Report

Harza
Engineering Company International

Model Tests on

BURFELL HYDROELECTRIC POWER PROJECT
ICELAND

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RIVER AND HARBOUR RESEARCH LABORATORY AT THE TECHNICAL UNIVERSITY OF NORWAY

A SINTEF associate laboratory

R E P O R T

HARZA ENGINEERING COMPANY INTERNATIONAL

MODEL TESTS ON

BURFELL HYDROELECTRIC POWER PROJECT
ICELAND

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PHOTO 1

MODEL I

PREFACE

This report summarizes a model study of certain problems involved in the Burfell Hydroelectric Power Project in Thjorsá river in Iceland.

The contract was set up between the River and Harbour Research Laboratory at the Technical University of Norway, Trondheim, and Harza Engineering Company International, Chicago, on behalf of Raforkumálastjóri (The State Electricity Authority of Iceland).

The first request to the laboratory was made in a letter from Raforkumlastjóri dated February 17th, 1964. A conference at the laboratory March 19th resulted in a preliminary outline and a summary of the necessary material for a possible study. This matter was further clarified in following discussions.

A contract proposal from the laboratory was finally submitted September 2nd, 1964. With a few amendments, the contract was signed by Harza Engineering Company International, September 21st, 1964.

The construction of the models started October 2nd, 1964, and the main model (model I) was finished January 26th, 1965. A secondary model (model II) had been finished and calibrated earlier to provide necessary data for calibration of model I.

The calibration of model I was completed March 15th, 1965. A third model was constructed during August 1965.

The tests were carried out continuously from March 1965 to April 1966. A few supplementary runs were made also after this period.

The contact between the client and the laboratory has been very good during the whole study. Dr. Gunnar Sigurdsson stayed at the laboratory as representative for the client in the periods September 27th to October 9th, 1964, and March 15th to April 8th and May 3rd to 8th, 1965. Shorter visits by various representatives for the client have occured March 18th - 19th, 1964, May 3rd - 4th, June 8th, July 5th - 6th, September 20th, October 13th - 15th, 1965, and January 26th - 28th, 1966. Various representatives for the Icelandic Allthing and Gouvernment visited the laboratory May 22nd, June 14th and August 19th, 1965.

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LIST OF SYMBOLS

A = area of pegs exposed to the flow

C_D = drag coefficient

D = depth of flow

d = grain diameter

d_{35} = grain diameter of the 35 per cent finer fraction

Fr = Froude's number

g = acceleration of gravity

I = slope

L = length

N = number of pegs per unit area

n = constant in Manning's formula

Q = total discharge

Q_{BC} = discharge through Bjarnalækur Canal

Q_D = discharge over the dam (Q_{BC} not included)

Q_{ICE} = transport rate of loosely packed ice (or model ice), porosity about 65 per cent

Q_{SE} = discharge through the sand excluder

Q_{ST} = discharge through the diversion canal

Q_{TR} = discharge through the trough

Q_1 = discharge over dam gate No. 1

q = discharge per unit width

q_B = sediment transport rate per unit width

R = hydraulic radius

R' = hydraulic radius with respect to skin friction

Re = Reynolds' number

t = thickness of pegs

v = velocity

v_y = vertical velocity

x = horizontal dimension

y = vertical dimension
γ_s = specific weight of solids
γ_w = specific weight of water
υ = kinematic viscosity

τ = shear stress
τ_o = skin friction
Φ = bed-load transport intensity
ψ = flow intensity

Subscripts:

m = model
p = prototype
r = scale ratio

SUMMARY

1. Introduction

This report summarizes a model study of certain problems involved in the Burfell Hydroelectric Power Project in Iceland.

The contract was set up between the River and Harbour Research Laboratory at the Technical University of Norway, Trondheim, and Harza Engineering Company International, Chicago, on behalf of Raforkumálastjóri (The State Electric Authority of Iceland).

The study has been carried out on three hydraulic models. The construction of two of the models was initiated in September 1964, and was completed in March 1965, while the third model was constructed during August 1965.

The tests have been carried out continuously from March 1965 to May 1966, and the report was completed in July 1966.

2. The project

The river Thjorsá upstream of the project is mainly wide and shallow, with long reaches having slopes just steep enough to prevent complete formation of ice covers during frost periods.

Recorded water discharges range between 72 and 1980 m^3/s with an average of 338 m^3/s. Normal winter discharges range between 100 - 200 m^3/s. The floods are usually of short duration, and may occur in any season, though most frequently in spring and fall seasons.

The ice production during frost periods is extensive. Based on calculations and a few measurements, the slush ice discharge under the most severe conditions has been estimated to about 40 m^3/s, loosely packed.

The floods carry significant amounts of sediments. Data on transport rates and composition of sediments are scarce, mainly based on samples from bank deposits.

The main idea of the project is to divert water from the river Thjorsá into an artificial lake from which the power plant can draw water through a tunnel system, discharging the tailwater into the small river Fossá, a tributary to Thjorsá, see fig. 1 and 2.

The project includes a dam across the river and dikes on both banks in extension of the dam. The diversion canal is provided with an inlet structure with submerged openings in order to prevent ice from entering the canal.

The ice is supposed to be flushed down the river or through the artificial Bjarnalækur Canal, which due to the topography can be given economically a steeper slope than the river itself.

The maximum discharge to the ultimate power station is about 250 m^3/s.

3. The problems

The primary purpose of the model has been to investigate the particular part of the ice problem concerned with movement of zero degree slush ice in the dam area, without consideration of freezing processes.

The accumulated amount of slush ice passing the dam site during a normal winter greatly exceeds the storage capacity behind the dam and in the artificial lake. Hence winter operation of the plant will require means to pass the greater part of the slush ice across the dam.

Ice production frequently coincides with periods of low discharge. With no excess water available, water used to flush the slush ice downstream of the dam means a similar cut in the power production. The necessary flush water should thus be reduced to a minimum.

A secondary purpose of the model has been to investigate deposition of sediments in the backwater of the dam and especially in the inlet area. Besides representing a transport problem in itself, the sediment situation was also implicit in the ice problem, since possible deposits would probably interfere with the ice movement.

4. The models

Since the study of the ice problems was restricted to include the movement of ice particles irrelevant of freezing processes, it was possible to apply ordinary principles for model investigation of sediment movement to the ice as well as the sand transport in the river.

The geometry and the flow conditions were modelled according to Froude's model law.

Three models have been chosen for the study:

Model I:	Horizontal scale ratio:	1:50
	Vertical scale ratio:	1:20
Model II:	Horizontal scale ratio:	1:500
	Vertical scale ratio:	1:50
Model III:	Scale ratio 1:40 (undistorted).	

The greater part of the study has been carried out on model I, which covered the river from the island Klofæy, about 1500 m upstream of the dam, to a section about 350 m downstream of the dam. It included the inlet and dam structures. Besides the water supply system, this model had facilities for feeding of desired amounts of sediments and artificial "ice" into the flow. The "ice" was reproduced by polyethylene shavings of similar bouyancy as natural ice. Model II covered a longer section of the river than model I, beginning about 500 m upstream of Klofæy. This model was used to study the partition of the flow through the two channels along Klofæy. The results were necessary to establish correct in-flow conditions in model I.

Model III was an undistorted version of the inlet and the gated dam section designed for study of the conditions downstream of the dam. It was also used to study flow conditions in front of the inlet, as a check of results from the distorted model I. Model III had no feeding arrangement for ice, but a few ice tests were performed by means of hand feeding.

The use of distorted models was in this case a consequence of the desire to test sediment conditions as well as the need to limit the size of the models while still keeping the water depths great enough to avoid scale effects from viscosity. Model I alone covered more than 600 m^2.

Distorted models are widely used in hydraulic model practice, particularly for movable bed studies. Experience has shown that reliable results can be obtained where the geometry of the models is not too complicated. In this study, check runs on the undistorted model III showed that even the conditions in the inlet area were reproduced to satisfaction or could be deduced from results in model I.

The basic concept for the study of ice movement was the assumption that ice particles in movement can be considered as sediments with negative submerged density. Well known methods for study of sediment movement were modified to fit the ice, and throroughly checked in advance under known prototype conditions before they were applied to the study of future conditions in model I.

A study of this kind will primarily give qualitative results. Where quantitative conditions were important, hence, care has been taken to keep the results somewhat on the safe side by applying unfavourable conditions or by extending the range of the study beyond the assumed limits.

Ice conditions, for instance, have been checked for discharges up to 55 m^3/s, well in excess of the expected maximum of 40 m^3/s, to adjust for possible deviations from the theoretical model conditions.

5. The study

The original design has been altered step by step as a consequence of the testing of numerous modifications suggested in close cooperation with the Consulting Engineers.

After a few introductory sand runs, the emphasis was put on the establishment of good ice passing conditions. The study was consequently for several months mainly concerned with the conditions in the corner area between the dam and the intake.

The original design had a tendency towards piling up ice in front of the inlet wall. The ice float thus forming gradually increased in thickness until its lower surface came flush to the upper edge of the inlet ports, whereupon ice intrusion into the diversion canal became significant.

The amount of flush water required to avoid this ice float formation was in excess of the assumptions upon which the project was based. Several attempts to improve the conditions by simple means failed.

In order to pass under the ice float and the inlet wall the flowing water has to change its direction from nearly horizontal to some angle with the horizontal. This vertical component of the flow and the vertical motion due to turbulence are responsible for transporting the ice downwards and into the diversion canal.

The model showed that if the trench in front of the inlet was widened and given a milder slope, the ice intrusion would decrease. An excavation which did not give excessive vertical velocities and also allowed already suspended ice a chance to surface in front of the ice wall, was designed in combination with a shorter inlet, as shown on fig. 18.

A definite improvement occurred upon installation of a jetty, as shown on fig. 19, directing the flow more parallel to the inlet wall. The situation was now considered acceptable, but the tests were continued in order to improve it further.

Finally, arrangement of a trough-like canal along the inlet wall proved effective to skim off the ice from the top layer of the water and carry it downstream with a minimum of water. This solution even gave good results without the jetty, but the jetty was still beneficial.

The ice runs were followed by sand runs, giving the tendency of the deposition during flood periods and the tendency of channel development during subsequent periods of low discharge. To the extent the sand could be kept from clogging the inlet area itself, the deposits seemed rather irrelevant to the efficiency of the plant. It is even possible that the deposits may mean a benefit to certain ice conditions, narrowing the watercourse during low flows.

Application of groins from the river banks proved to be a useful means to control the deposition pattern to some extent, if so desired.

The tests showed that a relatively great part of the sediments will be carried to the inlet region and tend to deposit in the trench along the inlet. If these deposits are not being removed, the velocities through the inlet will gradually increase and cause more and more of the sand to deposit in the diversion canal.

The bottom sluices of the original design proved insufficient to remove the sand from the trench. An apparently effective sand excluder system was therefore worked out and tested in the model.

The flow conditions in the inlet area and downstream of the dam were finally checked in detail in model III.

6. Synopsis of results

a. The model study has resulted in a design of the dam and inlet area which is able to by-pass the expected quantities of drifting zero degree slush ice in the river.

The final design involves alternative ice passing procedures, and a number of measures to attack possible ice jams.

The major features of the original design have been kept, but its performance has been greatly improved by the addition of 1) a jetty parallel to the inlet, 2) larger excavation in front of the inlet, 3) the trough in front of the inlet, 4) the runaway channel downstream of the dam, and 5) the sand excluder. The design of the inlet structure and the diversion canal has been modified.

b. Under conditions in the prototype corresponding to the model conditions, the necessary total flush discharge will not exceed 100 m^3/s for ice discharges up to 55 m^3/s (loosely packed, solid volume 20 m^3/s) and about 70 m^3/s for ice up to 28 m^3/s. Further decrease in the ice discharge will not lead to a proportional decrease in the flush discharge,since a certain water depth is required along the overflow crest to prevent ice from sticking to the crest, and a certain velocity is necessary to carry the ice towards the crest. To some degree the limits will depend on the varying properties of the drifting ice. Some additional flush water will be necessary if the trough is out of operation and the ice must be passed over the dam gates. Conditions for small ice discharges have been briefly studied. A safe lower limit of about 40 m^3/s of flush water can be indicated when relatively small though

significant ice discharges appear. (Careful operation may allow for reduction of the flush water). The need of flush water increases with increasing discharge to the station.

c. The deposition of sediments may require some control and maintenance operations, but represents no great problem to the operation of the plant. The sand excluder is effective, but requires up to 100 m^3/s of flush water for full operation. The sand load is considered small under low discharges, however. The sand excluder possesses some ability to clear itself if clogged by deposits when not in operation.

d. A draft of the basic operational guidelines has been worked out in the model. It is likely that revision of several details in the scheme will result from prototype experience.

1. INTRODUCTION

1.1. A hydraulic study of ice conditions

The ice situation in a river being observed at a certain moment is the result of many factors. Among them are the meteorological and hydraulic conditions by far the most important, but the picture can have been influenced also by other factors,for instance of geological or chemical nature.

The formation of ice means that thermal processes are active. The model laws governing the studies of hydraulic problems on a scale model can, however, not be extended to comprise also the thermal processes. Th a hydraulic model study must concentrate on the hydraulic scope of the problem alone. The effect of the thermal processes must be considered separately and if necessary and possible superposed upon the model results.

Even if thermal processes could be reproduced or simulated in a model study, it would be an overwhelming task to incorporate the many meteorological variables in the study, and all would have to be introduced as statistical variables. Hence, only qualitative results can be obtained even from the most comprehensive study.

It is of particular importance that the structural design resulting from a hydraulic study will represent the design of the optimal efficiency also when the influence of other factors are involved. The thermal processes may reduce the efficiency found by the tests or complicate the operation of the plant, but by experience and thorough considerations of all the pertinent factors we are convinced that a study including all these factors would not have resulted in a basically different and better solution.

While the qualitative flow data obtained from a separate hydraulic study must be used with due regard to possible modifying factors, hence, the study gives fully satisfactory solutions to structural questions, provided the hydraulic conditions adapted for the model study cover the full range of possible prototype conditions. In this respect the type and amount of moving ice particles are also regarded as part of the hydraulic conditions.

For the Burfell project future ice problems will depend almost entirely on the hydraulic conditions. This can be proved by the fact that ice jams never have been observed in the part of the river where the dam will be placed. The best possible hydraulic conditions can only be obtained through model studies. But even the best possible hydraulic design means an alteration of the original hydraulic conditions. The thermal processes are therefore to be expected to enter into the play and cause troubles, but only the future can tell how and to what extent.

1.2. Local conditions

The river Thjorsá originates south of Hofsjökull and flows mainly in a south-westerly direction, passing north-west of the volcanic mountain Hekla and finally discharging into the North Atlantic Ocean. The character of the river changes several times along its course, from gently sloping reaches to rapids and waterfalls. It has several tributaries, of which the Tungnaá- Kaldakvisl system is particularly important, originating from the Vatnajökull area.

Reference is made to the map, fig. 1.

Of particular concern to this study is the 17 km long reach between the confluence of Thjorsá and Tungnaá about 98 km from the sea and the waterfall Trollkonuhlaup. The average slope of this reach is 0.0037,and the river is wide and shallow. The upper eight kilometres of the reach have a milder slope than the rest, and a section from 3 to 5 km below the confluence is braided, with large banks of sand an gravel. Downstream of Klofæy, an island about 9 km below the confluence, the river constitutes a 4 km long, nearly uniform channel with almost rectangular 300 to 340 m wide cross sections and slope about 0.0035. Further down to Trollkonuhlaup the slope gradually increases.

The river bottom below Klofæy consists of head-size boulders resting on lava bed, while the banks are made up of deposits, mainly lapilli, a volcanic sand.

Along Thjorsá just east of Burfell, and between the river and Burfell, runs a small tributary, the Bjarnalækur Creek, nearly parallel to the river in a distance up to 1.2 km,separated from the river by a low ridge.The creek has a lower elevation than Thjorsá itself all the way down to the confluence.

A similar difference in level exists between Thjorsá and the river Ytri-Rangá just east of Thjorsá in the same region. Thus just upstream of Trollkonuhlaup Thjorsá runs along an about 2 km wide ridge between two other watercourses of lower elevation.

Thjorsá is mainly a glacier fed and direct run-off river, but spring water is of same importance. The spring water originates mainly from the Tungnaá tributary where it plays a major role.

Discharges measured at Urridafoss, 22 km from the sea, in the years 1947 to 1962, range between 81 and 2230 m^3/s, with a mean value of 380 m^3/s.Usually the peak floods appear in the period between February and May, while the rest of the year shows a gradual decrease in the mean discharge from 5 - 600 m^3/s in the early Summer season to about 200 m^3/s just before the flood period begins. There are many large deviations from this scheme, however. Discharge variations of the magnitude of 2 - 400 m^3/s within few days are very common.

Discharge data for the dam site have been computed based on assumed proportionality between drainage area and run-off:

Minimum:	72 m^3/s
Maximum:	1980 m^3/s
Average:	338 m^3/s

Computation of a probable maximum storm flood from available meteorological and hydrological data resulted in a design value of 7750 m^3/s.

The river carries large amounts of ice during frost periods, mainly frazil ice and moderate ice clusters. Drifting ice and low water frequently coincide, and ice discharge is rare when the total discharge is 400 m^3/s or more.

In addition to the drifting ice, bottom ice may cover great parts of the river bed in certain periods, the thickness being reported to rarely exceed 0.5 m. Local ice islands sometimes grow up from the bottom, however.

Ice bridges or jams are rare in the reach between Klofæy and Trollkonuhlaup. Upstream of Klofæy, more severe jams may occur, these are usually the direct cause of the recorded minimum discharges.

Narrow strips of thick bank ice use to form along the river during winter, covering only a small fraction of the width of the river.

The ice discharge varies greatly, and may reach 40 m^3/s or more (loosely packed) under severe conditions. Usually freezing and thawing occur several times during one winter.

The river carries a significant load of suspended material as well as bed-load during flood periods.

The amount and composition of the bed-load are not known. The composition has been estimated from deposits along the banks, giving a mean diameter of the bed-load about 5 mm.

Nine samples taken at Thjorsá bridge with a"depth-integrating sampler" at discharges varying between 207 and 588 m^3/s have given concentrations of suspended solids ranging between 102 and 453 ml/l as an average across the river.

The region is quite barren, and strong winds are prevailing. A continuous snow cover is rare in the winter, and the wind carries a significant load of sand particles as well as snow in the cold season.

1.3. Description of the project

The main idea of the power project is to divert water from the river Thjorsá into an artificial lake, called Bjarnalækur Pond, situated north of the Burfell mountain. From this pond a horizontal tunnel leading to vertical penstocks conducts the water to the power station,finally discharging into Fossá,a small tributary of Thjorsá. The difference between the normal headwater and tailwater levels is 118.5 m, and maximum discharge through the power plant is 250 m^3/s, ultimately.

Possible intermediate construction stages for the project have been considered.

Of concern to the model study were only the diversion structures, consisting of the following parts, see fig. 2 and 3:

a. A dam across the river, partly supplied with control gates, partly consisting of an overflow section supplied with flashboards.

b. Dikes extending from both ends of the dam to limit flood inundation and prevent flood waters from being diverted to neighbouring Ytri-Rangá river.

c. An inlet structure with submerged ports to skim surface ice from the diverted water.

d. An excavation to increase the depth in front of the inlet.

e. A canal leading from the western end of the dam to Bjarnalækur Creek. The canal is about 2 km long and supplied with a sluice structure in the upstream end with flap gates for surface water and ice, as well as bottom sluices for supply of flush water to the canal. The chief purpose of the canal was to conduct slush ice with a minimum of water, but also to flush away sand deposited in front of the inlet.

f. The diversion canal leading from the inlet to Bjarnalækur Pond.

1.4. The problems

1.4.1. Ice transport

The primary purpose of the model has been to investigate the particular part of the ice problem concerned with movement of zero degree slush ice in the dam area, without consideration of freezing processes.

The proposed structures will obstruct the ordinary flow of great amounts of drifting ice in the river frequently occuring during the winter season. The accumulated amount of slush ice passing the dam site during a normal winter greatly exceeds the storage capacity behind the dam and in the artificial lake. Hence, winter operation of the plant will require means to pass the greater part of the slush ice across the dam.

Ice production frequently coincides with periods of low discharge. With no excess water available, water used to flush the slush ice downstream of the dam means a similar cut in the power production. The necessary flush water should thus be reduced to a minimum.

1.4.2. The sediments

A secondary problem has been the sediment load in the river during flood periods. Though little is known about the rate of transport, it is evident that the dam will cause deposition in the backwater area until finally a more or less stable transport situation occurs.

Besides representing a transport problem in itself, the sediment situation is also implicit in the ice problem, since possible deposits are likely to interfere with the ice movement.

The model study should include possible means to control the deposition and improve the transport conditions.

1.4.3. Other problems

Flow conditions have been studied for several structural details not pertinent to the transport of ice and sediments.

The minimum structures required to divert sufficient water during intermediate construction stages have also been studied.

2. THE MODELS

2.1. Introduction

The problems to be studied were all concentrated in the vicinity of the dam site. Hence, the necessary reach to be covered by models was the dam and inlet area and the area immediately downstream of the dam, as well as the reach upstream of the dam so far as the flow conditions could be supposed to have any influence on the conditions in the backwater area.

The flow around the island Klofæy, about 1500 m upstream of the dam site, was supposed to have influence at least on the deposition of sediments. As very little was known about the flow conditions around Klofæy, modeling had to be extended even upstream of this island.

It was soon realized that using more than one model would be the most economical and convenient solution.

The scale reduction was restricted by two special conditions: the study should include details in the inlet and dam structures, and the river was very shallow, especially when ice might occur in the flow.

As a first approach two models were programmed, one main model covering the reach from downstream of the dam to Klofæy, including the dam and inlet structures, and a secondary small scale model of the whole reach, mainly designed to investigate the flow conditions at Klofæy. Fig. 4 - 5 and photos 1 - 2.

The contract also included a possible third model of the inlet and gate area,in case the main model should prove insufficient for certain detail studies. A model of this type as well as two other separate detail models were arranged during the study. Fig. 6 and photo 3.

