution

Leupold and Stevens state that the resolution is 1 part in 10,000. The following quotation is from a letter received by the Water Research Laboratory from Leupold and Stevens through A. G. Baker and Associates Pty. Ltd. of Melbourne.

"The Memomark Encoder will encode 100 feet of water (range 00,00 to 99.99'), into 10,000 discrete readings. Hence, it has an inherent capability of resolving 1 part in 10,000. Given adequate torque to drive the encoder (float or otherwise), the accuracy will also be 1 part in 10,000 <u>over the range</u>. If it is desired to encode to less than 0.01', different gearing could be supplied, but the torque requirements would increase substantially. For all of our previous applications the 0.01' accuracy was all that was required.

Table 2: Piezometric Level Variations and Upstream Waves

Test No.	1 Wave Length, ft.	2 Amplitude (Max. -Min. W. L.) ft.	3 Mean W. L. ft.	4 Manifold W. L. ft.	5 Std. Crump Tapping	Percentage variation in upstream gauged head		Level of upstream bed P(ft.)
						$\frac{4-3}{3-.819}$pc.	$\frac{5-3}{3-.819}$pc.	
353	1.5	0.038	1.698	1.695	1.699	0.40	0.06	0.516
354	3.0	0.036	1.772	1.772	1.774	0.0	0.21	0.516
355	2.0	0.054	1.823	1.822	1.824	0.10	0.10	0.516
356	2.1	0.054	1.856	1.855	1.860	0.0	0.39	0.516
357	-	0.036	1.911	1.914	1.917	0.23	0.50	0.516
358	3.2	0.030	1.983	1.981	1.983	0.19	0.017	0.516
359	3.3	0.068	2.070	2.067	2.055	0.25	1.21	0.516
601	-	0.013	1.2116	1.205	1.208	1.57	0.81	0.15
606	4.0	0.096	1.674	1.666	1.654	0.90	2.30	0.15
607	4.2	0.097	1.778	1.773	1.752	0.52	2.71	0.15
608	4.5	0.108	1.880	1.883	1.880	0.31	0.03	0.15
609	4.8	0.129	1.963	1.962	1.953	0.06	0.85	0.15
610	-	0.031	1.286	1.283	1.289	0.47	0.60	0.12
612	3.6	0.077	1.426	1.420	1.429	0.93	0.55	0.06
613	4.0	0.095	1.555	1.560	1.570	0.68	2.04	0.06
614	4.5	0.128	1.617	1.611	1.628	0.75	1.38	0.10
615	-	0.0306	1.755	1.759	1.761	0.43	0.64	0.10
616	-	0.199	1.910	1.902	1.916	0.73	0.55	0.10
618	3.9	0.082	1.445	1.444	1.441	0.16	0.64	0.12
619	4.7	0.169	1.914	1.928	1.945	1.31	2.86	0.12
620	4.5	0.155	1.867	1.874	1.867	0.70	0.03	0.12
633	4.0	0.019	1.527	1.526	1.528	0.21	0.07	0.203
634	4.0	0.034	1.530	1.528	1.528	0.28	0.28	0.203
635	4.25	0.207	2.003	1.991	1.986	1.01	1.44	0.203
636	4.0	0.129	1.947	1.941	1.931	0.51	1.40	0.203
637	3.9	0.098	1.905	1.900	1.894	0.43	1.00	0.203
638	3.9	0.073	1.843	1.837	1.834	0.63	0.92	0.203
639	3.8	0.07	1.773	1.768	1.770	0.58	0.37	0.203
640	3.1	0.10	1.688	1.677	1.657	1.29	3.59	0.203
641	3.0	0.074	1.610	1.611	1.605	0.15	0.61	0.203
642	3.0	0.023	1.5713	1.568	1.571	0.44	0.04	0.203
643	2.6	0.074	1.531	1.531	1.529	0.03	0.31	0.203
644	2.0	0.060	1	1.460	1.462	0.85	0.54	0.203
645	2.0	0.019	1.401	1.397	1.400	0.64	0.12	0.203
Mean percentage variation in upstream gauged head, 34 tests						0.521	0.858	

In our discussion in the bulletin, resolution refers to the least significant change in variable that the Encoder can recognise, 0.01: 100.00'.

The Encoder will also encode 100.00 feet, accurate to ± 0.01 feet, given adequate input torque. Float size affects overall encoding accuracy, as does line shift error, submergence of counterweight, temperature effects on floatline etc.

Reducing the number offeet to be encoded does change the accuracy and resolution. The Memomark will encode to an accuracy of ± 0.01 feet only, so in effect the accuracy and resolution decrease if less than 100' are measured.

Given a 50' range to be encoded, stainless steel beaded cable as the floatline, and neglecting temperature effects, a 16" diameter float should give ± 0.01' accuracy of encoding with the Memomark .

Very truly yours,

Carl W. Petty
Sales Manager,
Stevens Hydrographic Products."

It would seem that with a standard sized drum pulley, a standard 16" float, and for the stated accuracy then an impulse will be transmitted with a change in water level of ± 0.01 feet.

A resolution of ± 0.01 feet is sufficient for all practical purposes. The output will consist of a histogram with 0.01 feet vertical increments. The telemetering of level increments rather than time increements is to be greatly preferred since only very few signals will be transmitted at times of low flow with consequent economy in the use and storage of recording tape.

5.3 Accuracy

The accuracy of record can be affected by

(i) temperature effects on the floatline
(ii) mechanical hysteresis,
(iii) lag due to fluid suction in the tapping pipe,
(iv) levelling inaccuracy,
(v) inaccuracy of construction.

5.31 Temperature Effects in Floatline

The floatline is a stainless steel cable with small beads accurately spaced which cog into small holes on the drum pulley. At low flows, the maximum diurnal temperature range in the shallow river is from $50^{\circ}F$ to $90^{\circ}F$, a maximum probable change of $40^{\circ}F$. Assuming a co-efficient of expansion of steel of 0.07×10^{6} per $^{\circ}F$, and an effective length of cable of 40 feet, the approximate maximum probable diurnal variation in tape length is 0.01'.

5.32 Mechanical Accuracy (Hysteresis)

The source of error included here is the maximum probable in-accuracy due to blacklash and friction in the drum, floatline and float mechanism.

The torque required to actuate the drum pulley is provided by a force on the float which will result in a hysteresis difference between rising and falling water levels. Leupold and Stevens assert that the accuracy will be 0.01 feet for a 100 ft. range of water levels with a 16 inch pulley. To improve the accuracy for a 40 ft. range of levels a larger pulley and larger float would be required as well as instrument modifications. These modifications are not recommended.

Maximum probable error due to mechanical accuracy is 0.01 feet.

5.33 Lag due to Fluid Friction in the Tapping Pipe

This friction will give a small but negligible hysteresis lag to a hydrograph with quickly changing river levels. However, this source of inaccuracy will not affect the low flow performance for which the structure is designed, and no additional allowance is therefore made.

5.34 Levelling Inaccuracy

When the instrument is first set up and later when periodical adjustments are made it will be necessary to check the weir crest level, the piezometric level, and adjust the level telemetered from the site to correspond. The maximum error from this source is 0.01 ft.

5.35 Accuracy of Construction

A high order of dimensional accuracy should be obtained during construction. By far the most necessary requirement is that the crest be a straight line from one side to the other. Figures 26 and 26a show details of the recommended precast crest blocks (Refs. 3.8 and 3.12).

6. Three Dimensional Tests

The three dimensional tests may be conveniently divided into two series. In the first, conducted early in the investigation, a number of possible structures were tested. These tests and the two dimensional tests described earlier in this report led to the choice of a sloping Crump weir. The second series of three dimensional tests were an overall evaluation of this structure.

6.1 First Series of Tests

The structures tested in this series were:-

(i) a Parshall flume with a sheet pile cutoff wall
(ii) a Parshall flume with sheet piling and a stone apron
(iii) a Crump weir with a concrete apron

6.11 Model Scales

The first and second flood plains (Figure 2) will be influenced by the structure. The third flood plain which is remote from the structure begins to submerge at 70,000 c.f.s. Above this discharge there will be no increase in tractive stress on the lower flood plain. At 70,000 c.f.s. any structure will be well drowned out and not influencing water levels. For these reasons 70,000 c.f.s. was chosen as the maximum discharge. This will occur in the Hunter River once in every 12 to 15 years.

To use the available water supply of 7 c.f.s. (model) a discharge scale of 1:10,000 was needed. This requirement was met by the following length scales:-

(i)	Horizontal scale	1:48
(ii)	Vertical scale	1:36

The distortion was a compromise between requirements of tractive stress in the model and requirements of modelling the steeply sloping movable banks and shoals.

In order to obtain correct three dimensional flow patterns in the vicinity of the structure, the weir was modelled in the direction of flow to an undistorted scale of 1:36. The horizontal scale across the river was maintained at 1:48 as for the structure.

(i) Horizontal scale across the river	1:48
(ii) Horizontal scale along the river	1:36
(iii) Vertical scale	1:36

This change of scale at the structure amounted to a change of length of only about 3 inches in the movable bed model which was small compared to the size of the model.

Figure 37 shows the area modelled.

6.12 Model Construction

Except in the river the model was constructed in 8:1 mortar to within one inch of the final surface. A 3:1 mortar topping was applied over the whole area except for the river channel and given a rough brush finish. An area was left boxed out to accommodate the structures to be tested.

6.13 Test Procedure

The following test procedure was adopted.

(i) The bed channel was moulded in sand and the model verified for a moving bed and fixed flood plains.

(ii) The model was inserted and tested with a mobile channel and fixed flood plains for low discharges.

(iii) The whole area was covered with approximately 1 inch of sand and the model verified for high discharges.

(iv) The most promising model from the previous tests, the Crump weir, was then tested for high discharges. Hydrographs were used which, it is believed, gave the worst case of scour.

6.14 Sand Used

Early tests were run using Georges River sand. With this sand the model was not sufficiently rough and additional roughness had to be obtained by the addition of blue metal. The substitution of Nepean River sand led to much better verification. The grading curve (Figure 28) approximates the Kramer III criterion except that it has a lower modulus of uniformity. Some segregation occurred in the model but probably not in excess of the segregation that occurred in the prototype. In contrast to the Georges River sand there was hardly any rippling.

6.15 Scour Test of Parshall Flume and Sheet Piling

The Parshall flume with sheet piling was tested in the low flow range. At this stage Georges River sand was being used. At a discharge of 6200 c.f.s. the maximum scour occurred. This is shown in Figure 38. The depth of scour agrees with results obtained in the two dimensional tests. The addition of coarse sand did not alter the scour pattern but increased the time to reach equilibrium. Scour also occurred at the upstream approach. Scour was controlled in the model by introduction of curved approach walls.

Sediment which accumulated at drowned flows quickly cleared when the modular range was reached.

6.16 Scour Test with Parshall Flume, Sheet Piling and Stone Apron

The Stone Apron was constructed as shown in Figure 25. The base consisted of Nepean River sand; the 6" dia. stone was modelled with 1/4 inch blue metal followed by a layer of 3/4 inch blue metal and topped off with hand selected and hand placed 1-inch blue metal to represent the 3 to 4 ft. diameter stones. The appropriate scale was considered to be 1/36. Steady state discharges were run for 353, 530, 1370, 3930, 6200, 10,000 and 20,000 cusecs, and in addition typical hydrographs were also run.

Very little scour occurred below the structure for steady state discharges. The worst scour consisted of a shallow depression downstream of the apron at 10,000 cusecs. At discharges of 20,000 cusecs and above the sand tended to accumulate over the apron. Maximum sand accumulation to a level of +219 occurred at 70,000 cusecs.

However, one of the hydrograph tests proved to be disastrous.

A hydrograph was run representing a discharge increasing fairly rapidly from 5,000 to 10,000 cusecs. This meant that the tailwater level was low, although no lower than could be experienced in the prototype. The stone apron rapidly disintegrated commencing midstream at the outer edge. As the stones became dislodged, they rolled away into the scour hole. As the apron width decreased, the rate of scour increased and spread sideways. As the discharge increased the scour was finally reduced, and no further disintegration occurred after 20,000 cusecs when the structure drowned out.

The results of this test indicated caution would be necessary in considering the use of sheet piling with stone apron.

6.17 Scour Tests with Crump Weir and Concrete Sloping Apron

6.171 Low Flow Tests

The Crump weir and concrete apron performed well in the low flow tests. The maximum scour occurred just before the structure drowned out at 10,000 c.f.s. Scour also occurred at the curved approach wing wall. This was beneficial in keeping the tapping point clear of sediment although as testing proceeded the coarser fractions in the sand filled the scour hole to a level of RL. +213.

Sediment lodged on the downstream glacis but only downstream of the hydraulic jump and was quickly cleared as the jump moved downstream.

6.172 High Flow Tests

At the beginning of these tests the right hand end of the structure was buried by sediment moulded to existing flood plain levels.

Hydrographs were run intermittently with lower flow steady state tests in between to reproduce conditions to be expected in the Hunter River. Pronounced changes began to occur in the shape of the channel and first flood plain both upstream and downstream of the weir. The right end gradually became exposed and the weir had the effect of spreading the flow uniformly across its length. This reduced the sediment transporting capacity both upstream and downstream. Shoals formed in these areas as shown in Figure 39. Eventually the end of the weir was exposed and outflanking commenced producing the scour shown in Figure 39. Extending the weir length from 200 ft. to 360 ft. did not remove this difficulty.

6.18 Conclusions

The Crump weir was satisfactory in all aspects except that of outflanking. This could be overcome by constructing a sloping structure across the river which caused the least disturbance to the horizontal distribution of discharge in the river.

6.2 Second Series of Tests

This was an exhaustive series of tests of a sloping Crump section line crest weir. Most of the testing was done with a structure 200 ft. wide with one final test on a structure 150 ft. wide. Figures 40 and 41 show the two structures.

The requirements of the structure were

(i) it must remain clear of sediment for all discharges up to 600 c. f. s.

(ii) it should be capable of clearing itself of sediment during a flood recession so that it is clear at 600 c. f. s. even at high tailwater conditions;

(iii) it should be proof against structural collapse due to downstream scour even at low tailwater conditions;

(iv) the tapping pipe should not block and the apron should remain clear of sediment;

(v) it should not create any tendency to outflanking.

6.21 Model Scales

For the 200 ft. wide structure the model scales were as in the previous tests namely for the river;

horizontal scale	1:48
vertical scale	1:36

and for the structure itself

horizontal scale across the river	1:48
horizontal scale along the river	1:36
vertical scale	1:36

Testing of the 150 ft. wide structure at high discharges was achieved by using an undistorted scale of 1:36. No distortion at higher discharges was required as there was general bed movement in the undistorted model.

6.22 Model Verification

Reasonable verification was obtained for a range of flows. The model roughness was slightly too low at the scales chosen resulting in water level errors of approximately 6 inches at the model extremities. The tail gate was adjusted in all cases to give the correct stage at the structure.

Wire mesh was placed against the downstream left bank of the model to simulate the dense bush at that location. It was also necessary to add blue metal to roughen the upstream end of the Hunter River model in order to obtain bed stability and flow patterns corresponding with the prototype through the range of discharges under investigation. It was found possible to run the model with ease up to some 60,000 cusecs. Above 70,000 cusecs the model required maintenance at the upstream end, and frequent remoulding. The model was almost completely inundated at these high discharges so that the length modelled was consequently too short in the direction of flow to be expected to correspond closely with the prototype. At these high discharges, the sediment movement was considered unreliable because the scour pattern did not correspond closely with the evidence of scour at these discharges in the prototype river generally; however, flow patterns near the structure were of interest.

6.23 Test Programme

Figure 42 shows the form of the model flood hydrograph on which the test programme was based. The hydrograph conforms in general shape to the floods of August 1952 and June 1956. Steady flow tests were run for each step of the histogram, the flow was instantly cut off after each step and bed levels were then taken in the dry. It was therefore possible to obtain contours of scour as they were during the flood as well as after the flood. The increments of discharge where possible corresponded to recent historical flood stages for which detailed information is available. The increments were 350, 3930, 10,000, 24,000 and 50,000 on the rising limb. 70,000 cusecs was modelled briefly at the peak, but was restricted in duration to a few minutes because of bed instability in the regions of the head box and the tailgate. The recession steps were 50,000, 24,000, 10,000, 3,930 and 350 cusecs.

Water levels were measured by point gauge. Bed levels were measured by a point gauge fitted with a small 3/4 inch square foot. Velocities were measured using a DSIR miniature flow meter fixed in front of the point gauge. The point gauge vernier was mounted onto a staff bridge as shown in Photograph 13.

6.24 Test Results at Low Flows

6.241 353 Cusecs

Figure 43 shows the scour pattern after a test representing 353 c.f.s. and 12 days duration on the prototype. No bed movement was detected upstream of the structure after the model had settled down. The tailwater level was lowered to the minimum probable level and the downstream scour pattern then developed as shown. No sediment was transported onto the upstream apron, but if sediment was placed on the apron it remained as placed.

6.242 3930 Cusecs

Figure 44 shows the scour pattern and velocities after a test representing 3930 c.f.s. and 5 days duration on the prototype. Slight sediment movement occurred upstream especially in mid stream, and the upstream bed level was lowered by as much as three feet close to the structure, to the equilibrium levels shown. The tailwater level was lowered to the minimum probable level and the downstream scour pattern then developed as shown in Figure 44. At 3,930 c.f.s. a small eddy occurred upstream of the wing wall structure at the left bank. The upstream apron remained clear of any sediment deposition. Photograph No. 15 shows that a very small deposition of sediment occurred on the upstream apron at the stagnation point located on the left third of the apron length and therefore well away from the piezometric tapping orifice in the side wall of the left bank structure. The prototype Hunter River in flood at 3,930 cusecs is shown from upstream in Photograph 3.

6.25 Rising Limb of Flood Hydrograph

6.251 10,000 Cusecs

Figure 45 shows the scour pattern after a test representing 10,000 c.f.s. and 24 hours duration at prototype scales. At 10,000 cusecs the structure is fully drowned out with the tailwater curve at 1967 levels. The structure is not drowned out with the tailwater lowered to the minimum probable level and the scour pattern then developed as shown in Figure 45. The velocities and a qualitative description of sediment movement across the river at 10,000 cusecs is shown in Figure 49.

6.252 24,000 Cusecs

Figure 46 shows the scour pattern after a test representing 24,000 cusecs and 7 hours duration at prototype scales. The structure was fully drowned out at both normal and minimum probable tailwater levels. Sediment collected on the structure except at the left end quarter of the crest. The velocities shown on Figure 48 are in agreement with velocities observed during the fresh of 12th January 1968.

6.253 50,000 Cusecs

Figure 47 shows the scour pattern after a test representing 50,000 cusecs and 6 hours duration at prototype scales. Very active general bed movement was principally beyond the right bank end of the structure and no bed sediment movement was detectable near the left bank structure. The tapping and the approach apron remained clear of sediment, so that the piezometric capabilities of the structure remained unimpaired even though the rest of the structure was substantially covered by sediment.

6.254 70,000 Cusecs

At 70,000 cusecs the zone of very active general movement had passed beyond the left end of the gauging structure. The intensity of bed shear over the structure itself was apparently reduced as compared with tests at lower discharges. The right bank structure remained unaffected by bed sediment. The same portion of structure remained exposed and unburied as for 50,000 cusecs shown in Figure 47. The bed and surface velocities did not exceed 8 feet per second above the exposed structure.

6.26 Recession Limb of Flood Hydrograph

6.261 50,000 Cusecs

It is not feasible to isolate any differences between the rising limb and the recession limb for a discharge of 50,000 cusecs because the model required some remoulding after the peak discharge of 70,000 cusecs. Figure 47 therefore refers to both conditions, within the limits of accuracy possible on this model of the Hunter River. The test in recession represented a further 6 hours at prototype scale. No bed changes were observed near the exposed portion of the structure. Bed and surface velocities did not exceed 8 feet per second near the left bank structure. A small quantity of sediment settled or collected at the stagnation point between the upstream apron and the 1:2 upstream crest slope.

6.262 24,000 Cusecs

Figure 48 shows the scour and velocity pattern after a test representing 24,000 cusecs and 12 hours duration at prototype scales. General movement of sediment occurred midway along the line crest, where the bed velocity was at a maximum value of 8.56 feet per second. A shoal immediately downstream of the structure had accumulated and become armoured on the surface with larger sized sediment particles. This shoal would not move readily and resulted in higher downstream bed levels than in Figure 46. Similar shoal armouring has been observed in the prototype river. The tailwater was lowered to minimum probable levels but the downstream bed levels remained higher than in the rising limb of the hydrograph, (see Photograph 19).

6.263 10,000 Cusecs

Figure 49 shows the scour and velocity pattern after a test representing 10,000 cusecs and 2 weeks at prototype scales. General movement of finer particles occurs over the left half of the crest. The line crest gradually became more exposed as the downstream shoal was scoured away. The left bank downstream low flow channel did not establish itself as markedly as in proving tests without the weir structure in place, and consequently minimum tailwater levels immediately adjacent to the structure remained high. Downstream bed levels were lowered some two feet but the structure remained substantially buried. Adjustment of the tailgate and regulation of the tailwater level showed that scour downstream of the structure is very sensitive to tailwater level and also very sensitive to weir crest level.

6.27 Scour Envelope

Figure 50 shows the envelope of maximum scour for a 200 ft.

structure. The upstream scour pattern shows that a deep upstream toe is not warranted from considerations of scour and should be omitted unless required to reduce uplift. The downstream shoal remained stable throughout the test programme, and the persistent downstream shoal inhibited end scour at the right bank.

6.28 Discussion

The tests at a distorted scale of the 200 ft. sloping line crest structure show that the proposed structure is stable in the river . There is, however, a tendency for the structure to remain buried especially at the right bank. The end 30 feet of structure once buried remained buried, so that during subsequent low flows the precise length of weir which is likely to remain exposed is speculative. The results, therefore, indicate that the 200 ft. structure is adequate regarding stability against bypassing but longer than necessary.

6.29 Tests on 150 ft. Structure

6.291 Test Program

The river was moulded to a natural scale of 1:36 and it was found that the model roughness was slightly too low and as before the water levels were maintained correct at the structure. From the tests on the 200 ft. structure Figures 46 and 47 suggested that very active sediment movement would occur near the right bank end of a 150 ft. strucure for flows of 30,000 to 40,000 c.f.s.

The lowest probable tailwater rating curve was adopted and a below average sand feed was applied.

6.292 Test Results

Figure 51 shows the scour pattern after a test representing 37,400 cusecs and 18 hours duration at prototype scales. This discharge was found to be most destructive to the structure. Higher discharges produced less scour to the downstream shoal and consequently less scour occurred to the right of the structure. Lower discharges produced insufficient general sediment movement to the right of the structure. Photograph 20 also shows the resulting scour pattern.

A scour channel developed with an average bed level of RL. 216. After 18 hours the channel extended upstream and to the right of the structure but the downstream shoal was still effective in the prevention of undermining of the structure. The estimated recurrence interval for a flow of 37,400 c.f.s. is 3 to 4 years. The flood of 12th/13th January 1968 sustained its peak discharge of 24,000 c.f.s. for some 12 hours. However, greater floods can be expected to have steeper peaks so that a steady discharge of 30,000 to 40,000 c.f.s. for 18 hours is probably a rare event.

7. Conclusions and Recommendations

Both the Parshall flume and the Crump weir are adequate for gauging flows of up to 600 c.f.s. However, the Parshall flume is prone to problems of scour at higher discharges. This is shown in both the two dimensional and three dimensional tests. The sloping Crump weir by matching the flow distribution of the natural channel can tolerate high discharges and high sediment loads without scour or undue accretion.

It is recommended that a 165 ft. sloping crest Crump weir be constructed and that measurement of head be made in a float well by a Leupold Stevens memomark recorder which is suitable for recording or telemetry.

Figures 52 to 54 incl. show the recommended weir and Figure 55 is a theoretical rating curve.

References

1. Sediment Transport and River Morphology

1.1 Hans Albert Einstein "The Bed Load Function for Sediment Transportation in Open Channel Flows" Tech. Bulletin No. 1026, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C. September, 1950.

1.2 Bureau of Reclamation "Step Method for Computing Total Sediment Load by the Modified Einstein Procedure", U.S. Dept. of the Interior, July, 1955.

1.3 Geological Survey Water - Supply "Simplified Methods for Computing Total Sediment Discharge with the Modified Einstein Procedure" Paper 1593, U.S. Department of the Interior, 1961.

1.4 Kenneth C. Wilson "Bed-load Transport at High Shear Stress," Proc. A.S.C.E. Vol. 92 No. HY6 Nov. 1966.

1.5 Chien Ning "The Recent Status of Research on Sediment Transport" Trans. A.S.C.E. Vol. 121, 1956.

1.6 Geological Survey Water-Supply "Roughness Characteristics of Natural Channels" Paper 1848, U.S. Department of the Interior, 1967.

1.7 Blench, T. (1957)"Regime Behaviour of Canals and Rivers," Butterworths, London.

1.8 Einstein, H.A. and Chien, N. (1953) "Transport of sediment mixtures with large ranges of grain sizes Uni. Calif. Inst. Eng. Res., M.R.D. Sediment Series No. 2.

1.9 Lane, E.W. and Borland, W.M. (1953) "River-bed scour during floods" Proc. Amer. Soc. Civil Engrs.

1.10 Kuiper, E., "Water Resources Development" pp. 190-2, Butterworths, London, 1965.

1.11 Leopold, L.B., "Fluvial Processes in Geomorphology" W.H. Freeman and Co., San Francisco, 1964.

1.12 Henderson, F.M., "Open Channel Flow" Macmillan, London, 1966.

1.13 Hattersley, R. T., Munro, C. H. and Bonham, A. J., "Bed Sediment Load in Streams Tributary to Lake Burley Griffin" W. R. L. Report No. 96.

2. River Moving Bed Models

2.1 Einstein, H. A. and Chien N. (1956) "Similarity of distorted river models with movable beds" Trans. Amer. Soc. Civ. Engrs. Vol. 121, pp. 440-62.

2.2 Allen, J. "Scale Models in Hydraulic Engineering," 1947. London, Longmans Green.

2.3 Hattersley, R. T., "An appraisal of sands available in Sydney for use in hydraulic models" Uni. N. S. W., W. R. L. Report No. 50, Nov. 1961.

2.4 Yalin, M. S., "Use of models in the design of river works" River Engineering and Water Conservation Works, R. B. Thorn, Butterworths, London, 1966.

3. Gauging Structures

3.1 Elevatorski, E. A. "Hydraulic energy dissipators", 1959, London; McGraw-Hill.

3.2 Ackers, P. and Harrison, A. J. M. "Critical depth flumes for flow measurement in open channels" Hydraulics Res. Pap. No. 5, 1963, London, H. M. S. O.

3.3 British Standard 3680: Part 4 "Weirs and Flumes" B. S. I.

3.4 Hydraulics Research, 1962, London; H. M. S. O.

3.5 Parshall, R. L. "Parshall Flumes of large size" Bulletin 426-A Agricultural Expt. Stn., Colorado State Uni. Fort Collins.

3.6 Crump, E. S. "A new method of gauging stream flow with little afflux by means of a submerged weir of triangular profile". Proc. Instn. of Civil Engineers, Lond. March 1952 and discussion Nov. 1952.

3.7 Burgess, J. S. and White, W. R. "The triangular profile (Crump) weir: two dimensional study of discharge characteristics. Hydraulics Research Station, Wallingford, Report Int. 52, May 1966.

3.8 "Crump Weir Design" Water Resources Branch, Hydrometry Divn., Reading, U.K. 1966.

3.9 Pearson, G. "The Planning and Hydraulic Design of River Flow Gauging Stations" River Engineering and Water Conservation Works, Thorn, R.B., Butterworths, London, 1965.

3.10 Oates, F.L. "Measurement of River Flows" Assoc. of River Authorities Year Book, 1965.

3.11 Johnson, P., "Discussion on Problems of Measurement, Accuracy of Flow Measurement" Symposium on River Management, University of Newcastle upon Tyne, 1966.

3.12 "The Accuracy of Gauging Structures" Proc. I.C.E., July 1966.

3.13 Telemetry Symposium, J.I.W.E. Vol. 22, No. 3, May 1968.

3.14 Emmett, W.W. and James, R. Wallace "Errors in Piezometric Measurement" Jnl. A.S.C.E. Vol. 90 HY6, Nov. 1964.

3.15 White, W.R., "The Flat-Vee Weir" Hydraulics Research Station, Wallingford Report Int. 56 Dec. 1966.

4. <u>Miscellaneous</u>

4.1 E.C. research note No. 50.

4.2 Schlichting, H. "Boundary layer theory" McGraw-Hill, New York, 1960.

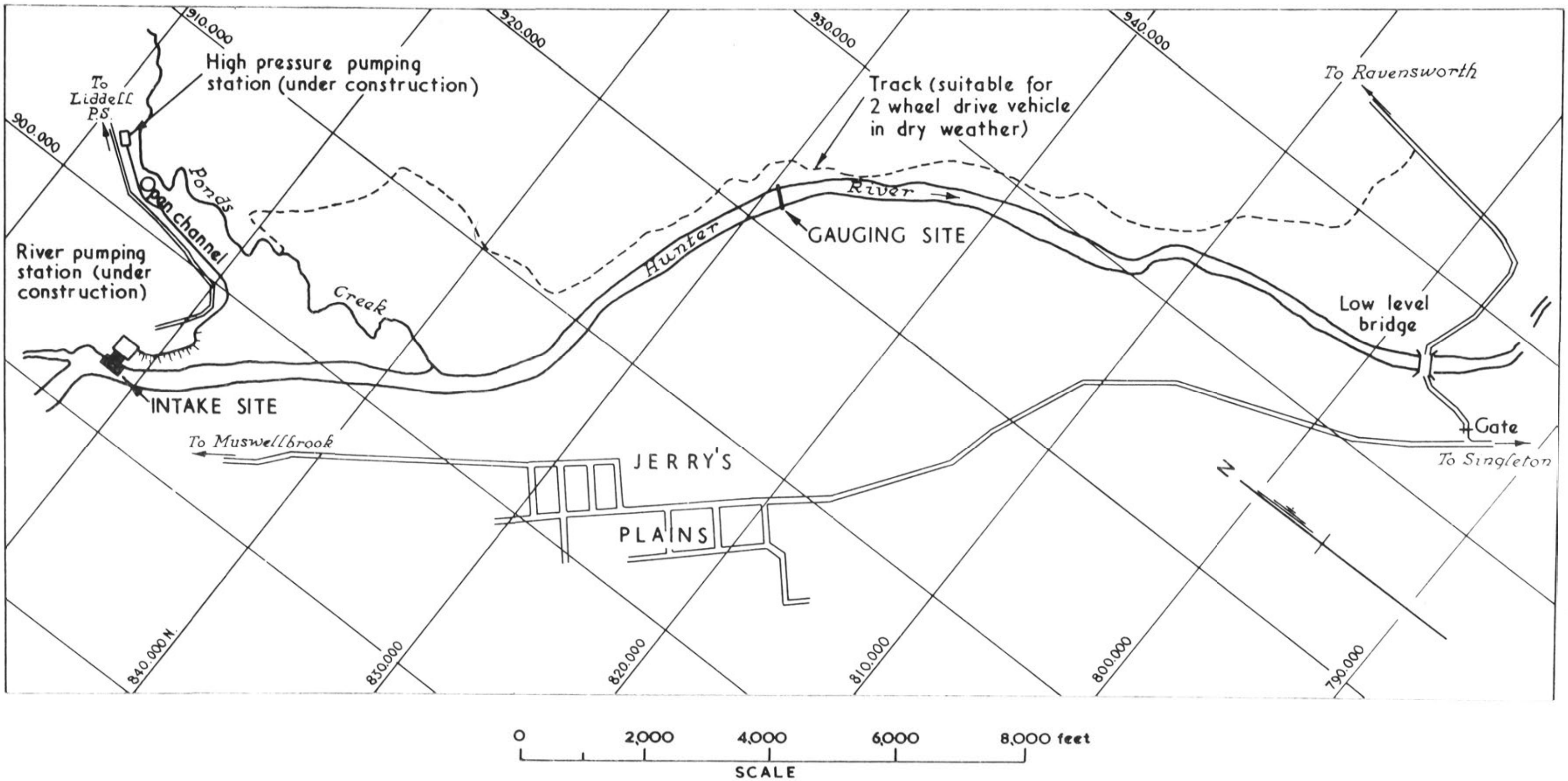


Fig. 1: Locality Plan.