2.2. Model laws and similarity

2.2.1. Similarity of flow

In hydraulic models where inertia forces are predominant, the conditions for similarity of flow are fulfilled when the Froude's number

$$Fr = \frac{v}{g^{\frac{1}{2}} \cdot L^{\frac{1}{2}}}$$

is similar for corresponding points in the model and the prototype, i.e. when

$$\frac{v_r^{\,2}}{g_r \cdot L_r} = 1$$

where

v = velocity
g = acceleration of gravity
L = length

and the subscript r indicates the model to prototype ratio.

Conditions for use of Froude's model law are that influences of surface tension and viscosity are negligible.

The surface tension may be significant in calm areas of small models, but can easily be reduced by adding small amounts of detergent to the water.

The viscosity forces are characterized by the Reynolds' number

$$Re = \frac{v\ L}{\upsilon}$$

where υ = kinematic viscosity. For Re > 2 - 3000 the influence of viscosity is negligible. This is usually the case in natural watercources, hence models must fulfill the same condition.

Distorted models, i.e. models where horizontal and vertical dimensions are reproduced to different scale ratios, can be used where the gravity forces are predominant compared to horizontal inertia forces. Denoting horizontal and vertical dimensions by x and y respectively, Froude's law for a distorted model yields:

$$\frac{v_r^2}{g_r \cdot y_r} = 1$$

The distortion is characterized by the ratio $y_r : x_r$.

2.2.2. Similarity of sediment transport

Several formulas exist for calculation of sediment movement in watercourses. For model studies, the bed-load concept by H.A. Einstein[x)] is convenient, characterized by the dimentionless parameters:

$$\psi = \frac{\gamma_s}{\gamma_w} \cdot \frac{d}{R' \cdot I}$$

and

$$\Phi = q_B \cdot \left(\frac{\gamma_w}{g}\right)^{\frac{1}{2}} \cdot \left(\frac{1}{\gamma_s \cdot d}\right)^{3/2}$$

where

γ_s = submerged specific weight of the particles

γ_w = specific weight of water

d = grain diameter (d_{35})

R' = hydraulic radius with respect to skin friction

I = slope

q_B = sediment transport per unit width

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x) H.A. Einstein: The Bed-load Function for Sediment Transportation in Open Channel Flow. U.S. Dep. of Agr. Tech. Bull. 1026. 1950

Similarity requires $\psi_r = \Phi_r = 1$

In comparatively wide rivers $R' \sim D$ (D = depth), and consequently $R'_r = y_r$. With $(\gamma_w)_r = g_r = 1$ then

$$d_r(\gamma_s)_r = y_r^2/x_r$$

and

$$(q_B)_r = y_r^3/x_r^{3/2}$$

For the suspended fraction of the sediment load, the equilibrium of a suspended particle is given by the equation

$$\frac{\Pi}{4} \cdot \frac{v_y^2 \cdot C_D \cdot d^2 \cdot \gamma_w}{2g} = \frac{\Pi}{6} \cdot \gamma_s \cdot d^3$$

where

C_D = drag coefficient

v_y = vertical velocity

C_D is approximately constant for $\frac{v_y \cdot d}{\upsilon} > 500$, in which case the similarity condition becomes:

$$d_r \cdot (\gamma_s)_r = (v_y)_r^2 = v_r^2 = y_r$$

The conditions are more complicated in the range where C_D is a function of $\frac{v_y \cdot d}{\upsilon}$, i.e. where influence of viscosity cannot be neglected.

Equating the two expressions for $d_r(\gamma_s)_r$ (when C_D = const.) gives the condition for similarity of both bed-load and suspended load:

$$y_r = x_r$$

Selecting $(\gamma_s)_r = 1$ gives further:

$$d_r = y_r = x_r$$

The horizontal velocity of suspended particles is close to that of the water, thus the scale ratio for transport per unit width is approximately the same as the similar discharge scale ratio of water, q_r, namely

$$q_r = y_r \cdot v_r = y_r^{3/2}$$

Equating q_r and $(q_B)_r$ again gives the condition

$$x_r = y_r$$

for similarity of the total load.

2.2.3. Similarity of movement of ice particles

The sediment transport concept is developed for particles heavier than water, but it can in principle also be adapted to buoyant particles as being sediments with negative submerged density.

Equivalent to the "bed-load" are then ice particles creeping underneath an ice cover or an accumulation of ice particles, while the term "suspended load" is self-explanatory also in this case.

Ice particles may in itself be so small that the condition C_D = constant is not fulfilled. But usually they appear in clusters which can be considered as the actual particles, with dimensions well above the critical limit even when scaled down to model conditions according to

$$d_r = y_r$$

Thermal processes pertinent to the ice conditions do not follow the hydraulic model laws and must be excluded from a model study of ice problems.

2.3. Selection of scale ratios

2.3.1. General comments

A direct use of the theoretical model laws is possible only in simple cases. Especially where sediments are involved, several, apparently contradictory requirements often have to be taken into account. World wide experience, however, has resulted in well established rules which allow model studies to be carried out also in these cases with reliable results.

2.3.2. Pertinent factors in this study

Pertinent to the selection of scale ratios in this case were the following factors:

A. Depth of flow: The wide and shallow river either required distorted or very large models to avoid influence from viscosity for the lowest discharges. The drifting "ice" also required sufficient depth to avoid interference with irregularities on the bottom not to be avoided in a small scale model.

B. Similarity of transport: Considering the transport of ice only, an undistorted model was preferable according to theory. The practical benefits compared to a distorted model would be limited, however, since real quantitative results could not be expected in any case. On the other hand, a distorted model would simplify and speed up the sediment transport study.

C. Roughness: The firm bed of the prototype requires in itself a rather rough model surface. In addition roughness, distortion and sediment transport conditions in models are interrelated. In distorted models, namely, additional artificial roughness is usually required to compensate for the increased slope which otherwise will lead to incorrect velocity and stage conditions. Where transport of sediments occurs,however, the bottom topography cannot be much exaggerated without seriously disturbing the transport pattern because of local turbulence. Also ice transport is likely to be disturbed by protruding roughness elements. A practical solution to the need of artificial roughness was therefore a major question to any model arrangement.

D. Accuracy: The required accuracy of the results determined a lower limit for the scale ratios. Structural details to be studied, might represent a limiting case, or the depth and velocity of flow where velocity measurements should be made. The accuracy of stage and discharge measurements are usually sufficient when other requirements are fulfilled.

E. Laboratory conditions: The water supply facilities and available floor area represented upper limiting conditions for the size and scale ratios of the models. The handling of sand and "ice" materials also had to be considered.

F. Operation time and cost: Both these factors usually increases with increasing size of the models.

2.3.3. Selected scale ratios

Based on a joint consideration of all the pertinent factors, the following scale ratios were chosen for the two original models as well as models introduced during the study:

	Model I	Model II	Model III	Sectional study of energy dissipator	Detail model of Bj.Canal
Selected scale ratios:					
Horizont.length	1:50	1:500	1:40	1:40	1:30
Vertical length	1:20	1:50			
Derived scale ratios:					
Velocity	1:4.47	1:7.07	1:6.32	1:6.32	1:5.48
Discharge	1:4470	1:177000	1:10119	1:10119	1:4930
Volum	1:50000	1:12500000	1:64000	1:64000	1:27000

2.4. Prototype data

2.4.1. Collection of data

All prototype information has been furnished by the client. Some data have been collected upon our demand, however. Only data pertinent to the model study will be mentioned here.

2.4.2. Discharge and stage characteristics

The discharge data for the dam site were given in section 1.2 and are only summarized here:

Flow water at dam site, calculated from recordings at Uriddafoss in the period 1947 - 1962:

Maximum:	1980	m^3/s
Average:	338	"
Minimum:	72	"

Design flood estimate: 7750 m^3/s

Attempts were made on our demand to measure the flow distribution past Klofæy. Unfortunately, occurence of bottom ice reduced the value of the results, which for a total discharge of 172 m^3/s showed a distribution of 55 and 117 m^3/s to the eastern and western channels respectively.

Stage measurements had been carried out close to the river banks in 15 different sections across the reach in question. Four different discharges, ranging from 166 to 900 m^3/s had been checked. Only eight of the sections were of direct concern to the study, however, and only four points had been measured for more than one discharge.

2.4.3. Ice and sediment characteristics

A few data from samplings of ice discharge were available, taken in February and March 1963 just above Thjofafoss. Based on these and theoretical heat loss computations, a probable maximum ice discharge of porous ice was estimated to 40 m^3/s.

Ice and water discharges were rather independent, but it was reported that ice rarely occured when the discharge exceeded 400 m^3/s.

The slush ice samples, with the free water immediately poured away, had a porosity of about 60 per cent.

To obtain an impression of the total ice flow during the winter season, a great ice jam usually occuring below Thjofafoss had been surveyed during the 1961/62 season. The seasonal accumulation of ice was estimated to 65 x 10^6 m^3.

No measurements of bed load rate or composition were available, and only a few measurements of suspended load, giving concentrations between 100 and 450 ml/l of suspended solids for discharges between 200 and 590 m^3/s, measured at Thjorsá bridge.

For estimation of the composition of the bed-load material, samples had been collected from bank deposits upstream of Klofæy and below the confluence of Thjorsá and Fossá. The samples did not differ much in composition, and can be characterized by:

Bank deposits upstream of Klofæy (16 samples):	$d_{10} \sim 0.25$ mm
	$d_{50} \sim 5$ mm
	$d_{90} \sim 45$ mm
Deposits downstream of Fossá - Thjorsá confluence (10 samples):	$d_{10} \sim 0.3$ mm
	$d_{50} \sim 2.5$ mm
	$d_{90} \sim 20$ mm

See also fig. 7.

An approximate lower limit of 400 m^3/s had been estimated for discharges carrying significant amounts of bed load.

2.4.4. The topography

For the dry part of the model area two series of maps existed, both to scale 1:20000:

a. Harza Engineering Company Internat.: Work sheet No. 33 - 34, 42 - 44, 51 - 53, 61 - 63.

b. Rafsmagnsveitur Rikisins Landmælingar 3640: Sheet No. 33 - 34, 42 - 44, 51 - 53, 61 - 63.

Cross sections were first available for approximately each 500 m of the reach in question below Klofæy, and also some sections 8 - 900 m apart upstream of the island. Additional surveys were made upon our request, until the greatest distance between the sections in the main model area was about 150 m (about 75 m in the dam area).

A certain difficulty was that due to the nearly flat river bed, local irregularities were of the same magnitude as the overall variation of the bed level. Location of possible depressions or stream channels could therefore be only roughly determined.

2.5. Preliminary studies

2.5.1. Ice and sediments

Little was known about ice studies in models when the present study was planned. Movement of sheet ice in models, using wax as substitute for the ice, had been studied occasionally. The present problem, however, required a granulated material with ability to simulate not only surface movement ,but also the motion of submerged ice particles in turbulent water.

It was convenient to use a material of specific weight equal to that of ice, i.e. 0.92, the diameter scale being then dependent only on the geometrical scale ratios of the model.

The wax previously used for sheet ice studies was rather soft and would be difficult to handle in bulk quantities with required accuracy. Use of polyethylene grains of specific weight 0.92 was therefore suggested.

This material was tested in a flume and compared to experiments with real ice recently carried out in a cold room flume. Known prototype situations were also studied under model conditions in the flume, and later in the main model without the future structures.

Further, the interaction between "ice" particles and the roughness pegs were checked and found negligible.

The diameter of the plastic grains ranged between 2 and 6 mm. Loosely packed the porosity of bulk volumes was found to be about 65 per cent, in good agreement with prototype data.

On the whole the plastic material was found to reproduce the motion of "passive" ice to satisfaction. Photo 4 gives a close up of the material. Sheet ice was simulated by pieces of wax. Photo 5.

Various materials were discussed for use as model sediments in the early stage of the planning. The final design of model I allowed for use of ordinary sand to reproduce the bed-load and the part of the suspended load which might cause trouble to the project.

The grain distribution of the model sand is shown on fig. 7. The average sample data from the bank deposits in the prototype are shown for comparison, and also shown reduced to model scale (d_r = 1:8). The model sand roughly corresponds to the fraction of the prototype sand between 0.8 and 24 mm.

The motion of the sand along the gravel bed of the model was checked in advance in the flume together with the roughness tests. (See item 2.5.2.3.).

2.5.2. Artificial roughness

2.5.2.1. General principles

Direct reproduction to scale in a model of the irregular details that constitute the roughness of a watercourse is usually neither necessary nor possible. It is satisfactory to use a surface that on the average has the roughness properties required to obtain similarity of flow.

Use of Manning's formula

$$v = \frac{1}{n} R^{2/3} I^{\frac{1}{2}} \quad \text{(metric units)}$$

usually assumes n to be a constant for each locality, independent of depth or velocity. Simple model/prototype ratios for n can then be established:

Undistorted models: $n_r = x_r^{1/6}$

Distorted models where R ≈ D: $n_r = y_r^{2/3}/x_r^{\frac{1}{2}}$

Hence, distorted models (with $y_r > x_r$) require more rough surfaces than undistorted models of similar scale ratios. In most distorted models a sufficiently rough surface cannot be obtained directly from the model material, and some kind of artificial roughness must be added.

Though nearly constant for most prototype conditions, the n-value in Manning's formula increases with decreasing depths. The effect is only significant for rather shallow depths, however. Models with much artificial roughness to be used for a wide range of discharges must therefore usually be calibrated for an average discharge, possibly with small deviations from the similarity conditions for other discharges.

2.5.2.2. Friction conditions in the Burfell study

In the Burfell project, the existing lava bed covered with boulders, is supposed to be more or less covered by deposits of sand and gravel in the backwater area upstream of the dam soon after its construction.

The roughness number of the existing bed was calculated from the stage-discharge data to be about n = 0.03.

The future roughness conditions of the sediments had to be estimated. Contributions to this roughness are partly the grain friction and partly the frictional effect of dunes or ripples forming in the surface of the deposits.

The grain friction can be well estimated from the grain size distribution of the bed material. Based on the existing data a probable value for the project was found to be n = 0.021.

A method to estimate dune friction is given by H. Einstein[x)], but it was doubtful whether it could be applied to this particular problem. In any case the dune figuration varies with velocity, slope and depth of the flow, giving a very complicated friction pattern.

A perfect design of model I should give correct flow conditions for all stages of the gradual development from a bed free from movable material, through conditions where sediments partially substitute the bed roughness with their own inherent roughness, to fully developed deposits. As data for calibration were only available for the condition with no deposits, it was necessary to concentrate the calibration procedure on this situation with due consideration to anticipated future bed conditions. A flexible solution providing for modifications of the roughness pattern was thus necessary.

Summarizing, the artificial roughness in model I had to fit into the following scheme:

Existing conditions:

Prototype:	Bed roughness:	$n_{p_o} = 0.03$
Model:	Bed roughness: Artificial roughness:	$n_{m_o} = n_{p_o} \cdot n_r$

Situation with deposits:

Prototype:	Grain roughness ($n \sim 0.021$): Dune roughness (varying):	$n_{p_1} \geqslant 0.021$
Model:	Grain roughness: Dune roughness: Artificial roughness not covered by the deposits:	$n_{m_1} = n_{p_1} \cdot n_r$

x) See reference page 20.

The selected scale ratios of model I gave $n_r = 1.04$ i.e. $n_{m_o} = 0.03 \cdot 1.04 = 0.031$, which was later found approximately valid for model depths down to 0.05 m, increasing for smaller depths.

From comprehensive computations and discussions it was found that the various requirements could be best fulfilled by applying a rather rough model surface (n about 0.025), if necessary by means of very low artificial roughness elements to avoid disturbance of the sediment pattern or interference with the movement of ice particles. The necessary additional roughness could then be provided by means of thin vertical pegs attached to removable wooden frames above the water surface resembling harrow-like units (fig. 8). The thin pegs were found to cause negligible disturbance to ice and sediment movements.

The contribution to the roughness from the pegs would only slightly be influenced by sediment deposits, but it would vary with depths and velocities of flow.

Model II was not supposed to be run with ice or sediments, and the artificial roughness design was ordinary routine. It was decided to use metal sheet baffles as roughness elements, (fig. 9), and the only requirement was that the height of the baffles had to be kept well below the water depth for all discharges.

2.5.2.3. Flume studies of bed roughness

Various ideas for bed roughness to be used in model I were studied in advance in a 0.6 m wide flume.

An attempt to use wire mesh spred on the concrete river bed was soon given up. The regular mesh pattern led to secondary currents which greatly disturbed the sediment transport, and even influenced the flow pattern for low discharges.

Another attempt to supply the bed with a pattern of tacks protruding 0.5 cm from the bed failed to give the desired roughness with a reasonable number of tacks.

Finally an excellent design was obtained by pasting gravel of 9 - 12 mm diameter to the concrete model bed.

The test procedure can briefly be described as follows:

Gravel was pasted on a 4 m long plywood board by means of concrete mortar covering the whole board.

The board was placed in the flume and given a slope of 0.01 (similar to the model bed).

Various discharges were run along the board and depths were measured along the flume for each 20 cm.

The local velocities and depths were then used to calculate a mean roughness number for each discharge, corrections being made for the influence of the glass walls of the flume.

The results of three different test series are given in fig. 10, one series with gravel diameter 4.7 - 9.8 mm and two series with diameter 9 - 12 mm, but with slightly different gravel density.

Some data from the model are also included in fig. 10. These results agree well with the flume tests, indicating that the slight non-uniformity of the flume flow has been of little consequence.

2.5.2.4. Pegs and baffles as artificial roughness elements

The number and size of pegs to be used in model I were estimated by equalizing the skin friction equivalent to the desired n-value

$$\tau_o = \frac{n^2 v^2}{R^{1/3}} \gamma \approx \frac{n^2 v^2}{D^{1/3}} \gamma$$

and the drag force exerted on the pegs by the flowing water

$$\tau = C_D \cdot A \frac{v^2}{2g} \cdot \gamma \cdot N,$$

where

A = area of one peg perpendicular to the flow

N = number of pegs per unit area.

Since $A = D \cdot t$ (t = thickness of pegs) in the case of pegs extending all the way from bottom to surface, we get

$$C_D t N = 2gn^2/D^{4/3}$$

From available data, $C_D \approx 1.5$ was estimated for pegs with square cross sections, giving for $t = 0.6$ cm N-values varying between 10 and 25 per m^2. The pegs actually used, however, had a nearly octogonal cross section, giving a lower C_D-value which later was checked in the model to be about 0.5. This increased the required number of pegs considerably, and complicated the calibration of model I.

An estimate of the number and size of the metal sheet baffles to be used in model II was based on an article by Sayre and Albertson[x)] giving test data for several types of baffles under various conditions. None of the data fit the actual case directly, but interpolation led to a design with 35 mm wide and 10 mm high baffles spaced 35 mm (free space) transversally and 45 mm longitudinally (fig. 9). This design later proved fairly good, needing only small adjustments for some discharges.

2.6. Similarity conditions and accuracy

2.6.1. Model I

Distorted models are widely used in hydraulic laboratory practice. Provided proper calibration of the models, good similarity is experienced for discharge, stage and current data describing overall conditions.

Some discrepancies must be expected for data related to details in the structures causing complicated flow patterns.

The available data for comparison of prototype and model conditions of ice movement indicate that the qualitative similarity is good in this study.

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x) Sayre and Albertson: Roughness Spacing in Rigid Open Channels. ASCE. HY3. 1961.

Of particular interest is the scale ratio for the transport rate of the ice per unit width. According to section 2.2.2, this ratio for ice suspended in free flow is $y_r^{3/2} = 1/20^{3/2} = 1/90$, while the similar ratio for the creeping transport under an ice cover is $y_r^3/x_r^{3/2} = 50^{3/2}/20^3 = 1/23$.

Using $y_r^{3/2}$ we get too good performance of ice transport capacity in the model, because some of the ice moves as "bed-load" according to $y_r^3/x_r^{3/2}$. On the other hand, if we look at "sediment time", i.e. the time required to fill a certain volume with ice, we get too short a time using $y_r^{3/2}$, and are on the safe side.

The time required to form an ice float in front of the inlet is therefore too low in the model. Once the ice float has formed, the rate of ice transport underneath the float is overrated.

Model results given as quantitative data in this report are based on transport scale $y_r^{3/2}$, hence, transport rates are on the safe side where "bed-load" are involved.

Only qualitative results of sand runs are given. Taking into account the scarce prototype data available for calibration, in addition to situations directly developed as a result of model runs, other possible situations have been pre-arranged in the model and tested for flow and ice conditions.

The accuracy of quantitative model data lies within the following limits:

Water level indication:	± 1 - 2 mm in turbulent flow, ± 0.2 mm in still water.
Discharge measurements:	± 1 per cent for water, ± 5 per cent (approx.) for ice.

2.6.2. Model II

This model was well fit for its main purpose to procure general flow data for calibration of model I, mainly in the Klofæy area. The similarity for details in the flow pattern was poor due to the large artificial roughness elements. These also made the model unfit for sediment and ice studies.

The accuracy of flow and stage measurements was less than for model I due to the smaller dimensions and the very rough flow, but still well comparable to the accuracy of good prototype measurements.

2.6.3. Model III and other detail models

The undistorted detail models were all designed for special purposes, and the scale and finish were chosen in each case to give satisfactory accuracy and similarity conditions. Results from these models are therefore of very good quality.

2.7. Description of the models

2.7.1. Model I

Model I (photo 1) was the main model, which covered the river from Klofæy, about 1500 m upstream of the dam, to about 350 m downstream of the dam. It included the dam, the inlet, and the adjoining parts of the diversion canal and the Bjarnalækur Canal. Besides the river itself most of the inundated area was covered.

Fig. 4 shows the original model plan.

The model was constructed to scale 1:50 horizontally, and 1:20 vertically, thus having a distortion of 2.5. It covered more than 600 m^2 of floor area in the laboratory hall.

The model was constructed in two parts. The lower part from just upstream of the inlet, including the dam, inlet and canal structures, was constructed directly on the floor, photo 6. Crushed rock was filled between the brick walls forming the boundaries of this model section up to slightly below the proper surface level. The surface was then finally finished by concrete mortar according to the river bed profiles and,for the dry areas, the aerial maps available. The dam and inlet structures were constructed as removable elements from wood and plastics, photo 7. To allow for calibration of the model the future structures were not put into it from the beginning, but provisions were made to make the arrangement of these elements as easy as possible when the calibration was once finished. Provisions were also made for moderate modifications of the original design, for instance the location of the dam and the inlet structures.