CE-E-7538

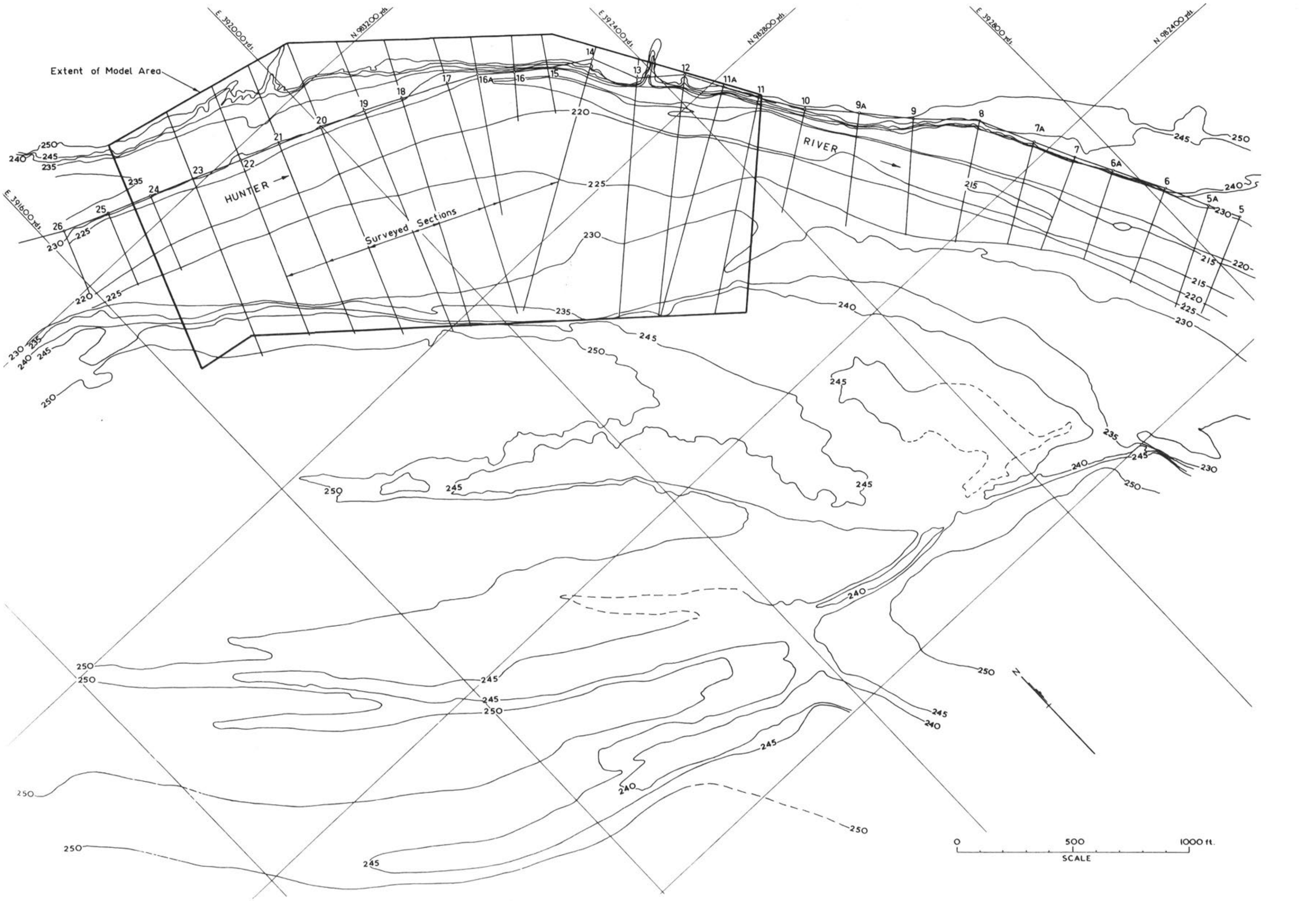


Fig. 2: Flood Plain Contours from Aerial Photos.

CE-B-7251

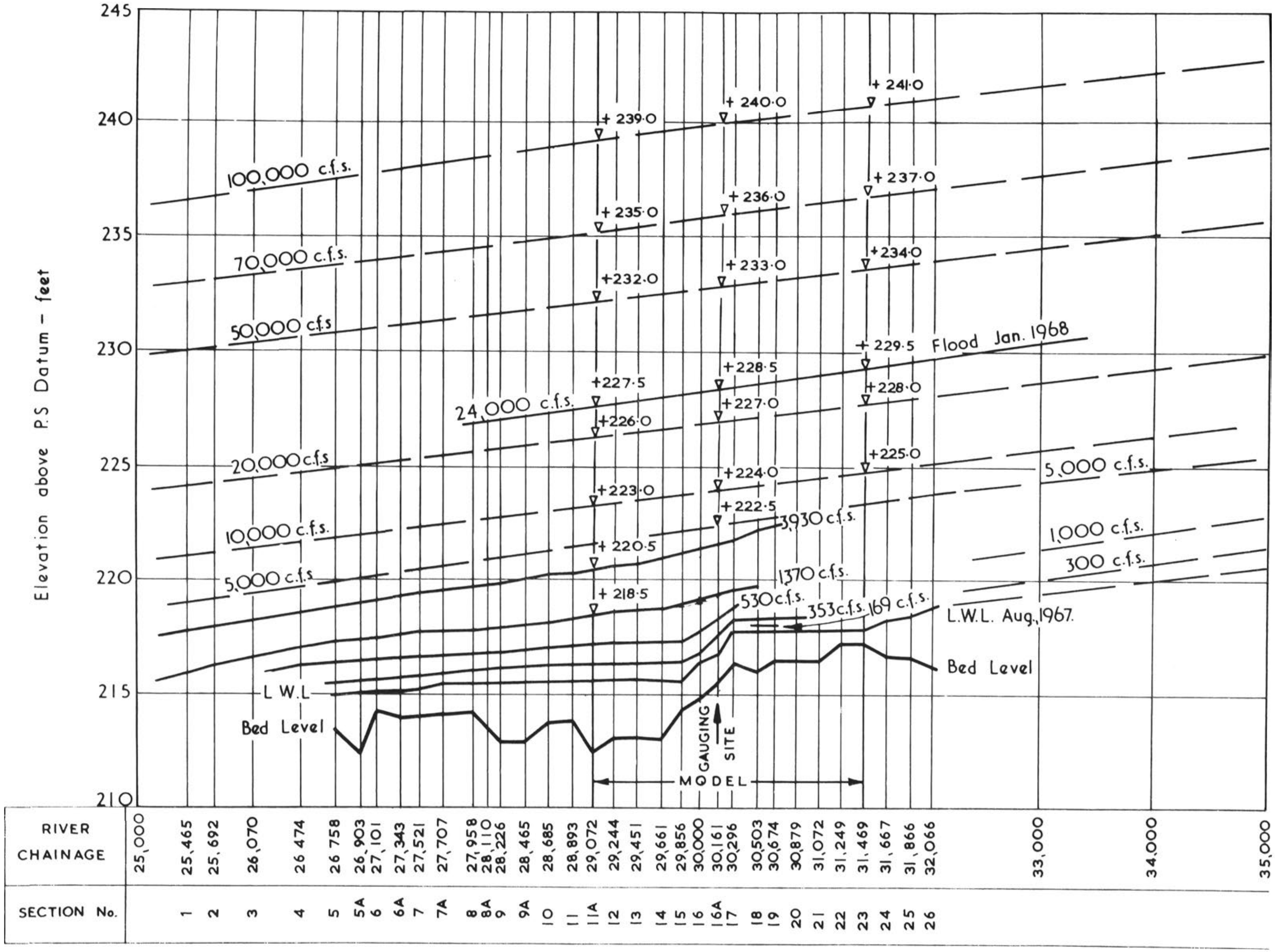


Fig. 3: Backwater Curves at Gauging Site.

CE-D-7390

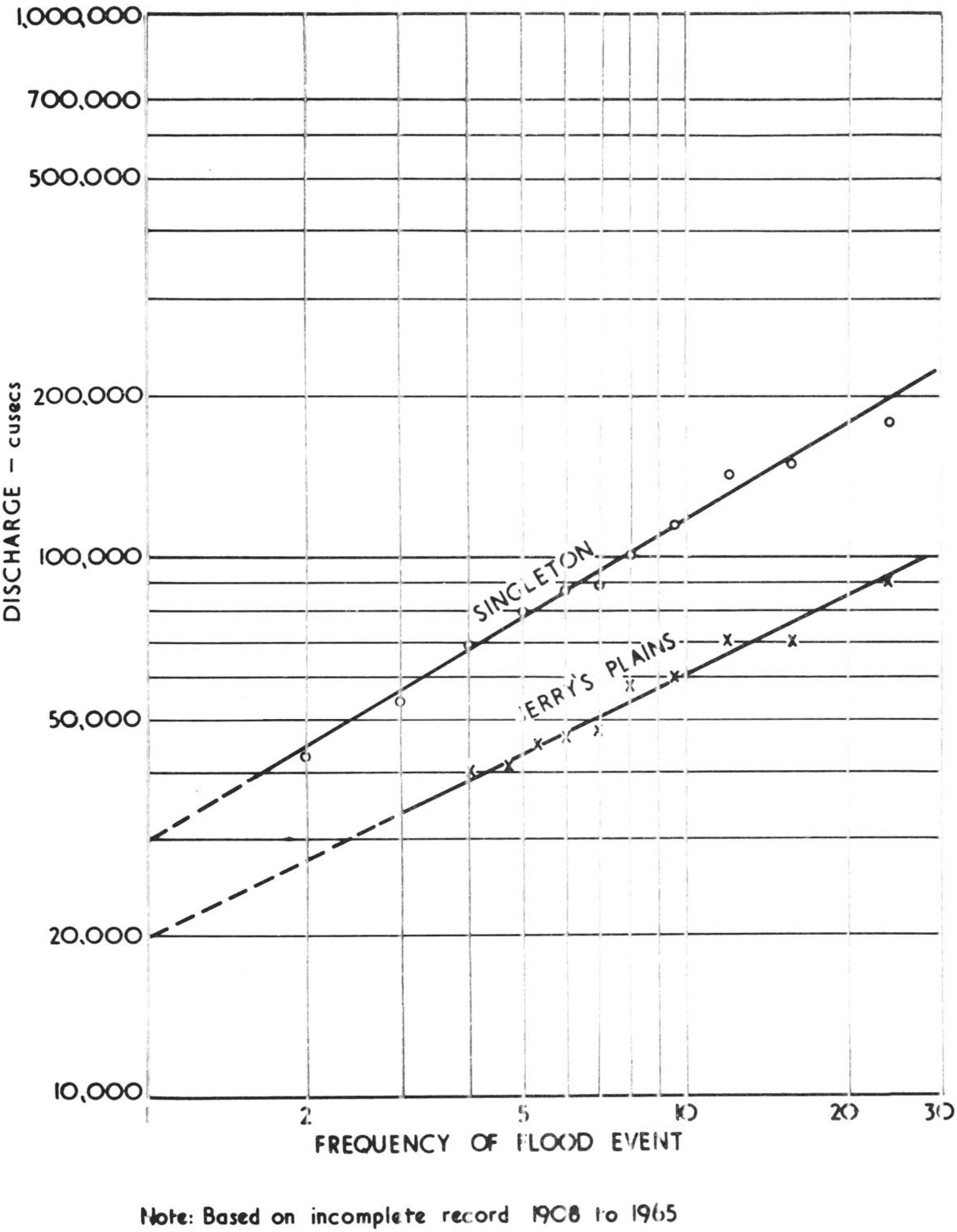


Fig. 4: Flood Frequency in Hunter River.

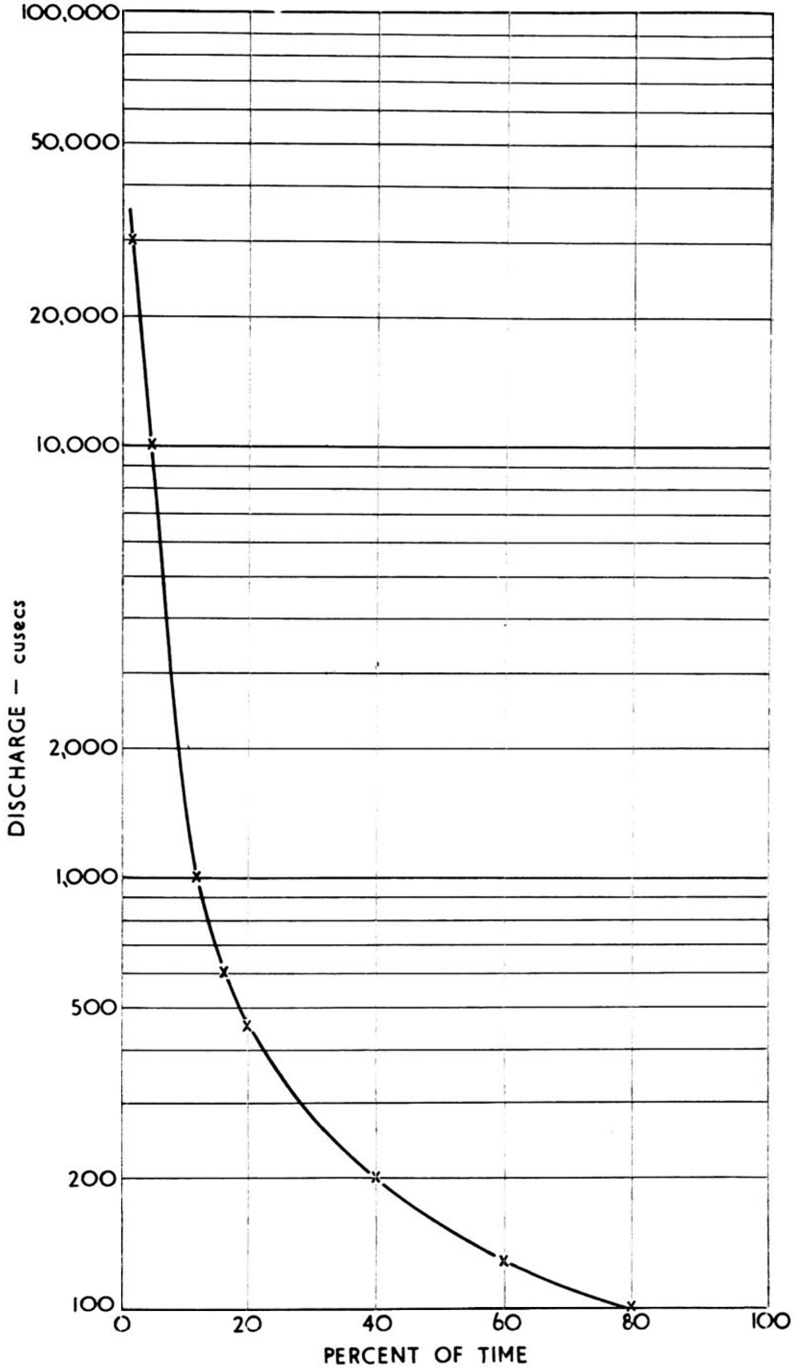


Fig. 5: Flow Duration Curve for Hunter River.

CE-E-7541

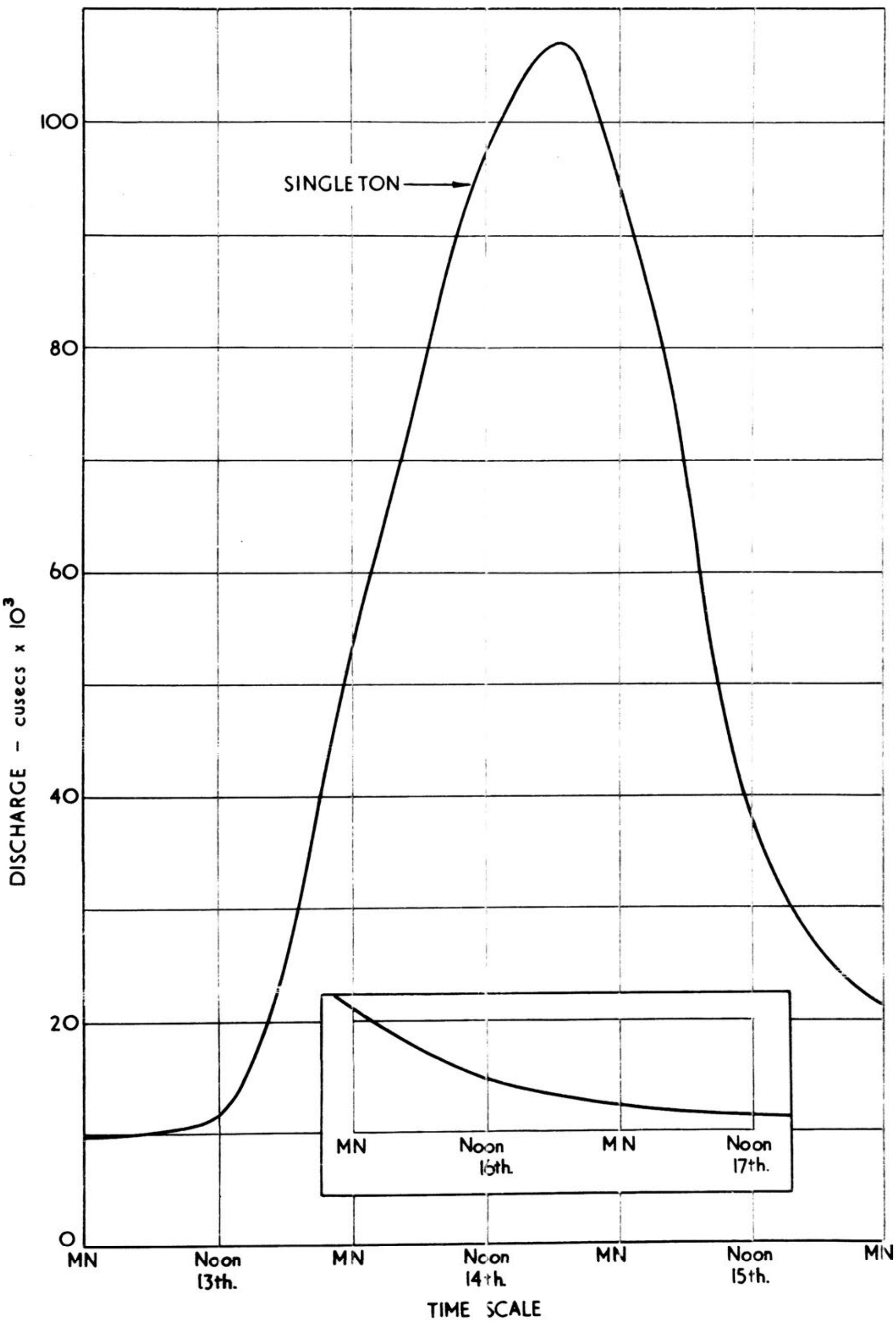


Fig. 6: Flood Hydrograph, August 1952.

CE-E-7542

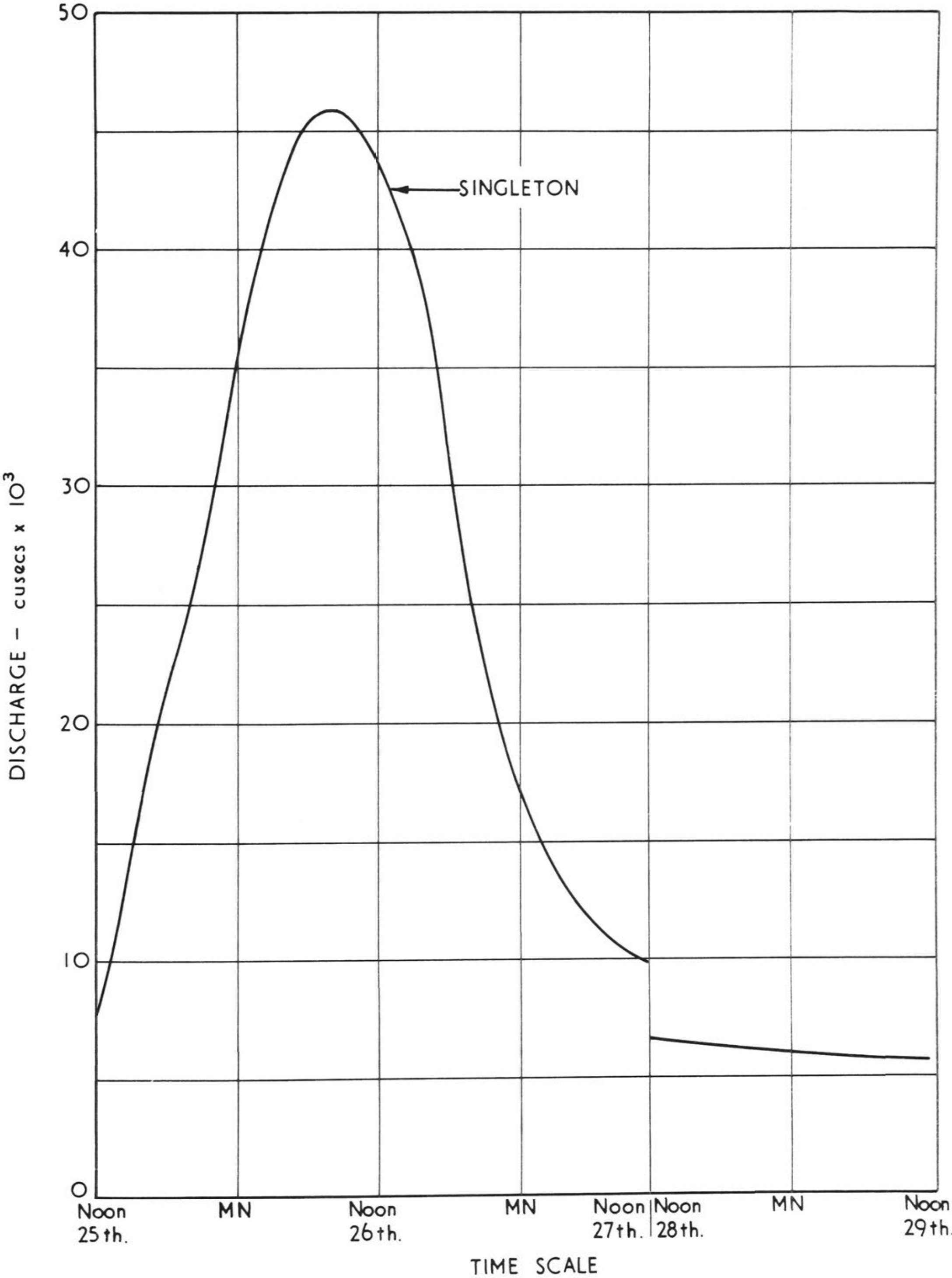


Fig. 7: Flood Hydrograph June 1956.

CE-E-7543

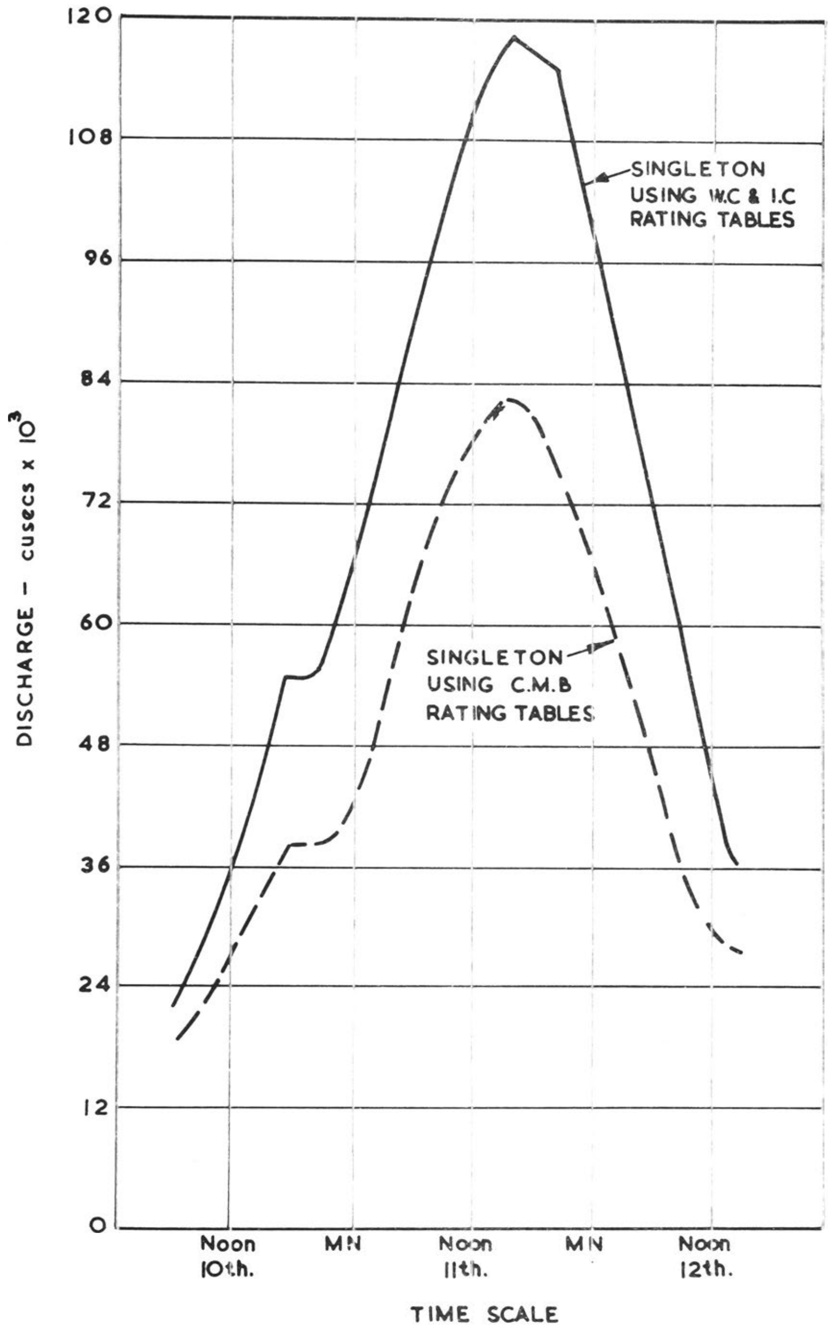


Fig. 8: Flood Hydrograph June 1964.

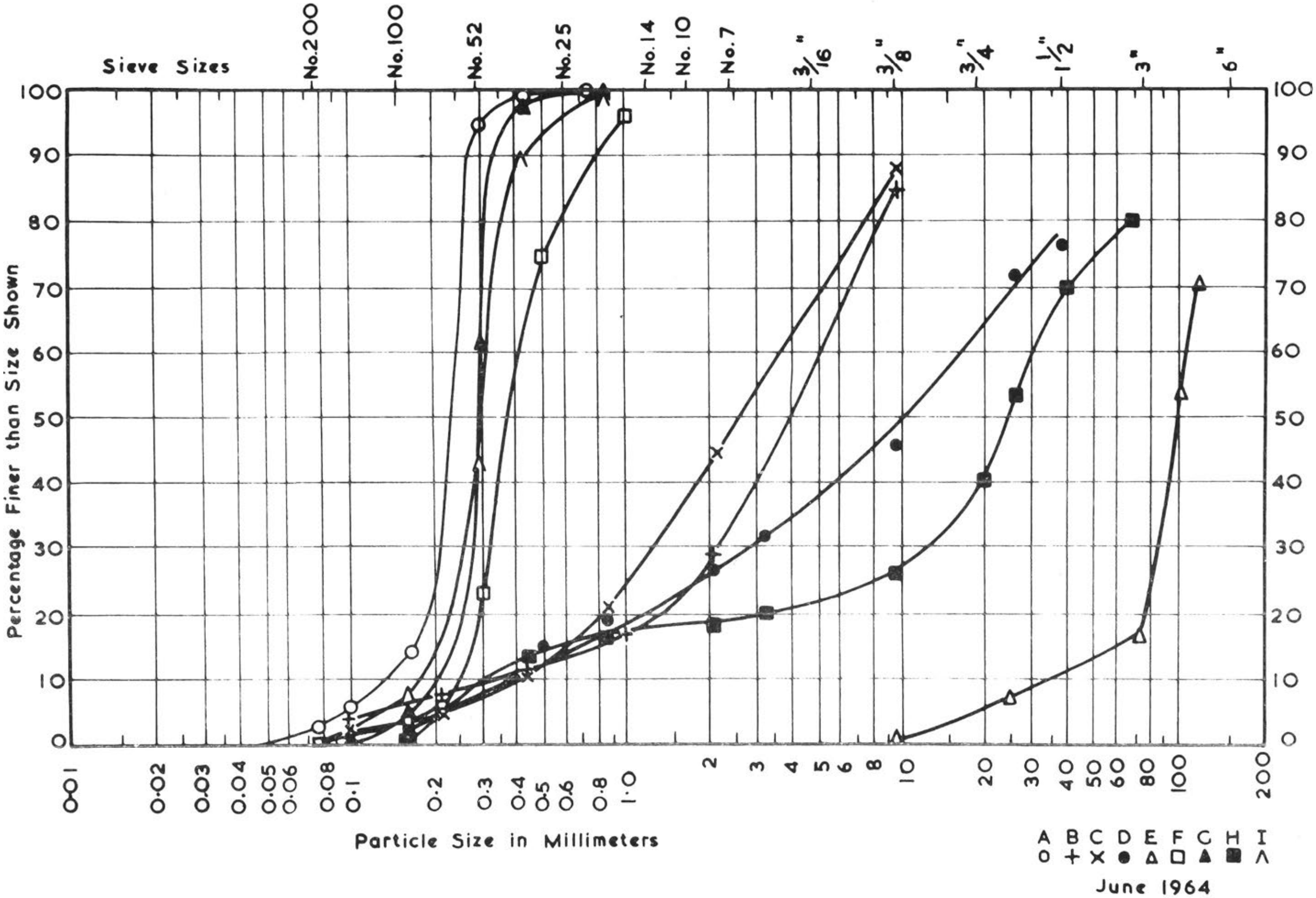
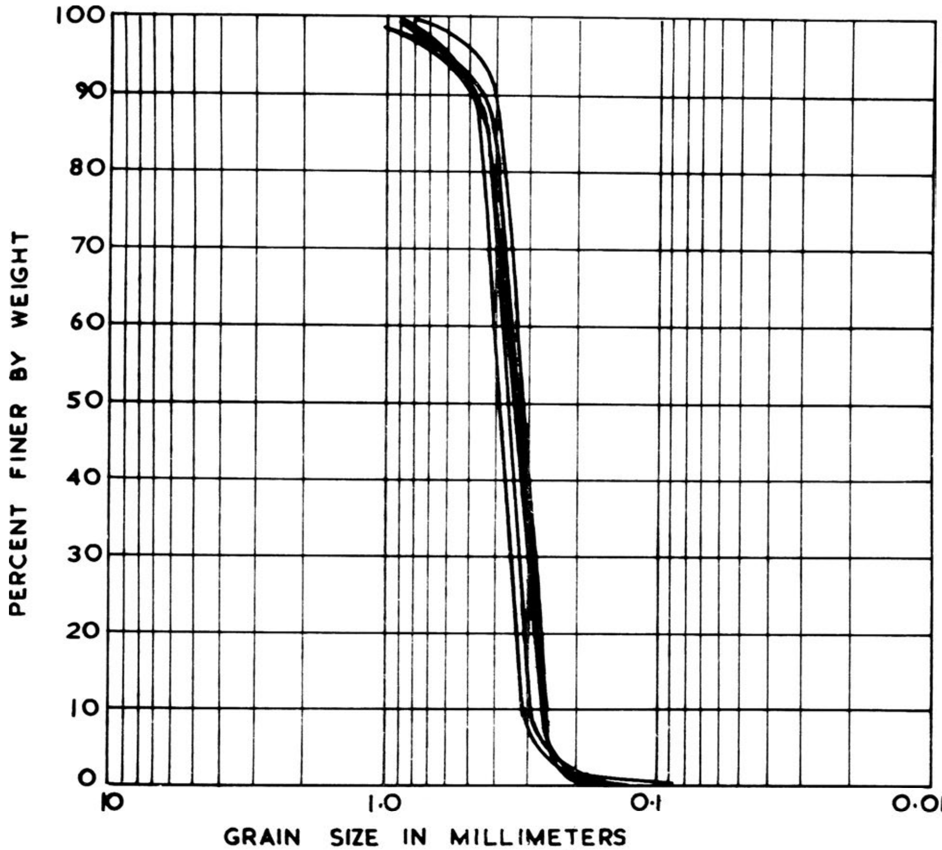


Fig. 9: Grading Curve, Bed Sediment at Intake Site.

CE-E-6290



D_{65} = 0.38 mm = 0.0013 ft. D_{35} = 0.31 mm = 0.0011 ft.

	SAND			
	Coarse	Medium	Fine	

Fig. 10: Grading Curves Bed Sediment at Gauging Site.

CE-E-7181

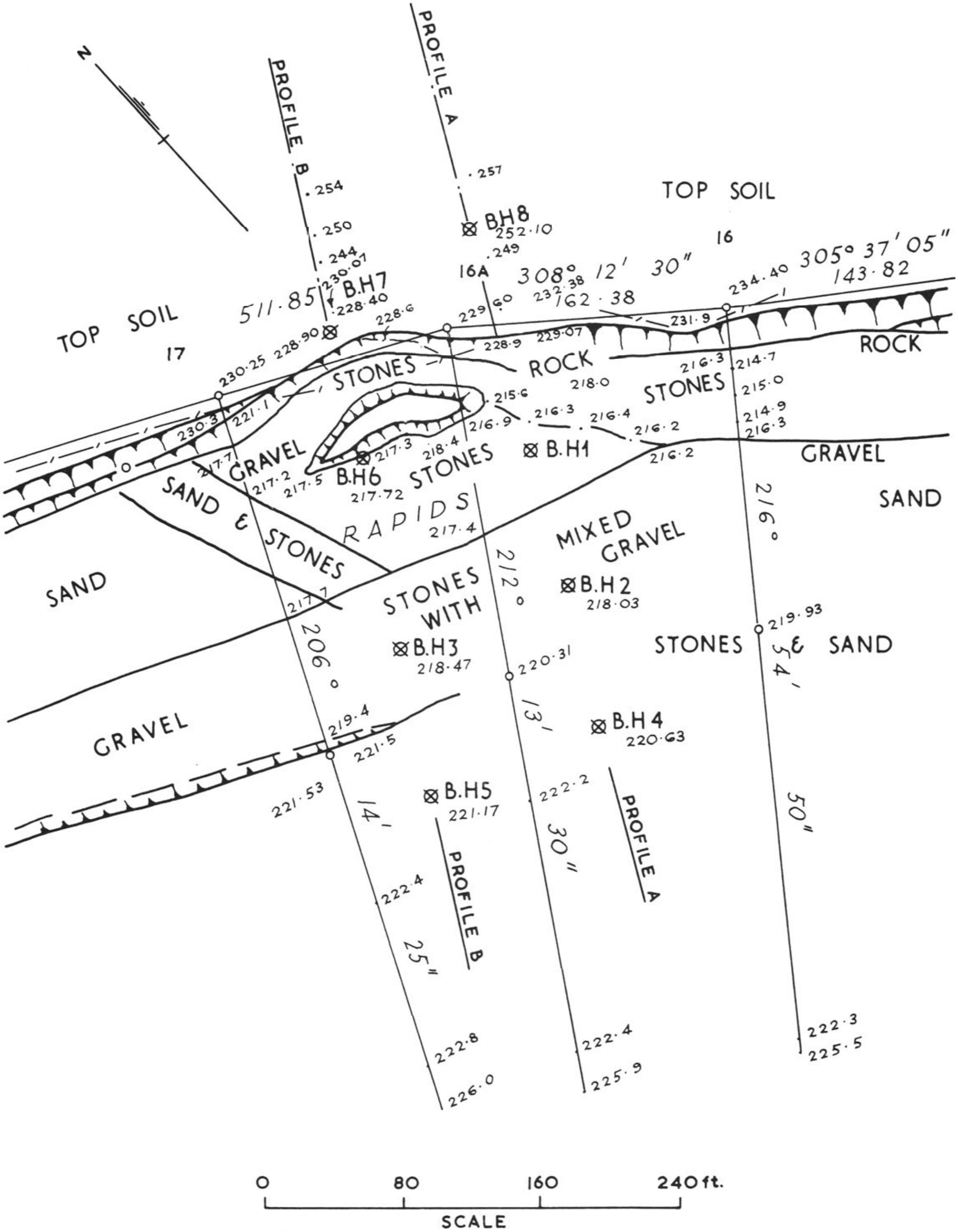


Fig. 11: Borehole Location Plan Surface Deposits.