The upper part of the model was constructed on a wooden slab. The contour lines of the dry areas and the river bed profiles were reproduced by pegs hammered into the slab until the top of the pegs were flush with the correct surface level. Rocks were filled in between the pegs, and the model was finally finished by concrete mortar flush with the top of the pegs.

The river bed area was finished 7 mm below the correct level, and a layer of gravel in the diameter range of 9 - 12 mm was spred on the concrete bed and pasted to it by cement mortar, photo 8. The necessary additional roughness was provided by thin pegs attached to wooden frames forming harrow-like units as previously described in section 2.5.2.2. These units could easily be removed from the model or rearranged.

Water to the model was supplied from the laboratory's recirculating system. The total discharge to the river was measured by three V-notch boxes at the upper end of the model. The water went into two stilling basins discharging to the two channels of the river past Klofæy. Filters were placed between the stilling basins and the model to provide smooth inflow. The discharge through the inlet was measured by two separate V-notch boxes, and controlled by valves, which also served to control the level in the diversion canal. The model also included facilities to control the tailwater level below the dam. The tailwaters from the main river bed and the inlet were returned to the recirculating system.

The ice and sediments were supplied to the model from a special feeder consisting of 6 m long and 2 m high storage bin suspended across the river with a discharge slot at the bottom controlled by a rotating shaft supplied with axial vanes, photo 9. The shaft was driven by an electric motor and supplied with gearing equipment to control the speed of rotation. Having passed through the model the sediments and ice were collected from the tailwater discharge by a sieve and carried back to the storage bin.

Rates of sediment or ice supply were computed from the number of revolutions of the feeder shaft. Quantity measurements elsewhere in the model were made as direct volume determination of time integrated samples.

The model was supplied with ordinary point gages for stage measurements. Gage points are indicated on fig. 4. Micro-propeller equipment for velocity measurements was available, but due to the contents of ice and sediments, velocity measurements usually had to be carried out by means of drifters and stop-watch. The drifting ice particles were excellent for photographic recording of surface flow patterns.

2.7.2. Model II

Model II, photo 2, covered a longer section of the river than model I, beginning about 500 m upstream of Klofæy, and extending to about 500 m downstream of the dam site. It was constructed to scale ratios 1:500 horizontally and 1:50 vertically, covering a floor area of about 20 m^2. Fig. 5.

The model was constructed in a similar manner as the upper part of model I. Photo 10. The artificial roughness consisted of sheet metal baffles of size and spacing as described previously in section 2.5.2.4.

The model was designed for calibration purposes and did not include the future structures, only the natural topography.

The supply and tailwater arrangements were similar to model I, but did not include any facilities for feeding and collection of sediments or ice.

The model was supplied with point gages for stage measurements, in points shown on fig. 5, and "ice" was available for studies of flow patterns.

2.7.3. Model III

Model III, photo 3, was an undistorted version of the inlet and the gated dam section, the upper part of Bjarnalækur Canal and surrounding areas. It extended from 200 m upstream to 100 m downstream of the dam, and covered about 1/3 of the width of the river.

The model was constructed in a permanent flume of width 3.6 m, supplied with permanent facilities for water supply and tailwater control. Most of the model was constructed in brick and concrete, but the inlet and dam structures were made of wood and plastics. The model was not equipped with facilities for ice and sediments, but a collecting screen was arranged in the tailwater to allow for hand feeding of ice if desired.

From a permanent bridge running on rails along the flume, the surface level of the water could be measured in any desired points. In addition, two fixed point gages were arranged, as indicated on fig. 6. For velocity measurements an electronic micropropeller instrument was available. Flow studies were supposed to be made by means of dye or confetti.

In addition to the permanent equipment of the flume arrangement, two V-notch boxes had to be arranged downstream of the model in order to measure the discharges through the Bjarnalækur Canal and the inlet respectively.

2.7.4. Other models

A sectional study to scale ratio 1:40 was arranged in a 15 cm wide flume to study the flow conditions in the runaway channel downstream of the dam.

To study the flow conditions in the plunge pool and upper part of the Bjarnalækur Canal a separate model to scale 1:30 was constructed. The part of the Bjarnalækur Canal supposed to be lined by concrete was reproduced in plastics, while the unlined part of the canal was made of wood with artificial roughness in the form of pegs.

2.8. Calibration of the models

The calibration of the models had to start with model II, since results from this model should be used for the calibration of model I.

The calibration of model II consisted of obtaining correct stage values along the river for various discharges between 100 and more than 2000 m^3/s. To obtain this it proved necessary to vary the artificial roughness slightly for different discharges. This was obtained by bending down the metal baffles according to a special system found by trial and error.

Model II was then used to find the correct division of the discharge to both sides of Klofæy, and these data were transferred to model I before it was further calibrated.

Also the calibration of model I aimed on obtaining similarity between stage data from the model and the prototype. This was achieved by varying the density of peg roughness elements for various discharges. The gravel roughness fit very well into the results found in preliminary flume studies, fig. 10. On the other hand, the effect of the peg roughness was less than expected, and the number of pegs per unit area had to be increased compared to the original estimate. As can be seen from fig. 11, the effect of the pegs was negligible for discharges less than about 300 m^3/s, while some improvement of the roughness conditions was obtained for the higher discharges.

The maximum peg density tested was 20 pegs/m^2. Fig. 11 shows that this did not exactly give the desired water levels, but the deviation was found of little significance. However, it was attempted to improve the conditions further by applying

horizontal steel rods across the river, attached to the pegs a few centimetres above the bed. The rods disturbed the sediment pattern significantly, as well as the ice flow for certain discharges. The benefit from the rods was slight, too, and it was finally decided to accept the calibration obtained with pegs only.

It was of particular interest that the pegs had so slight influence on the small discharges, since this ment that the pegs could just as well be removed for most discharges where ice conditions were to be studied.

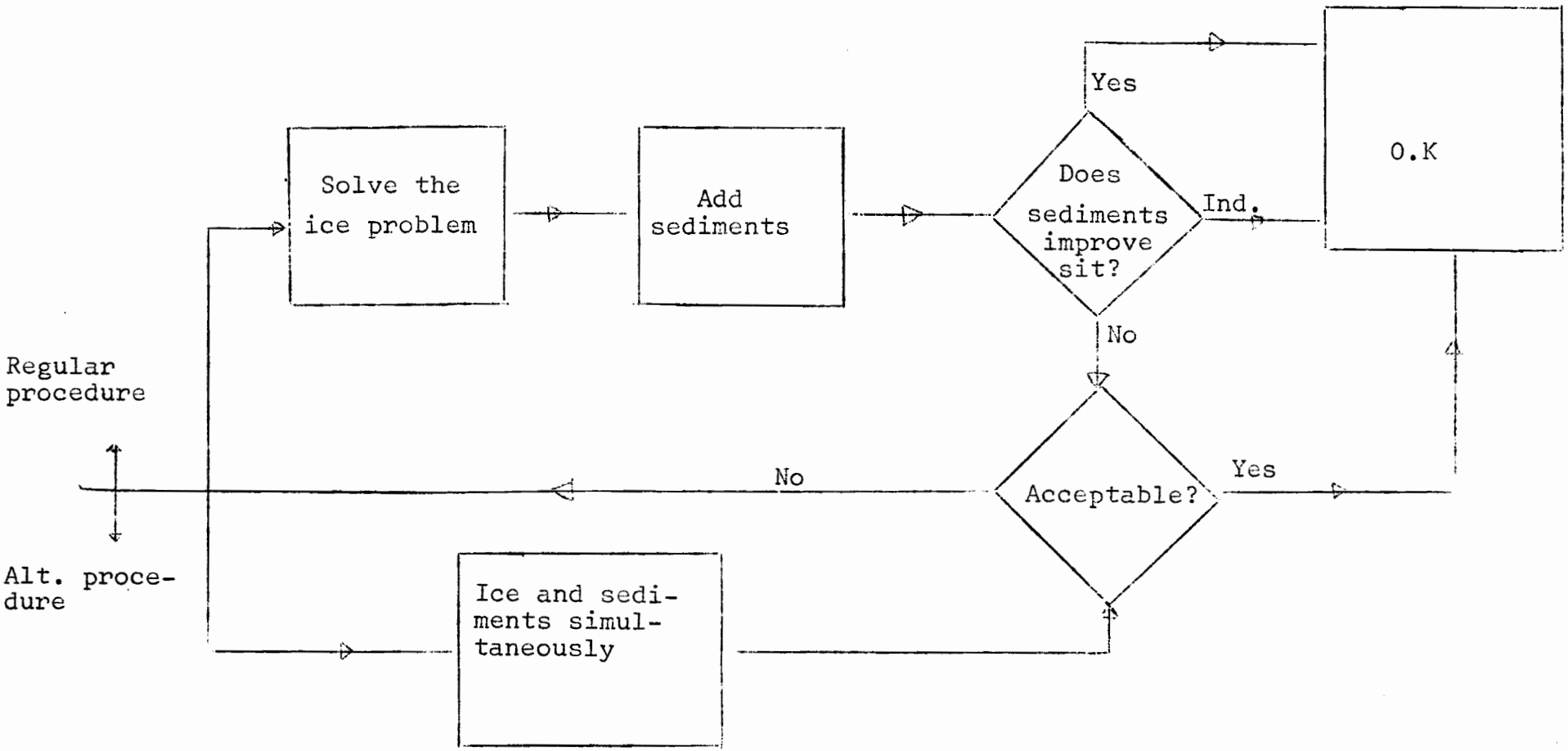
The calibration of model III and the detail models was very simple. According to calculations the flow in Bjarnalækur Canal would be sub-critical, and this had to be checked and adjusted in the models.

3. THE STUDY

3.1. About the program

The main program of the study can be explained from the flow sheet page 46. The primary purpose of the study was to solve the ice transport problems which would be introduced by construction of the dam across the river. Similar sediment problems which were likely to occur, were considered of secondary importance for the development of the study. Only if sediment deposits should prove to have major influence on the ice conditions, this part of the study had to be taken up in its full scope. Simultaneous use of ice and sediments in the model was needed, but since the sediment transport in the natural river is negligible for those discharges where ice drift is normal, simultaneous feeding of ice and sediments into the model was not necessary.

The model tests should begin with the design (fig. 3) given in Dwg. 290 Skc. 13 by Harza Engineering Company International, modifications or revisions to be introduced during the runs when necessary.


Solve the ice problem
Add sediments
Does sediments improve sit?
Yes
Ind.
O.K
No
Acceptable?
No
Yes
Regular procedure
Alt. procedure
Ice and sediments simultaneously

Model I was designed to act as the main model for the study, where the general conditions and the general lay out of the project should be investigated. The need of model III was anticipated from the beginning of the study, but its final scope and design were first determined late in the study. Model II should only provide the inflow conditions for model I.

Some limitations had to be adapted from the beginning of the study, for practical reasons. Thus, a discharge range up to about 2000 m^3/s, and a range of ice discharge up to about 55 m^3/s, loosely packed, were found satisfactory, referring to prototype data.

The range of sediment amounts to be used had to be estimated. To stay on the safe side the amounts of sediments introduced in the model were kept just below the limit where depositions would occur in the area of the river not influenced by the backwater of the dam.

3.2. The runs on model II

Only data for one particular flow condition were available for the division of the flow to both sides of Klofæy. The size of model II did not permit reliable velocity measurements. The study was therefore based on direct measurement of the discharge in the two channels.

With the flow going on both sides on the island, stage data in the western branch of the river were observed. The eastern branch of the river was then closed and the discharge reduced until similar stage measurements in the eastern branch as before were obtained. The reduced discharge was then assumed to be the fractional flow in this branch when both branches were open. To check the results, the fractional flow in the eastern branch was found in a similar manner. The two test series agreed well, as can be seen on the diagram given in fig. 12.

3.3. Development of the main study

3.3.1. The original design

The model was originally constructed according to Dwg. No. 290 Skc. 13 from Harza Engineering Company International. This design is shown on fig. 3.

Before the study was concentrated on the ice problems, a few sand runs were made to obtain an idea of the future pattern of deposits behind the dam. The tests were run with a total discharge of 1000 m^3/s, diverting 250 m^3/s to the station. The general flow pattern seemed to be little influenced by the inlet, except in the very vicinity of the inlet. The flow went mainly parallel to the river banks all the way down to the dam, but with a transverse component towards the inlet near the dam. (Compare photo 17, and section 3.3.2). The sediment deposits in the backwater area accordingly formed a nearly straight front moving with decreasing velocity towards the dam. Photo 11. In the vicinity of the inlet, however, the current carried great amounts of sediments down into the trench in front of the inlet, the more the nearer the sand front approached the dam. No test was continued until the whole backwater area was filled by sediments, but rather soon the upper part of the inlet trench was completely filled with sediments. Much sand was also carried into the diversion canal. See photo 12.

The transverse current along the dam caused skew flow conditions at the dam piers, with large contraction zones on their western sides,see photo 13. Widening the piers improved the flow, photo 14.

Although a major part of the sand settled down in the backwater, some of the sand was carried in suspension across the dam and over the gates. This sand did even include some of the coarser fractions.

Four samples taken from the sand deposits, and compared to the sand originally introduced in the model, showed only slight differences except for the finer fractions, which were more washed away from the deposits in the diversion canal and below the dam than in the main deposit upstream of the dam. See fig. 13.

The sand was cleared away from the model, and some preliminary ice runs with a discharge of 100 m^3/s were conducted. Various ice quantities were tried. The flow pattern of the ice was much similar to that previously described for the flow of water, the ice moving approximately parallel to the banks until near the dam before bending sharply towards the western bank and the inlet area.

It proved very difficult to avoid ice accumulation in front of the inlet, since most of the ice approached the inlet nearly perpendicular to the inlet wall. For the same reason only the two ordinary dam gates located nearest to the Bjarnalækur gate proved to some extent effective for ice flushing, while the eastern gate did not catch the ice flowing parallel to it. Photo 4 and 15. The gate leading to Bjarnalækur Canal was more or less ineffective for another reason, namely the great depth of water immediately upstream of the gate, which caused the gate to draw water from the lower layers while the surface water containing the ice particles did hardly move at all. Photo 16. The accumulated ice in front of the inlet gradually formed a stable float that finally covered the whole inlet area, growing thicker and thicker until ice went directly through the inlet ports, moving as "bed load" underneath the ice float. Gradually reducing the discharge to the station showed that a rather large fraction of the discharge had to be used for ice flushing to provide ice free inlet area.

3.3.2. The shortened inlet

As a consequence of the initial runs, a design of the inlet consisting of only the 6 inlet ports nearest to the dam was tested instead of the original design with the 12 ports. The length of the trench was shortened accordingly. This situation is shown on fig. 14.

Photo 17 shows streamlines for total discharge 1000 m^3/s with 250 m^3/s through the inlet, very similar to the pattern described in the previous section.

Photo 18 shows the result of an initial sand run, with a total discharge equal to 1000 m^3/s.

Some attempts were made to control the sediment deposits in the backwater area by means of groins extending from the river banks further up the river, see photo 19. Although some improvement was evident, it was not possible to avoid the sand from filling up the trench in front of the inlet.

The bottom sluices alone were not effective for removing sand from the trench, only a small area close to the sluice inlets was cleared. As it would be necessary to find a means to avoid sand from clogging the inlet, a sand excluder arrangement was installed in the trench, as shown on fig. 14.

The sand excluder consisted of a system of conduits with intakes along the whole length of the trench, discharing through the bottom sluices. The sand excluder proved very effective, and having thus proved that the sand problems could be handled to satisfaction, further sand studies were deferred to concentrate on finding a satisfactory design for the ice conditions.

Simultaneously with the installation of the sand excluder, a modification of the inlet was arranged. The six inlet ports now in use were doubled in height to 4 m and the bottom sill of the openings raised to el. 233.50 to give room to the sand excluder.

Ice runs showed a similar ice float formation as for the original design. A marked vortex formed in the corner between the inlet and the Bjarnalækur sluice for all the situations with the shorter inlet, causing much ice to pass through the inlet ports. Photo 20. The new design of the openings did not cause greater ice intrusion than the original design, however.

3.3.3. Excavations in front of the inlet. Various gate locations

In order to pass under the ice float and the inlet wall, the flowing water had to change its direction from nearly horizontal to some angle with the horizontal. This vertical component of the flow and the vertical motion due to turbulence was responsible for transporting the ice downstream and into the diversion canal.

To reduce the velocity of the ice approaching the inlet, and also allow already suspended ice to surface in front of the inlet wall, it was decided to try various excavations to widen the trench in front of the inlet.

As much of the ice approached the inlet along the dam, the first attempt consisted of digging a trench along the gated section of the dam, discharging into the trench along the inlet. Fig. 15. Although most of the ice could be flushed across the dam in this case, the movement of the ice was very slow, causing a permanent danger of ice bridge formation. The tendency of ice float formation in front of the inlet was still apparent.

Further increase of the excavation upstream and shortening of the inlet to four ports caused an eddy just in front of the inlet. Fig. 16 shows this situation.

To see if alternative locations of the flap gates could have any influence on the current pattern and the ice conditions, the model dam was redesigned and supplied with flap gates along its whole length across the river, with the sill of the gates at el. 240.50. Each gate had a length of 20 m, and could be operated independently. The top sluice to Bjarnalækur Canal was also widened from 12 m to 20 m, and supplied with three gate sections of 10, 5 and 5 m respectively. The situation is shown on Fig. 16 and photo 21.

No situation was found which could remove the eddy in front of the inlet and simultaneously provide favourable flushing conditions. The eddy would nearly vanish when flushing through Bjarnalækur Canal only, but the converging ice flow towards the canal gate soon caused complete clogging in front of the gate. Ice was also likely to accumulate in front of the inlet when flushing through Bjarnalækur Canal and the flap gates numbered 1, 2 and 3. The most favourable situation was obtained by keeping the crest of Bjarnalækur gate and the dam gates No. 1, 2 and 3 at el. 242.40, and dam gates No. 4 - 12 about 20 cm higher. In this situation the water surface was automatically raised when ice accumulation tended to clog the lowest dam gates. With the raised level more of the ice was carried over the gates at highest position, reducing the pressure on the inlet area. All the manipulations, however, only led to a delay of the ice float formation. Sooner or later the ice float would gain the necessary thickness to promote ice intrusion into the diversion canal.

3.3.4. Sheltering jetty and boom

Attempts were made to prevent the accumulation of ice in front of the inlet by a jetty extending from the western river bank just upstream of the inlet, providing a shelter for the inlet area, as shown on fig. 17. The jetty alone increased the ice troubles by causing a large eddy between the inlet and the jetty. Extending the jetty by a curved ice boom to the pillar between the Bjarnalækur Canal gate and the dam gates reduced the effect of the eddy, but even a rather deep boom did not prevent some ice from diving under the boom and accumulating in the sheltered area. The conditions were particularly unfavourable in the corner between the dam and the boom. Various gate operations were tried without further success.

3.3.5. The inlet ports moved upstream

To avoid the effect of the eddy in the corner between the inlet and the dam, the two inlet ports located closest to the dam were closed, using only the next four inlet openings. This modification indicated that some improvement could be obtained by moving the inlet ports still farther away from the dam. The model was consequently provided with an approximately rectangular excavation, the bottom extending about 120 m from the dam, as the original trench, but having a width of about 70 m, see photo 22. The inlet ports were now located between 60 and 120 m from the dam, and given a height of 5 m in the range between el. 232.50 and 237.50.

A promising ice run was made with the following specifications:

Total river discharge:	150 m^3/s
Discharge to inlet:	100 "
Ice discharge:	27 "

Gate No. 1, 2, 3 and the
Bjarnalækur gate open to el. 243.00

The water level in the inlet area at about el. 243.30.

The ice moved easily down to the dam and across the gates with little tendency to accumulate along the inlet wall. A thin ice cover gradually filled up the triangular area between the closed section of the dam and the eastern river bank up to point V-3. The part of the river available for ice transport was thus gradually narrowed into a channel about one third of the width of the river. This was apparently favourable to the ice movement, increasing the velocities. The velocities in front of the inlet were still low, however, and finally some ice accumulation appeared also in this case. Photo 23 - 28.

Repeating the run with water surface at el. 242.80 increased the velocities along the inlet wall slightly, reducing the tendency of ice accumulation further.

A third test was run with the gates No. 1, 2, 3, 4 and the Bjarnalækur Canal gate at el. 242.50, and the gates 5 - 12 at el. 242.60. Little ice entered the diversion canal, but a rather stable ice cover formed in front of the inlet, covering approximately the area of excavation. Some ice passed all the gates. The ice showed some tendency to stick to the gate flaps causing local ice accumulation.

A series of runs was made to find the most favourable shape of the excavation in front of the inlet. A shape as shown on fig. 18, was found to give good conditions. This figure also indicates two other studied arrangements.

Further study of various gate operations resulted in the following general rules:

Gates located where the current direction is more or less parallel to the gate sill are less effective for flushing of ice than gates approximately perpendicular to the flow direction, when the proportion of ice to water is considered.

Shallow water immediately upstream of the gates gives better ice passing conditions than deep water. In the latter case a great part of the discharge across the gates is drawn from the deeper layers of the water, resulting in poor efficiency of the flush water, since most of the ice is concentrated in the surface layers.

The wider the gated section is, the more water is needed to flush a certain amount of ice, because a certain water depth to prevent the ice from sticking to the gate crests. A width between 60 and 100 m will usually be sufficient.

3.3.6. Installation of a jetty parallel to the inlet

The observed benefits from the narrowing of the effective river width due to the formation of a stagnant ice cover in the eastern two thirds of the backwater led to the idea of promoting this development artifically. A few simple tests proved this to be the most promising idea of the study so far. A number of designs of a jetty extending upstream from the dam about parallel to the inlet wall, varied in length and alignement and in various distances from the inlet, led to the final design shown on fig. 19.

The jetty finally adapted leaves a 100 m wide gated dam section between the jetty and the inlet, including the width of the Bjarnalækur gate. The design led to much better ice flushing conditions than any other designs tested so far. The flow of ice between the jetty and the inlet was now nearly parallel to the inlet wall under most ice conditions. An exception was very small ice quantities when a return current along the jetty gathered some ice into a slowly rotating ice float. There was apparently no tendency for this float to grow and clog the whole area. The float was pushed over the gates as soon as the ice flow increased in magnitude. Photo 29 shows the situation at the beginning of a test run, before a stable condition has established in the inlet area, when the return current is still visible. Photo 30 - 33 show the further development.

Attempts to remove the return current completely under all conditions were not successful.

3.3.7. The inlet structures with-drawn into the diversion canal

The benefits from the jetty were so clear that it was likely to be included in the final design. However, before going into further detail studies of the jetty design, it was found reasonable to discuss possible other ideas for the inlet. An alternative design, shown on fig. 20, was built into the model and tested. The inlet structures were in this case with-drawn from the original river bank line into the diversion canal. Compared to the original inlet design, the six inlet ports farthest from the dam were closed, and the six ports nearest to the dam were turned 90° clockwise and moved 35 m into the canal, with a wall to close the gap. The height of the ports was now 5 m, and the width 10 m as before. The gate to Bjarnalækur Canal was moved about 60 m to the west, and the dam extended by a vertical wall of similar length to close the gap between the dam gates and the Bjarnalækur gate. The western

side wall of Bjarnalækur Canal was extended up to the western end of the inlet section, the inlet and the two new vertical walls thus forming a rectangular indentation from the river bank line. The water was supposed to enter the rectangular basin about parallel to the dam, and then follow an S-shaped course through the inlet and into the diversion canal.

Various designs of the excavation in the river bed in front of the rectangular basin were tested, but it was not possible to find a means to avoid strong turbulent motion in the basin. Ice carried into the basin would gather along the inlet wall if not carried directly through the inlet ports. Until all the rest of the basin was covered by ice, the water surface close to the vertical extension of the dam would stay free of ice, also when water was drawn over the gate to Bjarnalækur Canal. Even with the whole basin covered by a thick ice carpet, only small amounts of ice could be carried down the Canal.

All attempts to keep the ice out of the rectangular basin by means of ice booms or guide walls were in vain.

The design was also tested combined with the jetty described in the previous section, but did not give satisfactory results. Photo 34 shows a situation from the study.

3.3.8. Systematic investigation of the design of section 3.3.6, with respect to ice passing conditions

The design of section 3.3.6 (fig. 19) was found to be the most promising for further study. The influence of the following factors on the ice passing conditions was therefore studied:

Surface level in the pool:	Varied within the range of the flap gates
Ice discharge:	14 - 28 - 48 - 55 m^3/s
Water discharge:	75 - 350 m^3/s
Jetty:	Various designs
Inlet wall	More or less inclined to the bank line of the river
Inlet ports:	Vertical and horizontal position varied
Excavation:	Varied

Further, the effect of the roughness pegs was checked.

Results are given in tables 1 - 6 (pages 60 - 65), referring to fig. 21, and also in fig. 22 - 23.

The arrangement scheme 1, fig. 21, could be operated to prevent clogging and extensive ice intrusion into the diversion canal for any water discharge between 75 and 350 m^3/s carrying up to about 30 m^3/s loosely packed ice. 48 m^3/s ice discharge could be passed for water discharges 150 m^3/s and more. The combination of 48 m^3/s ice and 75 m^3/s water clogged the model river completely.

Each discharge situation required the pool level to be kept within a narrow range. Too low levels caused mixing of ice and water in front of the inlet. Too high levels increased the danger of ice bridging in the backwater.

The required pool level increased with total discharge from about - 243.0 for 75 m^3/s to about + 244.5 for 350 m^3/s.Fig. 22.

The acceptable pool levels were practically independent of the amount of ice in the water, but the conditions were more or less favourable for various ice discharges.

No fixed minimum water to ice ratio for passing of ice over the dam could be established. The ratio seems to increase with increasing total discharge and with decreasing ice discharge. For small ice discharges the necessary drag towards the gates and the depth on the gate crest determine the amount of water to be used for ice passage.

Visual observations indicated that modifications of the inlet wall as in scheme 2 and 3 or scheme 4 were of little or no benefit to the ice conditions. Scheme 5 showed worse conditions than scheme 1, introducing a deadwater zone along the wall just above the inlet openings.

Moving the six inlet ports 20 m towards the dam had apparently no adverse effect on the ice situation.

Parallel tests with and without pegs in the model showed that the pegs give lower values of the lower as well as the upper limit of the acceptable range of levels for a certain discharge situation.

The influence on the lower limits is small, as directly caused by the comparatively slight effect of the pegs on the velocities in the range of discharges in question.

The influence on the upper limits is greater, as the pegs increase the tendency for the ice to cluster and build up an ice bridge in the backwater area.

TABLE 1

Ice runs. Water discharge = 75 m^3/s

Scheme (fig.21)	ICE DISCHARGE m^3/s (loosely filled)			
	14	28	48	55
1	Q_{st} = 50 243.4 too high 243.2 clogged at dam 242.9 clogged at dam	Q_{st} = 25 242.8 - 243.5 acceptable 243.0 good	The whole river clogged	

EXPLANATIONS:

Q_{st} = discharge to diversion canal

Q_{BC} = discharge to Bjarnalækur Canal

$Q_1, Q_2, \ldots\ldots\ldots$ = discharge to gates 1,2, ... (see fig. 24) of dam

Discharge not spesified is equally divided on those of gates 1,2,3, and 4, and the Bjarnalækur gate not separately mentioned.

"Too high" indicates ice bridge in the backwater.
"Too low" indicates mixing of ice and water in front of inlet.
"Clogged" indicates accumulation of ice, usually as an ice float with gradually increasing thickness.

Water discharge is measured before "ice" is added.

The solid volume of ice is about 35 per cent of the "loosely filled in" volume.

Levels are measured at upper end of inlet. (Gage "Inlet", fig.54).

TABLE 2

Ice runs. Water discharge = 150 m^3/s

Scheme (fig.21)	ICE DISCHARGE m^3/s (loosely filled)			
	14	28	48	55
1	Q_{st} = 100 243.3 too low 243.8 good Q_{st} = 110 Q_4=0 243.85 high but acceptable	Q_{st} = 100 243.5 - 243.8 good	Q_{st} = 0 244.3 too high 243.8 good Q_{st} = 75 243.6 good	
2 A	Q_{st} = 100 244.2 too high Q_{st} = 115 243.5 clogged Q_{st} = 110 Q_4=0 243.65 good	Q_{st} = 100 243.65 good		
3		Q_{st} = 100 243.8 good		
4 + 8		Q_{st} = 100 243.8 high but good		
4 + 9		Q_{st} = 100 243.8 good		

TABLE 3

Ice runs. Water discharge = 250 m^3/s

Scheme (fig.21)	ICE DISCHARGE m^3/s			
	14	28	48	55
1	Q_{st} = 210 Not good	Q_{st} = 200 243.4 too low 244.3 too high 243.8 - 244.0 best conditions	Q_{st} = 165 244.3 too high $Q_{st}=Q_{BC}$ = 0 243.5 too low Q_{st} = 150 244.0 good	Q_{st} = 150 244.0 good
2 A		Q_{st} = 200 244.6 too high 244.0 acceptable		
2 B		Q_{st} = 200 244.6 clogged at inlet 244.3 clogged at inlet 244.0 too low		
3		Q_{st} = 200 244.2 clogged at inlet		
4		Q_{st} = 200 244.15 clogged at inlet Q_{st}=200 Q_{BC} =20 244.10 good Q_{st}=200 Q_{BC} =0 244.10 acceptab.		
4 + 8		Q_{st} = 200 243.9 clogged at inlet Q_{st}=200 Q_{BC} =0 244.0 clogged at inlet Q_{st} = 180 244.1 good		

continued .....

TABLE 3/continued. Water discharge = 250 m^3/s

Scheme (fig.21)	ICE DISCHARGE m^3/s			
	14	28	48	55
4 + 9		Q_{st} = 200 244.1 clogged at inlet Q_{st} = 200 $Q_1=Q_2+Q_3=Q_4=0$ 244.0 clogged		
5 + 9		Q_{st} = 200 244.0 - 244.5 clogged at inlet		
5 + 8		Q_{st} = 200 244.10 clogged at inlet 243.60 too low		

TABLE 4

Ice run. Water discharge = 350 m^3/s

Scheme	ICE DISCHARGE m^3/sec (loosely filled)			
	14	28	48	55
1		Q_{st} = 250 244.2. too low 245.0 too high 244.5 - 244.8 good		

TABLE 5

ICE RUNS

SCHEME 6 (fig. 21)

WATER DISCHARGE	ICE DISCHARGE m^3/s (loosely filled)	
	28	55
150	Q_{st} = 100 242.80 too low 243.35 good	
300	Q_{st} = 250 244.40 clogged at inlet 244.00 clogged Q_{st} = 250 $Q_{st}=Q_4=0$ 244.30 ice through inlet 244.10 clogged 243.70 clogged at inlet Q_{st} = 225 Q_4 = 0 244.35 too high Q_{st} = 225 Q_{BC} = 0 243.90 not good	

TABLE 6

ICE RUNS

SCHEME 7 (fig. 21)

WATER DISCHARGE	ICE DISCHARGE m^3/s (loosely filled)	
	28	55
150	Q_{st} = 100 Q_4 = 0 243.5 good	
		Q_{st} = 120 Q_4 = 0 243.80 good Q_{st} = 130 Q_4 = 0 243.80 acceptable
	Q_{st} = 250 Q_3 = Q_4 = 0 244.3 good Q_{st} = 250 Q_4 = 0 244.0 good Q_{st} = 225 244.05 good	Q_{st} = 200 Q_4 = 0 244.0 good

3.3.9. Study of possible channeling of the river caused by natural deposits

The approach direction of the water and ice upstream of the jetty was considered of some influence to the ice conditions between the inlet and the jetty. This flow direction is supposed to be influenced by possible deposits in the river bed. In the introductory runs it had been found that some control of the deposition would be possible. It would therefore be of interest to find if certain deposition patterns should be preferred when looking at the ice conditions. Instead of arranging real sand banks in the model, possible channels were simulated by long rock jetties extending most of the length of the model, as shown on fig. 21, scheme 8 and 9.

Two basically different channel patterns were studied, one being a rather straight channel along the western bank of the river (photo 35 - 37), the other following the eastern bank until about 375 m upstream of the dam, running in an S-curve across the river to the western bank just upstream of the upper end of the jetty (photo 38 - 40).

The two channels did not improve the situation in the inlet area, but seemed to reduce the danger of ice bridge formation in the backwater area, due to greater velocities.

Scheme 9 forced the ice towards the inlet wall, while scheme 8 showed rather small adverse effects in the inlet area. The latter development should therefore be preferred. The difference was of little significanse, however.

The results are included in table 2 and 3, page 61 - 63.

3.3.10. Effect of flow developers

Some simple tests were run to investigate the effect of flow developers on the great eddy between the inlet and the jetty. The flow developers were simulated by hoses with mouth pieces giving water jets of similar diametres, discharges and directions as for commercially available equipment. The tests showed that the return current along the jetty can be reduced to some extent by installation of several units along the jetty.

The mixing of ice and water caused by the resulting turbulence was local and did not influence the inlet conditions.

3.3.11. Introduction of the side runaway channel. The flume model

A discussion of the versatility of the project led to the idea of making provisions for the water passing the gates to be diverted into the Bjarnalækur Canal. A channel downstream of the gated dam section was suggested, running parallel to the dam into the Bjarnalækur Canal. The channel should not be used under normal conditions, hence a gate was necessary to close the channel outlet into Bjarnalækur Canal. Under normal conditions the channel should serve as energy dissipator for the ogee of the gated dam section. The original design is shown on fig. 25 A.

A structure of this kind was too complicated to be studied in a distorted model. The function of the channel as energy dissipator could be studied in a two-dimensional sectional study, but this would not be possible for the function of the channel to conduct water to the Bjarnalækur Canal. Hence, it was decided to construct a separate model for this purpose. This model would have to inlcude the gated dam section and at least some of the structures of the sand excluder and the inlet wall,

and of course the upper part of Bjarnalækur Canal. Thus most of the structures for which the need of an undistorted model study had previously been discussed, would have to be included. It would therefore be convenient, if possible, to combine these two studies.

To simplify the water supply and tailwater arrangements it was found convenient to construct the new model in the laboratory's permanent wide flume. This flume had a total width 3.4 m, which was found sufficient for the construction of a model to scale 1:40, including all the necessary details which were likely to be studied in an undistorted model.

The model (fig. 6) was constructed according to Dwg. 290 Skc. 135 from Harza. The first visual studies gave the following results:

a. The general idea of a combined energy dissipator and an emergency side runaway channel could be adapted.

b. The front wall of the side runaway channel should be straight rather than the original design with a bend, as the straight wall reduced a tendency of eddy formation near the pier between the dam and the Bjarnalækur Sluice. Achieving this by widening the middle part of the channel was found to be better than to narrow the western half of the channel.

c. Attempts to raise the crest of the channel front wall above el. 240.0 were not found advantageous.

d. The proposed cross section of the channel was found to be good. A vertical front wall was found to give definitely worse conditions, and a 3:1 sloping wall also apparently was no improvement compared to the proposed 4:1 slope.

e. With regard to the normal operation of the Bjarnalækur Sluices a symmetrical design of the plunge pool should be preferred. The proposed design involved formation of a great eddy when discharging over the crest as well as through the bottom sluices.

f. When discharging from the side runaway channel into the plunge pool, little or no benefit was found from the proposed inclined position of the side channel gate and sill to the axis of the Bjarnalækur Sluice structure, as compared to a position parallel to this axis.

These preliminary results led to a redesign of the side runaway channel as shown on Dwg. 290 Skc. 158, fig. 25 B.

3.3.12. The "hog trough"

Attempts to further improve the ice passing conditions led to installation of a trough-like flume along the inlet wall, discharging into the Bjarnalækur Canal. The flume was supplied with an overflow crest along the whole length of the inlet ports, supposed to skim off the ice from the surface of the water. The bottom of the flume was kept about at the crest level of the sill of Bjarnalækur Gate, well above the upper edge of the inlet ports. All the discharge through Bjarnalækur Gate had to pass through this flume. The first flume arrangement in the model is shown on fig. 26 A and photo 41.

The arrangement proved very promising. The first tests with total discharge 250 m^3/s and 28 m^3/s of ice, diverting 200 m^3/s to the station, showed no tendency for ice intrusion into the diversion canal. In these tests the trough was used simultaneously with two of the ordinary dam gates, but it was soon found that the trough alone would take care of the ice under many conditions.

With the flume installed it was found that a more inclined approach direction for the ice towards the inlet could be accepted, reducing the need of the jetty. However, the trough worked well with the jetty, too. By closing the Bjarnalækur gate,the dam gates could be operated as if the trough did not exist. A design with both the jetty and the trough would therefore represent a very flexible solution.

The preliminary studies of the trough idea led to the design of Dwg. 290 Skc. 158, shown on fig. 26 B. This trough design was also built into the undistorted flume model.

3.3.13. Initial construction study

3.3.13.1. Introduction

A number of various ideas for a reduced design as a first stage of the construction was tested in Model I. During these runs the inlet area was as shown on fig. 24, with the inlet ports located between el. 235 and 240. The trough was not installed during the tests described in sections 3.3.12.2 - 6.

3.3.13.2. The top of Bjarnalækur Sluice structure at el. 240.50. The top of the rest of the dam at el. 241.75

The maximum fractions of the total discharge which could be diverted to the station were studied, and results are given in table 7, page 71.

TABLE 7

INITIAL CONSTRUCTION STUDY. WATER DIVERTED THROUGH THE INLET FOR POOL LEVELS BELOW EL. 240.50.

Q_{total} (m^3/s)	100	200	300	400
$Q_{station\ max}$ (m^3/s)	54	94	134	172
Percent of water diverted	54	47	45	43

Increasing the pool level above el. 240.50 gradually reduced the available discharge to the station, primarily caused by water lost to Bjarnalækur Canal, later also by increased discharge over the main dam.

The fractions of the ice diverted to the inlet area are given in table 8.

TABLE 8

INITIAL CONSTRUCTION STUDY. ICE DIVERTED TO THE INLET AREA FOR POOL LEVELS UP TO SLIGHTLY ABOVE EL. 240.50 (ICE DISCHARGE 28 m^3/s).

Q_{water} (m^3/s)	100	300
Water diverted (per cent)	54	45
Ice diverted (per cent)	34	36

The various possibilities of ice treatment were observed visually:

a. The station kept in operation at pond level below el. 240.50, all the ice diverted to the excavation being passed into the pond:

   The head drop between the river and the excavation caused the ice to be well mixed into the water. A small float of ice accumulated in front of the inlet as long as the edge of the inlet ports were below the water surface.

b. The station kept in limited operation at pool level above el. 240.50:

   This situation allowed some of the ice diverted to the excavation to be passed over the Bjarnalækur Sluice structure. Due to the extensive mixing of ice and water in the excavation, a very great part of the water must be passed into the Canal to reduce the ice intrusion into the pond significantly.

c. The station closed:

   Apparently no ice troubles.

3.3.13.3. The top of the dam at el. 241.75 across the whole river

This represented a slight improvement as compared to the previous situation. The approach velocities to the excavation were still supercritical, however, when water was diverted through the inlet, and ice contents entered the excavation in suspension.

3.3.13.4. The western 100 m of the dam at el. 241.75, the rest of the dam at el. 242.50

Compared to the previous situations more water was diverted to the inlet area, providing comparatively more water for flushing purposes, and also reducing the head drop towards the inlet. Results are given in table 9.

TABLE 9

INITIAL CONSTRUCTION STUDY. SECTION 3.3.13.4.
Q_{ice} = 28 m^3/s

Q_{tot}	150	150	300
$Q_{station}$	75	50	90
El. at inlet	242.45	242.2	242.50
Comments	Some ice trough inlet. Ice cover in front of inlet	Good	Estimated 10 - 20% of ice through inlet

3.3.13.5. The western 100 m of the dam at el. 242.50, the rest of the dam at el. 243.25

Results from these runs are given in table 10, page 74.

TABLE 10

INITIAL CONSTRUCTION STUDY. SECTION 3.3.13.5.
Q_{ice} = 28 m^3/s

Q_{tot}	150	150	300	300	300
$Q_{station}$	75	50	100	150	200
El. at inlet	242.85		243.25		243.20
Comments	Great ice-float in front of dam and inlet	Acceptable	Good	Acceptable. Ice float at inlet	Estimated 15-20% of ice through inlet. No water over eastern dam section

3.3.13.6. A jetty construction included in the initial construction stage

This was only tried for a situation similar to the situation in 3.3.13.4. The jetty caused markedly worse ice conditions, increasing the approach velocities to the excavation so much that a great part of the ice was brought into suspension.

3.3.13.7. The inlet arrangements, including the trough, and the Bjarnalækur Canal and sluices completed, but the dam substituted by a temporary rock fill jetty

The situations A 1 and A 2, shown on fig. 27, have been tested for a water discharge 150 m^3/s with 28 m^3/s of ice added. With the trough crest at el. 243.0, situation A 2 seems to be advantageous to A 1 because of less tendency of stagnation in the continuous ice cover. Both situations show satisfactory performance when 75 m^3/s is diverted to the station. Situation A 1 was also tested for 100 m^3/s to the station with good results, but a tendency of accumulation of ice made this situation unsafe.

Situation A 2 was finally tested with trough crest at el. 242.75 and 75 m^3/s to the station. Compared to the crest at el. 243.0 this was a definite improvement.

3.3.13.8. The inlet and the trough completed, but the Bjarnalækur structures postponed, the trough thus being out of function. 80 m of the weir completed without installation of gates. The rest of the dam substituted by a temporary jetty

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Three various jetties were tested, two of them are shown on fig. 27 (B 1 and B 2), the third had an intermediate position. A summary of the results is given in table 11 (page 76), (only 28 m^3/s of ice was tested).

TABLE 11

INITIAL CONSTRUCTION STUDY. SECTION 3.3.13.8.

SIT. B 1:	Q_{tot} = 150 m^3/s Q_{st} = 70 " Weir crest el. 242.0	Some ice into pond. Small ice float in front of inlet.
	Q_{tot} = 150 m^3/s Q_{st} = 50 " Weir crest el. 242.0	Slight improvement compared to Q_{st} = 70
	Q_{tot} = 300 m^3/s Q_{st} = 75 " Weir crest el. 242.0	No ice accumulation at inlet. Some ice into pond.
	Q_{tot} = 150 m^3/s Q_{st} = 70 " Weir crest el. 242.5	Ice float greater than for el. 242.0. Very little ice into pond.
SIT. B 1:	Q_{tot} = 150 m^3/s Q_{st} = 70 " Weir crest el. 242.0	No ice intrusion. Large ice float in front of inlet.
Intermediate location of jetty:	Q_{tot} = 150 m^3/s Q_{st} = 70 " Weir crest el. 242	No ice intrusion. Less ice accumulation than for Sit. B 2.
	Comparison of Q_{tot} = 150 m^3/s Q_{st} = 70 m^3/s and Q_{tot} = 300 " Q_{st} = 140 " showed very little difference in ice conditions.	

3.3.14. Modification of the inlet design

Installation of the trough required some structures to support it. It was first suggested to extend the division walls between the inlet ports to serve as supporting walls. However, studies in the main model as well as in the flume model showed that the flow pattern through the inlet ports was not quite satisfactory, as the water had to turn from its original direction nearly parallel to the inlet wall when passing through the ports. In addition to adverse influence on the flow pattern, this also apparently reduced the effective area of the inlet ports, causing higher velocities through the ports than originally expected.

The inlet structure was originally designed with provitions for a possible gate arrangement,to be able to close the inlet ports under certain run conditions, requiring that the structures should be able to sustain a head difference from one side of the inlet to the other. This gate arrangement was later substituted by a proposed separate control structure in the diversion canal. Substitution of the division walls between the ports by columns became then possible.

Check runs in the models showed that columns instead of the division walls would improve the flow conditions, in particular if the alignement of the diversion canal was also modified towards a design more inclined to the river bank than before.

3.3.15. Sand runs

Having now arrived at a design which in its main features was supposed to be the final, a series of sand runs was carried out to check that the sand deposits would not seriously affect the operation of the project. It was also desired to establish guide lines for the best operation of the plant under various flow conditions, to reduce the sand problems to a minimum.