CE-E-7555

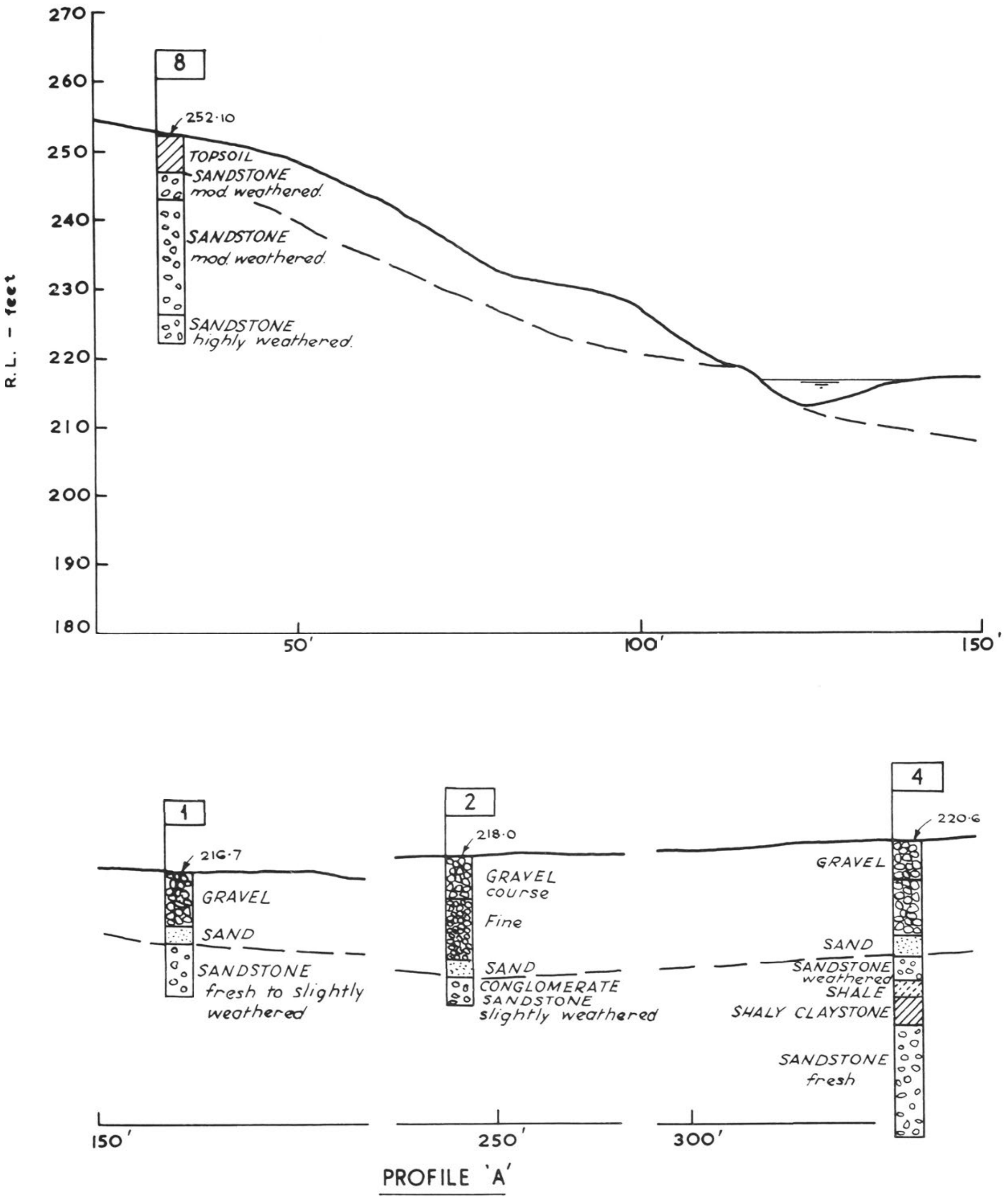


Fig. 12: Borehole Log Profile 'A'

CE-E-7556

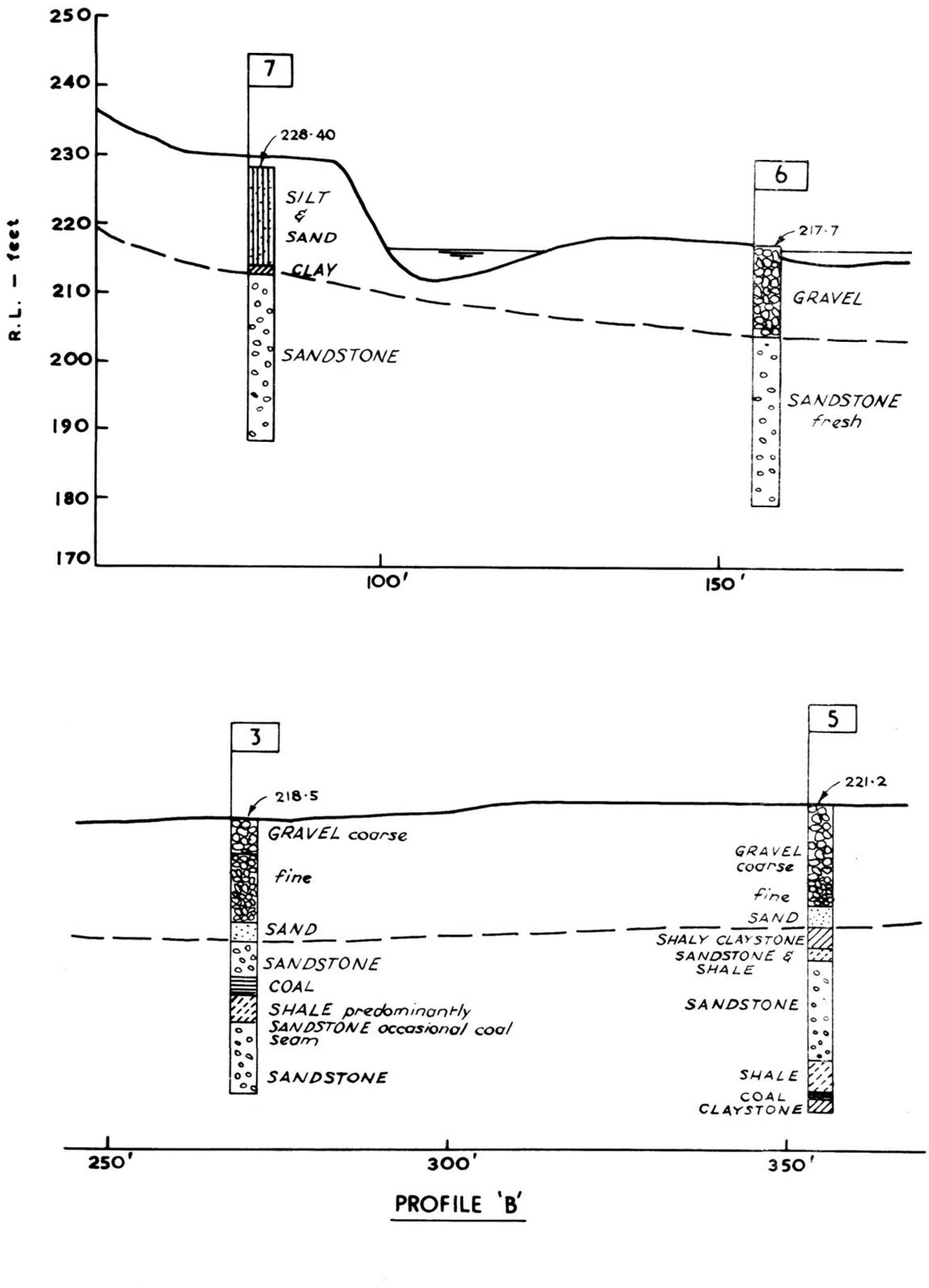


Fig. 13: Borehole Log Profile 'B'.

CE-E-7557

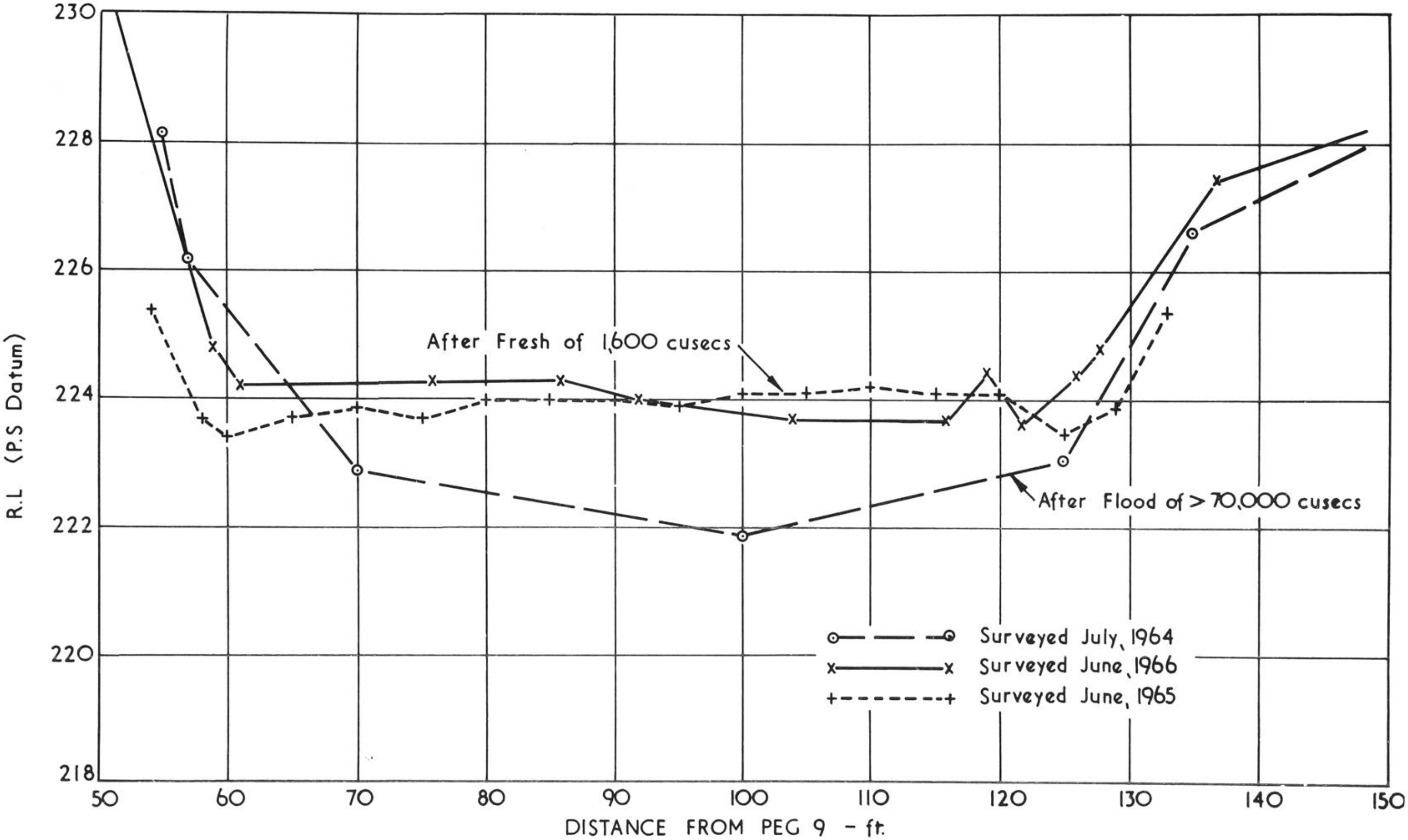
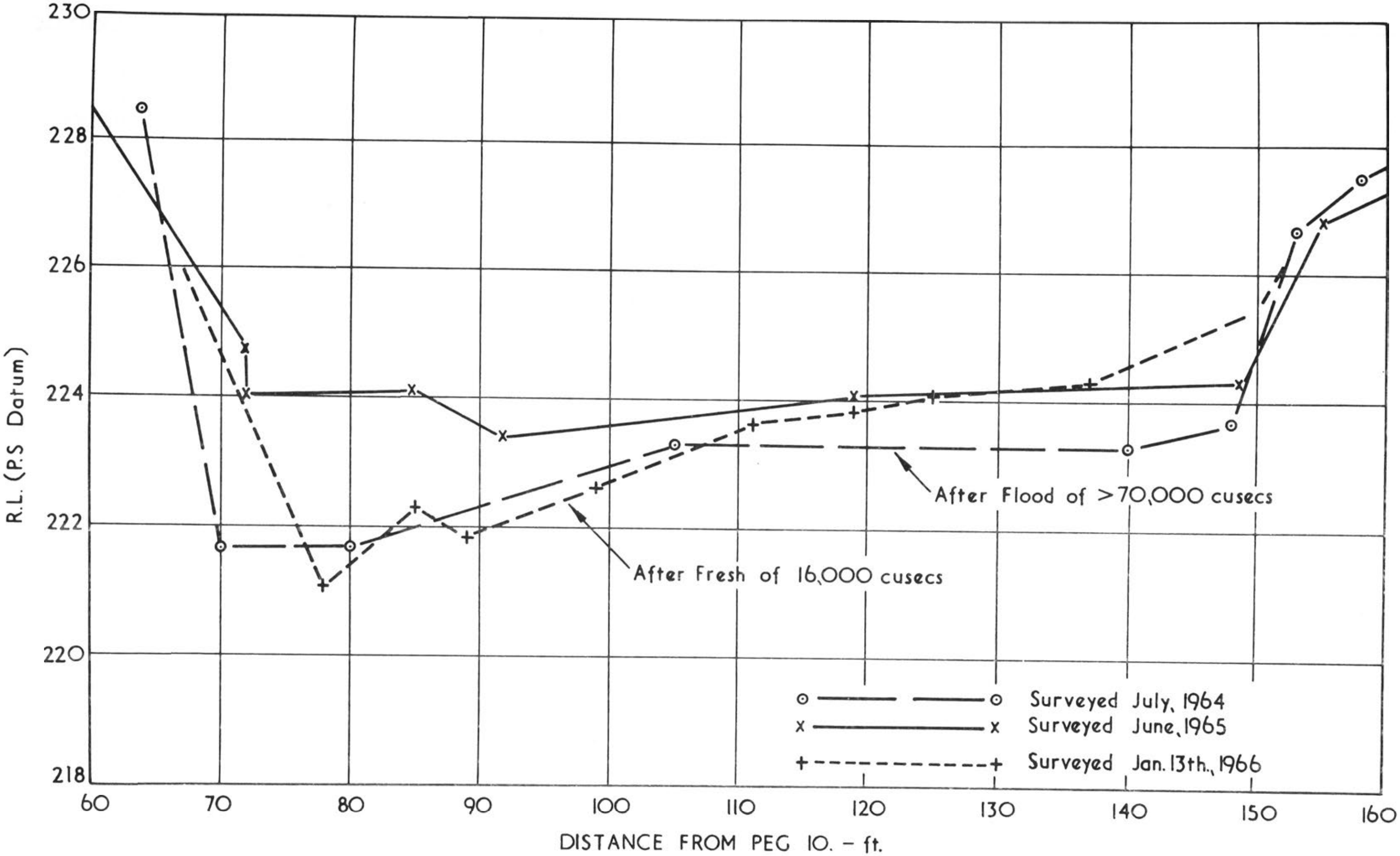


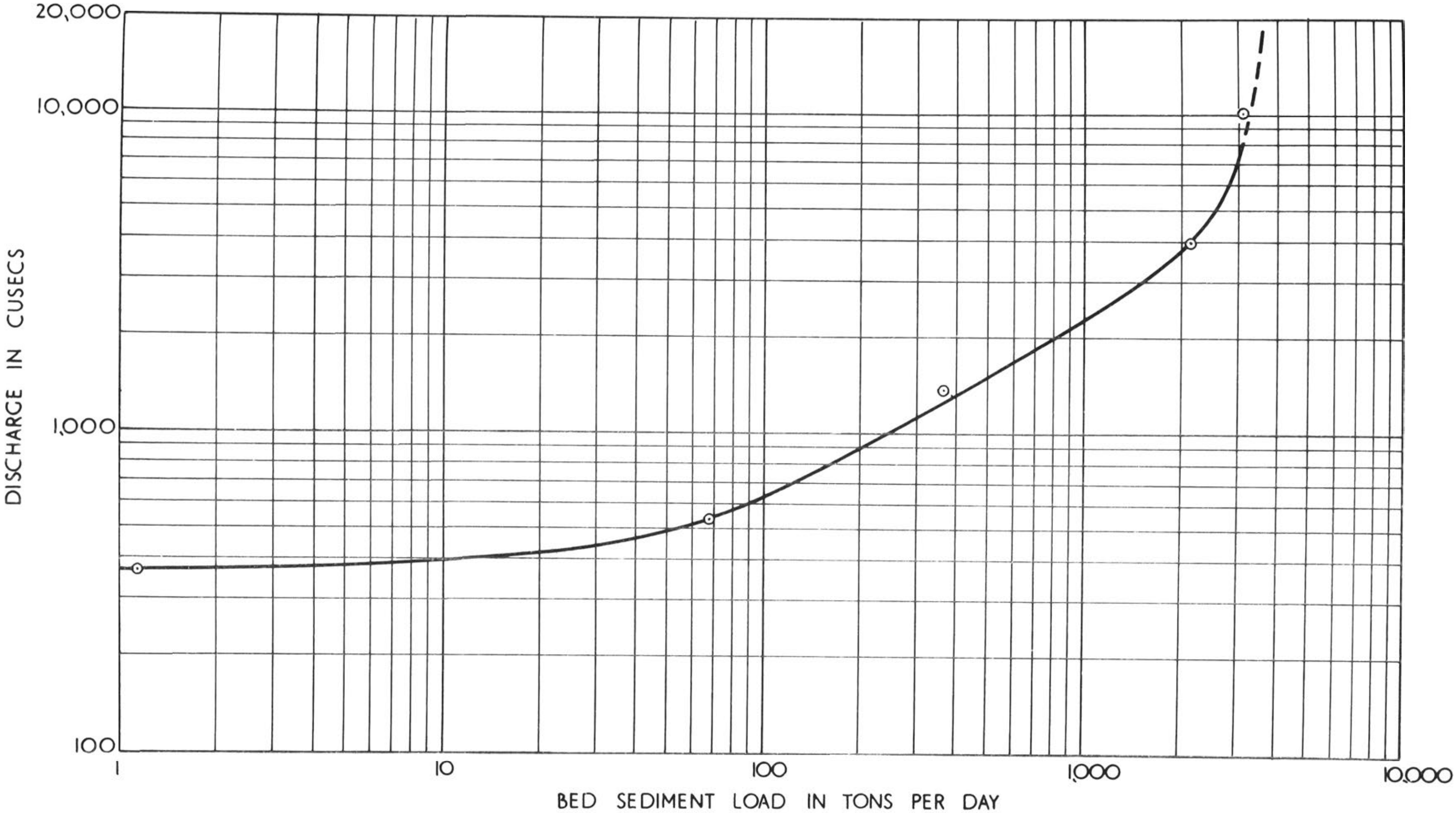
Fig. 14: Bed Changes at Intake Site - Section 9.

CE-E-7547



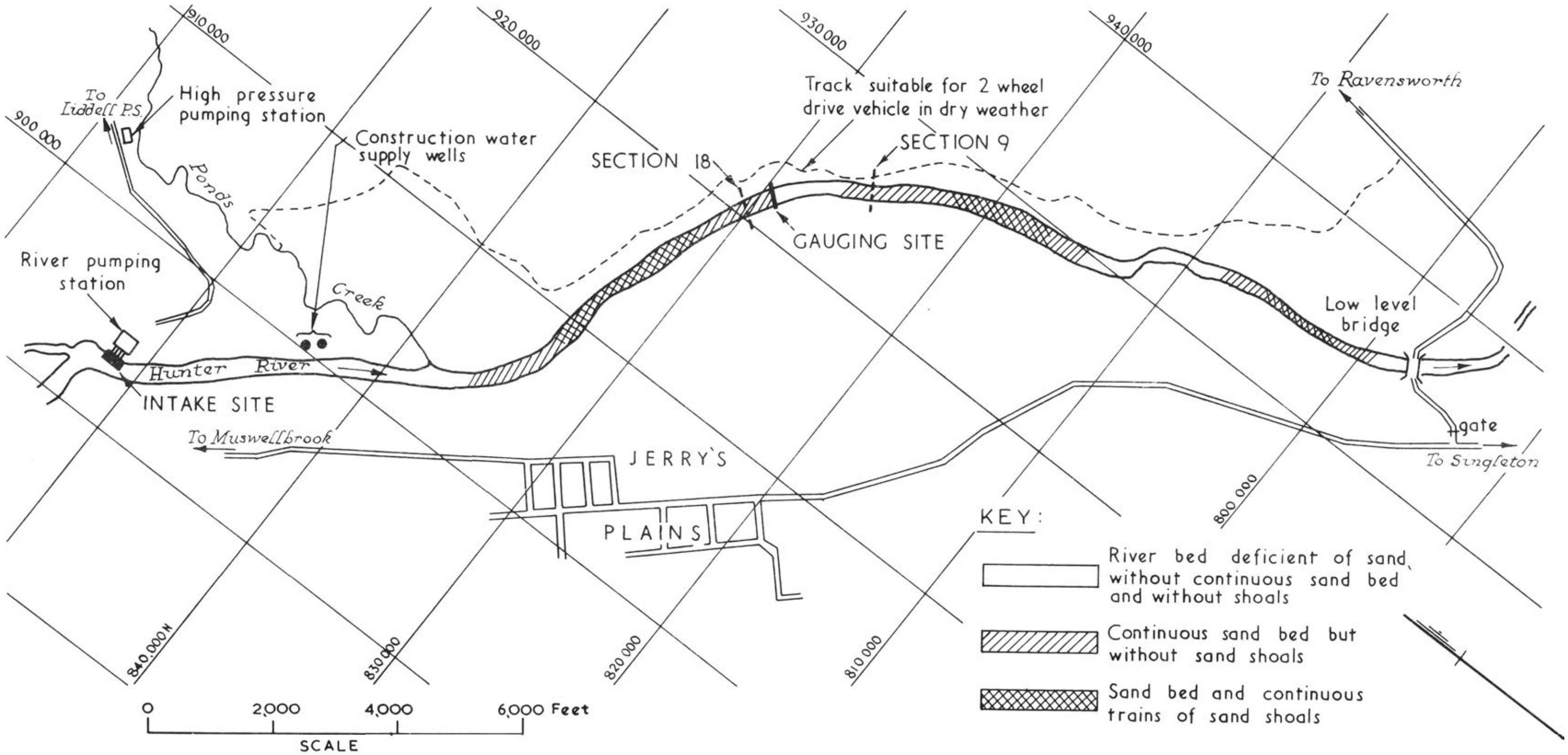
CE-E-7548

Fig. 15: Bed Changes at Intake Site - Section 10.



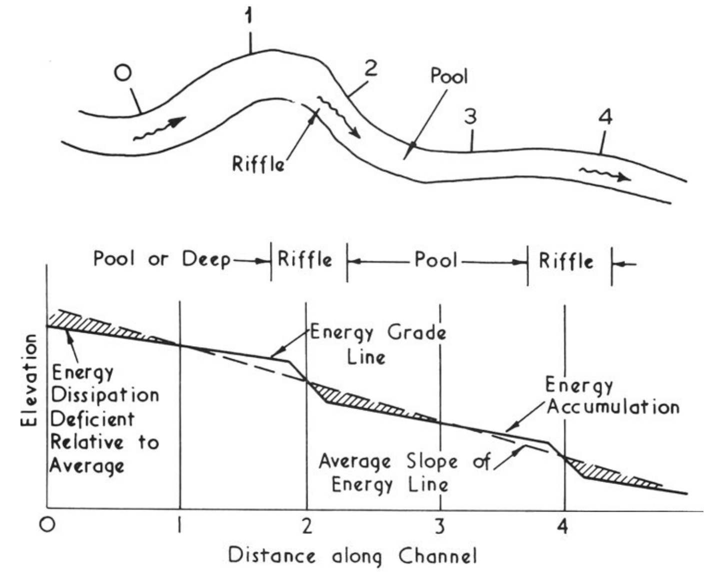
CE-E-7182

Fig. 16: Bed Sediment Rating Curve for Sand.

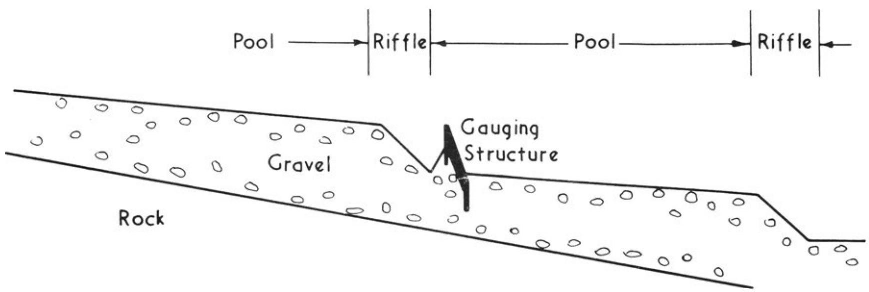


CE-E-7351

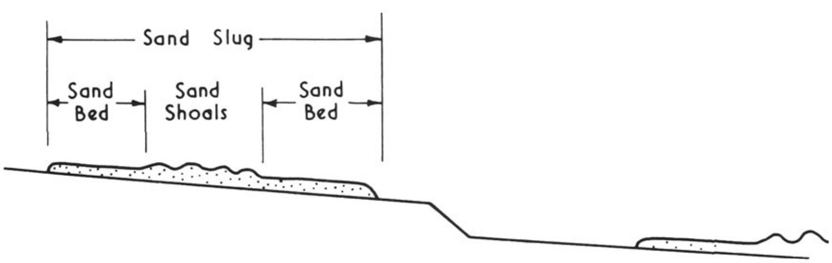
Fig. 17: Plan showing areas of Bed Sand/ Bed Shoals
10th April 1968.



PLAN AND PROFILE OF HYPOTHETICAL CHANNEL SHOWING LOCATIONS OF POOLS AND RIFFLES. (After Leopold)

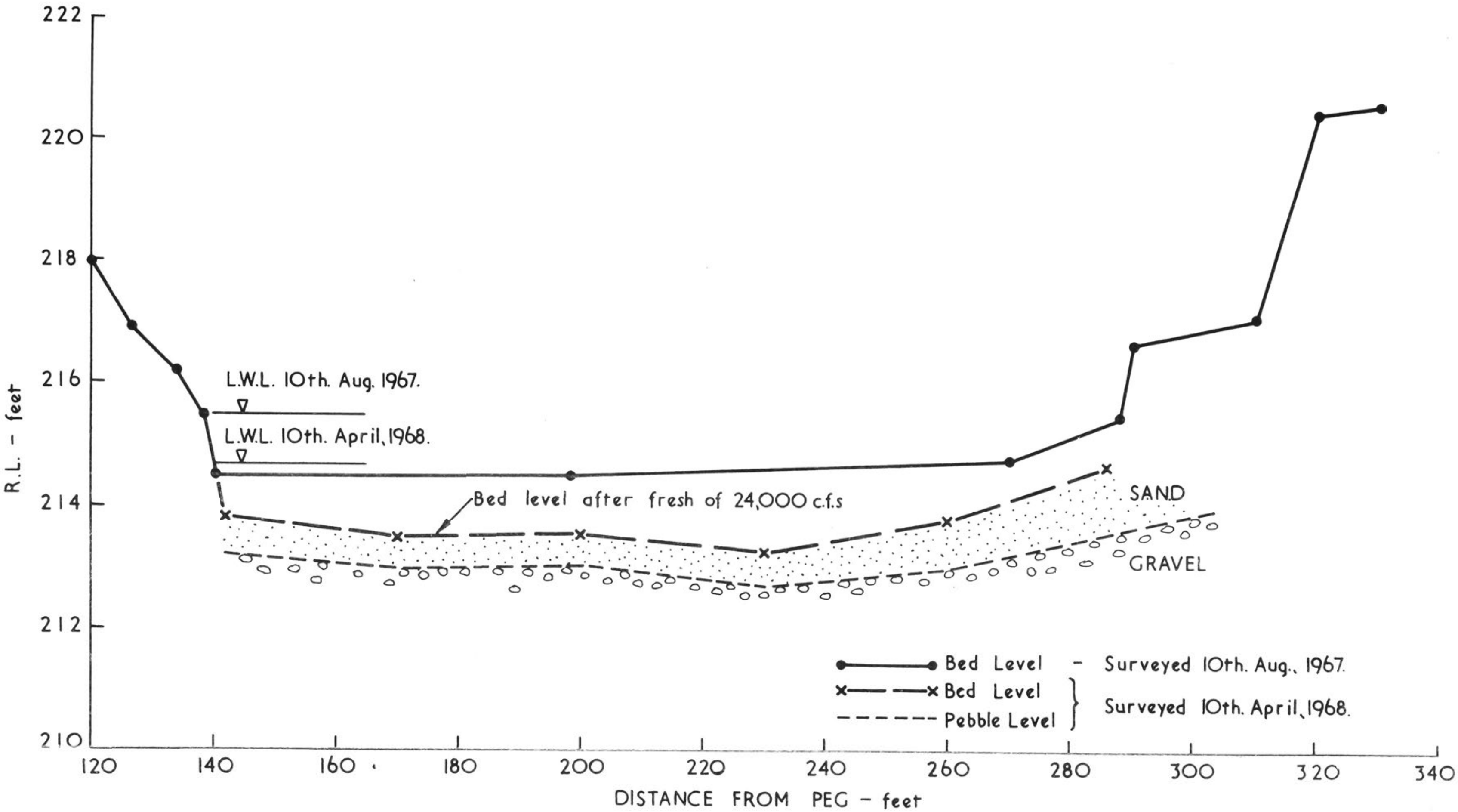


PROFILE OF PROBABLE POOL AND RIFFLE FORMATION AT GAUGING SITE IN HUNTER RIVER AT JERRY'S PLAINS



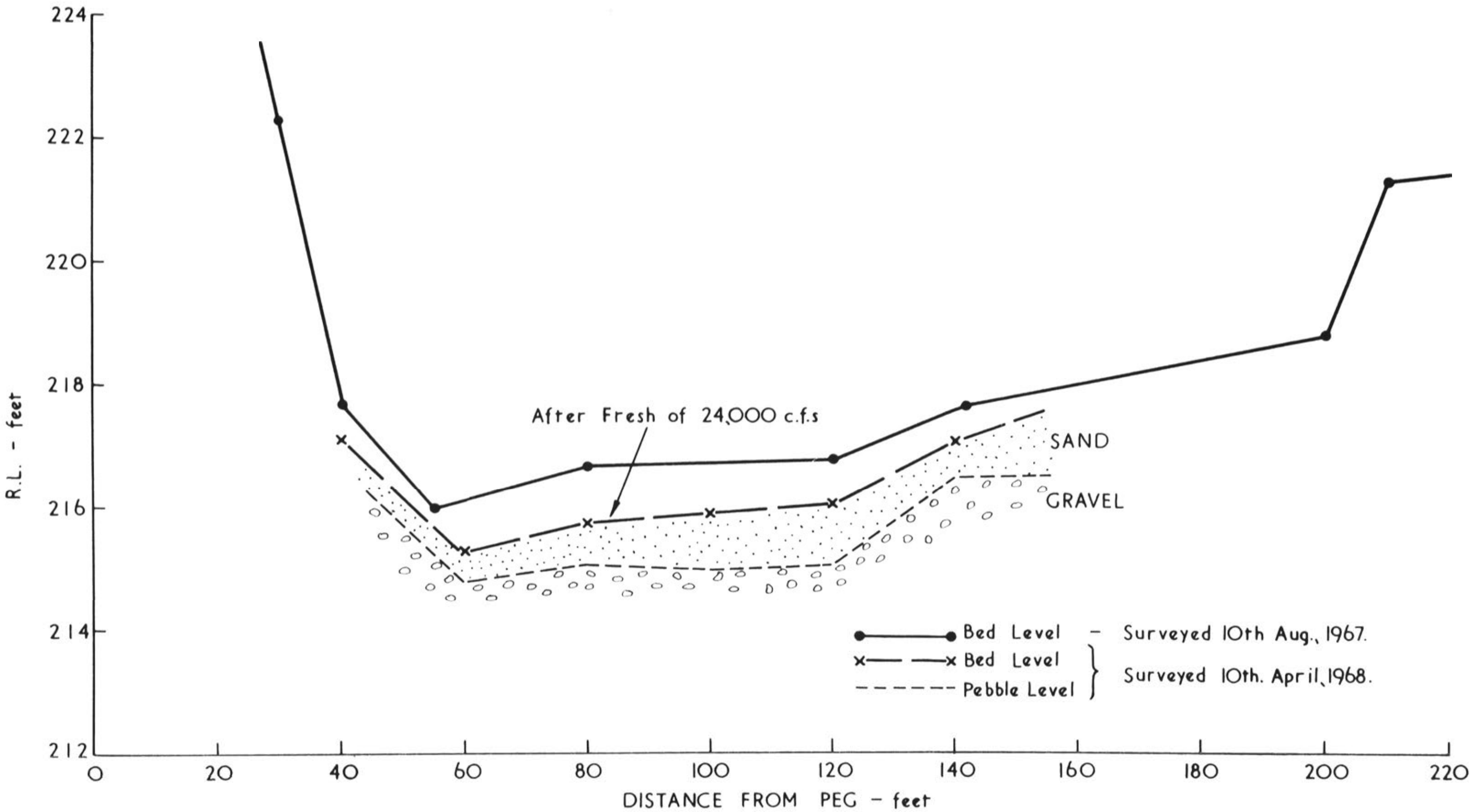
PROFILE OF POSSIBLE SAND SLUG FORMATION IN HUNTER RIVER AT JERRY'S PLAINS

Fig. 18: Pool and Riffle type of Gravel Bed.