Referring to photos 42-63, the run conditions and the results can be summarized as follows:

Run 1 a: No pegs in the model. Total discharge 600 m^3/s, 250 m^3/s to the station. The Bjarnalækur flap gate closed. The other gates and the fixed dam crest at el. 244.5. The bottom sluices open (95 m^3/s). Sand added at a rate of 15 l/min.

1st day: 2 hours run (1.75 with sand addition). Photo 42 shows dried model.
2nd day: (Continued from 1st day) 6.5 hours run (5 hours with sand addition). Photo 43.
3rd day: (Continued)5.5 hours run (4.5 hours with sand addition).Photos 44 - 46 give final results of flood period.

Run 1 b: Discharge reduced to 200 m^3/s (stepped down during interrupted runs over several hours), 120 m^3/s to the station, the rest (80 m^3/s) through the bottom sluices. Rapid erosion in the inlet area, resulting in development of a channel system in the backwater deposits from run 1 a. Photo 47 shows the result of 3.5 hours steady run (plus the step down period).

Run 2 a: No pegs in the model. Total discharge 1000 m^3/s, other conditions as for run 1 a.

1st day: 6 hours run (sand 4 hours). Photo 48.
2nd day: 2.5 hours run (sand 2.5 hours).
3rd day: 6.5 hours run (sand 3.75 hours). Photos 49 and 50.

Run 2 b: Discharge reduced in steps to 250 m^3/s, with 150 m^3/s to the station, the rest through the bottom sluices. Photos 51 - 54 show the situation after 9.5 hours run including 2 hours step down period. Photo 55 shows the result of 0.5 hour additional run with discharge 250 m^3/s, with 200 m^3/s to the station.

Run 3 a: Pegs in the model (20 per m^2), other conditions as for run 2 a.

The result after similar 3 days run as for run 2 a is shown on photos 56-58. (The pegs were temporarily removed when the pictures were taken).

Run 3 b: The pegs replaced in the model. The total discharge stepped down to 250 m^3/s and the discharge to the station stepped down to 150 m^3/s, during 2.5 hours run. Photos 59 - 60 show the result. Photos 61 - 63 show the final result after 6 hours further run with total discharge 250 m^3/s and 150 m^3/s to the station.

Runs 1 a and 2 a showed very similar deposition patterns in spite of the great difference in discharge over the dam crest (250 and 650 m^3/s respectively) while the discharges to the station and through the under-sluices were about the same.

Comparison of runs 2 a and 3 a shows that the latter got somewhat thinner deposits, probably due to increased turbulence in the flow and thereby increased capacity to carry sediments. It was observed that the propagation of the sand front was more rapid with pegs in the model, giving a flatter deposition pattern through-out the whole run period.

Comparison of runs 1 b, 2 b and 3 b shows the common result that rather flat sand deposits will be eroded to a channelled or braided river bed soon after the discharge and the water level are being decreased. Resulting channel systems may vary (compare run 1 b and 2 b with rather similar test condition, the latter with slightly greater discharge, however). Whether the difference between the results of runs 2 b and 3 b is due to the pegs or simply shows alternative deposition patterns, is not quite clear from these runs. It was found likely that the pegs had some effect, however.

3.3.16. Detail studies

3.3.16.1. The sand excluder

The sand excluder described in section 3.3.2 consisted of two separate conduits. Based on hydraulic computations, a design with three conduits was later proposed and tested in both model I and model III. As in the original design, the entrances to the three channels were evenly distributed along the length of the port section of the inlet, and given a design which was likely to trap most of the sand carried into the excavation in front of the inlet. The two original bottom sluices in the Bjarnalækur structure had consequently been substituted by three gated sluices discharging into the plunge pool below the ogee. Fig. 28.

The ability of the sand excluder to trap and carry away the sand was tested with various discharges in the river and in the Bjarnalækur Canal, and for various amounts of sand to be trapped. For simplicity the sand was dumped by hand just upstream of the excavation at rates exceeding conditions which were likely to appear in the river. As long as the bottom sluices were open, there was no tendency of clogging, and the sand excluder immediately removed any amount of sand carried into the excavation, only negligible amounts of the very finest particles were carried into the diversion canal.

Photos 64 - 65 show the result of two runs with the following specifications:

Sand added:	2 litres/min (model data)
Period:	2 - 3 hours
Discharge to station:	250 m^3/s
Discharge through bottom sluices:	95 m^3/s
Water level at inlet:	244.8
Total discharge:	Photo 64: 600 m^3/s
	Photo 65: 900 m^3/s

In the 900 m^3/s run, a significant part of the sand passed over the dam gates, though much less than through the sand excluder. Similar results were also obtained from a few runs in model III.

The sand excluder was also tested with only one or two of the bottom sluices open. This of course led to clogging of the closed conduit and to some sand intrusion into the diversion canal, but the situation did not appear dangerous to the operation of the plant. By reopening the closed gates, sand which had settled down in or in front of the closed conduits was soon carried away.

The sand showed no tendency to deposit in the plunge pool or in the Bjarnalækur Canal when all three bottom sluices were open. When discharging through only one or two of the bottom sluices, some deposits gathered in the outlet of the closed conduits.

The bottom sluices can only be continuously used during flood periods with enough excess water available. It was found likely that some sand might gather in the excavation during low flows, clogging all three conduits. To get an idea of the ability of the sand excluder to clean itself under such conditions, the entrance to the conduits and the area in front were

filled with sand and well packed. After having filled the model with water again, the bottom sluices were suddenly opened, which in all cases resulted in reopening of the sand excluder. Not only the sand used for ordinary sand runs in the model was tested, but also a graded material of composite masses with poor permeability and good packing properties.

2.3.16.2. The side runaway channel

Two series of detail runs concerning the side runaway channel were requested.

A study of the longitudinal section of the channel and its flow capacity without overtopping of the crest of the front wall was carried out in model III. A design of the channel with maximum capacity without overtopping about 75 m^3/s was first found, see fig. 25 C.

A capacity of 100 m^3/s was then requested. A number of various elevations and slopes of the channel bottom were tested, and also various crest elevations of the sill of the side channel gate at the confluence of the side runaway channel and the Bjarnalækur Canal, resulting in the design of Dwg. 290 S 34 R 1, see fig. 25 D.

Having found a satisfactory design for the channel to function as emergency conduit for water to the Bjarnalækur Canal, its function as an energy dissipator was rather poor. A sectional study of the channel was therefore arranged in a 15 cm wide flume to study the influence from the shape of the cross section on the energy dissipation. The two-dimentional model, as previously described in section 2.7.4, was made to scale in 1:40.

Six test series have been run, dealing with:

1. The design of Dwg. 290 Skc. 158 (fig. 25 B) with variations.

2. Ice conditions in the channel and some means to improve it.

3. Variation of the channel width, based on design Dwg. 290 S 35 R 1 (fig. 25 D).

4. Pressures on the concrete apron downstream of the channel.

5. Sand deposits in the channel.

6. Influence of downstream river bed elevation on the conditions in the channel.

Results are given in tables 12 - 17, page 86 - 91.

Photos 66 - 81 show some characteristic situations.

On photos 66-77, the downstream bottom has been kept at el. 240.0 approximately, partly to simulate a scour or excavation in the present bed, partly to compensate for the lowering of the prototype tailwater due to the three-dimensional spreading of the flow in the prototype. Photos 78 - 81 show some runs with the bottom at el. 241.0 for comparison.

The photographs demonstrate the different flow conditions for 6 and 12 m wide channel sections, and for the flap gates in upper and lower positions.

Photo 70 indicates the effect of a baffle to direct the flow along the surface for low discharges, compare table 13.

The following conclusions were deducted from the study:

1. The channel design of Dwg. 290 S 35 R 1 has a satisfactory effect as energy dissipator under most flow conditions. A wider channel will improve the energy dissipation slightly, particularly for flood discharges.

   Variation of the channel bottom within el. 237.0 and 239.0 has only slight effect on the energy dissipation.

   Excavation of the downstream bed level below the existing el. 241.0 (approx.), has some influence on the water level in the channel provided the crest of the downstream channel wall and the concrete apron are kept at el. 240.0. The character of the flow conditions is, however, nearly invariant.

   Raising the crest and apron to el. 241.0 makes the water level in the channel independent on excavations giving downstream level below el. 241.0.

   Keeping the downstream crest of the channel at el. 240.0 seems slightly beneficial to the flow conditions as compared to el. 241.0.

2. For most flow conditions a submerged roller appears in the channel. This is always the case when the gate is in upper or intermediate positions. Provided the gate can be lowered below the point where its influence on the discharge ceases, the flow conditions in the channel can to some extent be controlled by using the gate as a guide vane. For a narrow range of gate positions, the flow will then shoote across the channel as a surface current, while all other gate positions will maintain the roller.

3. Negative pressures on the downstream slab have not been found.

4. Certain flow conditions will cause some sand to deposit along the upstream wall of the channel, the wider the channel or the coarser the sediments, the more deposits. The deposits do not tend to fill a greater part of the channel, and can more or less easily be removed by manipulating the gates during low flows.

5. Referring to item 2 above, conditions with a roller in the channel are not beneficial to ice transport across the channel. Some ice accumulation is likely if the gates cannot be lowered to create a surface current. However, it may be possible to control the situation by interrupted operation of the gates to clear the channel when necessary.

6. Some improvement of the ice conditions of the design of Dwg. 290 S 35 R 1 can be obtained by introducing a rather large baffle along the downstream face of the ogee. The baffle must have an upper horizontal face at el. 240.25 and reach to a point 8 m from the gate hinge to be effective for all gate positions.

TABLE 12

SIDE RUNAWAY CHANNEL

SERIES 1: VARIATION OF THE DESIGN OF Dwg 290 Skc 158

Bottom width of the channel = 9 m

Bottom el. of channel	Crest el downstr. of chan.	Gate crest elevation	Discharge per gate (m^3/s)	Water levels		REMARKS
				Upstream	Downstream	
236.0	241	242.50	250	245.80	244.50	Ondulating flow shooting across the channel
		242.50	150	244.80	243.50	Smooth flow from the channel
		243.50	150	245.50		Ondulating, rough flow across the channel
		244.50	150			Smooth flow from the channel
		244.50	50	245.50	242.50	Good energy dissipation
		Varied	50			Good energy dissipation
	240	Varied	50			Good energy dissipation
		Varied	150			Less energy dissipation than for channel crest el. 241.0
		242.50	250	245.80	244.50	Ondulating flow across the channel
		244.50	250			Ondulating flow across the channel
237.0 238.0	Varied	Varied	Varied			Variation of the bottom level has slight effect on energy dissipation

TABLE 13. SERIES 2: ICE CONDITIONS IN THE CHANNEL AND SOME MEANS TO IMPROVE IT
Desired conditions: <50 m^3/s per gate: Surface flow across the channel
>50 m^3/s per gate: Energy dissipation

SIDE RUNAWAY CHANNEL

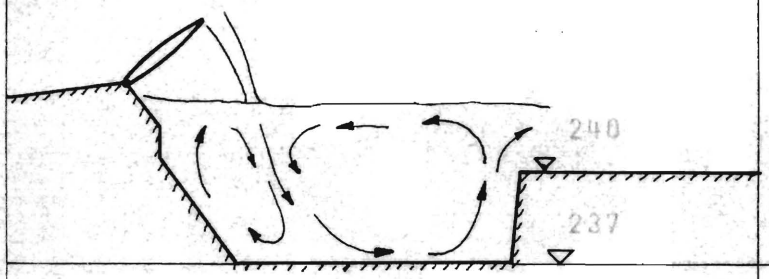
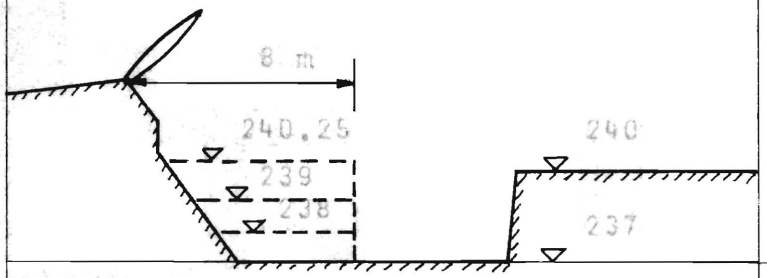
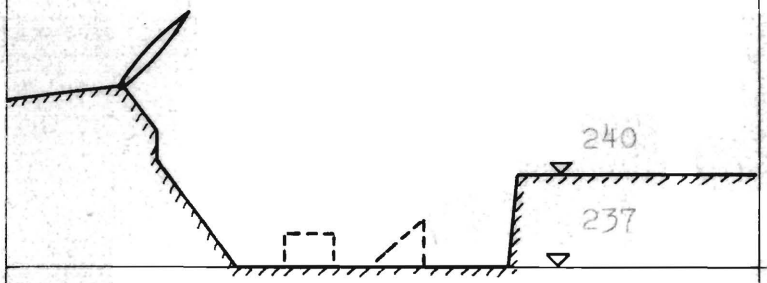
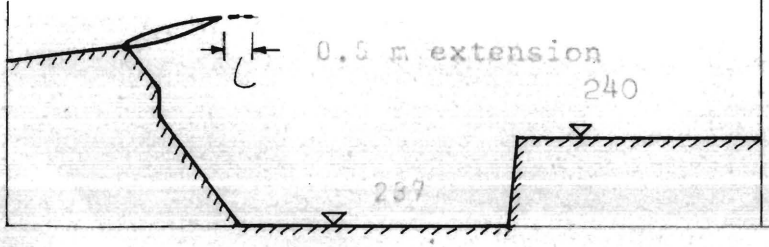
	The design of Dwg 290 S35 R 1: Varied: Tailwater level (242.5-244.0), gate elevation, width of channel (6 - 12 m) Basically similar current patterns (as sketched) obtained for all conditions, except for certain low positions of the gate producing a shooting current across the channel, beneficial to ice transport
	Various baffles in the channel intended to promote ice transport across the channel for low discharges and various gate positions: Baffle to el. 240.25 gives approximately the desired effect. Lower baffles are ineffective, causing a roller downstream of the baffle. The length of the baffle must be about 8 m measured from the hinge of the gate to function properly for various gate positions.
	Various types of baffles on the bottom of the channel: Little or no effect on the flow conditions
	Extension of the flap gate by 0.5 m: The ice conditions were slightly improved when the gate was in low positions, but the change had no effect when the gate was in higher positions.

TABLE 14

SIDE RUNAWAY CHANNEL

SERIES 3: VARIATION OF THE CHANNEL WIDTH

Bottom of the channel at el. 237.0. Design of Dwg 290 S35 R 1

Discharge per gate (m^3/s)	Downstream crest el. of channel	Gate crest elevation	Water level downstream	Most favourable width (m)	R E M A R K S
250	240	242.50	241.50-242.50	18	The tailwater level influences the conditions in the channel
	241		241.50-242.50	18	Level in the channel independent of the tailwater level
150	240	242.50	241.50-242.50	13	Conditions influenced by the tailwater level
	240	244.50	241.50-242.50	16	
	241	Varied	241.50-242.50	13-16	Level in the channel independent of the tailwater level
50	240	Varied	241.50-242.50	7 or greater	Good conditions for most situations
	241	Varied	241.50-242.50		
				Testing of average width as design	
250	240	242.50	242	10	Acceptable conditions. Water level in channel approx. el. 243.0
	240.50	242.50		10	Worse conditions than with crest el. 240.0. Water level in channel higher than 243.0
	241			10	

TABLE 15

SIDE RUNAWAY CHANNEL

SERIES 4: Pressures on the concrete slab downstream of the channel.
The pressures have been measured in point 0.8, 2.8, 6.0 and 10.0 m from the crest of the channel, marked No 1, 2, 3 and 4 respectively.

Slab elevation 240.0. Bottom of the channel at el. 237.0

Bottom el. downstream of slab	Width of channel bottom (m)	Discharge per gate (m^3/s)	Tailwater level 36 m downstream of the gate	Measured pressures ($tons/m^2$)				
				No 1	No 2	No 3	No 4	
~ 240.5	6	50	241.5	1.0	1.4	1.45	1.45	Gate at lowest position
		150	242.0	1.15	1.9	1.8	1.9	Gate crest el. 244.5
		250	242.2	1.25	2.3	2.6	2.2	
		400	242.8	0.2	2.9	3.0	3.0	
	12	50	241.5	0.9	1,3	1.4	1.3	
		150	241.5	1.4	1.8	1.2	0.9	
		250	242.0	1.9	2.2	2.7	1.8	
		400	242.8	2.2	3.5	3.5	2.9	
240.0	12	50	241.2	0.7	0.8	1.0	1.0	
		150	241.2	1.4	1.65	1.3	1.0	
		250	241.8	2.0	2.4	2.4	2.2	
		400	242.8	2.3	3.5	3.6	3.2	

TABLE 16

SIDE RUNAWAY CHANNEL

SERIES 5: SAND DEPOSITS IN THE CHANNEL

Bottom of the channel el. 237.0. Slab el. 240.0.
River bed downstream el. 240.5

Width of channel bottom (m)	
6 m	Flood discharges form a deposit along the upstream wall of the channel when the gate is lowest. Reduction of the discharge to 50-60 m^3/s per gate causes most of the sand to disappear. Manipulations with the gates more or less clean up the channel in any case. Fractions tested are 1-2.5, 2.5-4.7, 4.7-9.4 and 1-9.4 mm. The coarsest fractions cause most deposits and are most difficult to remove.
12 m	Deposits appear for discharges greater than 200 m^3/s per gate, and disappear for discharge about 50 m^3/s per gate. Manipulation with the gates may keep the channel clean also with greater discharges. Deposits of the coarser fractions (4.7-9.4 and 1.0-9.4 mm) are more difficult to remove than with 6 m width of the channel.

TABLE 17

SIDE RUNAWAY CHANNEL

SERIES 6: INFLUENCE OF DOWNSTREAM RIVER BED LEVEL ON THE ICE CONDITIONS IN THE CHANNEL

The river bed varied between el. 240.0 and 241.0

The discharge varied up to 50 m^3/s per gate

El. of downstream crest of channel	R E S U L T S
241.0	The conditions in the channel independent on the downstream bed level
240.5	The conditions in the channel independent on the downstream bed level
240.0	The water level in the channel decreases with the downstream bed level. The ice conditions do not vary significantly

3.3.16.3. The Bjarnalækur Canal

The Bjarnalækur Canal will be blasted through basalt rock and is supposed to be unlined in its whole length except for the expanded section at the upper end including the plunge pool, where the walls are supposed to be concrete lined.

The character of the basalt rock is such that it may be eroded in places where strong turbulence or pressure fluctuations occur. The flow conditions in the upper part of the channel were therefore studied in a separate model, previously described in section 2.7.4. Possible pressure fluctuations were too rapid to be studied with the equipment at hand, but the character of the flow was studied by means of dyes, which made the stream lines of the flow visible, and velocity measurements to study the distribution of the flow. The results are given in fig. 29 - 40. Except in the plunge pool just below the Bjarnalækur Gate, the flow conditions in the channel seemed smooth, and no further attempts to study the tendency of erosion were found necessary.

Photos 82 - 85 from model III show the flow conditions in the plunge pool when discharging only through the bottom sluices. Discharge also over the flap-gate improved the flow picture.

The situation when discharging from the side runaway channel is shown on photos 86 - 87.

3.3.16.4. The "hog-trough"

The first design of the trough that was tested in the model, shown on fig. 26 A, was considered rather acceptable. However, it seemed likely that improvement of the flow conditions along the trough was possible. The design of Dwg. 290 Skc. 158 (fig. 26 B) showed better velocity distribution. A test series was run in model III to further improve the conditions.

It was found that the part of the trough located downstream of the overflow section, could be modified to a design with parallel walls, 14 m apart, as shown on fig. 26 C. This modification led to a shortening of the Bjarnalækur Gate, and a narrowing of the upper part of the Bjarnalækur Canal by 4 m. Also the under-sluices had to be redesigned accordingly to maintain symmetry of the outlets.

To improve the flow in the upstream end of the trough, the designs shown on fig. 26 D - F were tested. The slope of the bottom along the overflow section was increased, and the upper part of the overflow crest was given a lower elevation than the rest of the crest as an attempt to increase the velocity in the upper part of the trough. However, it was found that only a small gap at the upper end of the trough could be left with a lower crest. A longer low section either required too much of water to keep the ice flowing over the crest along its full length or caused the ice to stick to the highest part of the overflow crest.

The elevation of the highest section of the crest was varied between 243.0 and 242.75. It was found that the highest elevation would be beneficial to most flow conditions. A lower crest was preferable for very small discharges, however. It was therefore suggested to design the overflow crest with provisions for a simple means to vary some parts or all of the overflow crest between el. 242.6 and el. 243.0 according to experience in the prototype. Alternatively a fixed lower crest elevation could be adapted with provisions to allow for possible permanent addition to the crest up to el. 243.0. The final design is shown on fig. 26 G.

Photos 88 - 91 show some situations from the tests.

3.3.16.5. Currents in the inlet area

The various designs of the excavation, the "hog-trough", and the inlet have been studied visually by means of dyes, or for the surface currents by means of the artifical ice or confetti. To obtain some quantitative data for velocities in the inlet area, micropropeller measurements of local velocities were run in model III in 4 sections perpendicular to the trough, and also in some points in the trough itself, for seven different flow situations. The velocities were measured parallel to the local flow directions found by visual methods. Fig. 41 - 47.

Confetti pictures of the surface flow were taken simultaneously for the same seven situations, photos 92 - 98.

The location of the separation line along the front of the trough were checked. All the tested situations with discharge through the trough and to the station simultaneously, showed a rather horizontal separation line located within a 1 m zone along the midline of the trough front.

3.3.16.6. The piers

The flow conditions in the vicinity of the dam piers were checked in model III for various discharges on one or both sides of the sentral pier of the gated section. Some results are shown on photos 99 - 101. The conditions were found fairly good, and no further modifications were tested.