CE-E-7352

Fig. 19: Bed Changes at Section 9.



CE-E-7353

Fig. 20: Bed Changes at Section 18.

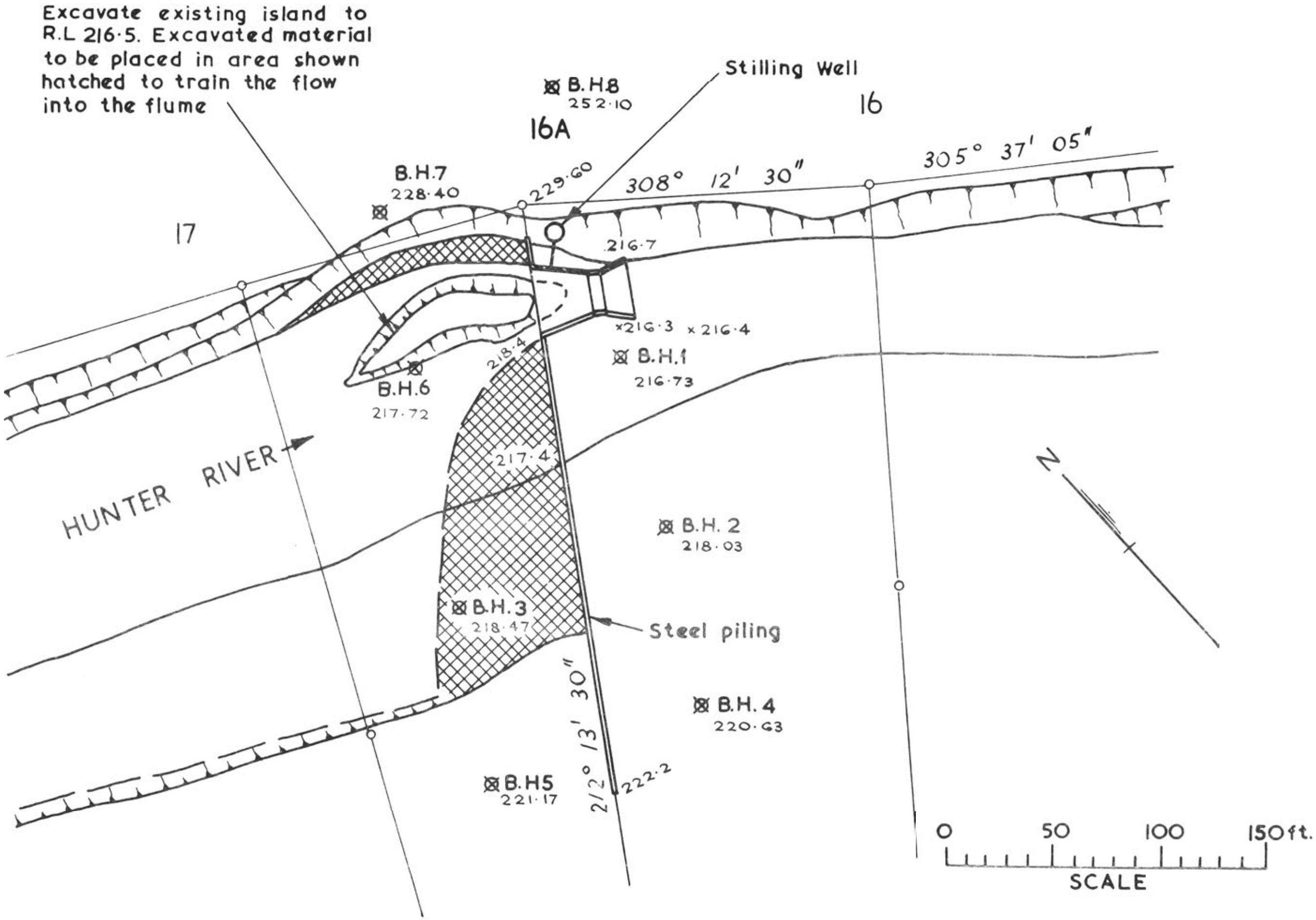
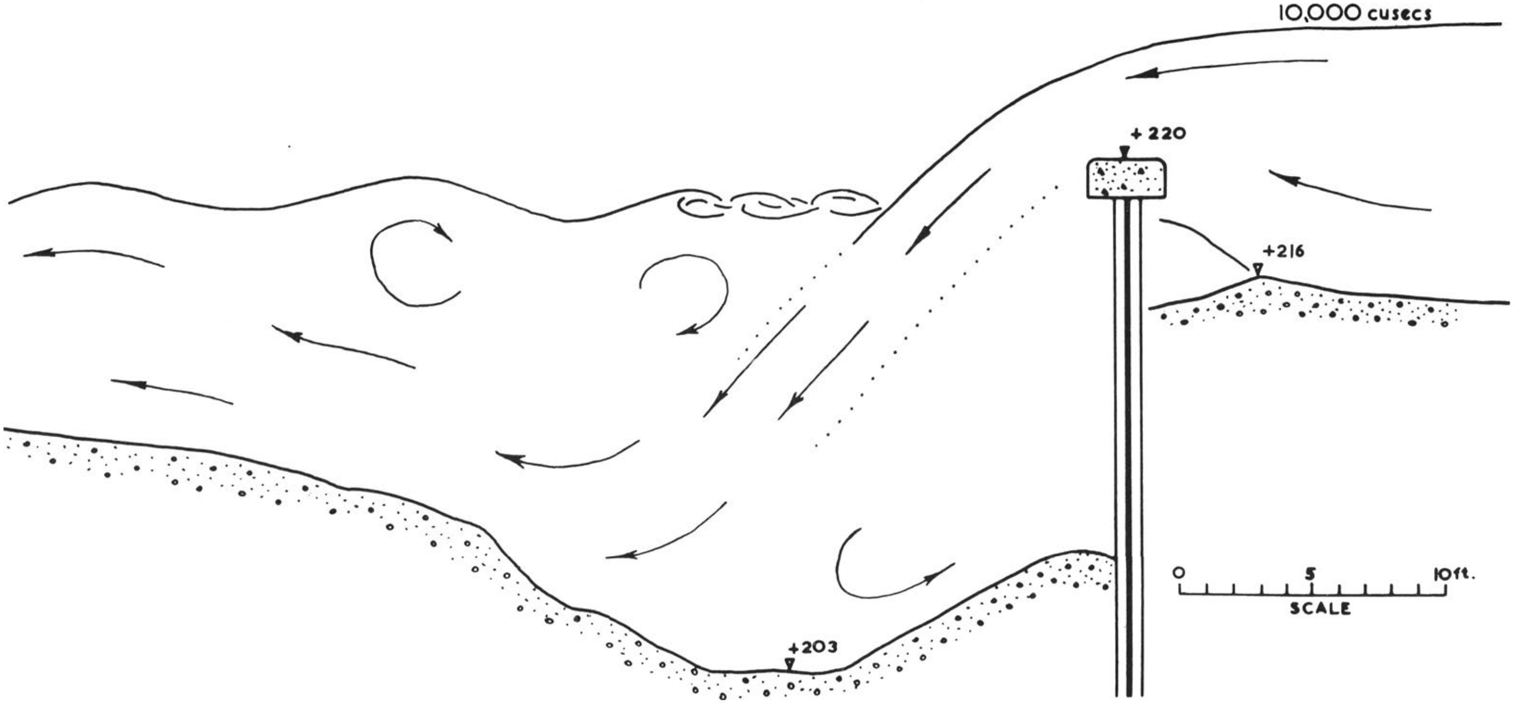


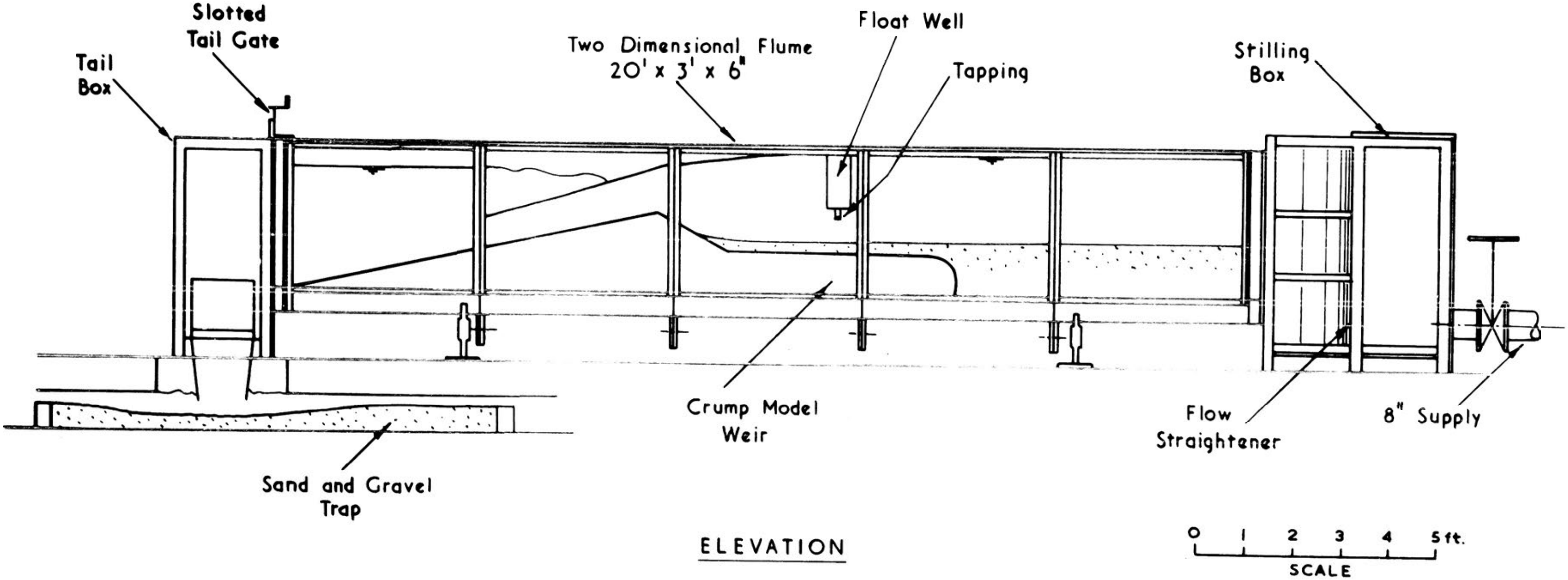
Fig. 21: Proposed Parshall Gauging Structure.

CE-E-7545



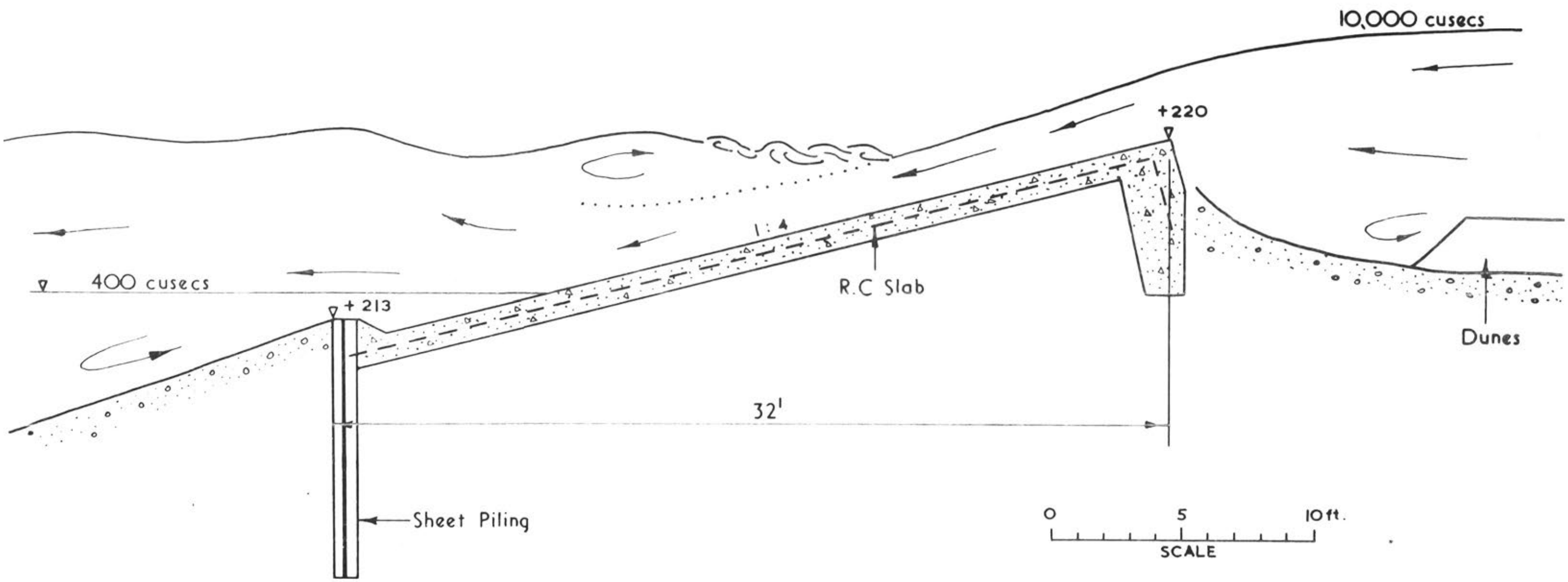
CE-E-7551

Fig. 22: Scour below Sheet Piling.



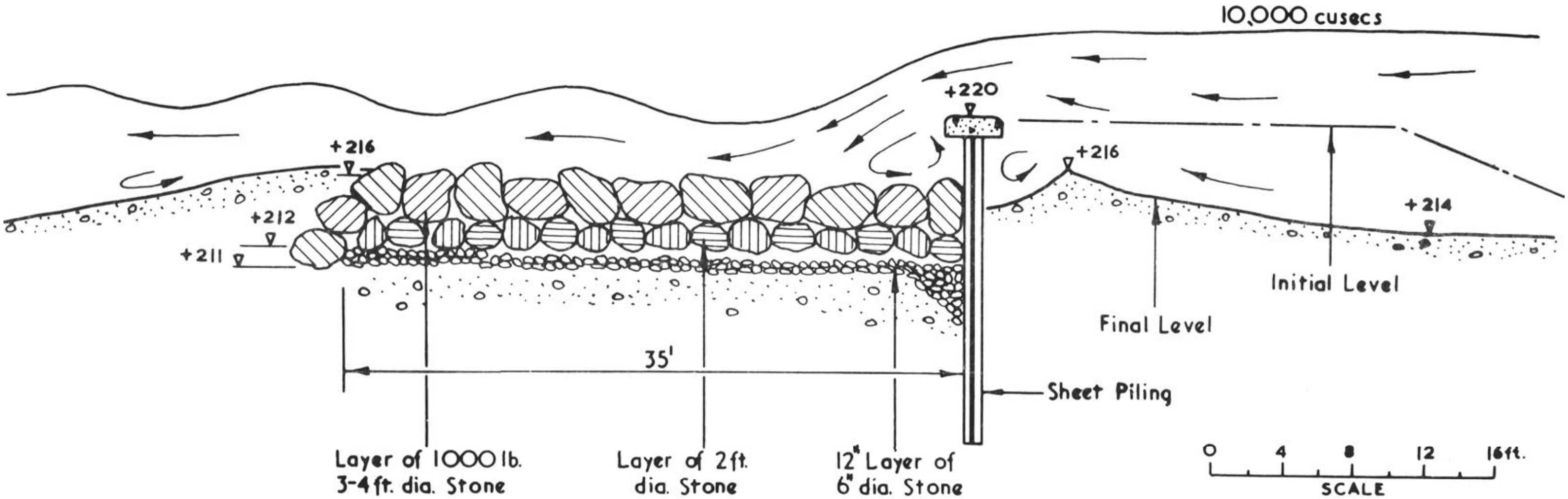
CE-E-7405

Fig. 23: Two Dimensional Flume.



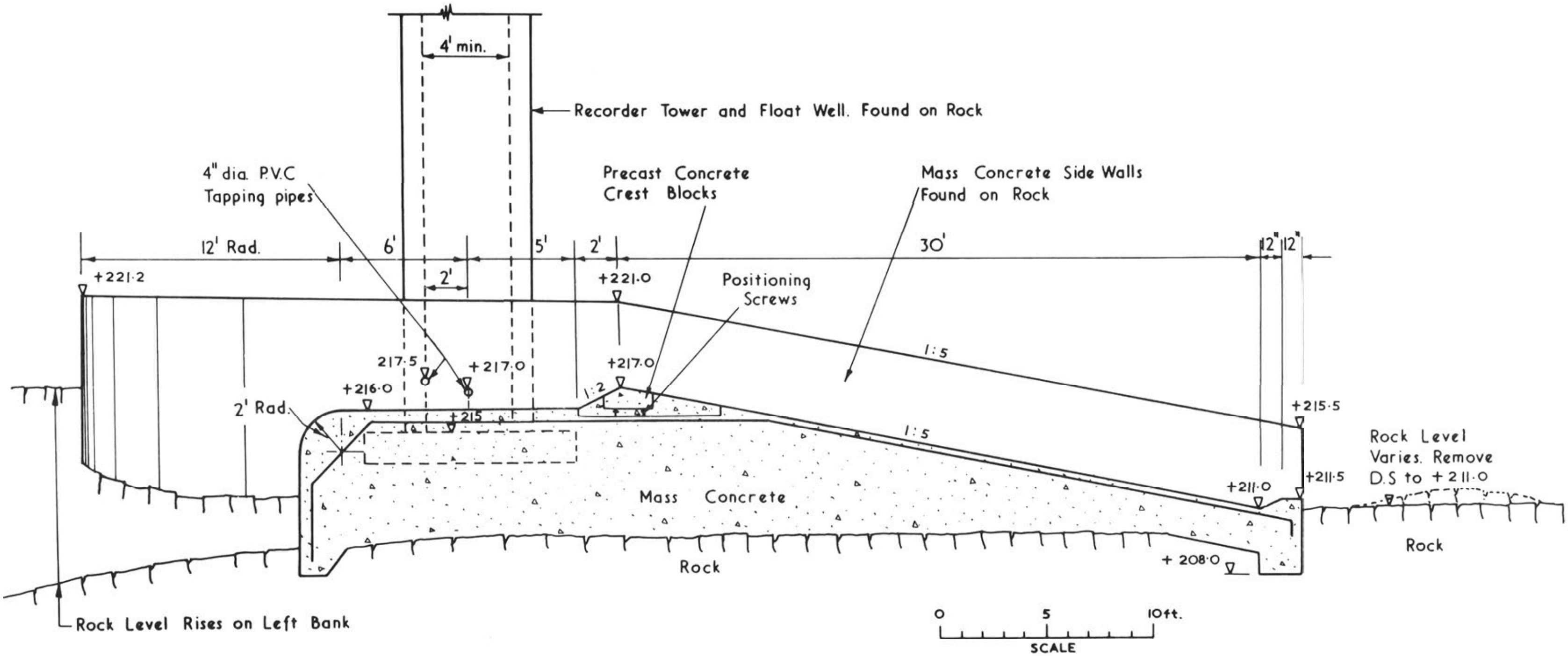
CE-E-7553

Fig. 24: Concrete Sloping Apron Alternative.



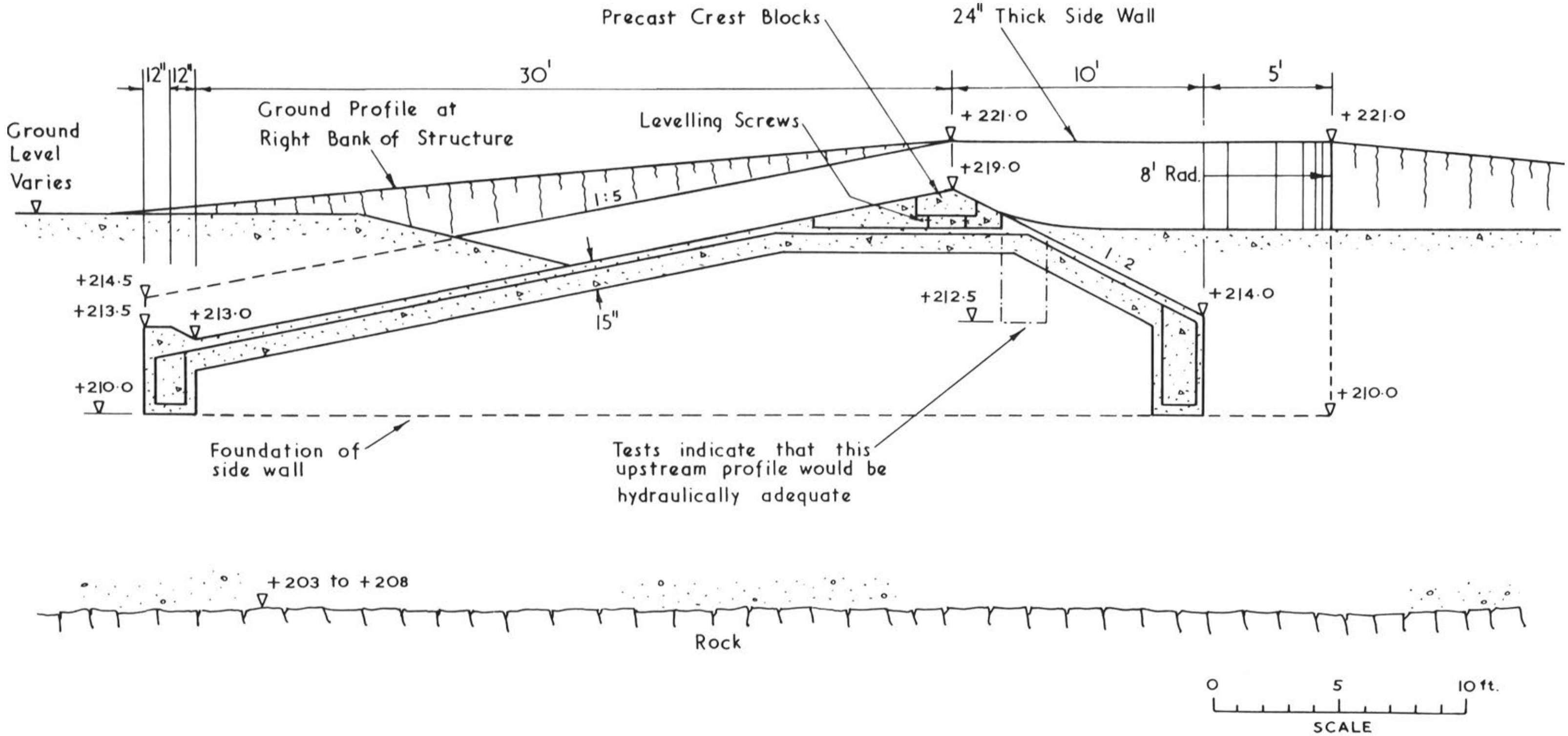
CE-E-7552

Fig. 25: Stone Apron to Piling.



CE-E-7401

Fig. 26: Crump Weir Left Bank Structure Tower.



CE-E-7402

Fig. 26a: Crump Weir Right Bank Structure.

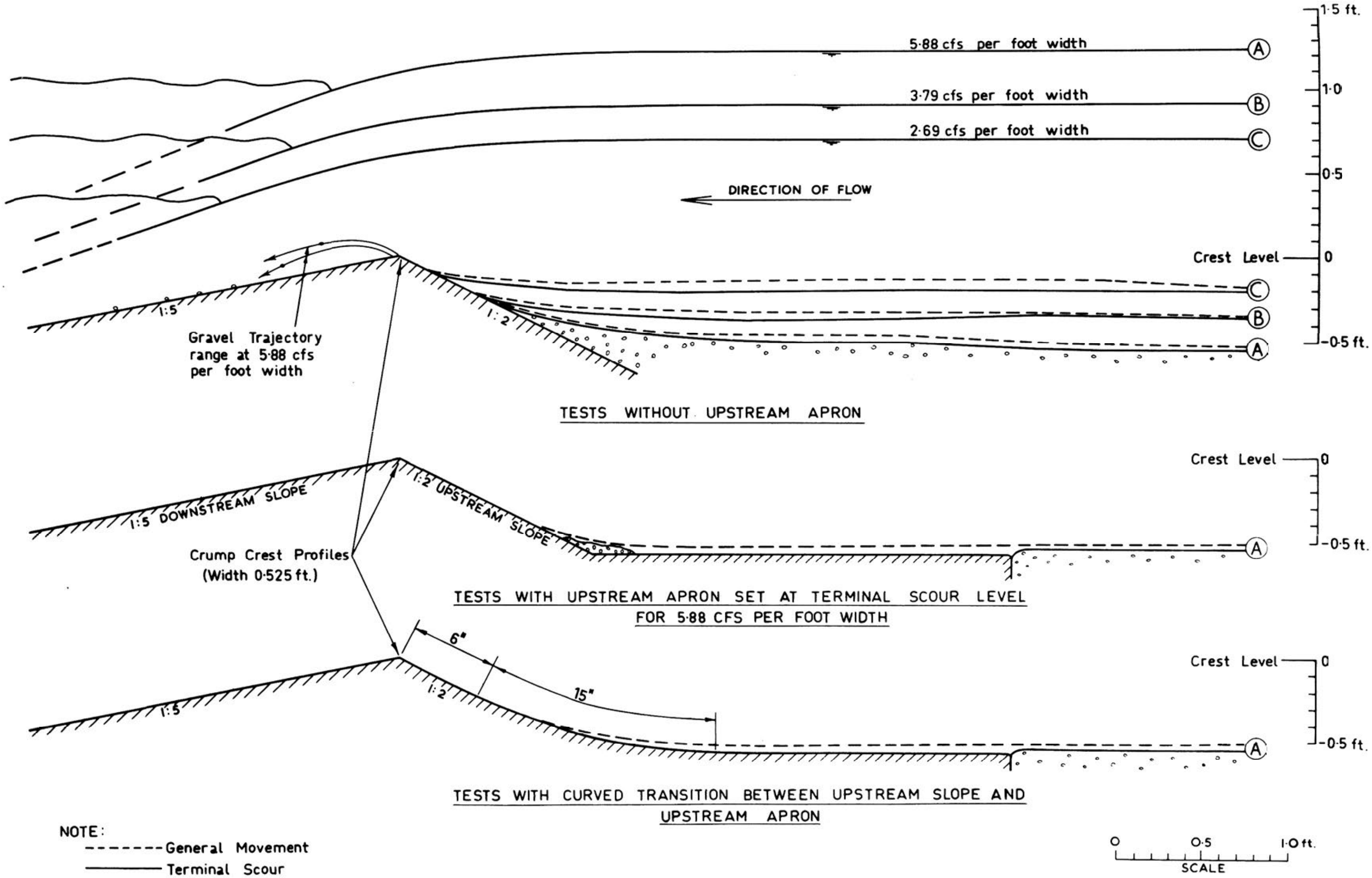


Fig. 27: Tests with Crump Weir 3/8" Gravel.

CE-E-7406

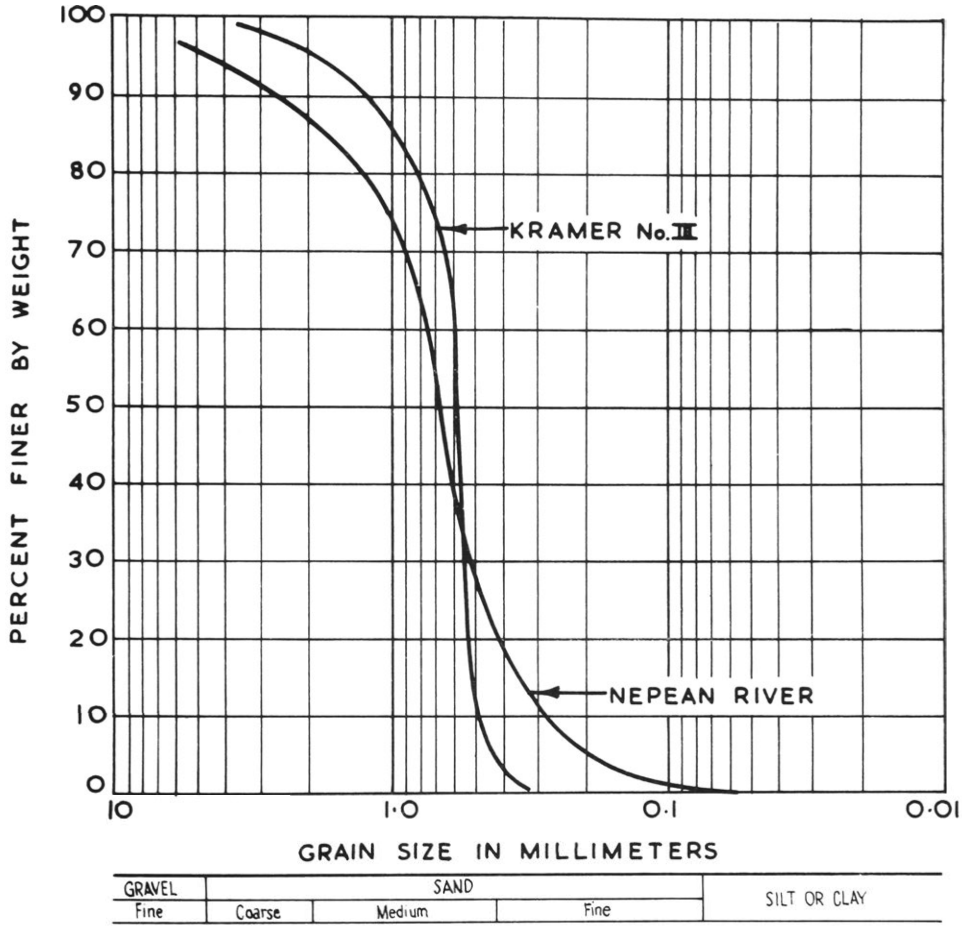


Fig. 28: Grading Curve for Nepean River Sand.

CE-E-7558

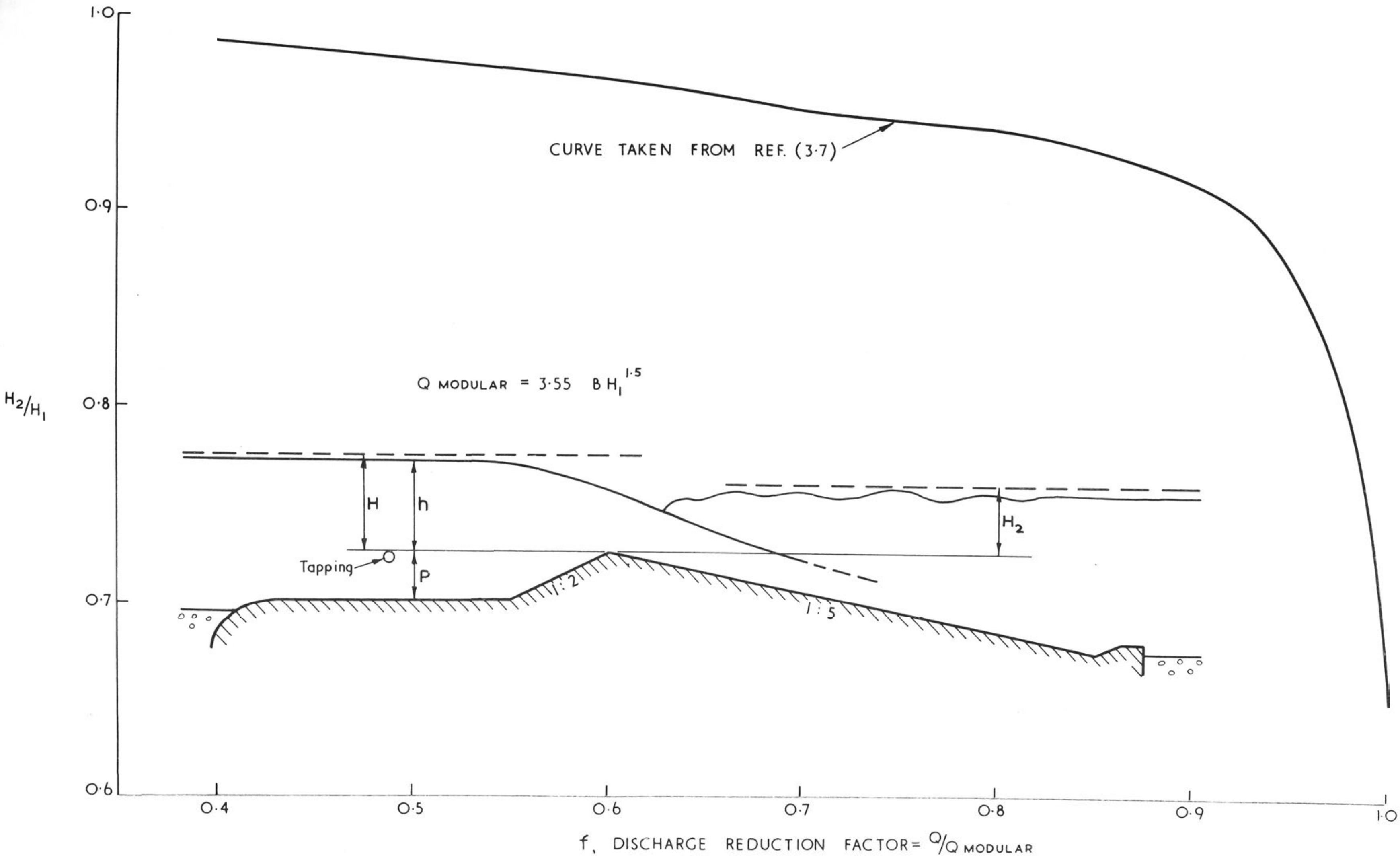


Fig. 29: Discharge Reduction Factor for Drowned Condition.