3.3.16.7. The auxiliary water supply

To provide water to flush the Bjarnalækur Canal if the conduits of the sand excluder were clogged, an auxiliary intake for water from the diversion canal had been included in Dwg. 290 S 35 R 1 (fig. 48 A). This design was tested in

model III, and produced a rather wide eddy above the entrance, as shown on photo 102. A similar solution, but with the bottom of the entrance at el. 234.5, is shown on fig. 48 B. A strong air-sucking vortex still remained. The effect of a shelter wall (fig. 48 C) is apparent from photo 103. Several solutions of this type were tested, all with bad outcome. Finally the pocket in the channel wall was abandoned and the entrance to the tunnel was placed directly on the straight channel wall as shown on fig. 48 D. In this case the eddy did not appear as long as the entrance was located near the bottom of the channel, el. 234.5, about flush with the top of the sand excluder. By raising the bottom of the tunnel entrance to el. 240.0, the eddy reappeared. See photos 104 and 105.

All these tests were carried out with the water level in the pool as low as possible, determined by the overflow of the top crest of the gate sills, as this would mean the adverse condition for eddy formations.

3.3.17. Final design runs

3.3.17.1. Ice passing conditions

The systematic study of ice conditions described in section 3.3.8 was based on a design rather different from the final design shown on fig. 49. Consequently a similar, but less comprehensive investigation of the last design was carried out. It was of interest to see if the trough with its fixed overflow section had any adverse effects on the operation of the project. It was further of interest to see if the trough design would allow passing of small ice quantities with less water than the approximate lower limit found in the previous tests.

In the first part of the study the dam gates were closed, discharging only through the trough. The results are given in table 18, page 97 and fig. 50 - 51 and show good agreement with the previous tests. The fixed elevation of the overflow section of the crest, however, required slightly higher pool levels for small discharges. This apparently caused no troubles to the ice movement, since the velocities in the trough could easily be controlled by the Bjarnalækur flap gate.

Ice discharges down to 7 m^3/s were tested. For such small amounts the length of the overflow crest determines the necessary amount of flush water.

To utilize the small length of the Bjarnalækur flap gate as the determining crest, the water level in the trough had to be raised to at least el. 243.30, well above the overflow crest of the trough. For such high levels in the trough in particular, but also generally for small discharges in the trough controlled by the Bjarnalækur flap gate, the movement of the ice was slow and irregular with tendency of bridging. Provided ice bridging in the trough could be avoided, possibly by careful, interrupted manipulations with the flap gate, it was possible to pass small amounts of ice with less flush water than previously found for a design without the trough.

By closing the Bjarnalækur gate and operating the dam gates, a similar situation is obtained as previously described in section 3.3.8. A few runs for comparison of these two situations showed negligible differences. If the Bjarnalækur Canal or the trough should be clogged for some reason, it is therefore possible to run the project as if the trough did not exist, under acceptable conditions similar to those found before the installation of the trough.

TABLE 18

FINAL DESIGN. ICE RUNS.

Water dis-charge	ICE DISCHARGE m^3/s		
	7	28	55
100	Q_{st} = 85 243.00 acceptable 243.15 good	Q_{st} = 60 243.23 good 243.60 acceptable Q_{st} = 70 243.15 excellent	
150	Q_{st} = 130 243.25 good	Q_{st} = 110 243.25 good Q_{st} = 100 243.80 not accept. Q_{st} = 50 243.80 excellent	Q_{st} = 80 243.50 good
250	Q_{st} = 180 243.40 good	Q_{st} = 150 243.55 good Q_{st} = 160 243.30 accept. 243.82 not accept.	

TABLE 18/continued

Water dis-charge	ICE DISCHARGE m^3/s		
	7	28	55
260		Q_{st} = 210 / 244.22 acceptable	
270		Q_{st} = 210 / 244.05 excellent	
300			Q_{st} = 220 / 243.95 good

3.3.17.2. Runs with sheet ice

Sheet ice was simulated in a few runs by means of flat pieces of wax of specific weight 0.92. Photo 5. The area of each fragment was up to 25 m^2, and the thickness varied up to 40 cm (prototype measures). To pass ice fragments of this size required greater water depth along the gate crest than for ordinary slush ice. If the ice fragments were not too many, they could be passed by temporary lowering of the gates, either for each single fragment or for clusters of fragments gathered in front of the gates during a short period.

It was possible to pass reasonable amounts of sheet ice through the trough to the Bjarnalækur Canal, but this required a rather high water level to keep the ice from getting stuck along the overflow crest of the trough. The small water velocities in the trough under such conditions involved tendency for ice bridging and jamming in the trough, requiring constant attention from the operator.

Large amounts of ice fragments required much flush water.

3.3.17.3. The influence of sand deposits

Most of the ice experiments so far had been run without regards to possible influence from sediment deposits. An exception was the study described in section 3.3.9 of the effect of a channelling of the river caused by deposits. All the detail studies of conditions in the inlet area had been run with the original bed level of the river maintained. A test series was therefore run to study how the ice conditions would be influenced if the bed level of the approach area was raised as a cause of deposits.

The river bed of model I was covered by layers of sand and gravel to simulate the bed level raised by 1.0 and 1.5 m respectively. Repeated ice runs showed that raising the bed level increased the turbulence in front of the inlet. This could be compensated to some degree by raising the water level in the pool accordingly, but besides being limited by the upper position of the flap gates, this involved transport complications caused by reduced velocities in front of the dam and in the trough. As soon as local erosion had levelled down the deposits just upstream of the excavation, the mixing of ice and water was reduced to a degree more like the conditions with the original river bed, and acceptable ice conditions were obtained, photo 106.

Some data from the runs are given in table 19, page 100.

TABLE 19

FINAL DESIGN. ICE RUNS WITH SAND DEPOSITS

	Q	Q_{ice}	Q_{st}	Surface levels "Inlet"	Surface levels "Dam"	Comments
1.0 m sand deposits	100	28	60	243.70		Not acceptab.
		15 - 20	60	243.70		Good
		28	50	243.85		Small velocities in trough Poor condit.
		28	40	243.80		Good
	260	28	225	244.30	244.57	Some ice through inlet. Trough clogged. Not good
		28	225	243.50	244.30	Much ice through inlet, but majority through trough Acceptable
		28	200		244.30	Some ice through inlet, but else good
		28	155	243.90	244.35	No ice through inlet. Good
	300	55	100			Supercritical flow along inlet. Much ice through inlet. Not acceptab.
		55	150	244.15	244.53	Some ice intrusion. Rather good
1.5 m sand deposits	100	28	60	243.70	244.20	Good
	250	28	170	243.80	244.58	Ice through inlet.
		28	150	243.60	244.58	Ice through inlet.
		28	130			Not good.
		28	100	244.05	244.63	Some ice through inlet. Else good.
		55	100	244.05	244.63	Some ice through inlet. Else good.

3.3.17.4. The water level

Fig. 52 - 53 give stage-discharge curves for the final design. The locations of the gages are given on fig. 54.

Curves for V-2 and H-2 have been omitted for situations S.1 and S.2, as they nearly coincide with the curves for S.3 and S.4, the differences being only 5 - 10 cm for great discharges and less for small discharges.

4. DISCUSSION AND COMMENTS

4.1. Flow studies

4.1.1. The natural conditions

The flow situation in the natural river is clearly demonstrated by photo 107, showing a run in model II with a flood discharge (1500 m^3/s). The stream lines run parallel to the river banks, nearly unaffected by the slight bends. This effect is even overemphasized in the models, due to the distortion.

The conditions for rather small discharges are slightly more irregular, because slight channel-like depressions in the river bed become of some importance, see photo 108 (200 m^3/s).

4.1.2. The influence of the dam

The flow conditions upstream of the back water area of the dam were not influenced by the structures. Even in the upper part of the back water area the general direction of the flow lines was mainly parallel to the banks, but depending on the proportion of the water diverted through the inlet and the distribution of the overflow water along the dam, the stream lines near the dam were more or less bent towards the inlet. A particular situation is shown on photo 23 (total discharge 150 m^3/s, discharge through the inlet 100 m^3/s, water level at inlet 243.0). The resulting strong flow component perpendicular to the inlet was a disadvantage to ice conditions.

Widening the gated section of the dam or moving it to other locations along the dam had only slight influence on the general flow pattern as long as a major part of the discharge was diverted to the inlet.

For flood discharges, when the inlet discharge was a small fraction of the total discharge, the general pattern of the original flow conditions was more or less resumed, compare photo 17.

Except very close to the gates and the inlet the surface flow pattern was representative for the whole flow from surface to bottom.

4.1.3. Influence of jetty installations

The flow pattern could be changed more or less at will by installation of groins or jetties from the banks. Attempts to obtain different current patterns on the surface and along the bottom, at least in the back water area, beneficial to both ice and sand conditions, were not successfull.

The jetty shown on fig. 19 served to move the backwater and the transverse currents away from the dam and the inlet area, and curve the flow to an about parallel direction to the bank and the inlet wall, while the flow conditions further upstream were not much influenced. The flow now being only slightly inclined to the inlet wall meant that the water had to make a rather sharp turn to enter the inlet ports. This disadvantage was later reduced by additional excavation in front of the inlet and finally by the substitution of the division walls between the inlet ports by columns.

The jetty design involves a velocity concentration around the upper end of the jetty and just upstream of the inlet. The transition zone between the original river bed and the excavation, where separation of the flow from the bed may occur,is a source for suspended ice conditions in front of the inlet. This effect decreases with increasing size of the excavation, up to a certain limit.

Under conditions with a great fraction of the total discharge diverted through the inlet, a separation of the flow along the jetty seems inevitable, causing a return current along the jetty. This produces an eddy in the corner between the jetty and the gated dam section, even when the whole gated section is in use as spillway.

The flow concentration around the upper end of the jetty and the flow along the jetty were to some extent dependent on the shape and the length of the jetty. In general a longer jetty improved the conditions, and fairly good conditions were obtained by extending the jetty across the river to the eastern bank. Various means to reduce the great eddy were tried. The eddy was fairly consistent, however, and could not be destroyed by operation of the gates, except in such rare occations where the discharge through the inlet was only a minor fraction of the total flow in the inlet area.

Some reduction of the eddy seemed possible by use of flow developers placed along the jetty. Other ideas were roughly tested, like leaving small openings in the jetty to release small currents from the eastern side of the jetty against the direction of the eddy, or applying division walls in the basin between the jetty and the inlet parallel to the jetty. As such arrangements would involve complications to ice conditions, they were considered of little interest, however, although they reduced the eddy considerably.

For alternatives of the inlet where the inlet ports extended all the way down to the dam, a vortex in the corner between the inlet and the dam disturbed the flow conditions for the inlet as well as for the flap gate to Bjarnalækur Canal.

4.1.4. The final design

The flow conditions of the final design, fig. 49, involves the eddy previously described, as can be seen on the flow pictures, photos 92 - 98. The approach conditions to the gates and the trough are good, however, and so is apparently also the flow through the inlet. The turbulence conditions in front of the inlet depend much on the water level, but can under most conditions be controlled to satisfaction.

Due to the jetty, causing a flow about perpendicular to the dam, the flow pictures around the piers were very satisfactory, photos 99 - 101.

After the recommended revision, the inflow conditions to the auxiliary water supply were good, photo 104.

4.1.5. Downstream conditions

The river bed downstream of the dam is slightly deeper along the eastern bank. Hence the rather small discharges below the dam during future winter conditions, will follow this channel in the river. As the dam gates are on the other side of the river, a transverse current along the dam will result. For the original design (fig. 3) this was clearly demonstrated in the model, the current following the energy dissipating apron just beneath the ogee. Ice transport was therefore unfavourable, increasing the danger of a gradual accumulation of ice downstream of the dam. The conditions can easily be improved by channelling the western part of the river below the dam gates, either by excavation or by construction of a low jetty parallel to the bank.

The first design did only include a continuous concrete apron along the whole dam, serving as an erosion control measure for the dam foundations than as energy dissipator. The final design maintained the apron along the ungated section of the dam, while the apron was substituted by the side runaway channel along the gated section. Under normal conditions, this channel will serve as energy dissipator. Only the flow conditions in this channel has been studied in detail in the models.

The character of the structure and the tailwater conditions is such that very little energy dissipation can be obtained for flood conditions. Under such conditions the dam only increases the flow velocities across the dam while nearly natural flow conditions are resumed shortly below the dam, independent of energy dissipating structures. Some energy dissipation can be obtained for small discharges, though, and the side runaway channel gives satisfactory conditions. The flow in the channel when used as side spillway to discharge into the Bjarnalækur Canal, is satisfactory, and so is the flow conditions in the plunge pool and the upper part of the channel for the various situations run in the model. The bottom sluices should be operated symmetrically as much as possible.

Negative pressures have not been observed on the concrete slab downstream of the runaway channel.

4.2. Sand studies

4.2.1. The natural and model conditions

The river reach in question is not subjected to sediment deposits, except in some local areas along the banks. The sediment transport in the river is known to be extensive

during flood flows, however, but data on the sediment transport rate were not available. To be on the safe side in the model runs, a sediment rate near to the maximum transport capacity of the river had to be tried. This was achieved by increasing the feeding of sediments until some deposition occurred. The sand was then fed into the model at various smaller rates. It was found that a feeding rate about 15 litres/min could be used for various flood conditions, though this amount was much below the maximum capacity of the greater floods. The deposition pattern in the backwater was apparently rather independent on the feeding rate within wide limits, only the rate of growth was significantly influenced.

4.2.2. Model results

The construction of the dam caused the sand to deposit in the back water area. The deposition pattern was studied for a number of various discharge conditions, various gate locations and other structial details.

The general development of the deposits was nearly independent of the various modifications in the inlet area. As described in section 4.1.1., the flow lines under flood conditions were nearly parallel to the river banks. This caused the sand to form a deposit with a front nearly perpendicular to the river banks, moving slowly towards the dam while growing in thickness. The surface of the deposit would if extended down to the dam, approximately touch the dam crest. Only in the vicinity of the inlet and the gated dam section this general development was disturbed. As can be seen from photo 46 and 49, about similar conditions resulted from runs with various discharges.

Only small amounts of sand were carried across the dam until the front of the deposits had approached the dam within a distance of one to two metres in the model. The transport of sand down into the excavation started much earlier, and constituted fully developed a fraction of the total sediment transport about proportional to the fraction of water diverted to the inlet. The bottom sluices in their original design were not effective as sand excluders, and consequently the inlet was soon partially clogged by sand. This in turn led to increased velocities with ability to carry sand into the diversion canal where large deposits gradually formed. Photo 12.

When a sediment deposit had once developed during a flood period, a considerable amount of sand was carried down into the excavation by erosion of the nearest part of the deposits also for rather small discharges. This erosion produced a channel in the deposits, as a result of the lower water surface in the inlet area under such conditions. Photo 60. When this channel was fully developed down to the original river bed, further sediment transport into the excavation was small, and the flow conditions in the inlet area approximately resumed similar characteristics as without sediments in the river.

Installation of the sand excluder in the bottom of the excavation removed the problem of clogging the inlet, and only sediments completely suspended in the water were transported into the diversion canal.

Looking separately at the fraction of the sand carried between the jetty and the western river bank, most of it went to the excavation. The amount of sand carried along the jetty or passing through the excavation before being carried across the dam gates was not negligible, however, even when the discharge over the gates was small, see photos 64 and 65. This means that not all the sand carried between the inlet and the jetty during low discharges will deposit in the inlet area when the sand excluder cannot be operated.

The channel eroded in the deposits during low discharges tended to follow along the western river bank under most conditions, but sometimes a secondary channel running from near point V-2 on the eastern bank in an S-curve towards the inlet area became the main channel. It is not sure wheather the two types of channels are of equal chance or wheather each of them belongs to certain flow conditions. Photo 47 and 53 show the results of runs with 200 and 250 m^3/s of water in the river respectively. Judging only from these two runs there is a tendency towards the curved main channel alternative for increasing discharges, but the runs are too few for a certain conclusion.

The deposits in the back water area can to some extent be controlled by construction of groins extending from the banks, see photo 19. By means of such structures it should at least be possible to create a fairly stable channel in the deposits.

There is no tendency of sand deposits in the upper part of the Bjarnalækur Canal and the plunge pool, except when the discharge through the undersluices is asymmetrical. Such deposits are, however, very small and can be easily removed by changing the discharge conditions slightly. Also in the side runaway channel, there is a tendency of deposition along the upstream wall for certain discharge conditions, but also these deposits can be removed by manipulating the gates or changing the discharge conditions.

Tests in the model indicated that the undersluices possess good ability to clear itself if the conduits are clogged by deposits when not in operation, simply by opening the sluice gate. It is uncertain, however, to what extent model results of this kind can be transferred to prototype conditions.

4.2.3. Comment

Based on the model tests two alternative modes of development of the sediment conditions can be sketched, depending on the rate of sediment transport in the river.

If the average annual deposition during flood periods exceeds the corresponding erosion during low discharges, a rather stable sediment pattern will develop during one or, more probable, several years, growing during floods and decreasing due to erosion during low discharge periods. Under such conditions there may be some sense in attempts to influence the channel development by means of mechanical methods like drag line excavations in the backwater area.

The other possibility is that sediments deposited during the flood period are more or less completely removed in large parts of the deposition area during the subsequent low discharge periods. Under such conditions application of mechanical methods to influence the sedimentation must be repeated at intervals, and permanent installations may be preferable.

4.3. Ice studies

4.3.1. General results

Some general design rules for obtaining good ice conditions, found or confirmed on the model, are listed below:

a. The velocities should not be too low, to avoid irregular ice movement or ice bridging.

b. The velocities should not be too fast in areas where suspension of ice particles into the water is disadvantageous.

c. Sudden changes in the flow direction tend to pile up ice and are usually disadvantageous. In particular it should be avoided at the inlet.

d. Sudden velocity reductions are unfavourable, increasing dangers of ice bridging.

e. Accellerations usually improve the ice conditions, provided the accellerations are not obtained by a sudden decrease in the depth, in which case pure water can be drawn from the deepest layers and the ice movement in the surface retarded.

f. The necessary flush water to pass ice over a crest is not only determined by the rate of ice to be transported, but also by the necessary depth to prevent ice lumps from sticking to the crest, and the need for sufficient surface currents to carry the ice towards the gates, compare item e.

g. The horizontal pressures in the ice float in a zone of retarded motion usually lead to different flow pictures in the surface when ice is present than without ice.

4.3.2. General conditions in the river

The construction of the dam will produce a backwater area with small velocities compared to the natural conditions. As long as the ice movement is not disturbed by significant freezing processes, the general ice flow pattern will be similar to the flow described previously in section 4.1. However, certain differences appear where the flow is converging. Here the ice float either must be compressed or increased in thickness, meaning increased danger of ice bridging.

An ice bridge in the backwater area will usually grow slowly against the current until the velocities in the upper part of the backwater area are great enough to carry the ice particles under the ice float. Most of the ice particles will then be transported directly to the downstream end of the ice bridge, where some may deposit, extending the bridge downstream. Some of the ice may deposit under the ice carpet, however, increasing its thickness until the velocities are great enough to prevent further thickening. The initial formation of an ice bridge will thus usually cause a gradual filling up of great parts of the backwater area by ice.

As the tendency of ice bridging depends on the velocities,it can be avoided by keeping sufficiently low pool levels behind the dam and providing enough flush water to get rid of the ice as soon as it approaches the dam.

It is beneficial to the transport of ice, if it can be concentrated into a more narrow channel than the whole width of the river. The construction of the dam will to some extent result in a channel of this kind by the natural development of an ice situation. Ice will accumulate in the stagnant area between the dam and the eastern bank, and form a gradually increasing ice float until it covers most of the triangular area between the dam and the bank up to about the upper end of the backwater area. This channel formation can be promoted by installation of groins from the river banks upstream of the backwater area, but the most simple and effective method was found to be the construction of a jetty parallel to the inlet wall extending from the dam as shown on fig. 19.

Clogging of the eastern channel past Klofæy seemed to have little effect on the conditions in the backwater area.

Sand deposits in the river are likely to improve the ice conditions upstream of the inlet area, by producing a natural channeling of the river bed. Much will depend on the actual development of the sand conditions in each case, compare section 4.2.3. A sand deposit can only develop all the way down to the excavation under conditions with high water levels in the pool. The possibility of a rather rapid local erosion by a lowering of the pool level is therefore always present. If this can be achieved between a flood period causing such deposits and the subsequent period with ice in the river, ice problems can be treated in a similar manner with sand deposits in the river as found for the original river bed. Coincidence of sand deposits down to the excavation and ice conditions in the river will require more flush water to control the situation.

The location and width of the dam gates are of some significanse to the general ice conditions in the river. Spreading of the flush water along a long overflow crest is not beneficial, as this counteracts the formation of a relatively narrow transport channel. Also division of the flush water on two or more separated gate sections along the dam is not recommendable for similar reasons. The section of the dam nearest to the inlet is therefore the natural place for the ordinary dam gates. It is not possible to change the ice conditions significantly by drawing some water and ice through gates in the eastern part of the dam.

4.3.3. General conditions in the inlet area

The original design with its rather strong flow component against the inlet wall, as described in section 4.1, involved a tendency of ice accumulation in front of the inlet wall. The ice float forming tended to increase in thickness and size until it finally covered a large part of the inlet area

and extended down to the upper edge of the inlet openings. The flow being thus forced to dive under the ice float, carried ice directly through the inlet ports, while the hydraulic conditions for flushing of ice across the dam gates became very poor.

The rather strong current along the dam had a rather poor ice carrying effect to the eastern dam gates, as these gates sucked most of the discharge from the bottom layers of the water. Due to a vortex formation in the corner between the inlet and the dam, the Bjarnalækur flapgate was very little effective for ice flushing.

Moving the inlet away from the dam was an improvement, but rather satisfactory conditions were first obtained when the jetty shown on fig. 19 was installed. The velocities became now strong enough to carry even a rather thick ice carpet continuously between the jetty and the inlet wall towards the gates. This motion even destroyed the eddy caused by a return current along the western side of the jetty, except when the amounts of ice were small. A slight tendency of ice accumulation along the inlet wall still appeared under certain ice conditions, however.

Various attempts to remove the eddy also for small ice discharges were not successfull. Some effect was obtained by use of flow developers, however.

The installtion of the trough (fig. 26) finally removed the tendency of ice accumulation along the inlet wall.