CE-D-7428

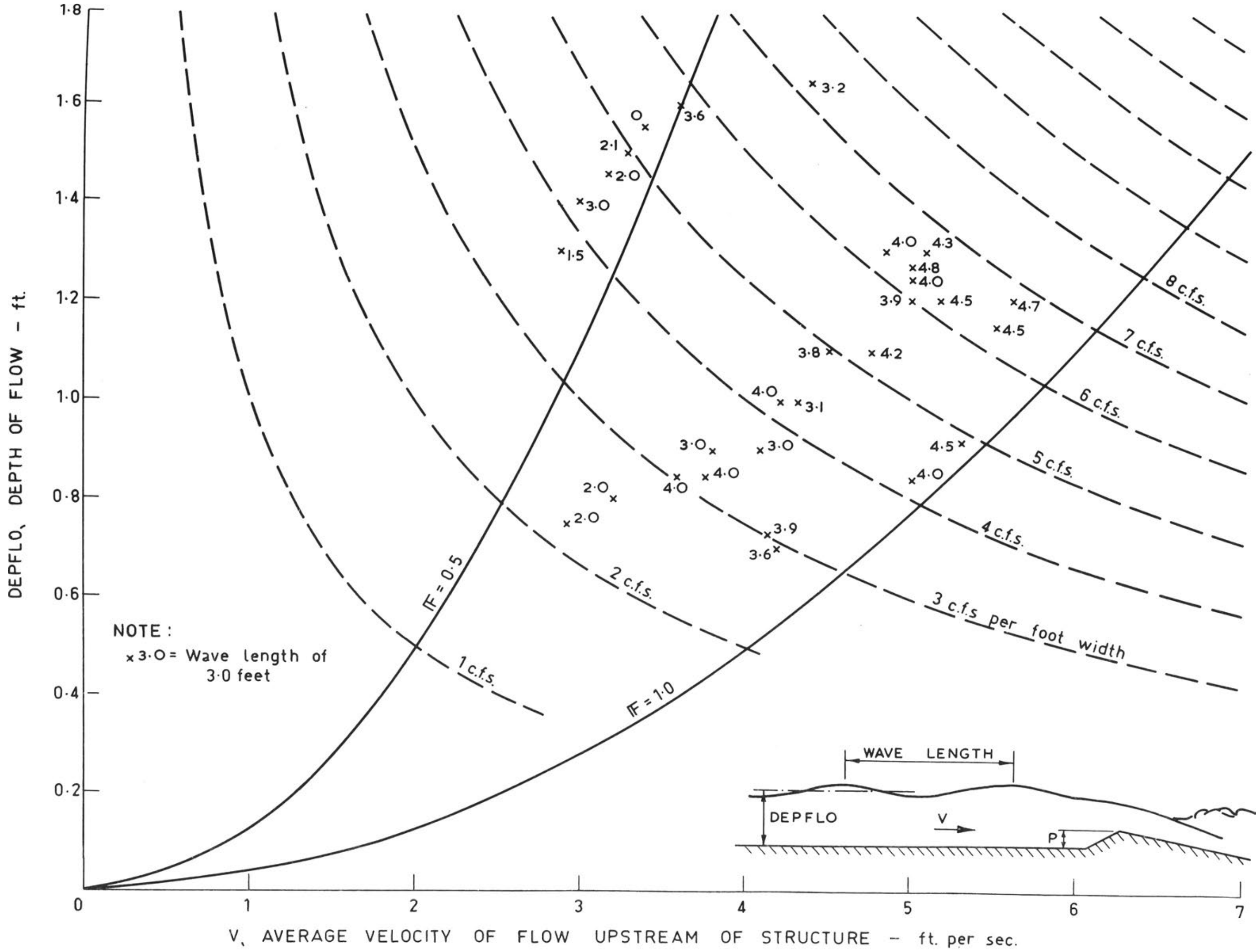


Fig. 30: Upstream Waves at High Froude Number.

CE-D-7602

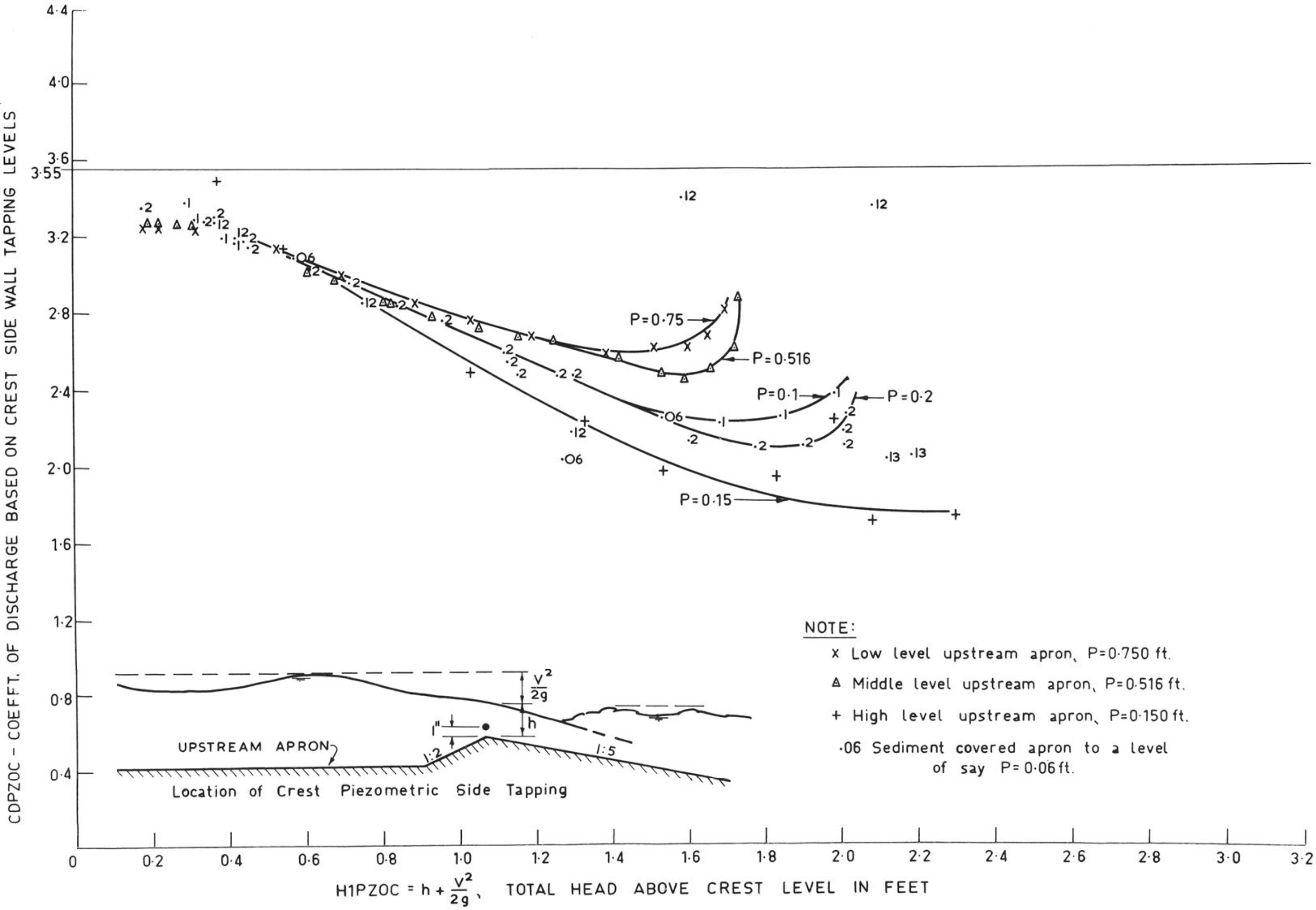


Fig. 31: Crest Side Wall Tapping Levels.

CE-D-7603

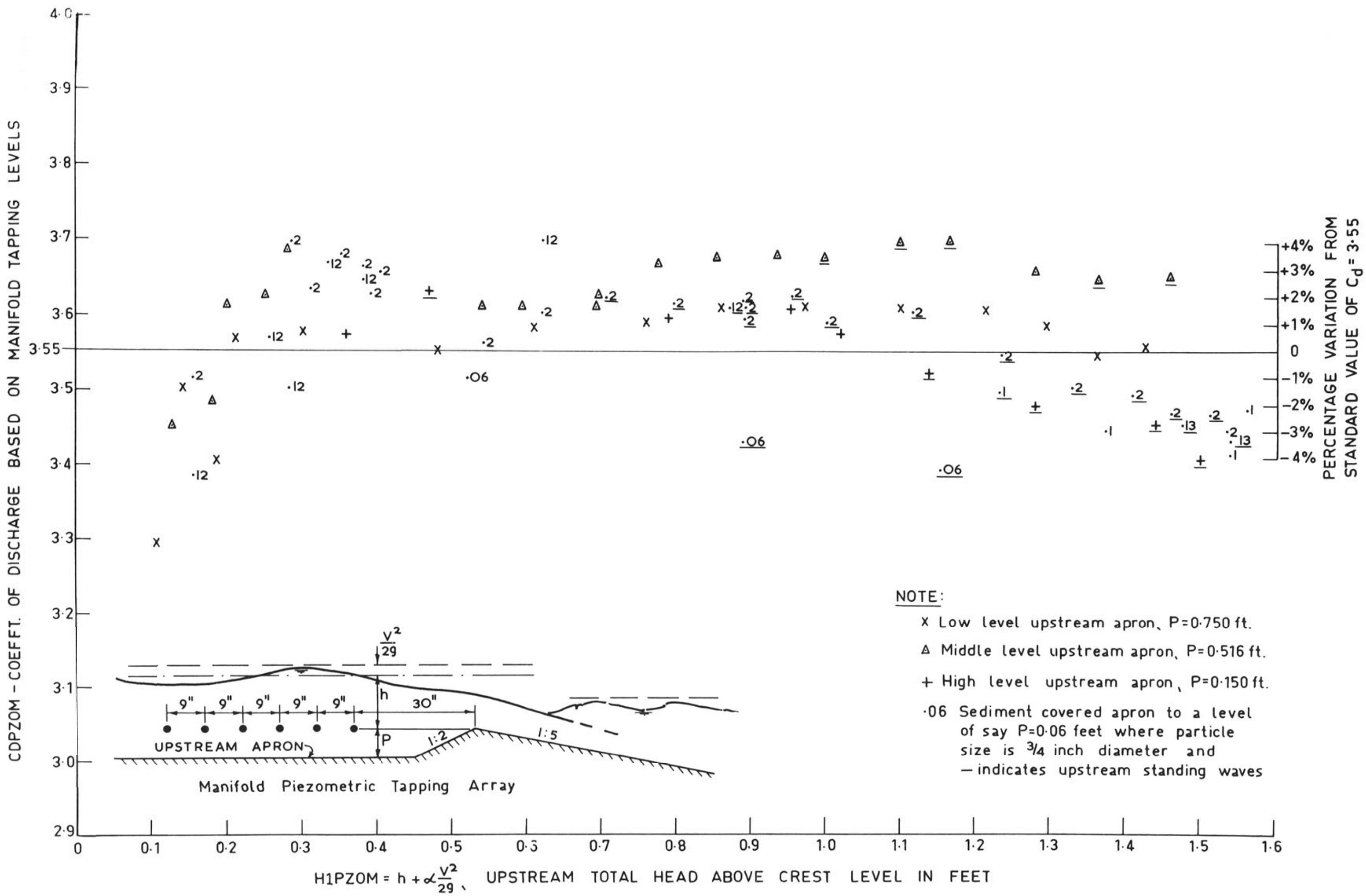


Fig. 32: Manifold Tapping Levels.

CE-D-7505

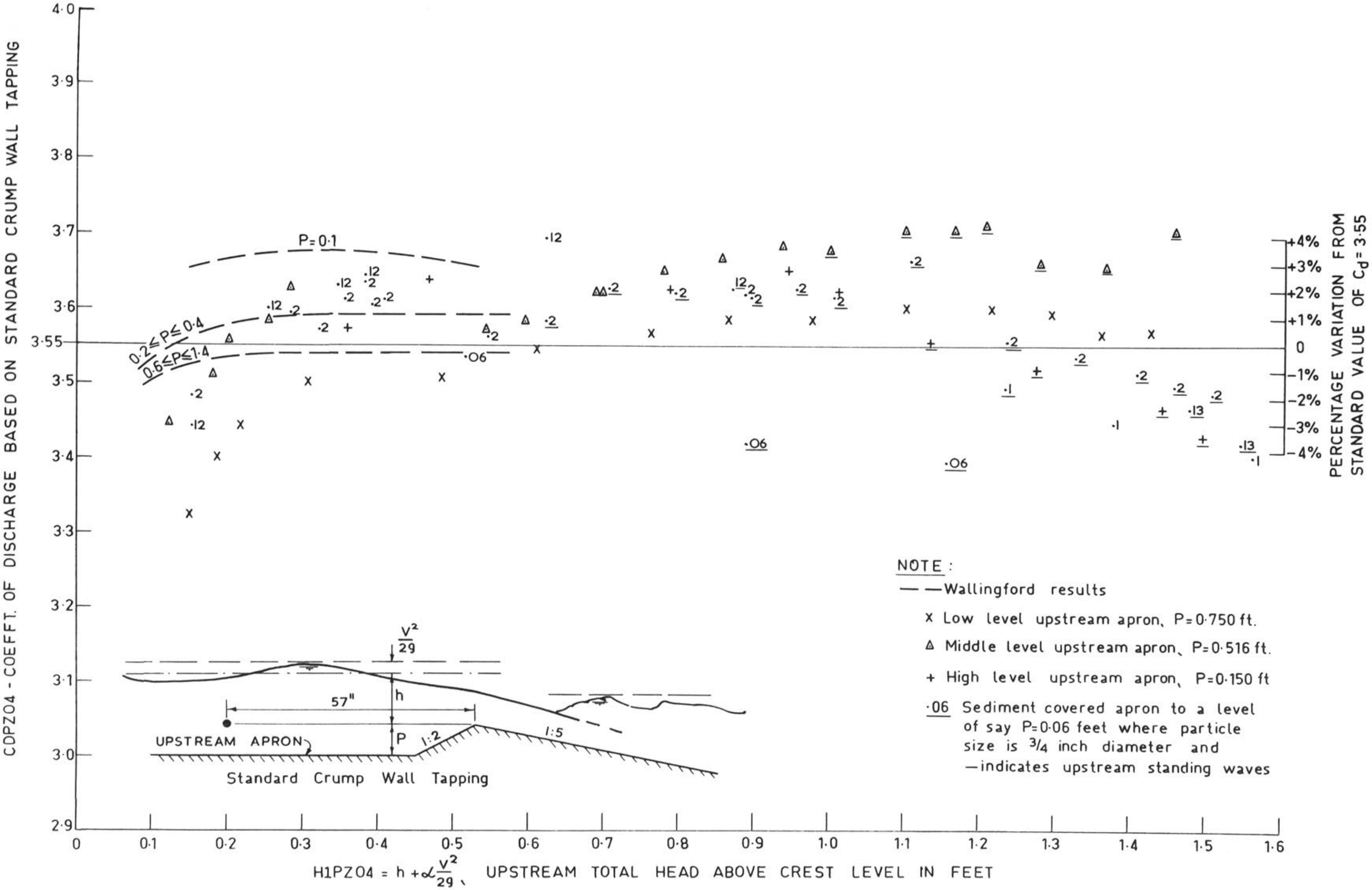


Fig. 33: Standard Crump Wall Tapping.

CE-D-7507

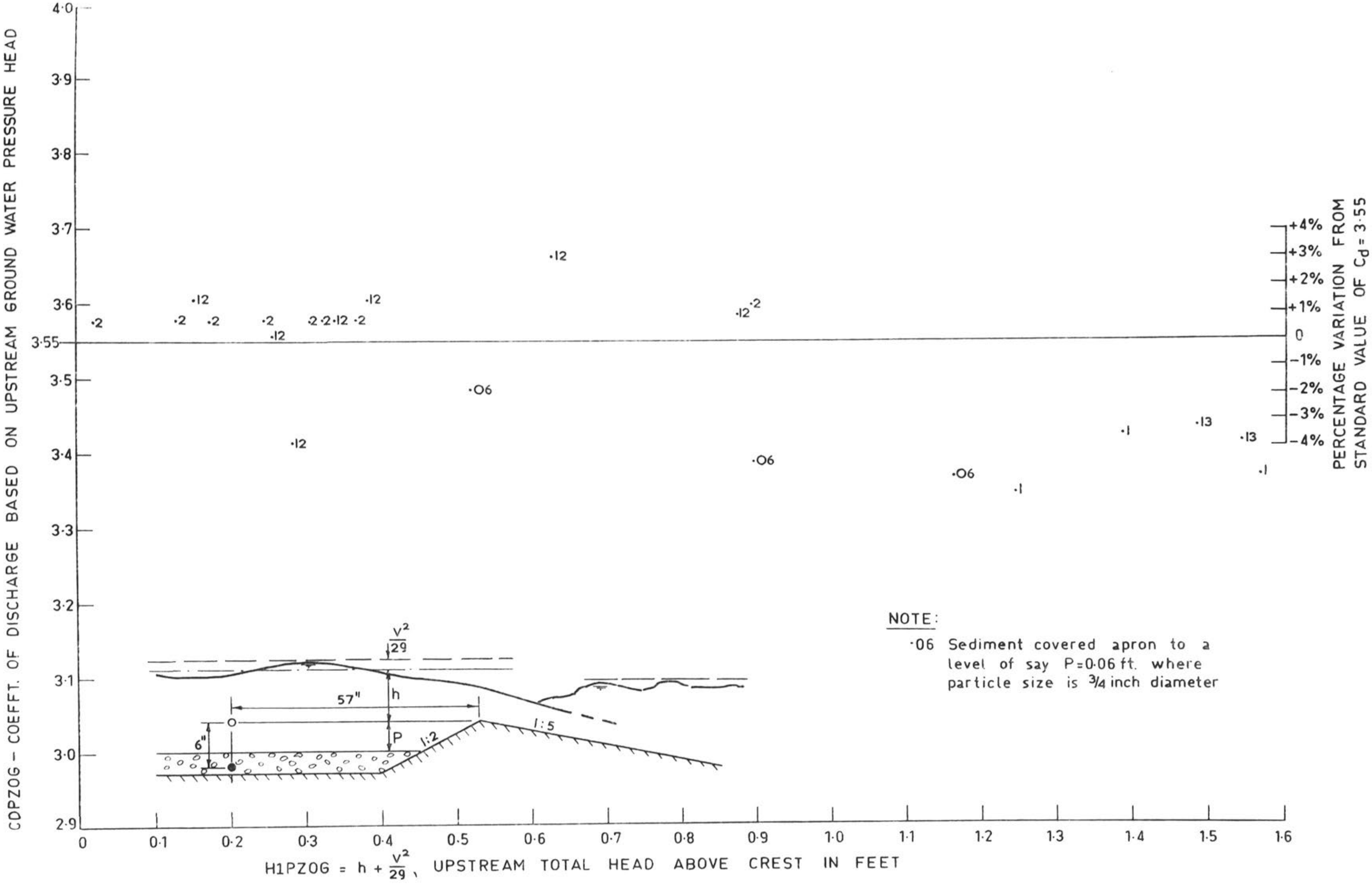


Fig. 34: Upstream Ground Water Pressure Head.

CE-D-7609

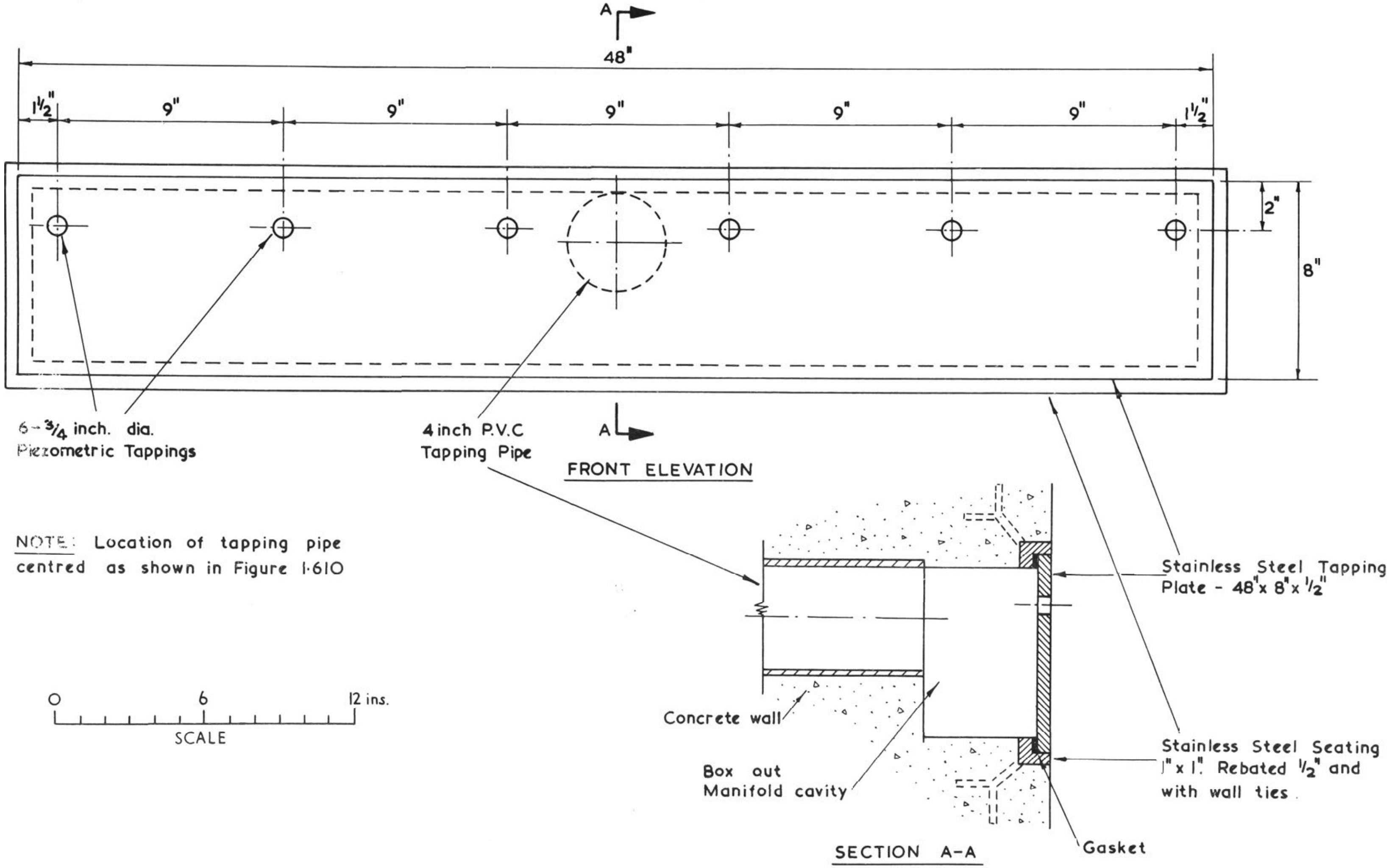


Fig. 35: Wall Manifold Tapping Plate.

CE-D-7565

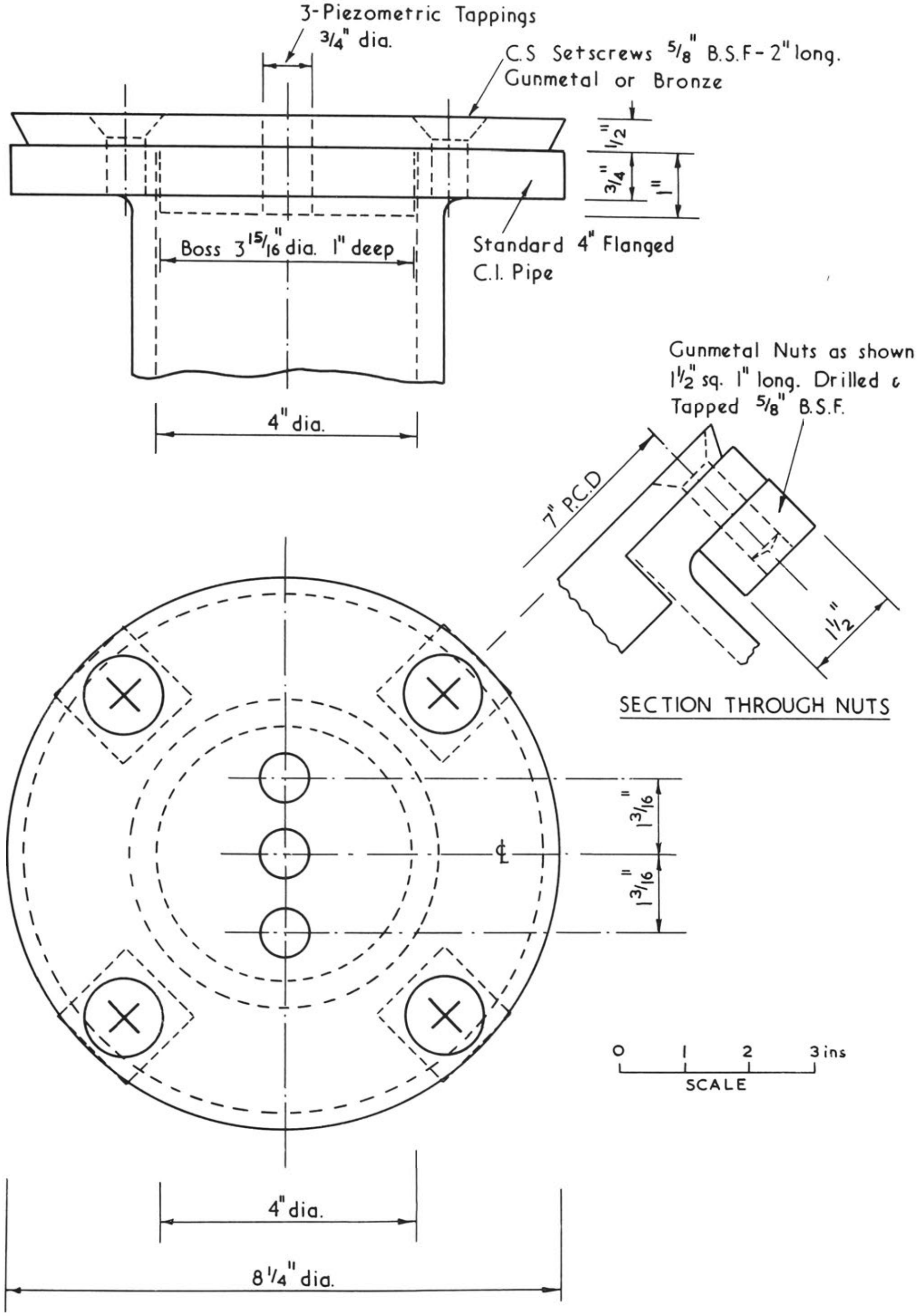


Fig. 36: Wall Tapping Plate.

CE-E-7403

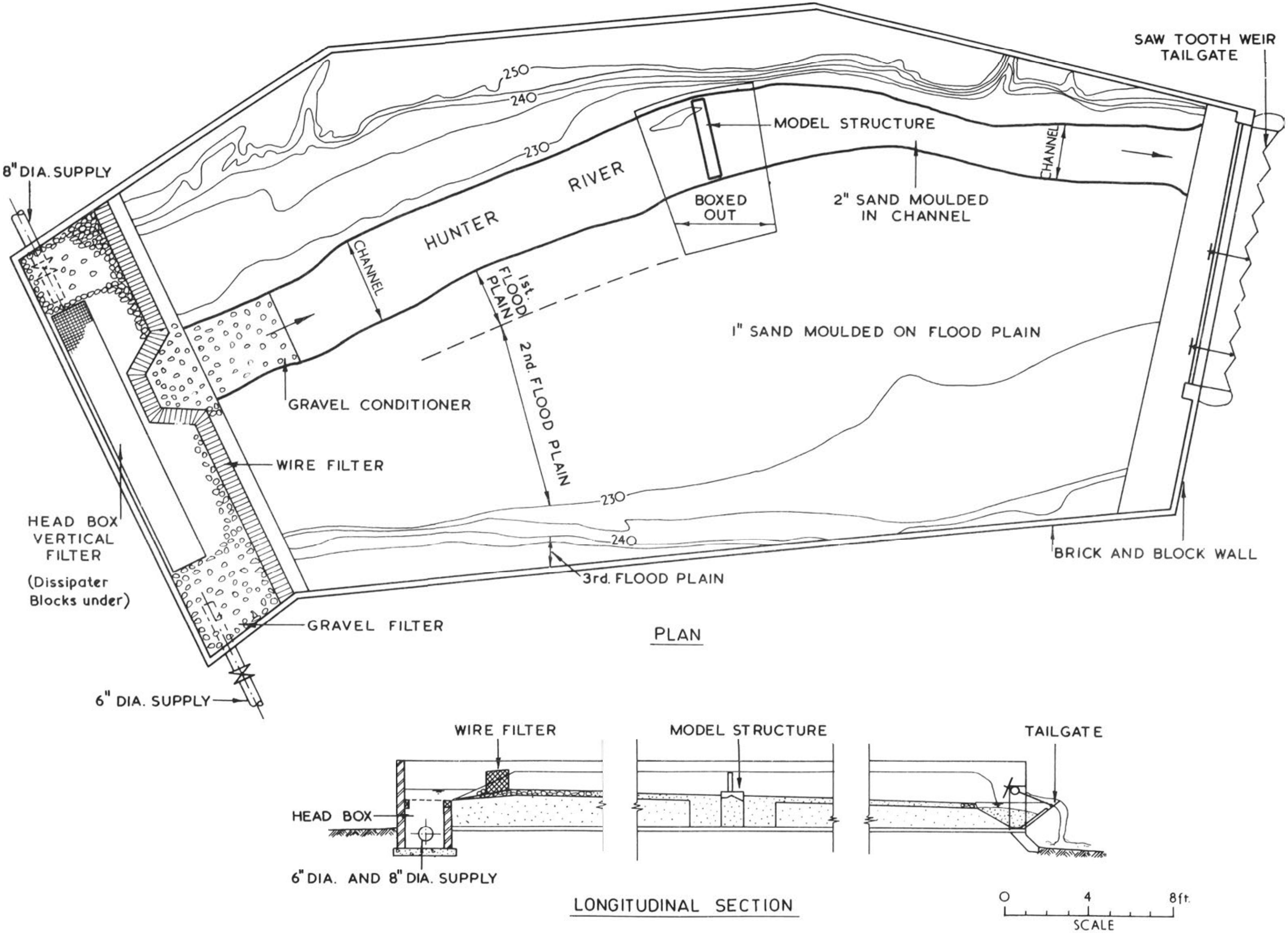


Fig. 37: Hunter River Model.

CE-D-7407

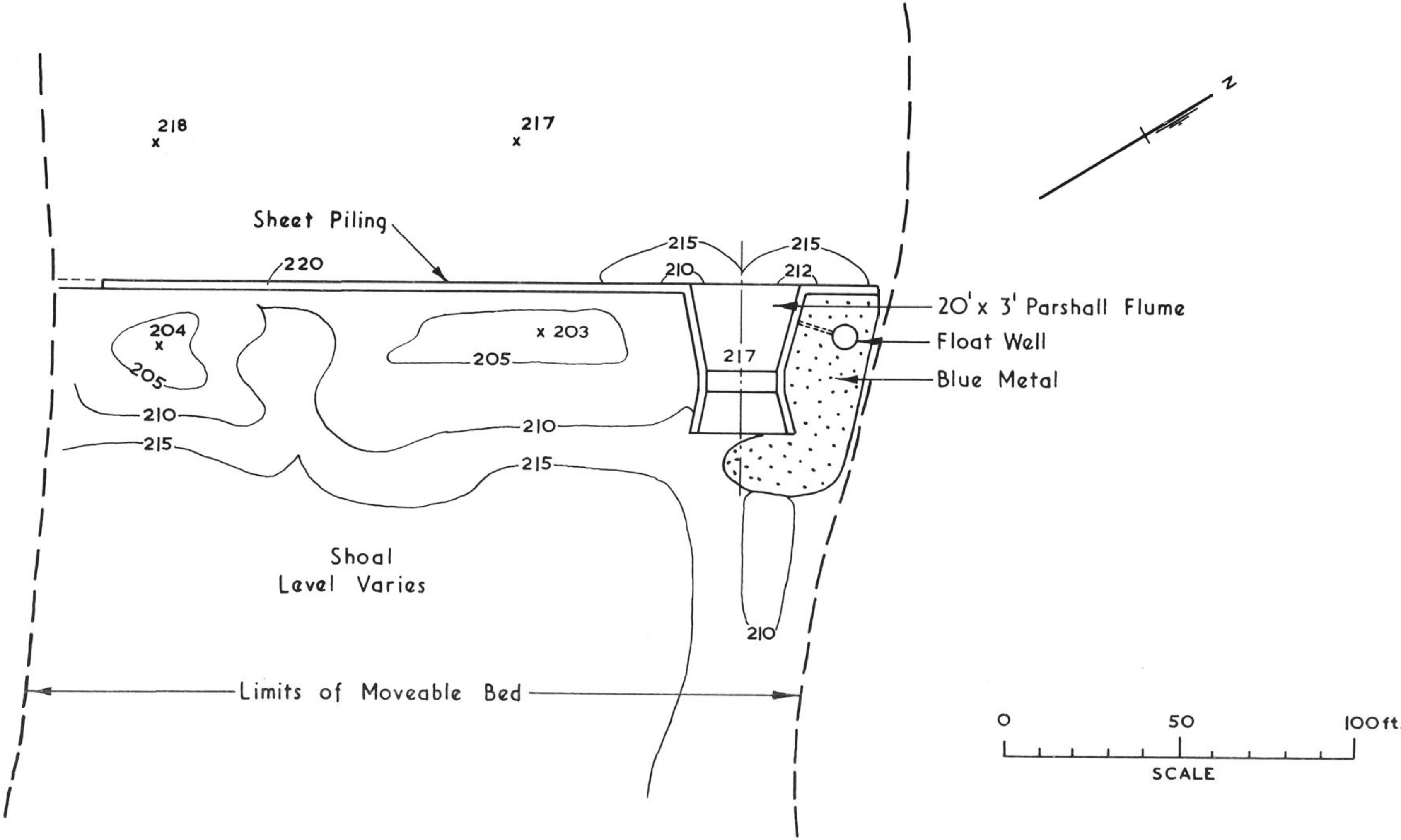
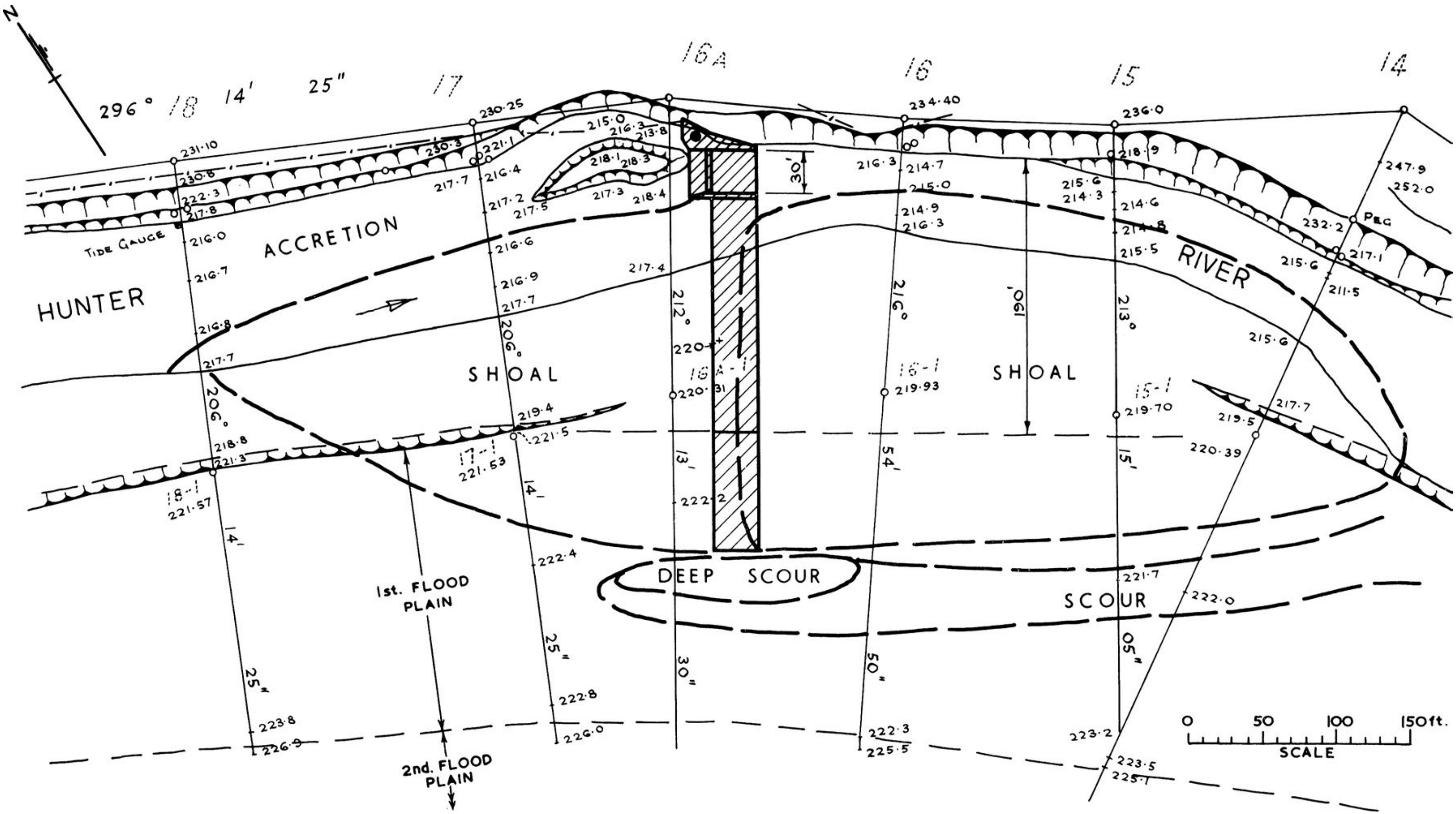


Fig. 38: Scour Parshall Flume and Sheet Piling.