The design on fig. 49 is the result of comprehensive tests to find the best design of details like the jetty and the excavation with the trough installed. As long as the general idea of the design is maintained, the ice conditions are not sensitive to small modifications of such details. A major

requirement to the excavation is that it should exceed a certain size to allow the ice brought into suspension by the sudden change in flow conditions at the edge of the excavation to surface before reaching the inlet wall. A very narrow excavation as the trench of the original design, also leads to vertical velocity components along the wall of such a magnitude that they directly increase the tendency of ice intrusion into the diversion canal.

The ice conditions in the inlet area depend to a large extent on the water level in the pool. Optimum conditions are obtained only within a narrow range of water stages, varying with the distribution of the discharge to the gates and the inlet. Results of comprehensive tests are given in figures 22 and 50, and summarized in Appendix A to serve a quide lines for the operators.

The flow pictures, photos 92 - 98, serve as illustration of the ice conditions for small amounts of ice. More heavy ice conditions are shown on photos 24, 31, 41 and 106.

Experiments with ice booms in the inlet area showed that a depth of 1 - 2 m was necessary to prevent ice from diving under booms even fairly parallel to the current. Deeper booms did not improve the conditions significantly. The final design of fig. 49 can in fact be considered more as a permanent boom structure than as a real inlet structure.

If sand deposits in the river cause a raised bottom level around the entrance to the channel between the inlet wall and the jetty, increased turbulence in the excavation may result, causing some or more of the ice to be mixed with the water when carried to the inlet. This effect can be avoided either by raising the water level in the pool accordingly, or by increasing the amounts of flush water, compared to the data given for the original bed. Some data for this particular situation is given in table 19.

4.3.4. Conditions at the inlet wall

The various factors influencing the conditions at the inlet wall can be characterized as follows:

The approach direction of the flow against the inlet wall is of great importance. A velocity component perpendicular to the wall causes the ice to accumulate and gradually build up an ice float in front of the wall, increasing in size and thickness until it covers a great part of the inlet area and extends down to the inlet ports. Having reached this stage, ice carried below the ice float will move directly through the inlet into the diversion canal. The less inclined to the inlet wall the flow is, the less is the tendency of ice float formation and the slower is its growth. Even a nearly parallel flow direction along the wall does not completely remove the ice accumulation, however, as long as a plane wall like the original design is maintained.

The turbulence conditions in front of the inlet are significant for the amount of ice carried in suspension into the diversion canal. For this reason it is recommended to use an excavation in front of the inlet great enough to allow the ice to surface in rather calm water before reaching the inlet structure.

The vertical velocities along the inlet should be smaller than the rising velocity of the ice particles in still water, to prevent the ice from being carried down to the ports. A widening of the excavation is beneficial to this conditions.

The distance from the dam to the nearest inlet port is of significance. With the inlet ports all the way down to the dam, complicated flow conditions appear in the corner between the dam and the inlet with a tendency to create a strong vortex (photo 20) with ability to suck much ice through the

inlet. These bad conditions are improved, but not removed when discharging over the Bjarnalækur flapgate. It was found that the minimum distance from the dam to the nearest inlet port should be 30 - 40 m due to vortex rows from thetip of the jetty. Varying the horizontal angle between the dam and the inlet wall does only slightly influence on the ice conditions. Only variations within ± 15° have been studied, however, see fig. 21.

The installation of the trough, fig. 26, to skim off the ice approaching the wall, removed the tendency of ice accumulation along the inlet completely, as long as enough water was discharged through the trough to take care of the ice. Neither was there under such conditions any tendency towards sucking ice from the water surface down through the inlet instead of skimming it to the trough. Even with a large degree of turbulence, much ice can still be flushed through the trough or over the dam gates.

The depth and size of the inlet ports are of less importance than originally expected. No adverse effects could be found from raising the upper crest of the ports from the original el. 234.5 to at least el. 237.5, without the trough installed. The installation of the trough changed the situation even more for ordinary runs. However, even without using the trough the depth of the front wall of the trough down to el. 239.1 is sufficient to avoid significant ice intrusion as long as ice accumulation in front of the wall does not appear. If an ice float has once started to grow in front of the wall, it is only a question of more or less time before its thickness will increase down to the inlet ports independent of the elevation.

The total area of the inlet ports, and consequently the velocity through the inlet, are within reasonable limits of little importance for the ice conditions.

4.3.5. Conditions at the dam

The effect of the dam for ice flushing purposes depends on various conditions:

The width of the gates in use and the discharge over the dam must be adjusted to provide sufficient water depth across the gates to avoid the ice to get stuck at the edge of the flap gates.

The depth of water immediately upstream of the dam gates is important for the ability of the gates to draw ice from the surface layers in the pool, being the most effective use of the flush water. With a nearly vertical wall upstream of the gates, a large fraction of the water passing the gates will be drawn from deeper layers containing very little ice, while a sloping wall, or better, a horizontal bottom about at the original river bed level, gives much better ice passing conditions. In general the approach velocities to the dam should not be too small.

The desire of a shallow area in front of the dam can be obtained by moving the inlet ports and the excavations in front of the inlet away from the dam as in the recommended design fig. 49. The shape of the excavation is of some importance to avoid stagnant areas causing uneven ice approach to the gates.

To ascertain maximum of ice content in the flush water, an approach direction of the flow as much perpendicular to the gates as possible should be preferred. The location of the gates along the dam has some influence on these conditions, as a spreading of the gates will result in weaker transverse current along the dam than in the original design. Spreading the flow across the river without widening the total width of the gates means adverse tendency of ice bridging, however. The jetty solution solves the problem in a much easier way.

4.3.6. Downstream conditions

Only the conditions immediately downstream of the dam and the Bjarnalækur sluice have been tested in the models.

The transport capacity for ice in Bjarnalækur Canal is sufficient for any quantities of slush ice used in the model. Under conditions as tested in the model, the ice shows no tendency to accumulate in the plunge pool, neither when discharging over the flap gate nor when using the side channel gate in an emergency situation.

A few tests with sheet ice showed that large ice floes combined with protruding irregularities in the canal surface represent a danger for ice jam formation.

The various provisions for flushing additional water to the canal represent a safe guard against complications especially in the plunge pool because of the wide variety of flow conditions which can be created.

The ice conditions below the gated dam section are not studied in detail. The natural river bed downstream of the dam causes the ice to move across the river to its deepest area along the eastern bank of the river. If desired, this eastward transverse flow in the river can easily be avoided by digging a shallow channel downstream of the gated dam section or constructing a low jetty downstream of the dam from the eastern end of the gated section parallel to the river banks. Structures of this kind have not been studied in the model, however.

The flow conditions in the side runaway channel, when used as energy dissipator for flow down the river, are not particularly good for ice transport, as some of the ice will stay for a shorter or longer time in the roller created in the channel, particularly for small water discharges. If this

should create a dangerous situation, however, it is always possible to manipulate the gates temporarily to remove the roller and clear up the channel.

The side runaway channel seems to work well in its function as an emergency arrangement.

4.3.7. The development of an ice situation

A period of ice transport in a river can generally be divided into three stages. Based on experience from the model study, the following probable development can be sketched for the prototype.

The first stage, beginning with an ice free river, is characterized by a gradual filling up of ice in areas of the river with stagnant water or very small velocities. The ice carpet forming in such areas will act as border for the areas with freely flowing surface ice, and gradually increase until the ice flow is confined to fairly stable channels in the river.

The second stage of the ice situation is characterized by the stable conditions resulting from the first stage.

The third stage of the ice situation is characterized by a gradual reduction of the ice transport in the river, either due to increasing temperatures ceasing the ice production, or possibly temporary ice accumulation further upstream in the river. This stage may also include the breaking up of the ice carpets of the stagnant areas, bank ice etc.

The complete ice runs in the model have included all three stages, with the emphasis of the study placed on the threat of the second stage.

The development of the first stage may vary considerably. Usually this period of an ice situation will involve less problems than in a fully developed ice situation. As some of the ice will be accumulated in the stagnant areas, the amount of ice to be flushed across the dam will be less than in the fully developed stage, gradually increasing to its full value. The handling of the ice during this stage will, however, greatly influence the future development.

The operation of the plant during the first period was particularly significant for the original design of the project. The rather strong flow component perpendicular to the inlet wall gradually decreased during the first stage of the ice situation and it was therefore very important to avoid extensive accumulation in front of the inlet during this period. As soon as a stagnant ice carpet in the eastern part of the river was fully developed, it was much easier to maintain a constant ice flow along the inlet and avoid extensive ice intrusion.

With the installation of the jetty, the parallel flow situation was immediately obtained, and the first stage of a run became of less importance. In the area between the inlet and the jetty the great eddy was gradually reduced with increasing ice discharge, until finally it could not be traced on the surface, and the ice was moving evenly parallel to the inlet wall across all the width of the inlet area when discharging over the gates. When discharging into the trough, however, the ice accumulating in the area between the jetty and the gates of the dam finally formed a triangular stagnant carpet, with its edge acting as a guide wall to conduct ice into the trough.

In the stable stage of a run, the moving ice forms a more or less continuous moving carpet, not only in the parts of the river with rather constant flow velocity, but also in stagnant areas and between the inlet wall and the jetty. The variation in the ice velocity is apparently less than the similar variation in the flow velocities, indicating that the horizontal pressures in the ice layer were significant.

The third stage of an ice run is mainly characterized by the gradual reduction of the rate of approaching ice to the inlet area, reducing the horizontal pressures in the ice carpet causing the velocities of the ice in the inlet area to decrease. In many runs this led to considerably worse conditions for ice intrusion than in the steady part of the run. Much of these problems disappeared, however, with the installation of the trough.

The development of an ice situation will to a great extent depend on the operation of the gates. Based on the model runs, some general rules for the operation are collected in the Appendix A. Due to the qualitative character of the study, the rules will have to be checked in the prototype and revised by experience, but they should serve as general guidelines until prototype experience is available.

4.4. Final remarks

4.4.1. About the final design

The model study has resulted in the design of the dam and inlet area shown on fig. 49. A description of the main features of this design is given in Appendix B. All the major features of the original design have been kept, but its performance has been greatly improved by the addition of a jetty parallel to the inlet, larger excavation in front of the inlet,

the trough in front of the inlet, the runaway channel downstream of the dam, and the sand excluder. Also the design of the inlet structure and the diversion canal has been modified.

The final design involves alternative ice passing procedures, and a number of measures to attack possible ice jams, as described in the table 16, fig. 55 and Appendix A.

The design is able to bypass the expected quantities of zero degree slush ice in the river.

Various preliminary construction stages have been studied. Some designs are shown on fig. 27.

4.4.2. The application of the model results

Referring to section 1, the aim of the study has been limited to qualitative results. This is a normal restriction to model studies where sediment transport is involved. In this case the problem of finding representative data for the porous ice lumps of very variable size also was a further barrier to quantitative studies.

The model results are further limited to the conditions for transport of zero degree slush ice, neglecting the influence from freezing and thawing processes.

Trondheim, July 12th, 1966

Einar Tesaker

Einar Tesaker

ET/RB

A P P E N D I X A

GUIDE LINES FOR THE OPERATOR

1. Passing of ice

1.1. General conditions

The numerous alternatives for passing of ice and attacking possible ice accumulations are visualized on the flow sheet, fig. 55, and in table 20, page 131.

1.2. Normal operation for slush ice conditions

Guide lines for the operation under normal conditions are given in fig. 50 and 51. Fig. 50 gives recommended water levels at upper end of inlet for various discharges, while fig. 51 gives the approximate minimum need of flush water for various rates of ice transport, provided the water level is in agreement with fig. 50.

If the trough is out of use for some reason, fig. 22 and 23 give similar quide lines for passing the ice across the dam gates.

1.3. Passing of ice floes

The passing of large ice floes or lumps may require more flush water than given by fig. 51 or 23. Ice floes should normally be passed over the dam gates, possibly by interrupted operation of the gates either for each floe or for clusters of floes gathered together at the dam at intervals.

1.4. Emergency operations

1.4.1. Jams in the inlet area and the trough

Rapid manipulations with one or several gates may break up accumulations of ice near the gates.

Increasing the flush water by reducing the water to the station is the most effective method to attack ice jams in the inlet area, possibly combined with alternating gate operations.

More permanent lowering of the dam gates should be avoided if possible, unless it is combined with a decrease in the discharge through the inlet to avoid lowering of the pool level. Otherwise the temporary increase of the flush water will be of short duration, and the final inlet conditions may be worse than before. Once lowered, it is very different to raise the pool level again under severe ice conditions. If it is necessary to raise the level, the gates must be operated very slowly.

Discharge through the bottom sluices has adverse effect, if any, on ice in the inlet area. Passing of ice through the bottom sluices may only be recommendable under possible hypothetic extreme conditions.

1.4.2. Jams in Bjarnalækur Canal or Creek

The following sequence of operations is recommended:

Manipulations with the flap gate.

Flushing through the bottom conduits.

If the upper parts of the conduits are clogged by sand it is possible to use the auxiliary water supply from the diversion canal.

Flushing through the Bjarnalækur Pond outlet.

Benefits only if the jam is in the creek downstream of the canal.

It is also possible to pass flush water over the dam gates and through the side channel gate.

Interrupted flushing with full head will usually be beneficial compared to continuous flushing with a reduced head, especially when attacking ice in the upper end of the canal. If necessary, additional flush water can be obtained by decreasing the water to the station.

1.4.3. Jams in the river downstream of the dam

Manipulations with the dam gates should be used, possibly combined with a reduction of the water to the power station.

1.4.4. The trough and the river clogged while the dam gates and the Bjarnalækur Canal are clear, (or a situation opposite to this)

Open the side channel gate. The discharge in Bjarnalækur Canal should not exceed about 200 m^3/s if possible, usually less, except in short periods.

2. Control of sand deposits

2.1. Normal operation

No ice in the river:

Keep the bottom sluices open whenever excess water is available. All dam gates should be put in operation before using the Bjarnalækur flapgate, possibly in order from east to west.

Ice in the river:

If not necessary for attacking ice jams in the canal, the bottom sluices should be closed, leaving possible sand to deposit in the excavation.

If the amounts of deposited sand prove to be significant, flushing through the bottom sluices at intervals may be recommended.

2.2. If flood deposits of sand develop in or just upstream of the inlet area

As soon as the discharge permits, lower the pool surface to erode a channel in the deposits. If ice appears before the channel is finished, the water level must be balanced as to give moderate turbulence and ice intrusion through the inlet, while still providing further erosion of the deposits. Some model data given in table 19, page 100, may be useful.

2.3. If the sand excluder is filled by sand while not in use

Open the bottom sluices. If necessary, raise the pool elevation. If still clogged, try to clear out the lower part of the conduits first by using the auxiliary water supply.

2.4. If sand deposits in the plunge pool of Bjarnalækur Canal

The reason is usually asymmetrical discharge through the bottom sluices, and the deposits can be cleared away by changing the discharge conditions slightly.

2.5. If sand deposits in the side runaway channel

Small deposits forming along the upstream wall of the channel during flood periods will be cleared away by a discharge of 50 - 60 m^3/s over each gate, or by manipulating the gate flaps irregularly.

Possible sand deposits will soon be cleared away by the flow when using the channel as side spillway to the Bjarnalækur Canal.

A P P E N D I X B

THE FINAL DESIGN

The design shown on fig. 49 consists of the following main structures with functions as indicated:

STRUCTURE	FUNCTIONS
1. Dam and dikes	Raise the head and divert water to the station.
2. Dam gates	Pass flood water or flush ice. Adjust the head for ice and sediment control.
3. Flash boards	Emergency measure for great floods.
4. Inlet structure	Avoid ice intrusion into the diversion canal.
5. Diversion canal	Conduct water to the intake reservoir.
6. Trench along the bottom of the diversion canal	Emergency conduit for the water in case of ice clogging the upper wide part of the canal.
7. Bjarnalækur Canal	Carry ice and sediments during normal conditions.
8. Trough	Skim ice and carry it to Bjarnalækur Canal.

STRUCTURE	FUNCTIONS
9. Bjarnalækur Flap Gate	Control the flow in the trough.
10. Sand excluder	Avoid deposition of sediments along the inlet structure.
11. Bottom sluices	Provide flush water to Bjarnalækur Canal. Control the sand excluder.
12. Auxiliary water supply	Emergency supply to the bottom sluices if the sand excluder is clogged.
13. Side channel gate	Divert water from the river to Bjarnalækur Canal or from the Canal to the river in certain situations of ice jamming.
14. Side runaway channel	Normal: Energy dissipator. Emergency: Conduct water and ice to Bjarnalækur Canal.
15. Jetty	Direct the flow to the inlet along the western bank.
16. Bjarnalækur Pond outlet	Supplementary flush water supply to Bjarnalækur Creek.

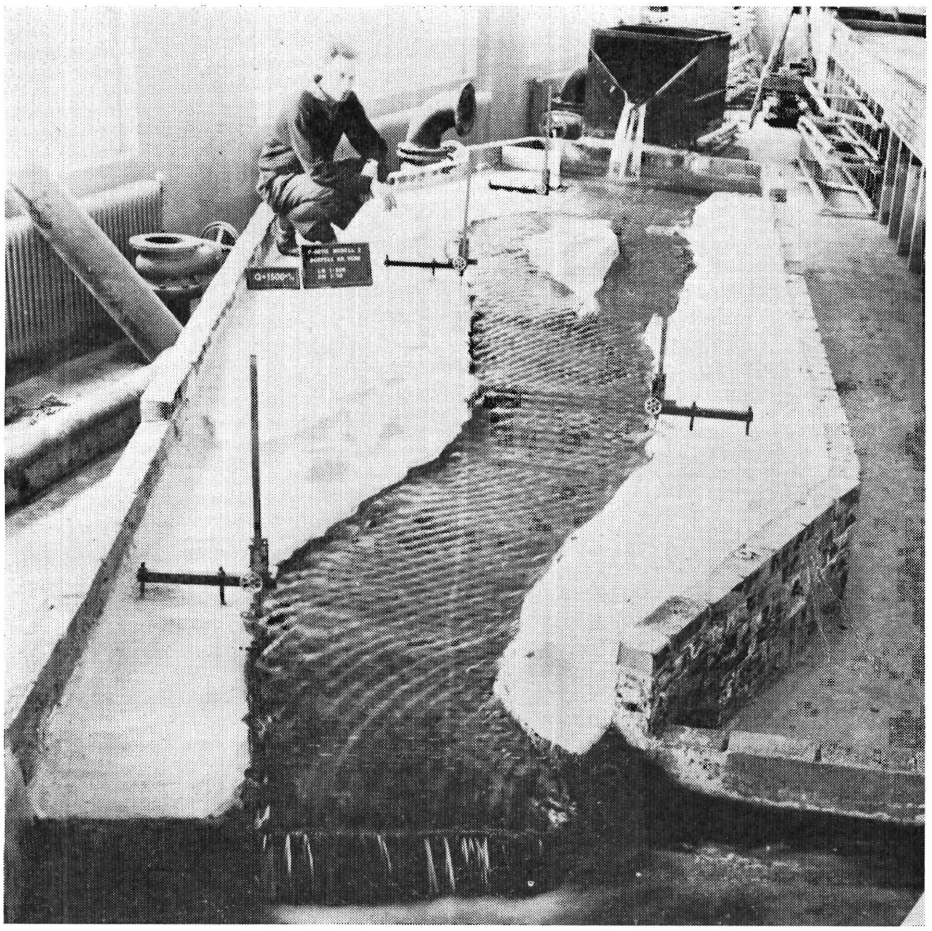
The locations of items 1 - 15 are marked on fig. 49, item 16 is marked on fig. 2.

A particular quality of the design is its versatility. Possible operations under ice conditions are drafted in table 20.