CE-E-7560



CE-E-7561

Fig. 39: Outflanking of Single Level Cut Off Weir.

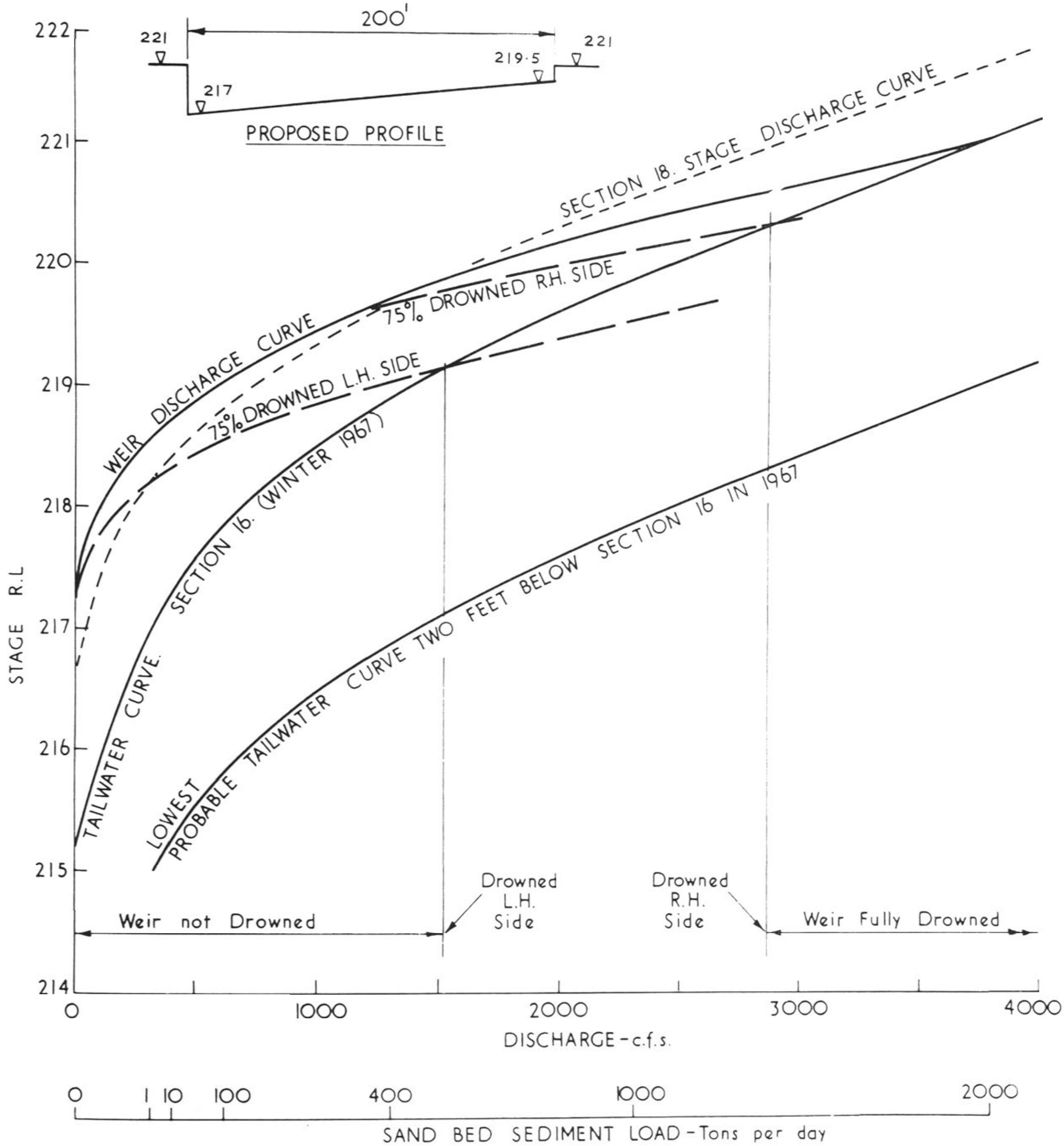


Fig. 40: Design Curves for Crump Weir 200' Wide as Proposed.

CE-E-7393

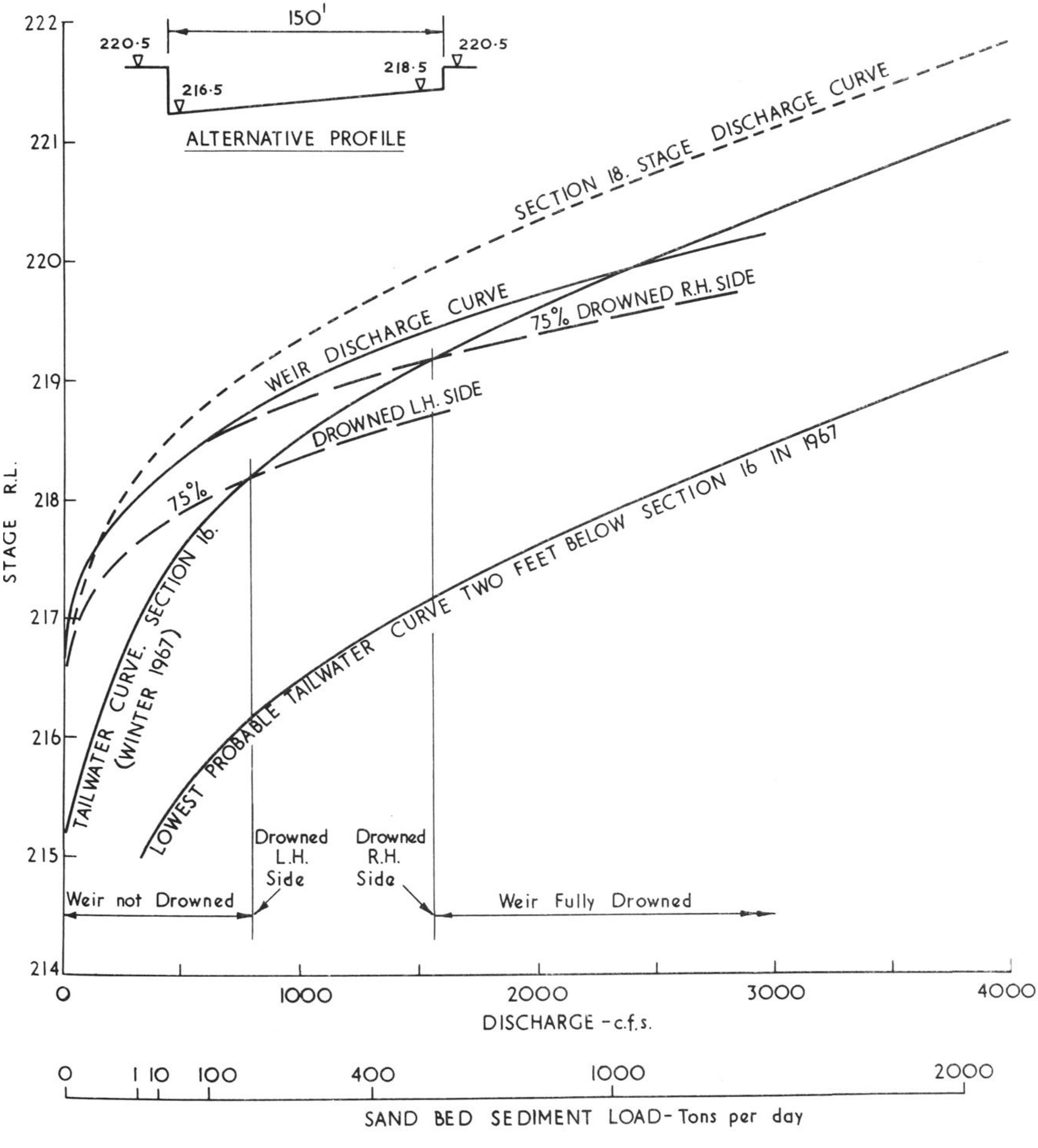


Fig. 41: Design Curves for Crump Weir 150' Alternative.

CE-E-7395

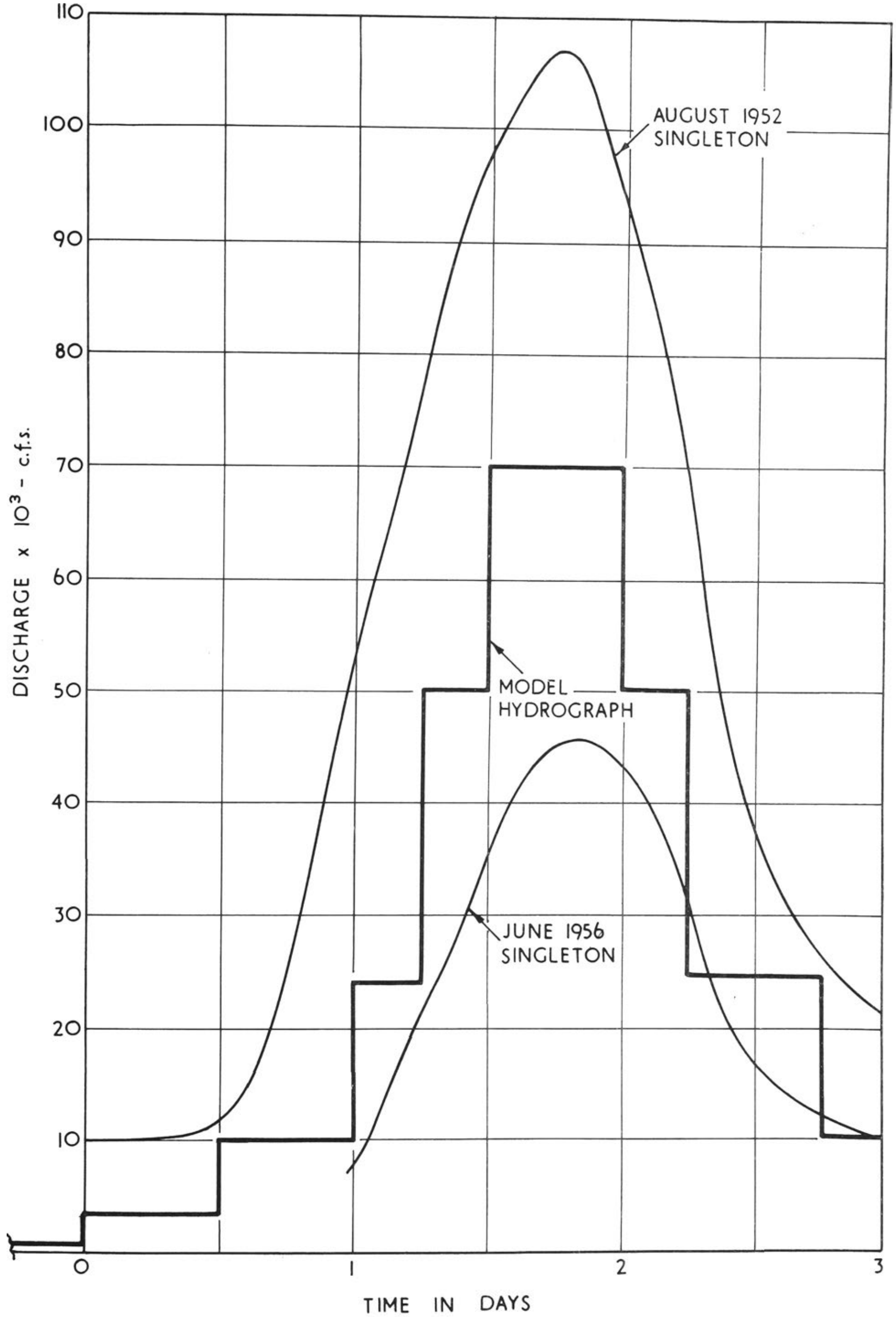


Fig. 42: Model Flood Hydrograph.

CE-E-7408

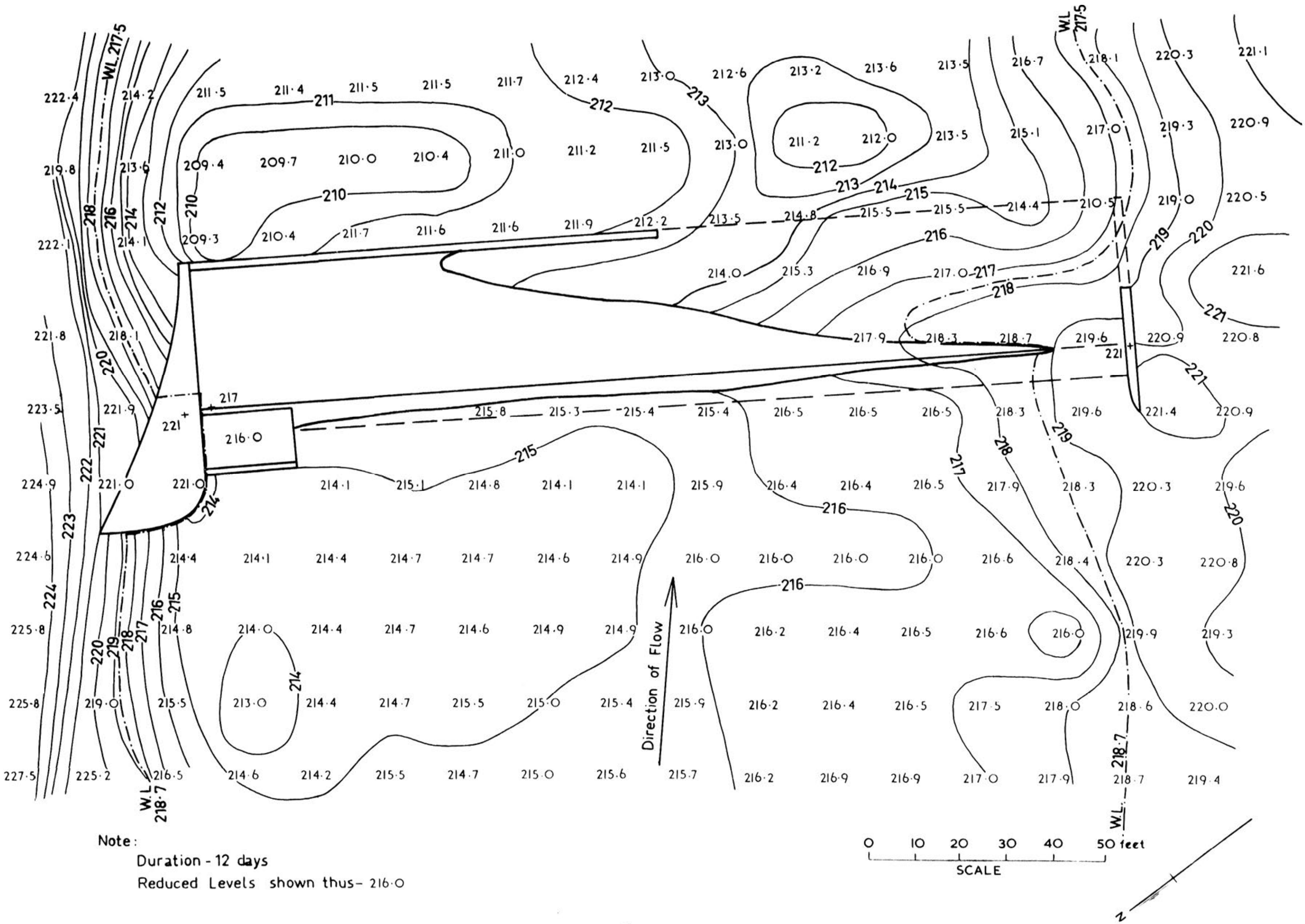


Fig. 43: Scour 353 c. f. s.

CE - C-7409

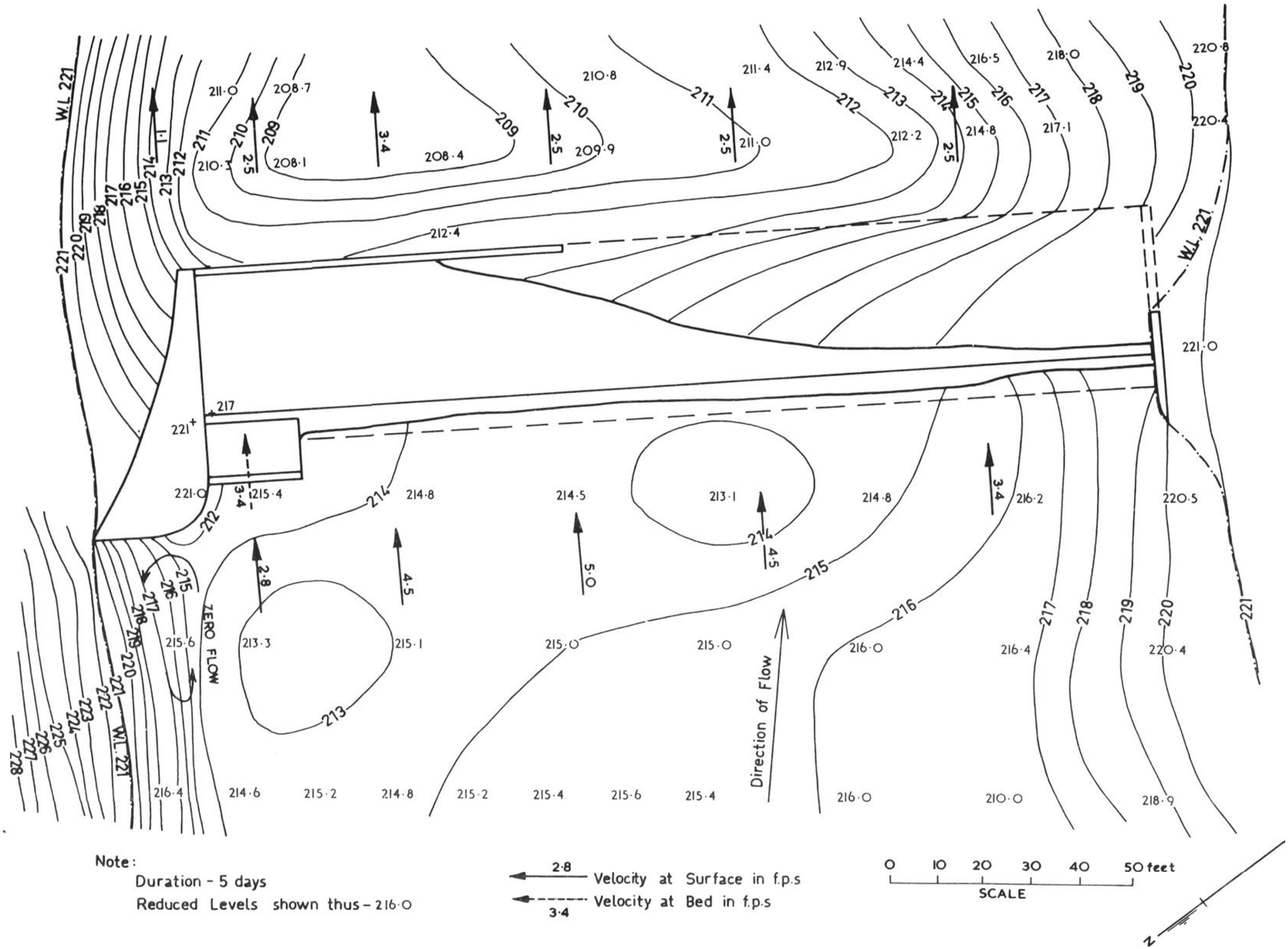


Fig. 44: Scour 3930 c. f. s.

CE-C-7410

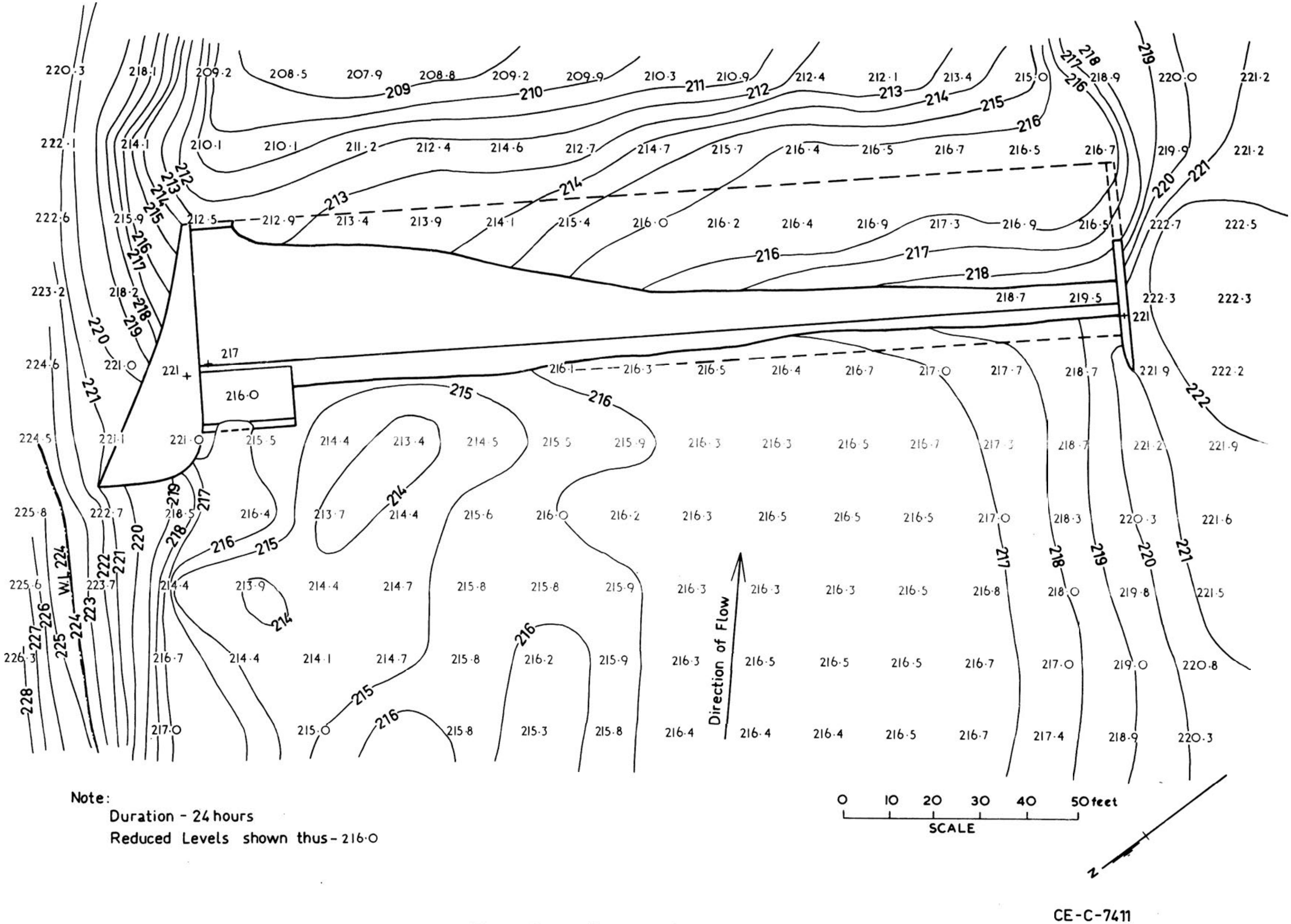
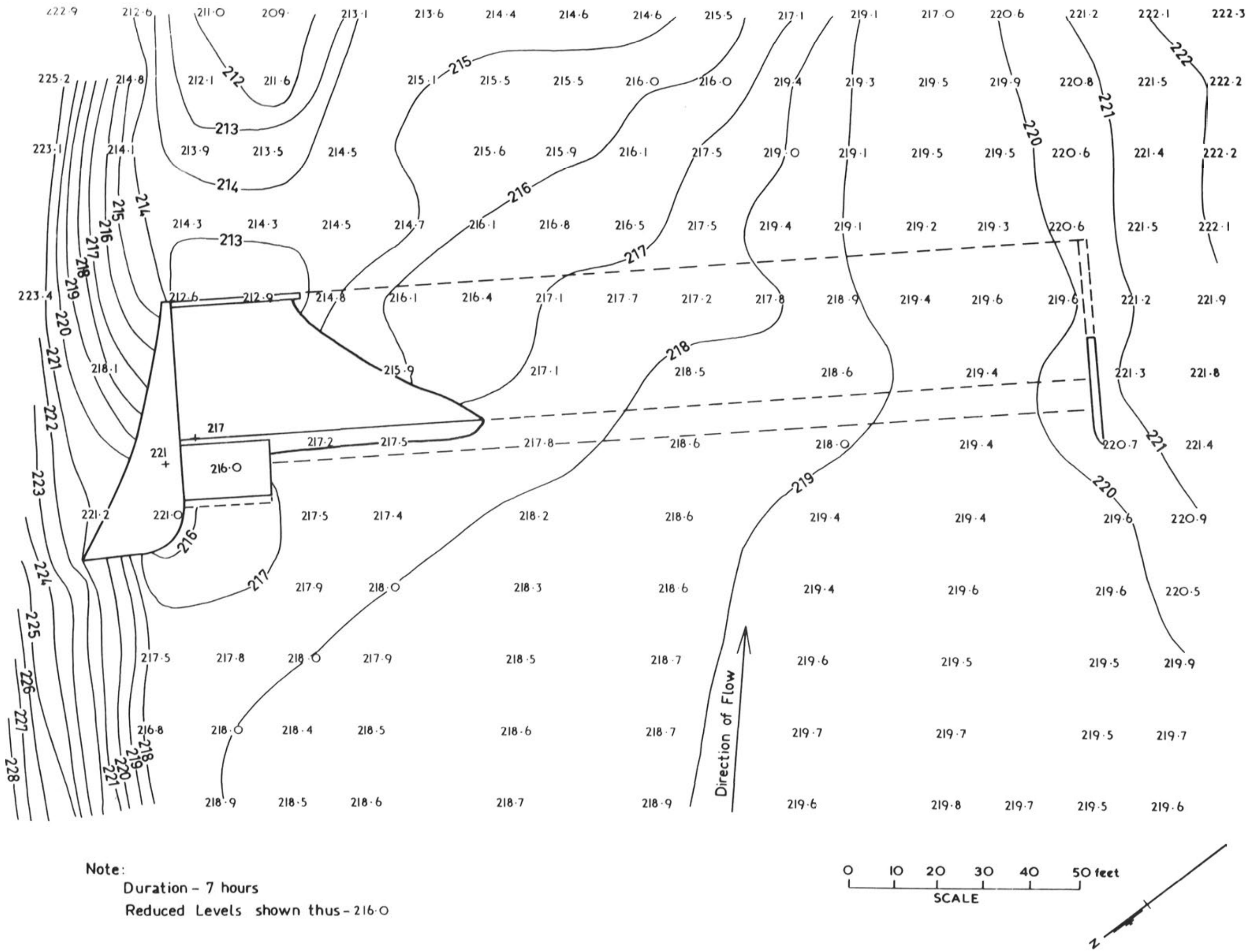


Fig. 45: Scour 10,000 c.f.s.



Note:
Duration - 7 hours
Reduced Levels shown thus - 216·0

Fig. 46: Scour 24,000 c. f. s.

CE-C-7412

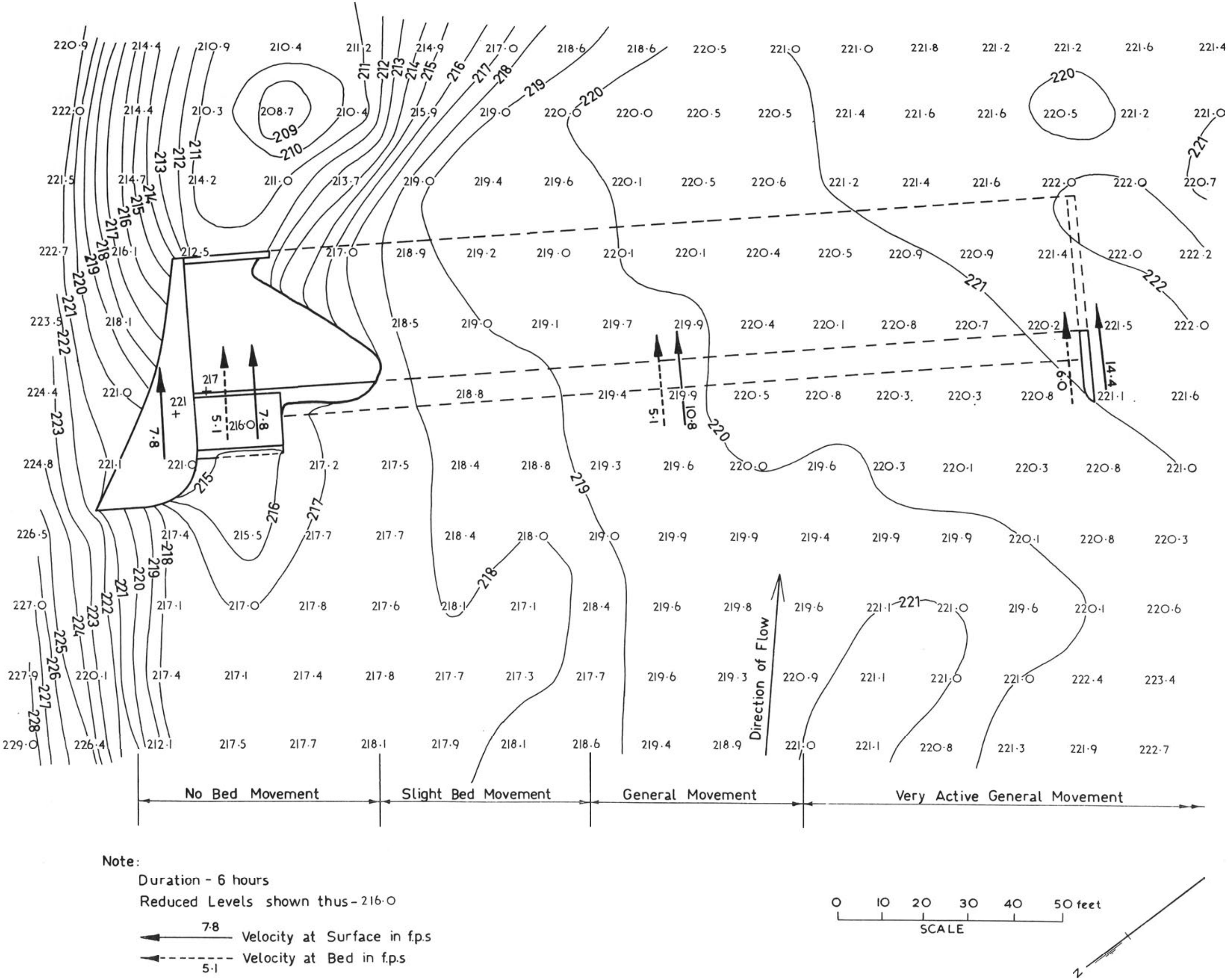


Fig. 47: Scour 50,000 c.f.s.

CE-C-7413

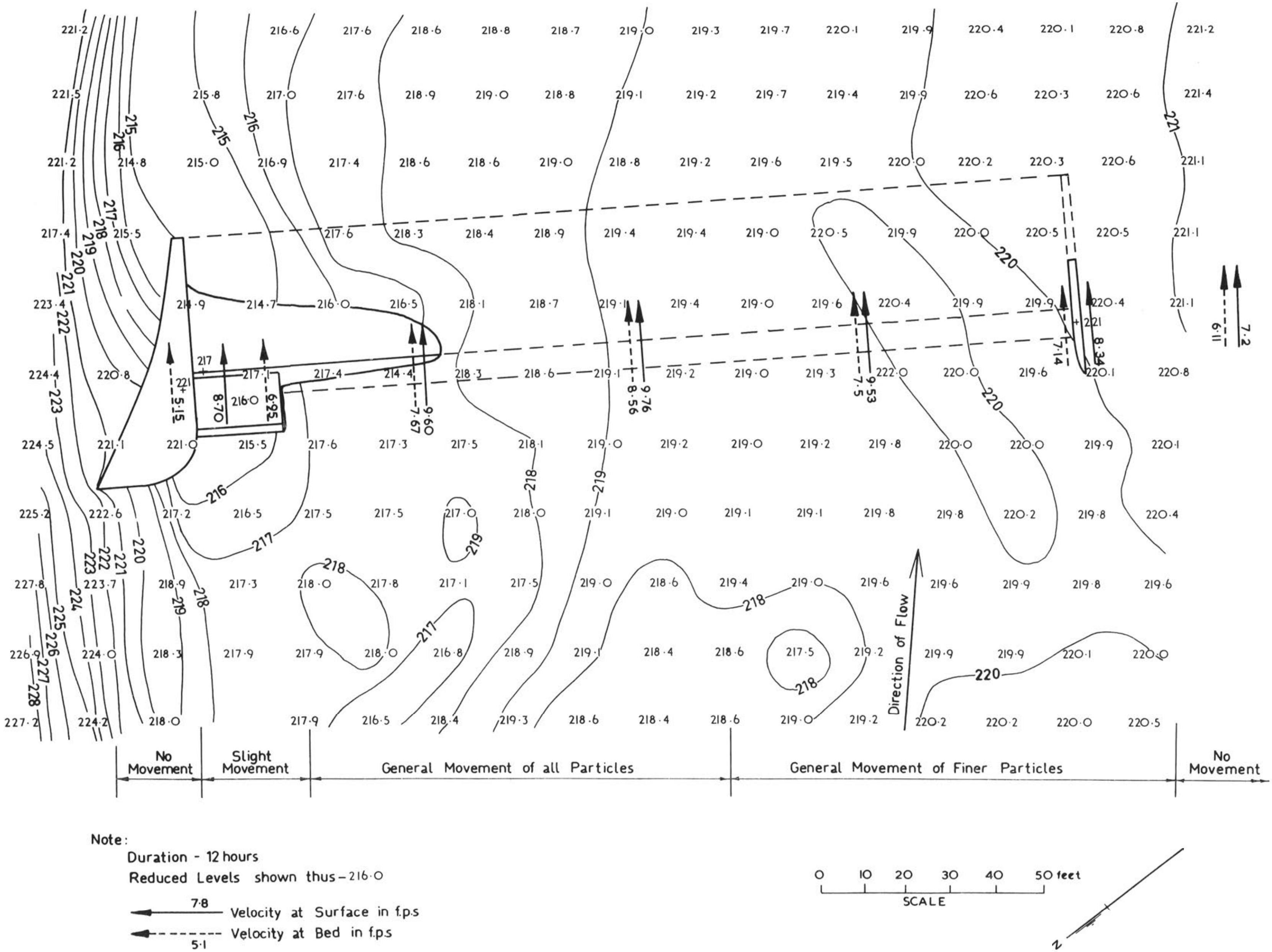
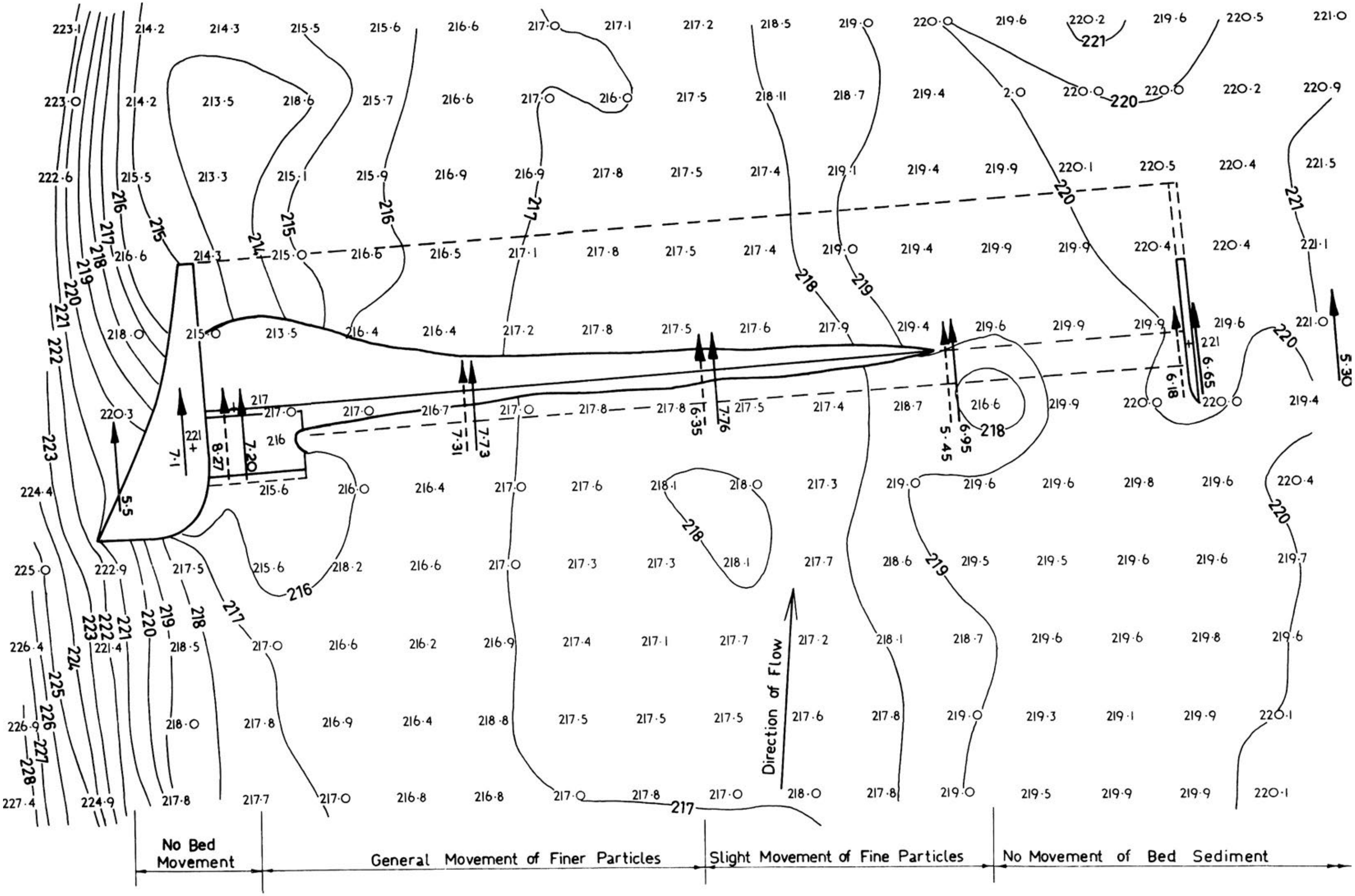


Fig. 48: Scour 24,000 in Recession.

CE-C-7414



Note:

Duration - 2 weeks

Reduced Levels shown thus - 216·O

7·8 Velocity at Surface in f.p.s

5·1 Velocity at Bed in f.p.s

O 10 20 30 40 50 feet

SCALE

N

Fig. 49: Scour 10,000 in Recession.

CE-C-7415

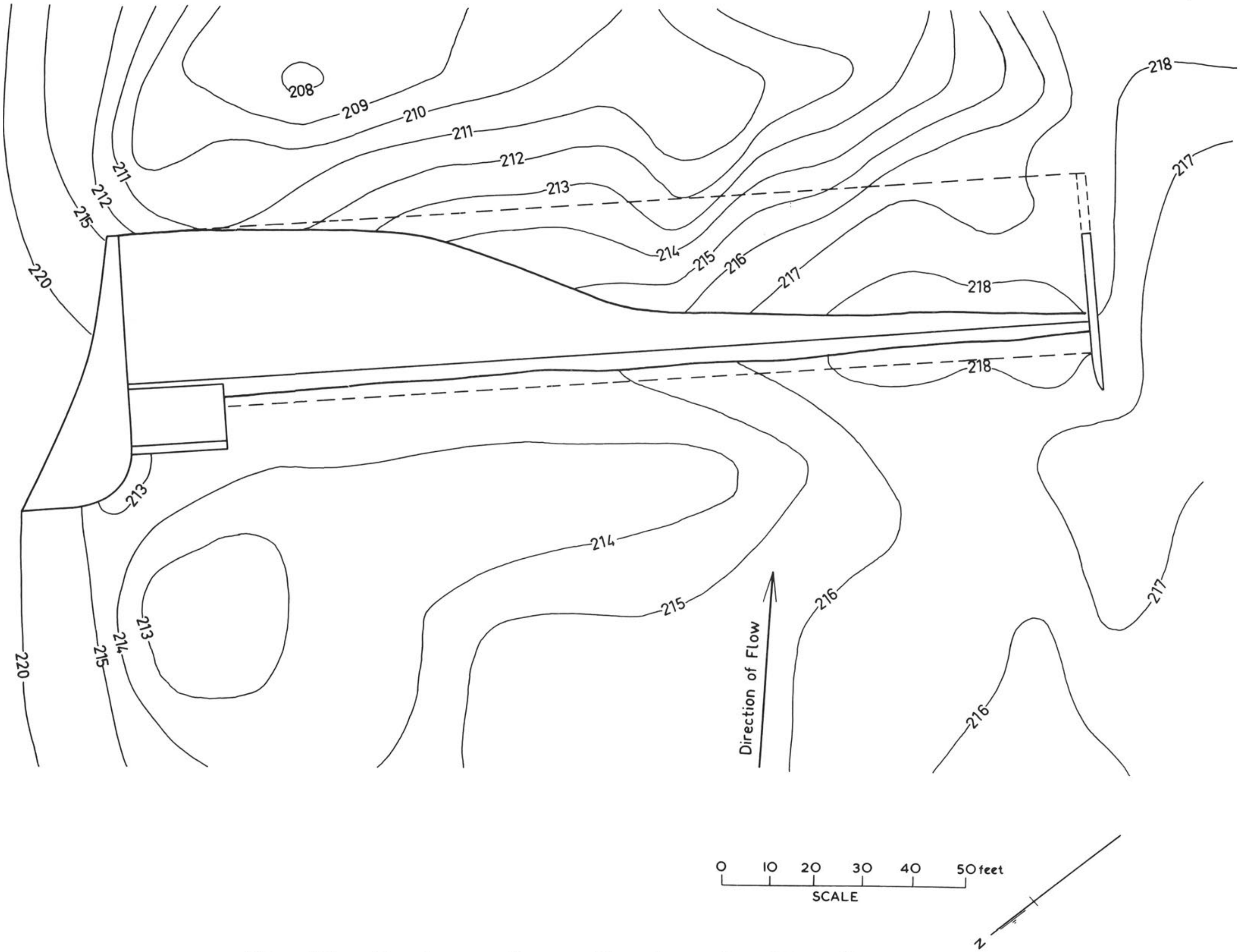


Fig. 50: Maximum Scour Envelope for 200' Structure.

CE-C-7416

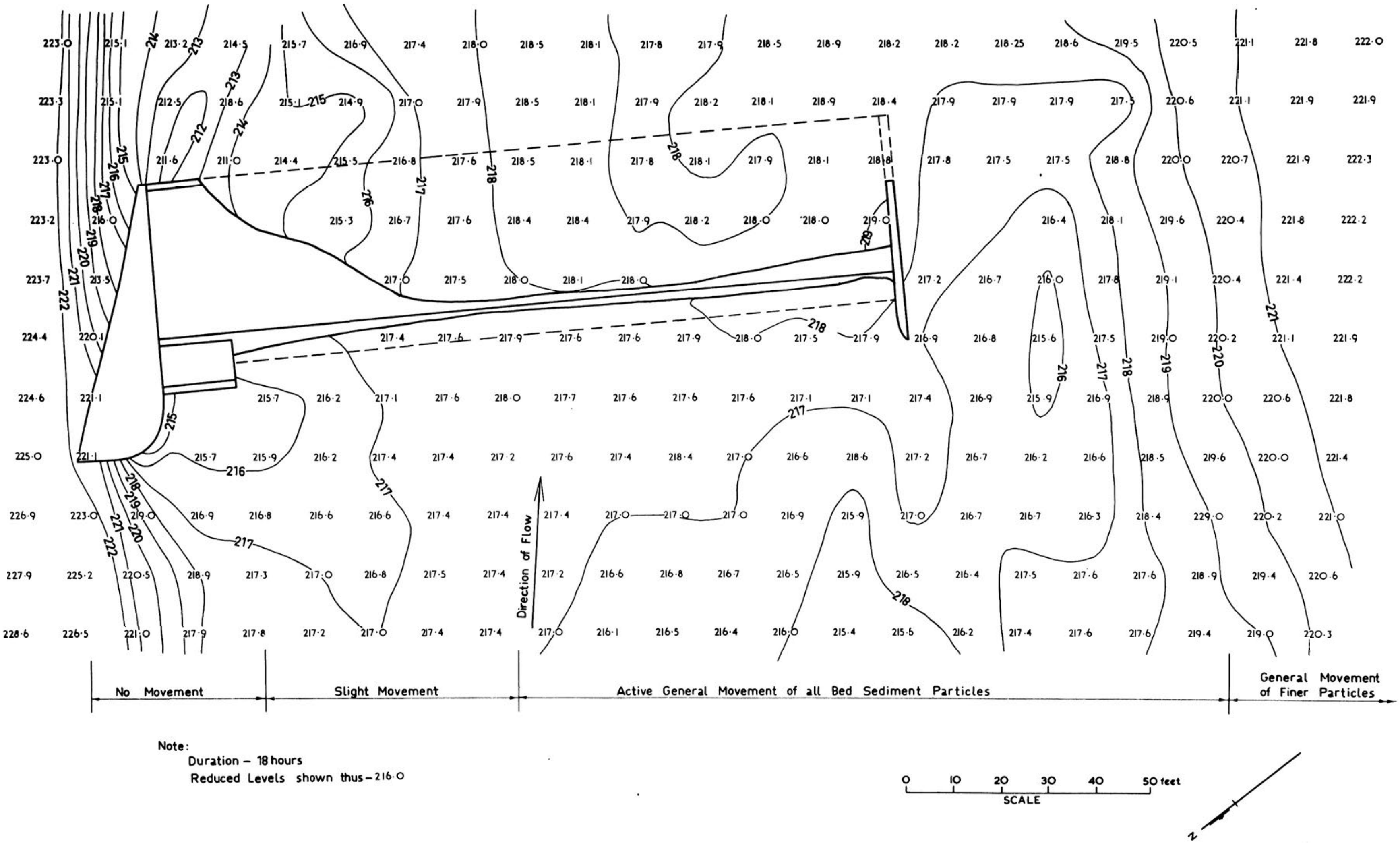
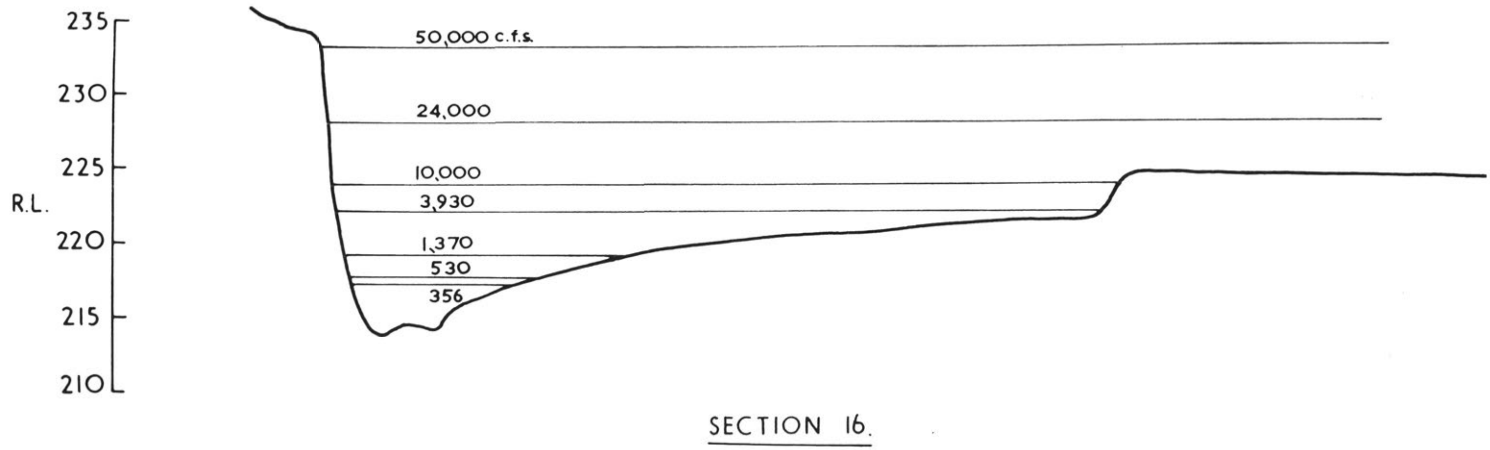
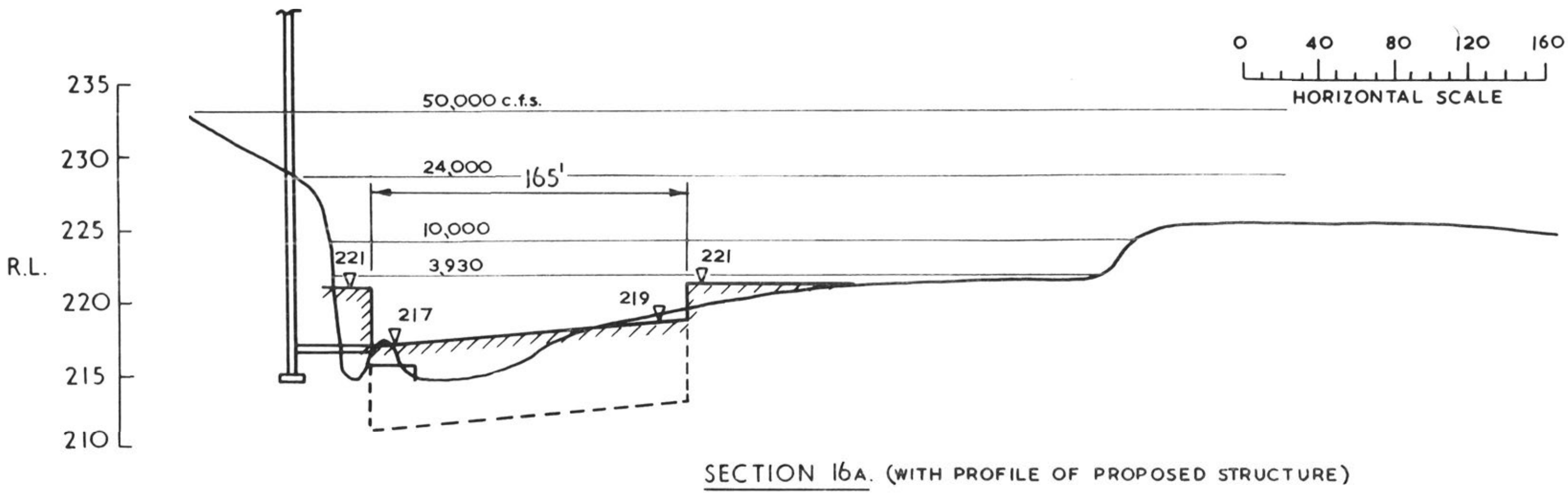


Fig. 51: Scour 37,400 c.f.s. 150' Structure.

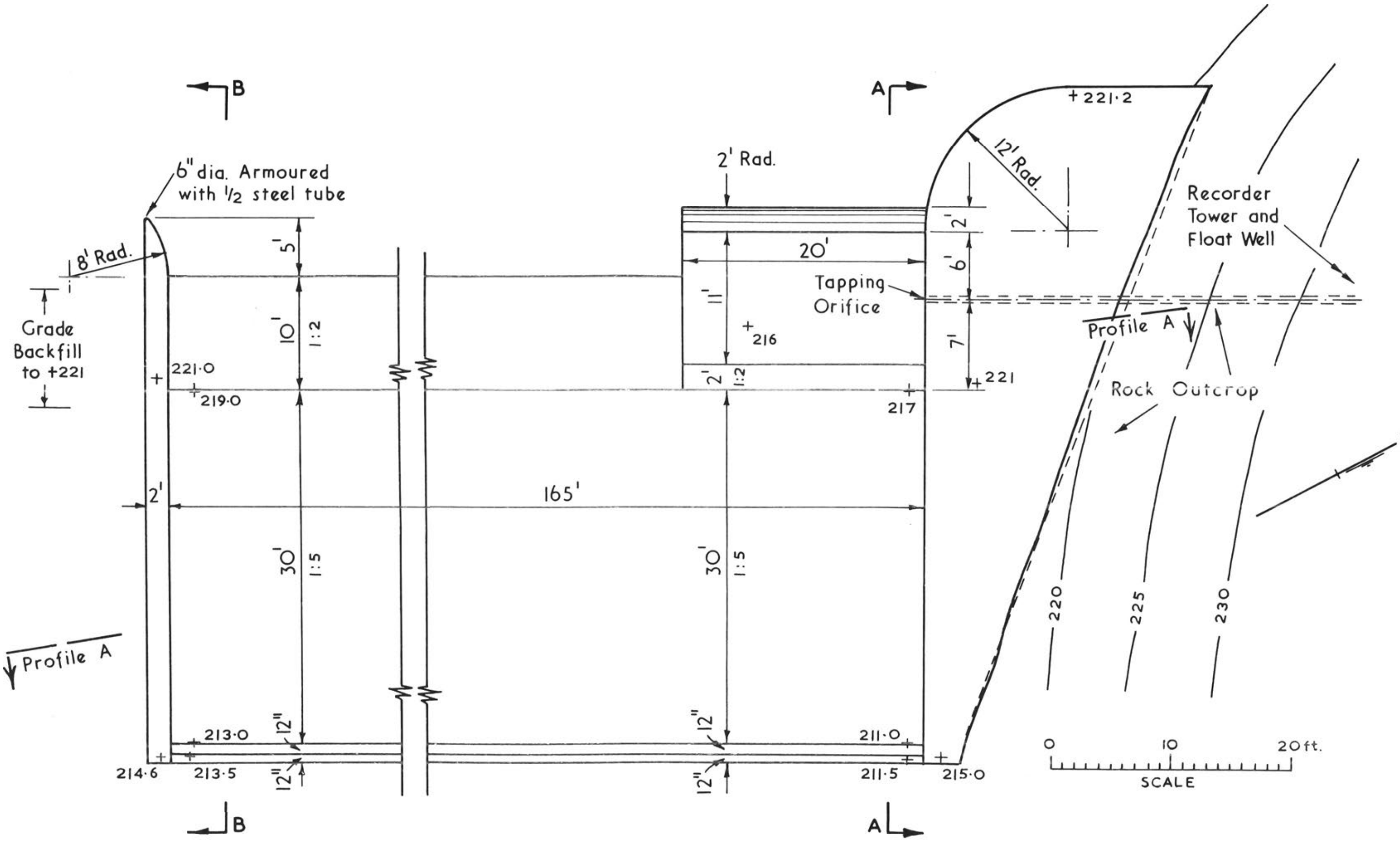
CE-C-7417





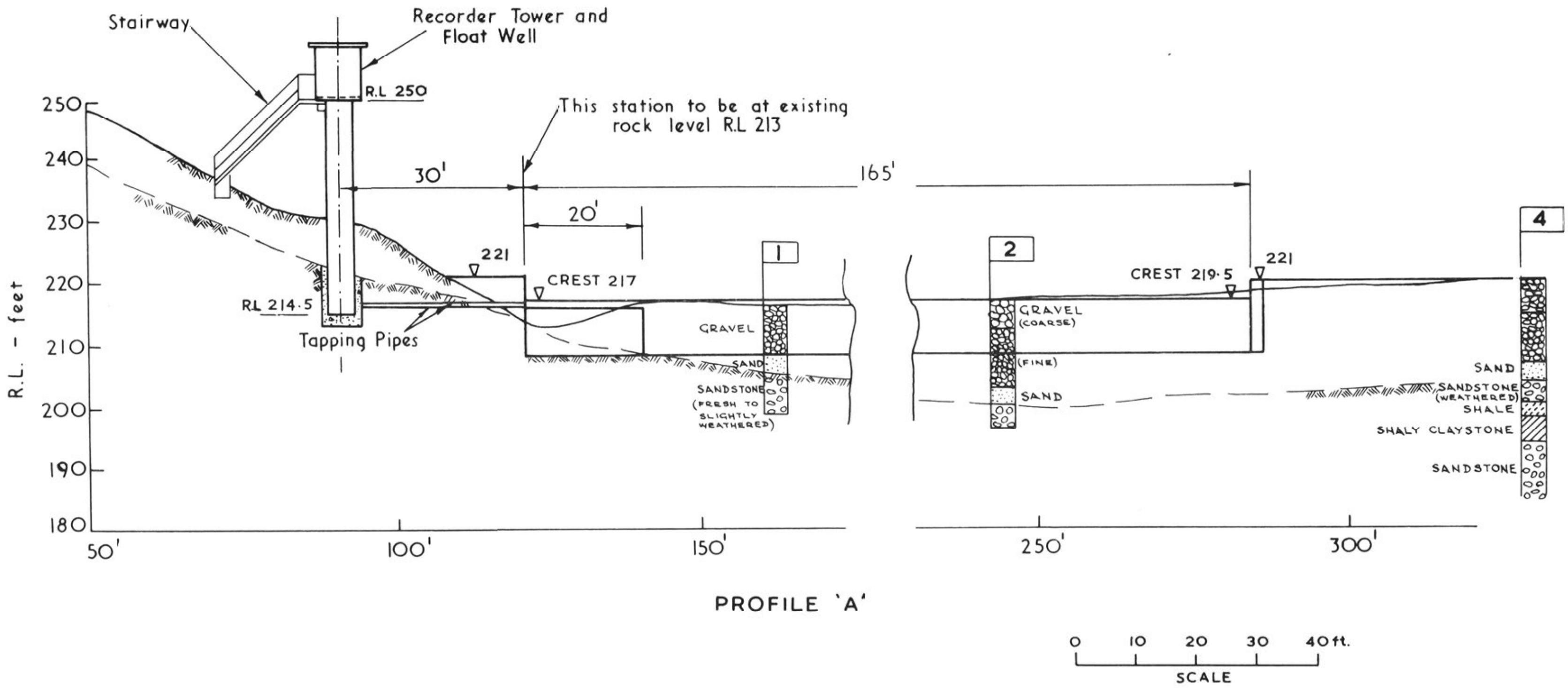
CE-E-7397

Fig. 52: River Cross Section 16 and 16a.



CE-E-7418

Fig. 53: Plan of Proposed Weir Structure.



CE-E-7400

Fig. 54: Profile 'A' Crump Weir.

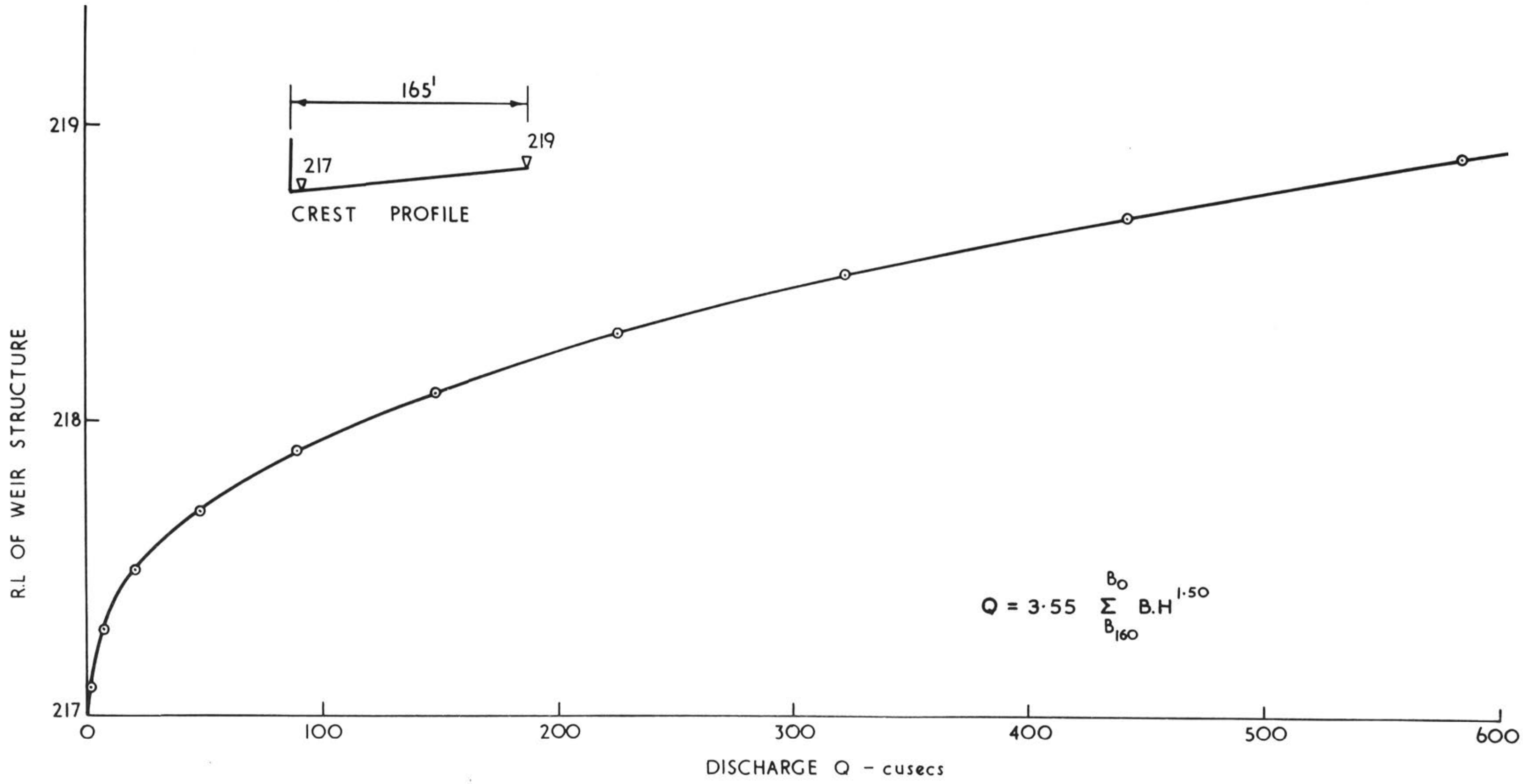


Fig. 55: Theoretical Rating Curve - 165' Crump Weir.

CE-D-7427

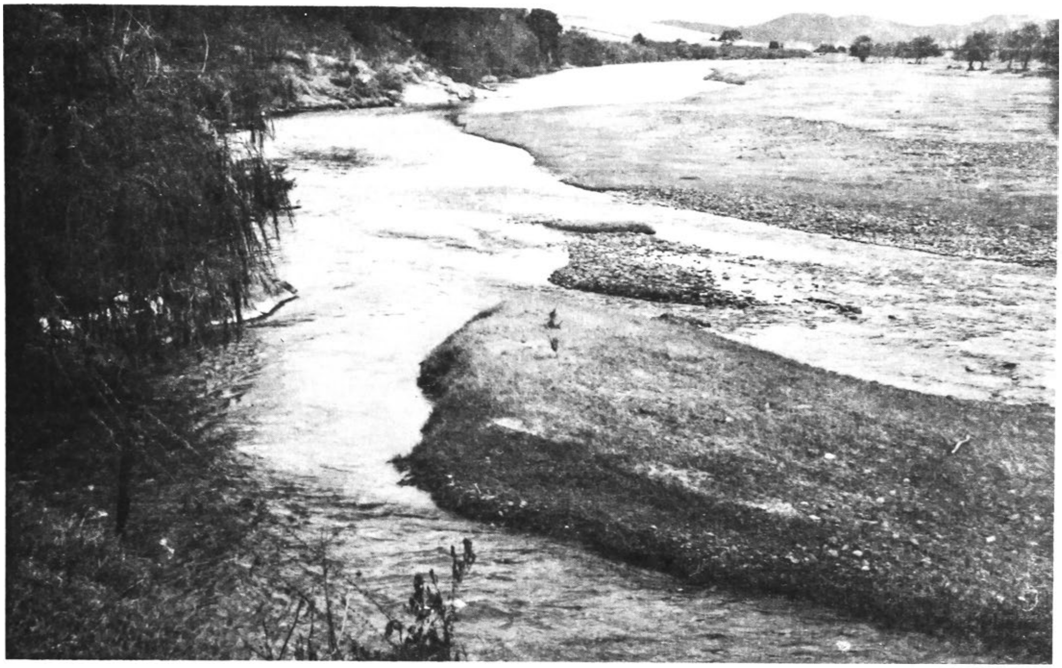


Photo 1: Weir Site from Upstream
169 c. f. s.

Photo 2: Weir Site from Downstream
169 c. f. s.

Photo 3: Weir site from Upstream
3930 c.f.s.

Photo 4: Weir Site from Downstream
3930 c.f.s.

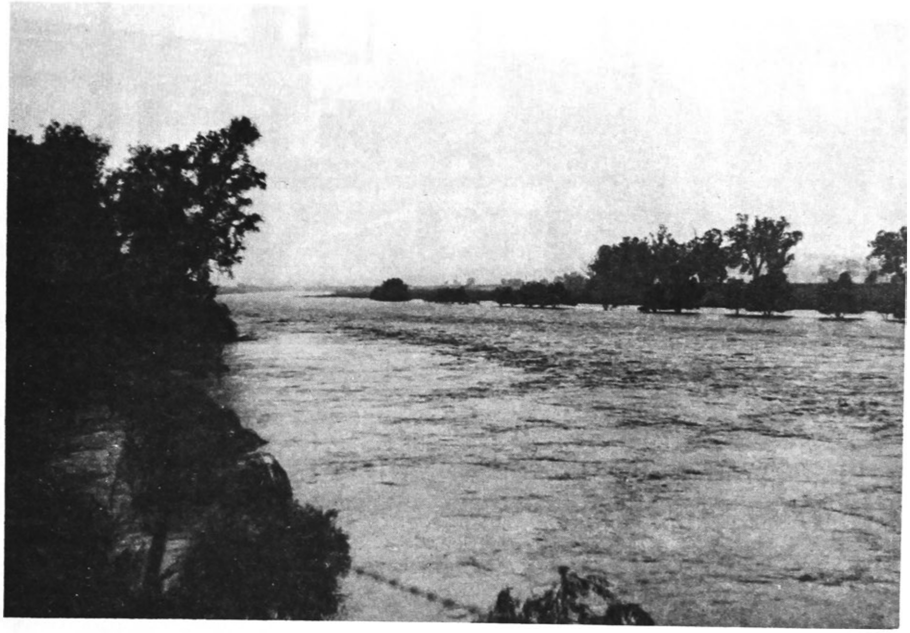


Photo 5: Weir Site from Upstream
24000 c. f. s.

Photo 6: Weir Site from Downstream
24000 c. f. s.

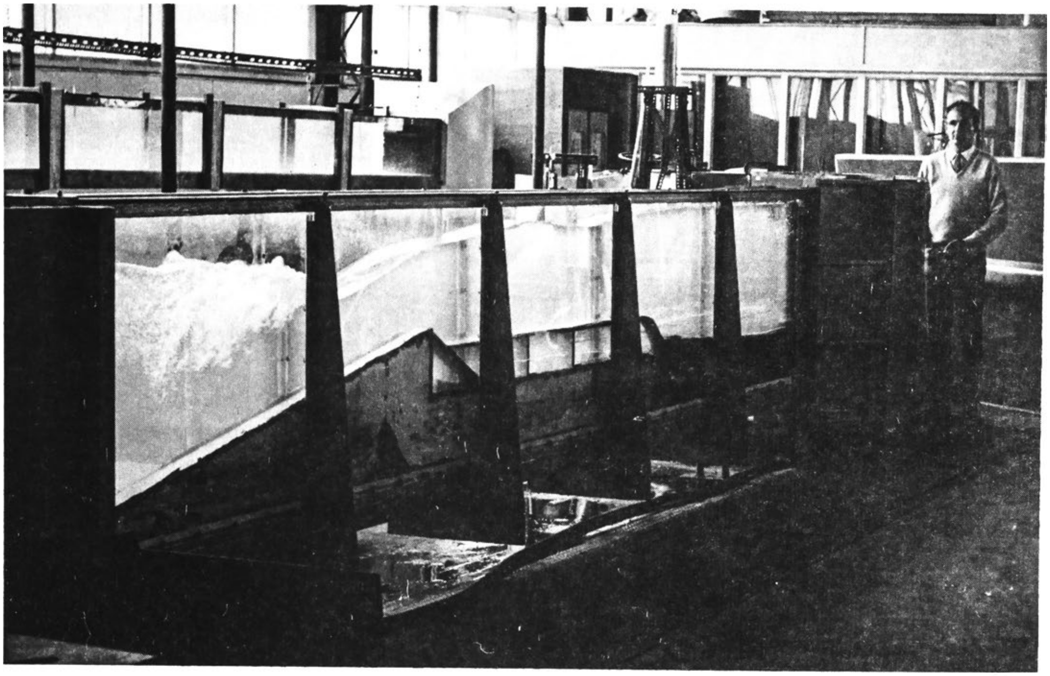


Photo 7: Two Dimensional Flume

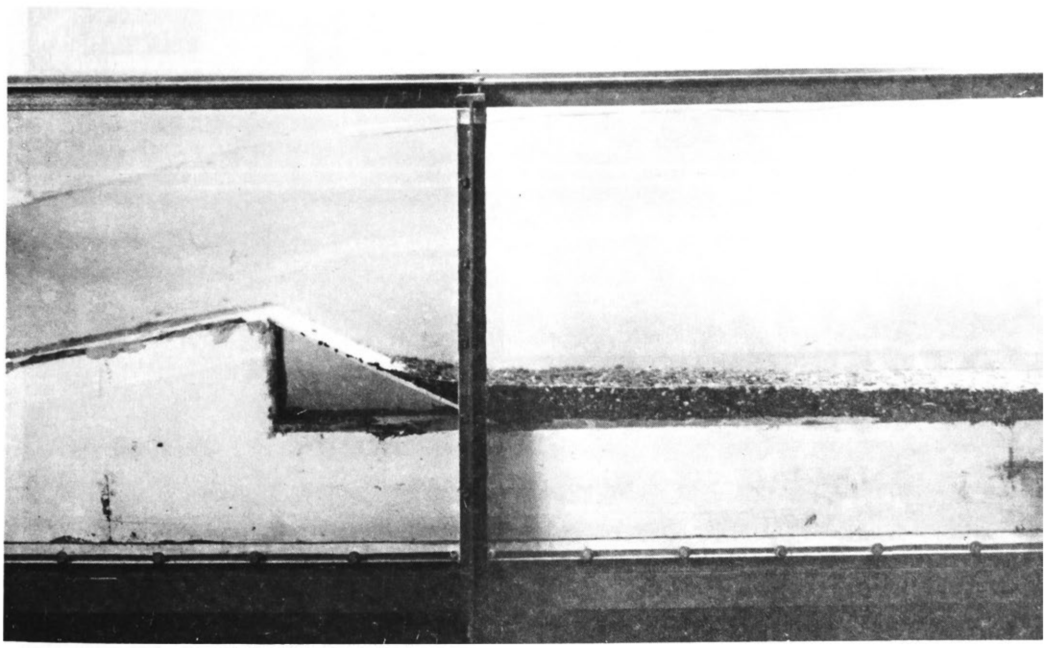


Photo 8: Two Dimensional Test
3/8" Gravel and Low Apron.

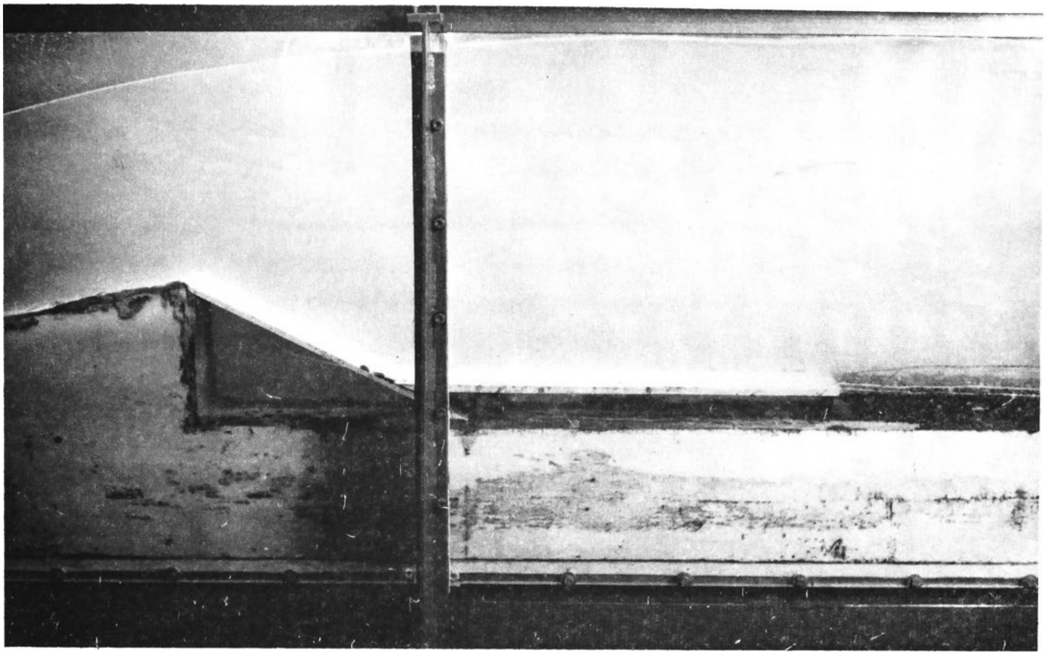


Photo 9: Two Dimensional Test
3/8" Gravel and Mid Apron.

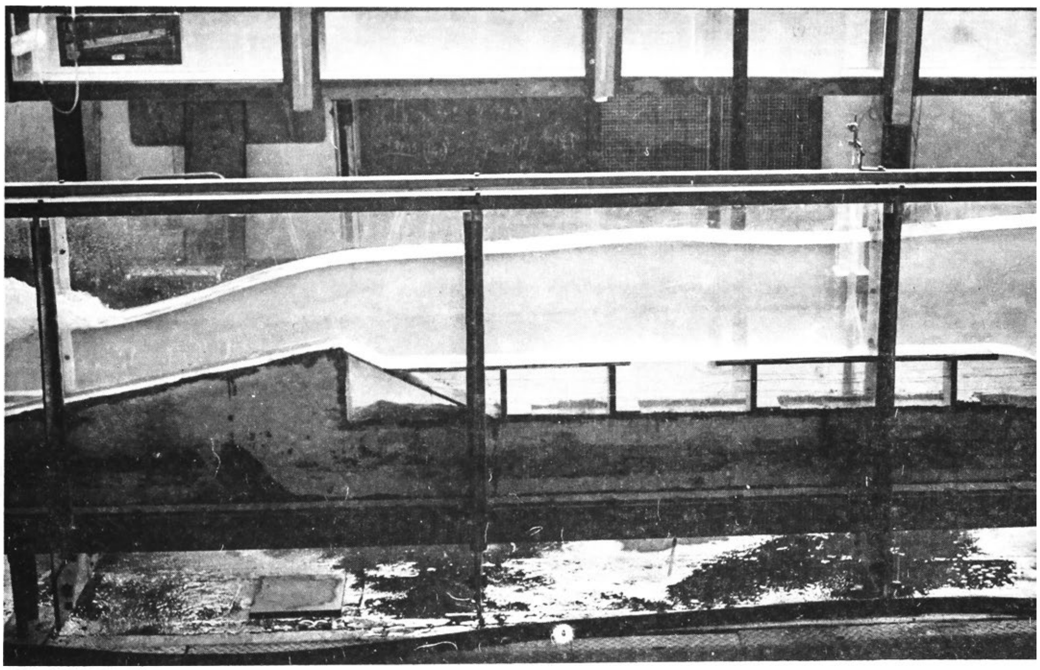


Photo 10: Two Dimensional Test
High Apron with Standing Waves.

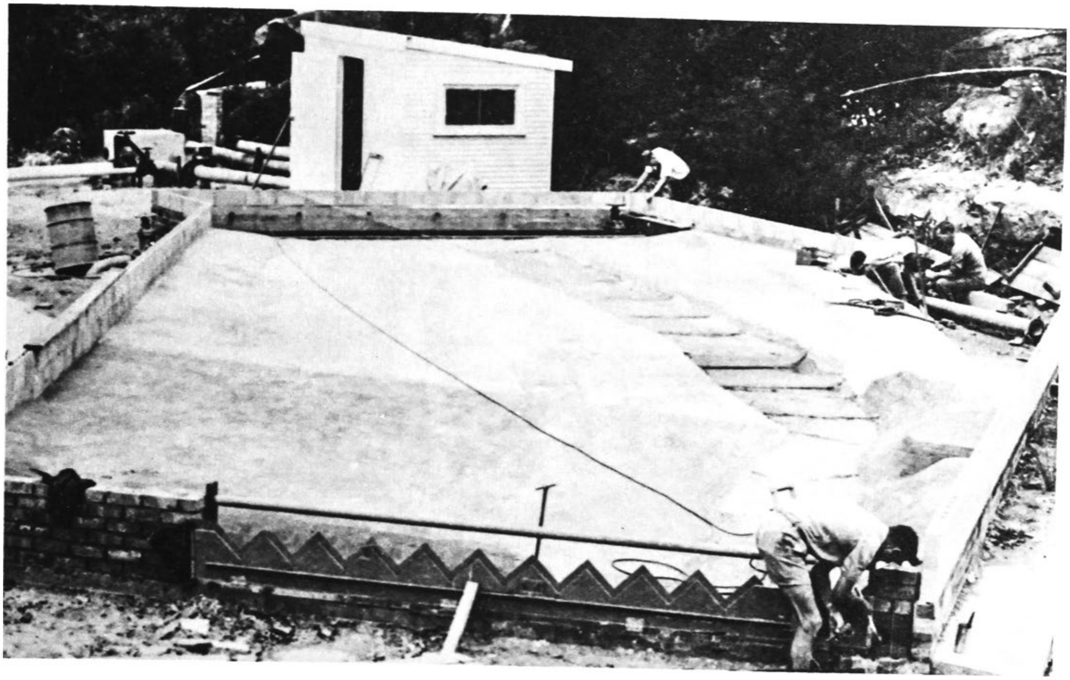


Photo 11: Construction of Three Dimensional Model.

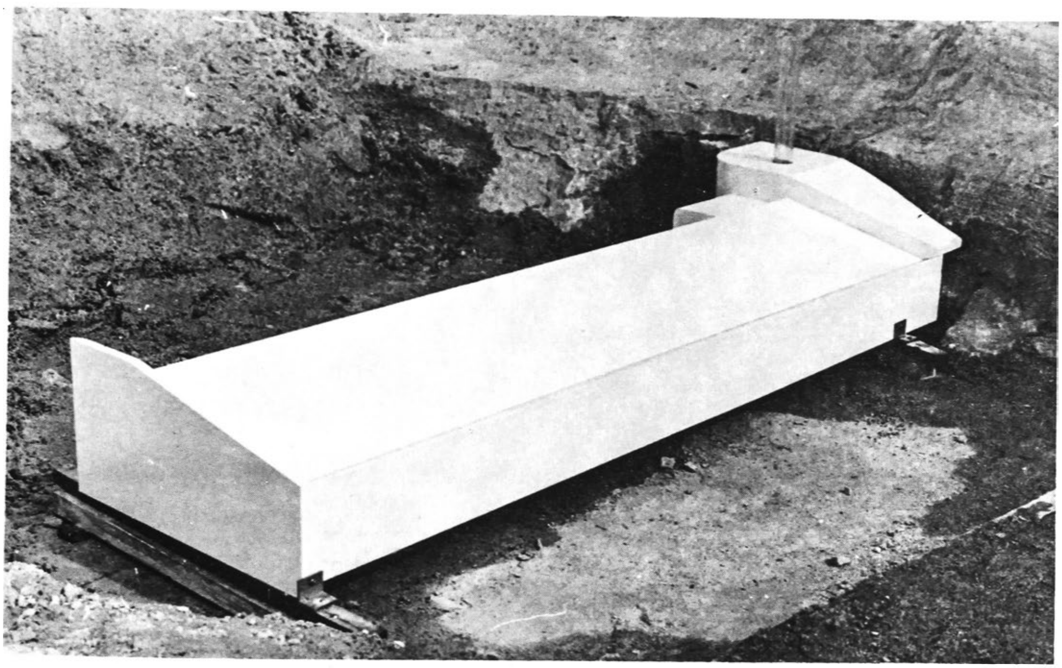


Photo 12: Model of Weir for Three Dimensional Tests.

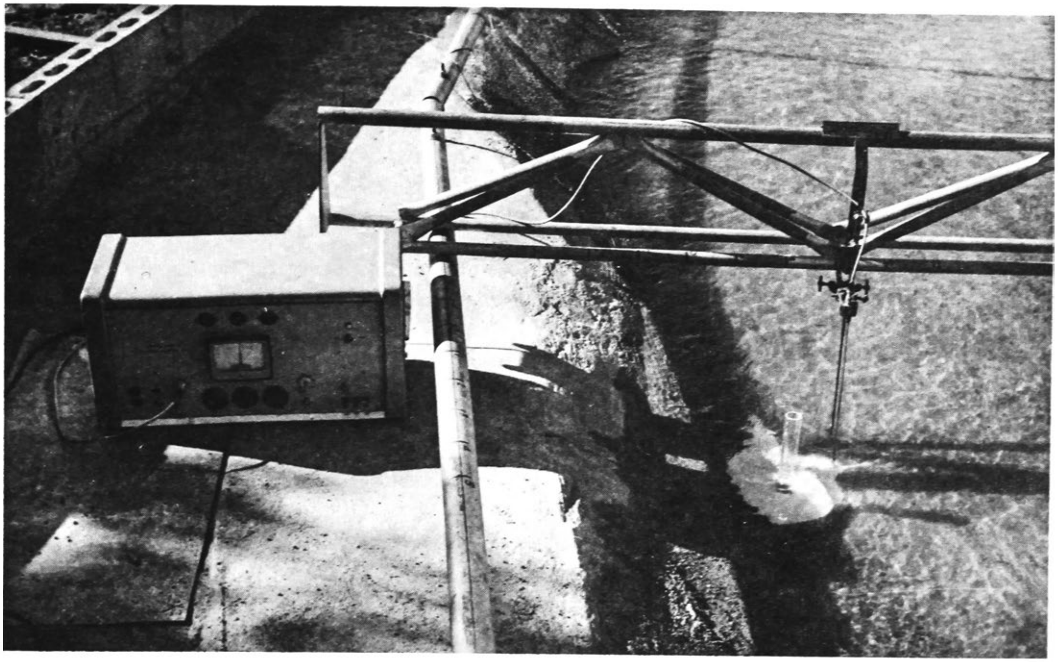


Photo 13: D. S. I. R. Miniflow Meter.

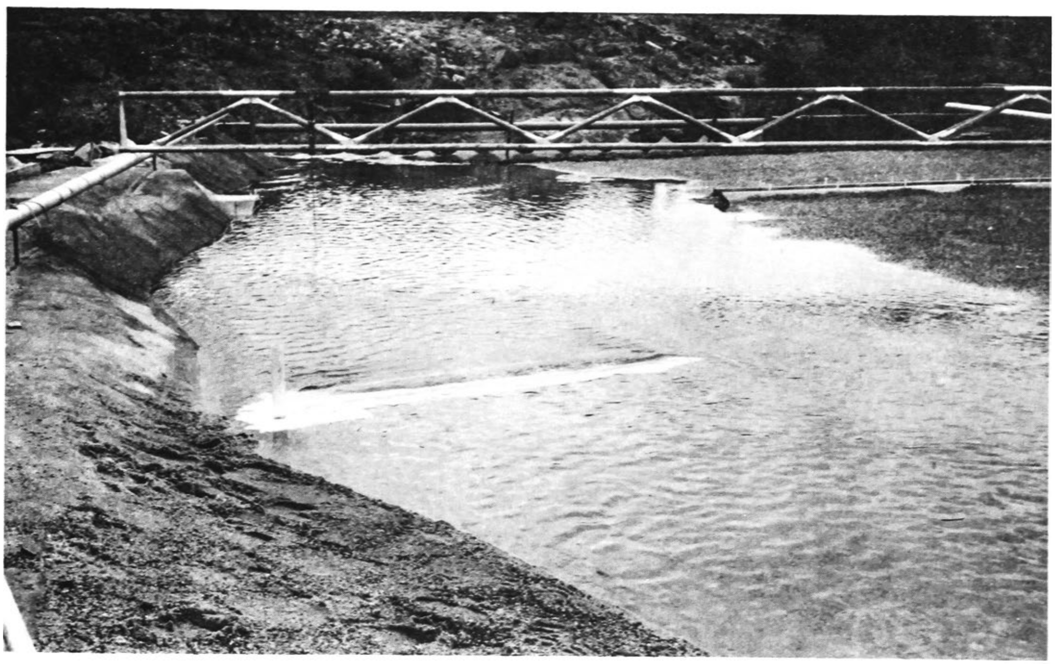


Photo 14: General View of Weir Model.

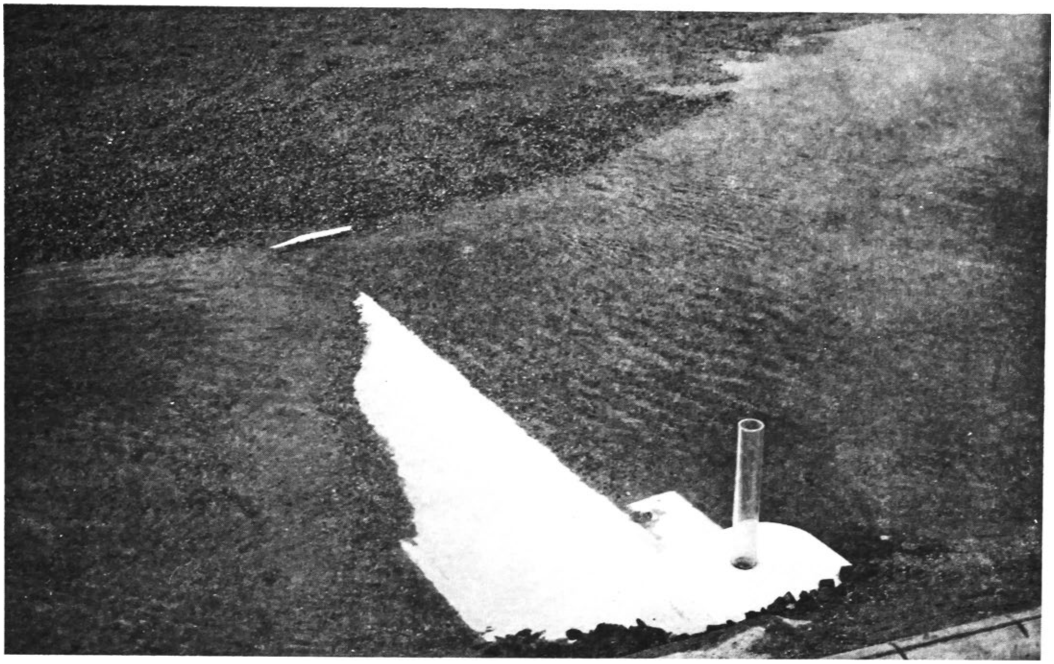


Photo 15: Sediment Test on 200 ft. Weir
3930 c.f.s. Low Tailwater.

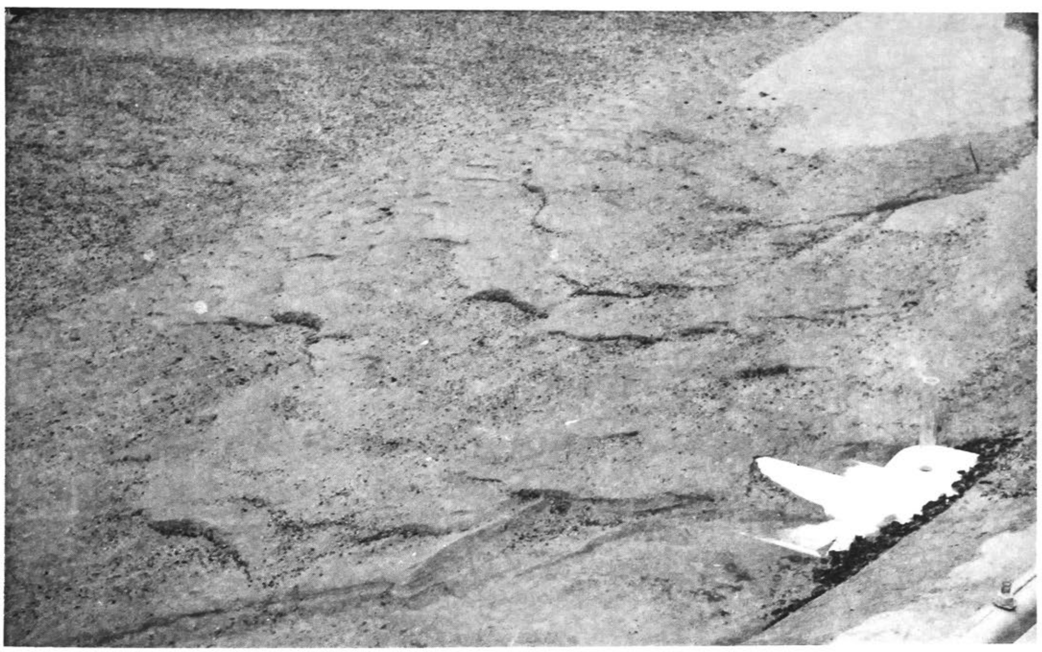


Photo 16: Sediment Test on 200 ft. Weir
showing Sand Slug covering Structure.

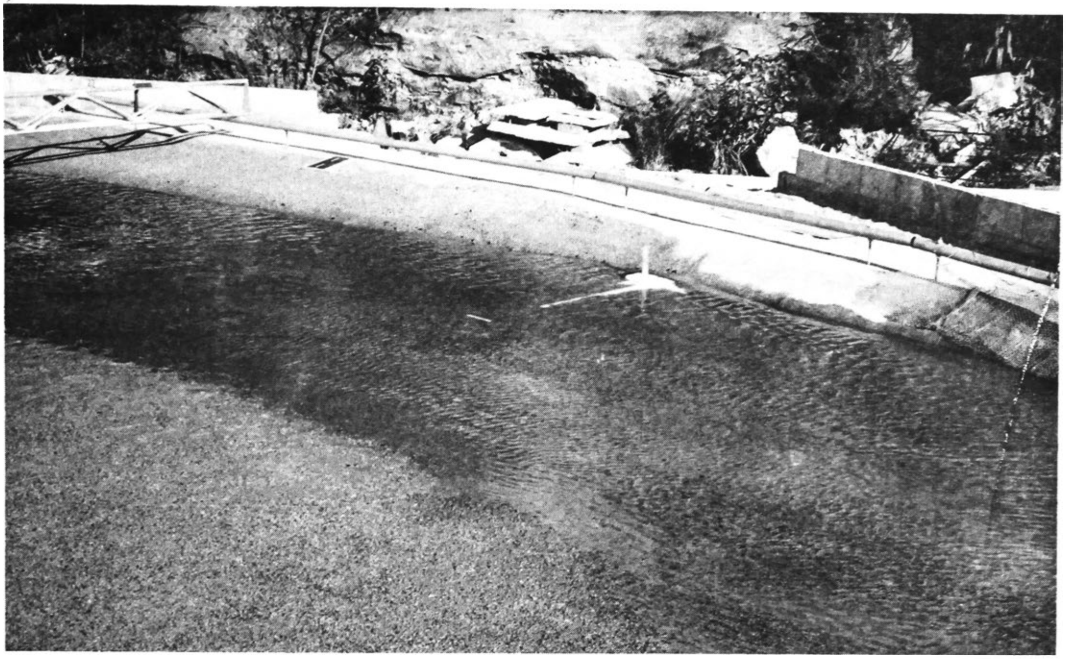


Photo 17: Sediment Test 10,000 c.f.s.

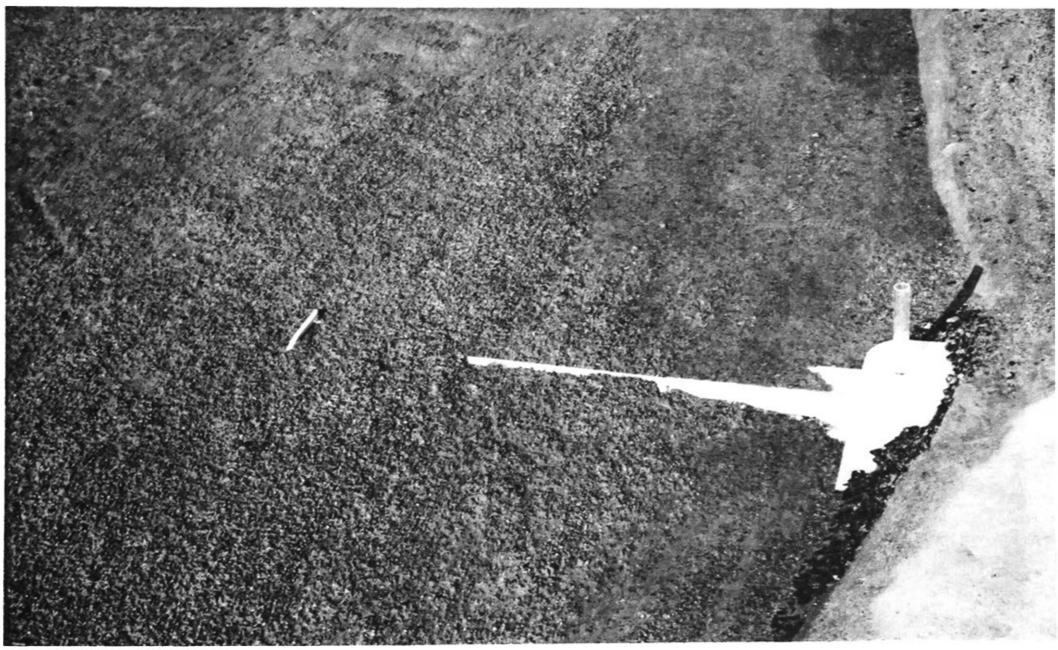


Photo 18: Shoaling after 10,000 c.f.s. for one week.

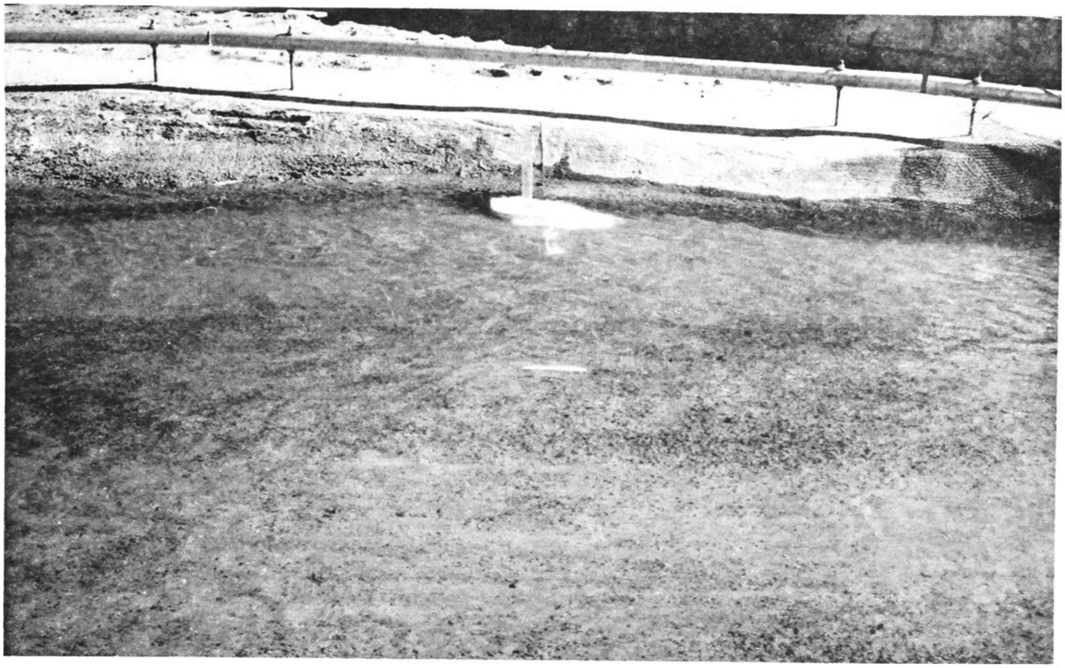


Photo 19: Sediment Test on 200 ft. Weir
24,000 c.f.s.

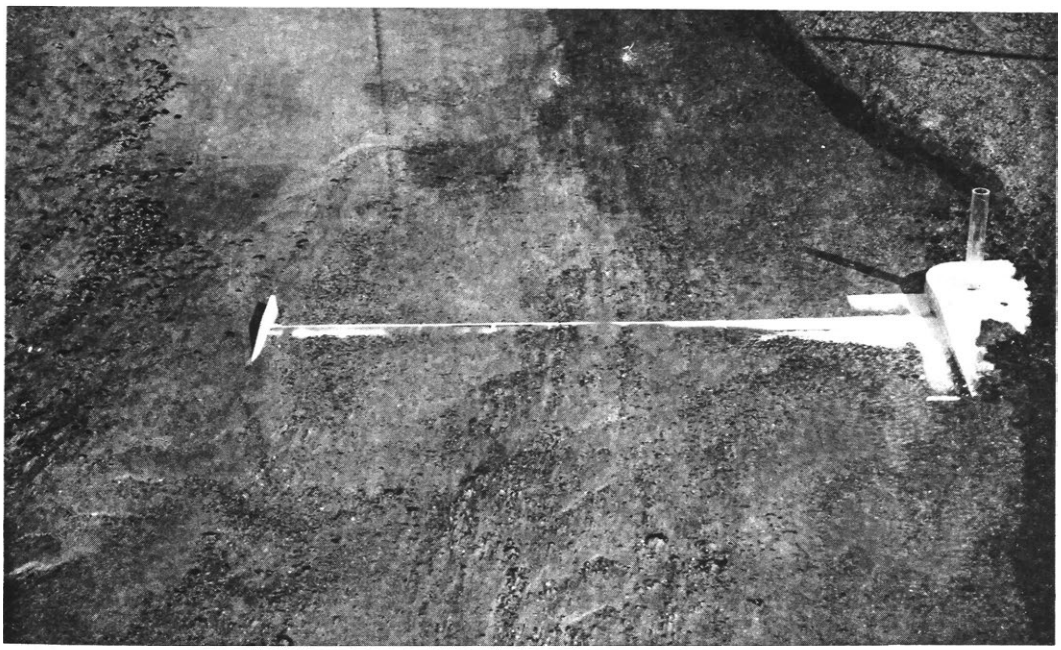


Photo 20: Scour Pattern on 150 ft. Weir
37,500 c.f.s. for 12 hours.