TABLE 20

ICE TRANSPORT VERSATILITY

Normal operation		Pass the ice through the trough to Bjarnalækur Canal
Supplementary or alternative operation		Pass the ice over the dam gates to the river
Emergency operations	if the trough and the river are clogged	Open the side channel gate to pass the ice over the dam gates to Bjarnalækur Canal
	if the dam gates and Bjarnalækur Canal are clogged	Open the side channel gate to pass the ice through the trough to the river
	if the river and Bjarnalækur Canal or the trough and the gates or all are clogged	Pass the ice through the inlet into the pond. (The storage capacity is limited)
Measures available for reopening of clogged ice transport facilities	The trough, the dam gates or the river is clogged	Manipulate the gates
		Reduce the discharge to the station
	The Bjarnalækur Canal or Creek is clogged	Manipulate the flap gate
		Flush through the bottom sluices
		If the sand excluder is clogged, use the auxiliary water supply
		Flush through the Bjarnalækur Pond outlet
		Reduce the discharge to the station

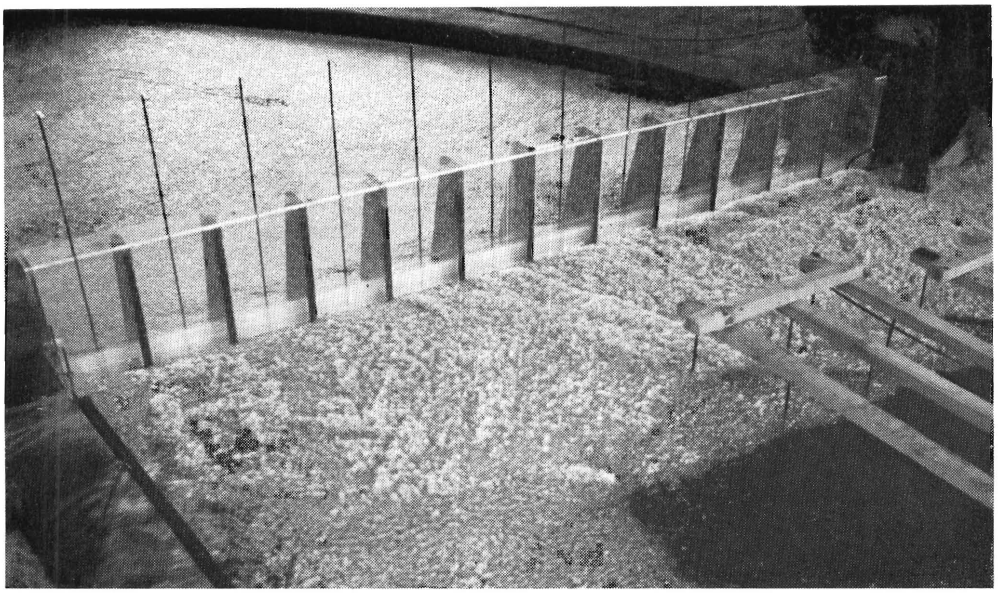


MODEL II

PHOTO 2

MODEL III

PHOTO 3

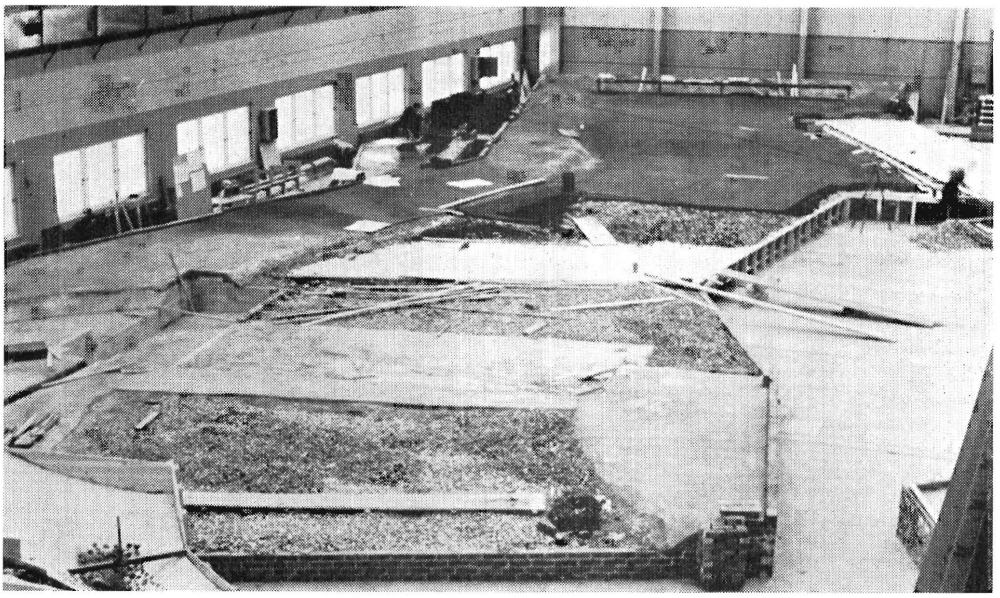


THE
PLASTIC
ICE

PHOTO 4

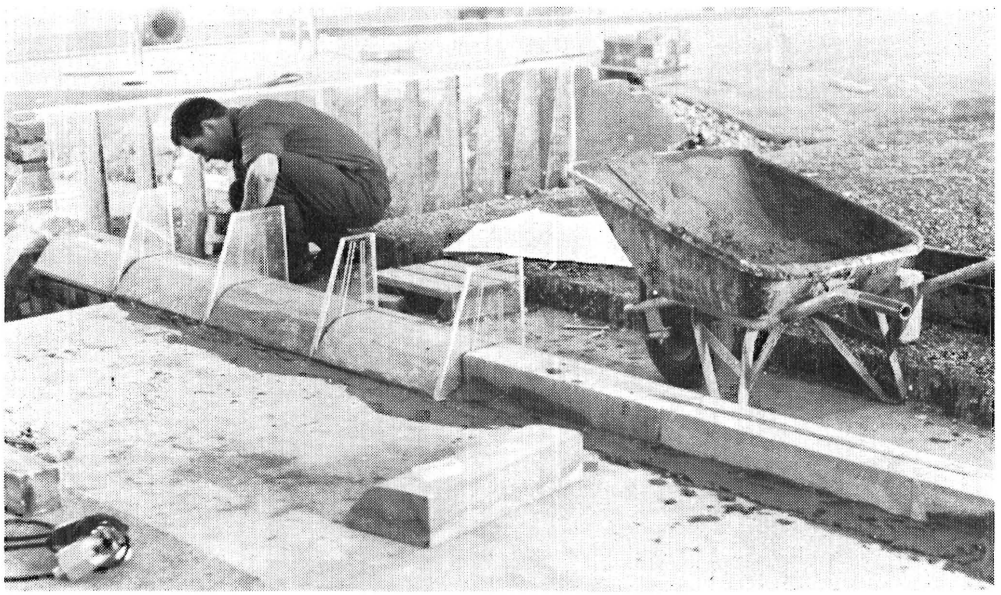
THE WAX ICE

PHOTO 5



MODEL I UNDER CONSTRUCTION

PHOTO 6

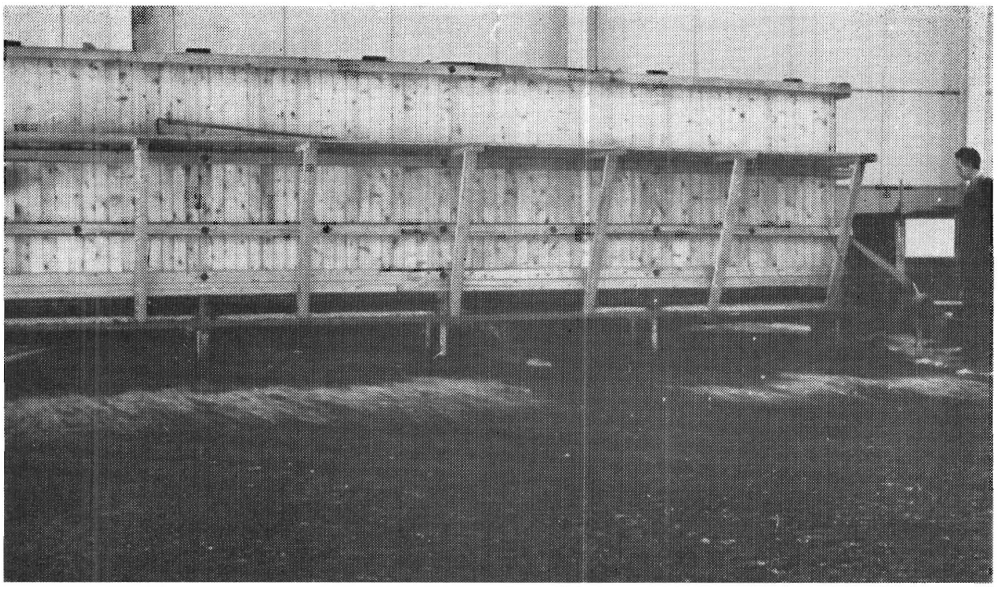


CONSTRUCTION OF DAM AND INLET

PHOTO 7

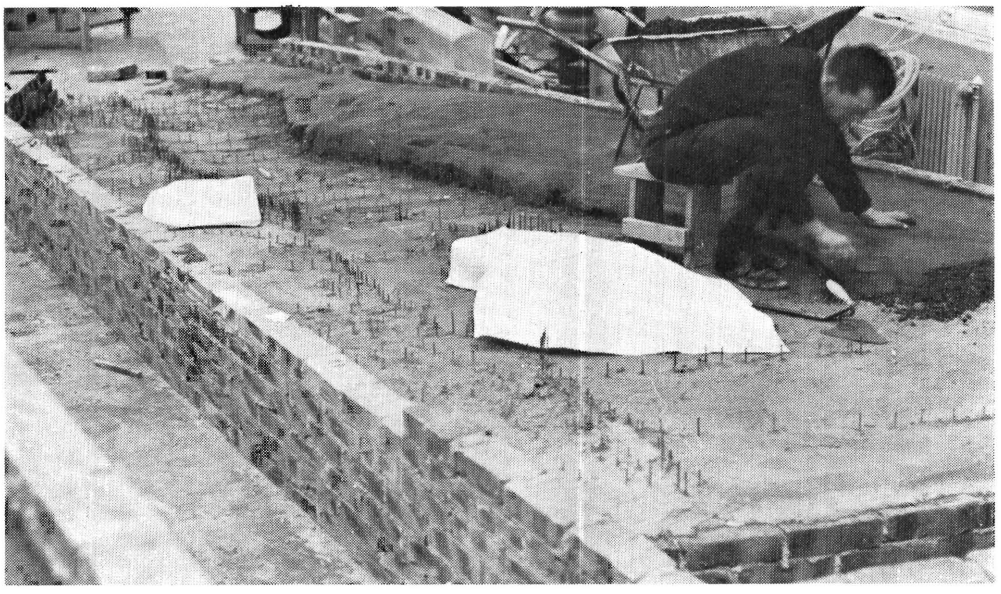
THE GRAVEL BED IN MODEL I

PHOTO 8



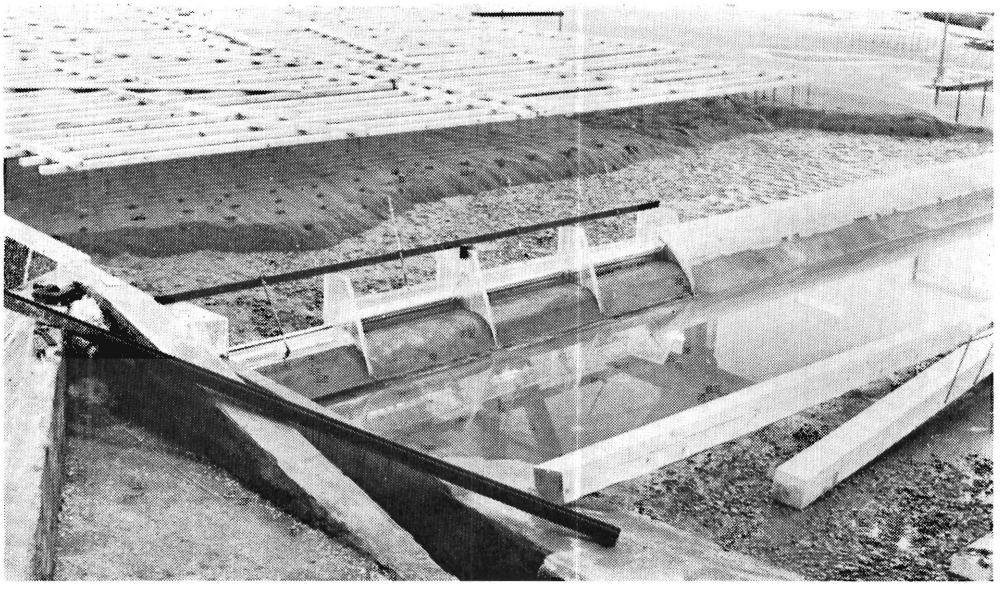
THE SEDIMENT FEEDER

PHOTO 9



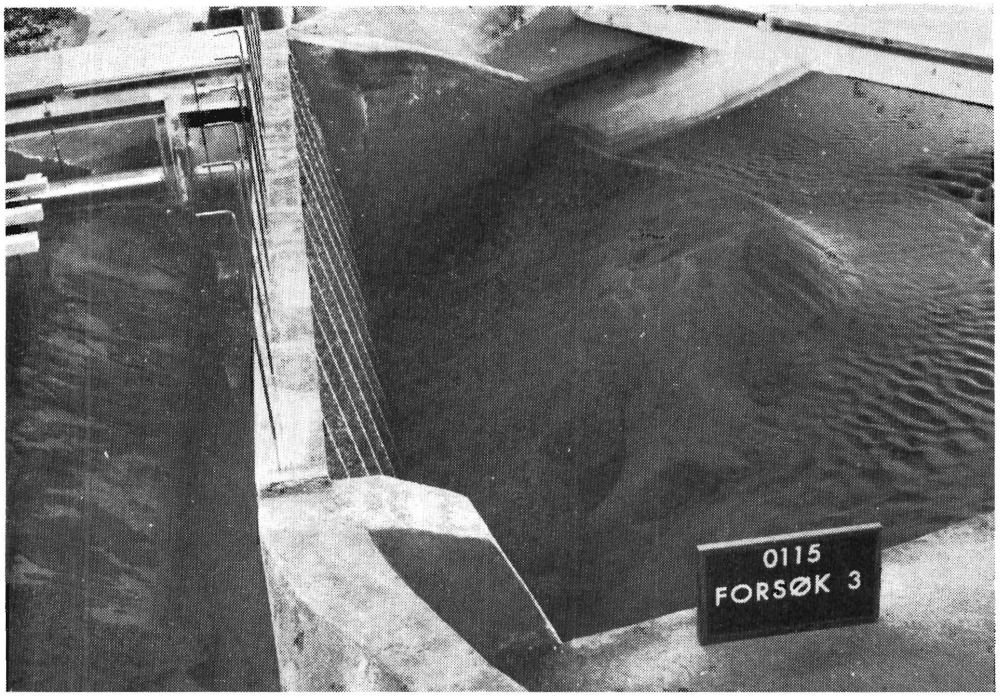
MODEL II UNDER CONSTRUCTION

PHOTO 10



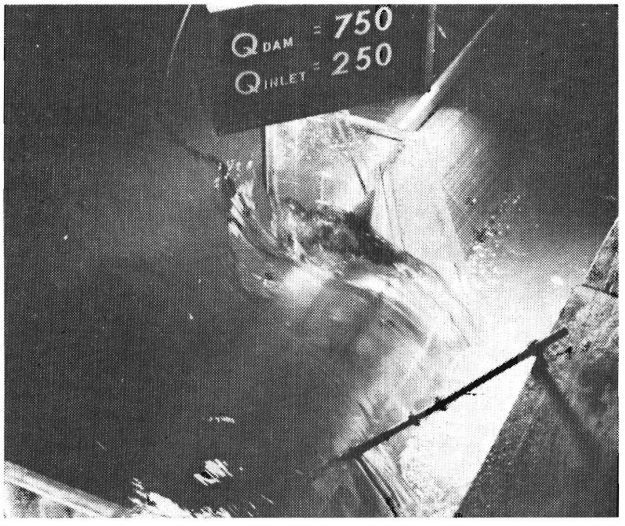
THE SAND FRONT AFTER FIRST RUN

PHOTO 11



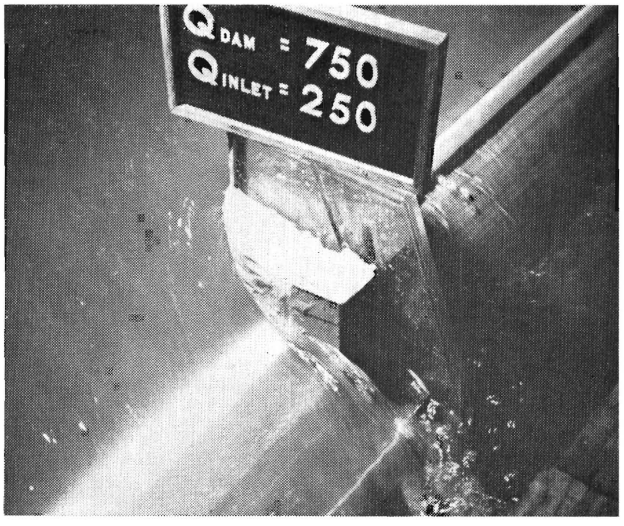
THE DIVERSION CANAL AFTER FIRST RUN

PHOTO 12



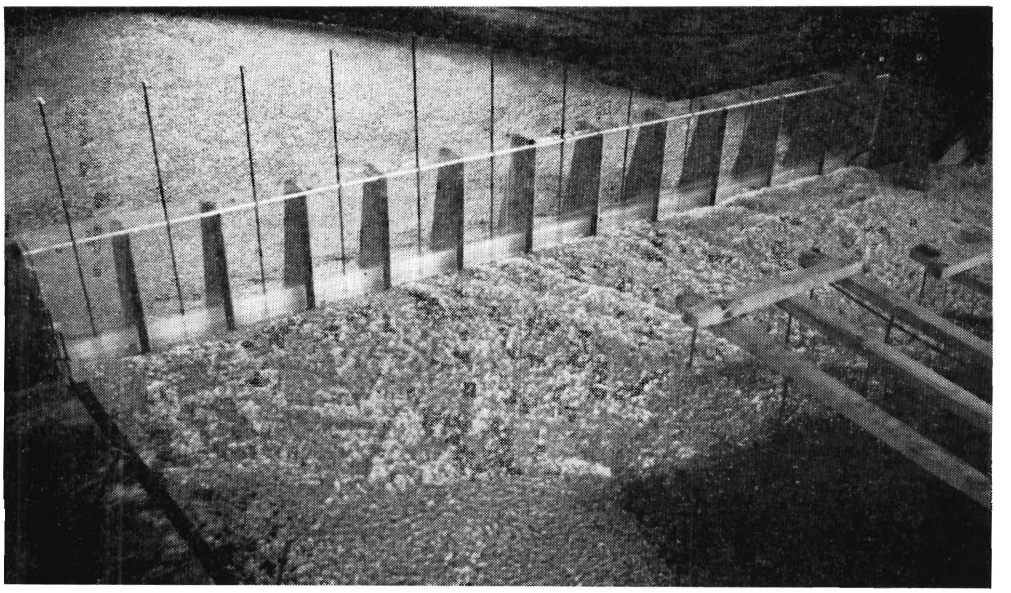
THE ORIGINAL DAM PIER

PHOTO 13



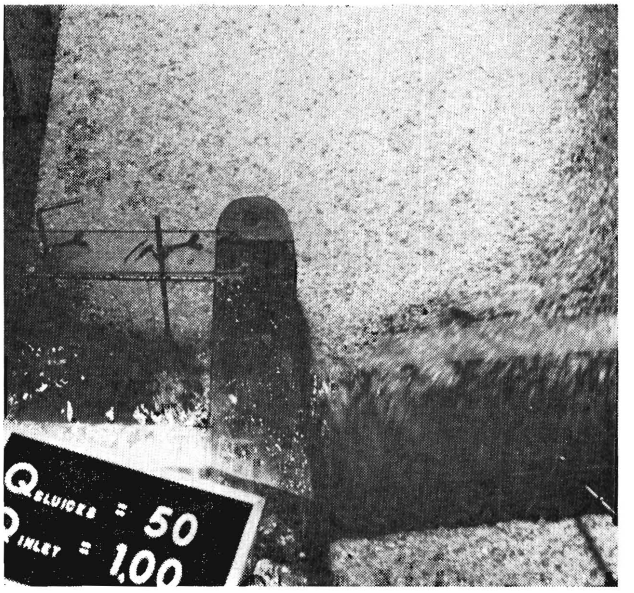
IMPROVED DAM PIER

PHOTO 14



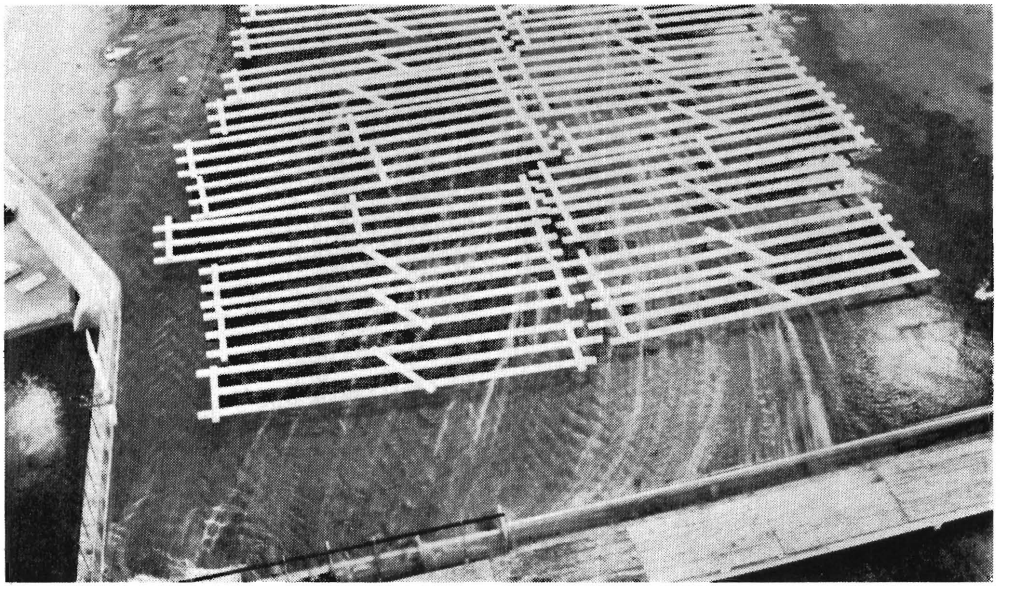
ICE FLOAT
AT INLET.
ORIGINAL
DESIGN

PHOTO 15



ICE BRIDGE AT
BJARNALÆKUR
FLAPGATE.
ORIGINAL DESIGN

PHOTO 16

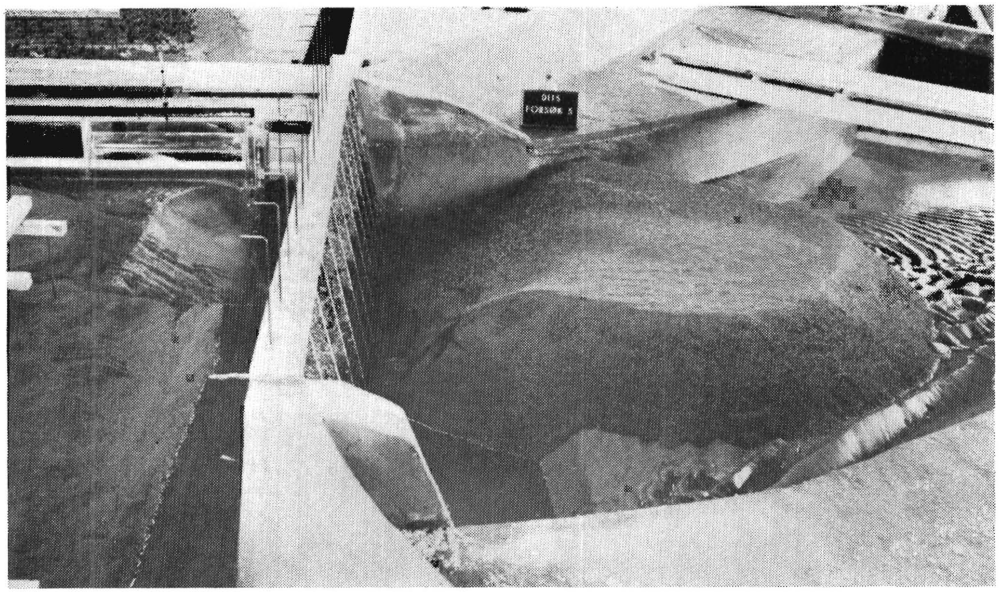


STREAMLINES

Q = 1000 m^3/s
Qst = 250 »

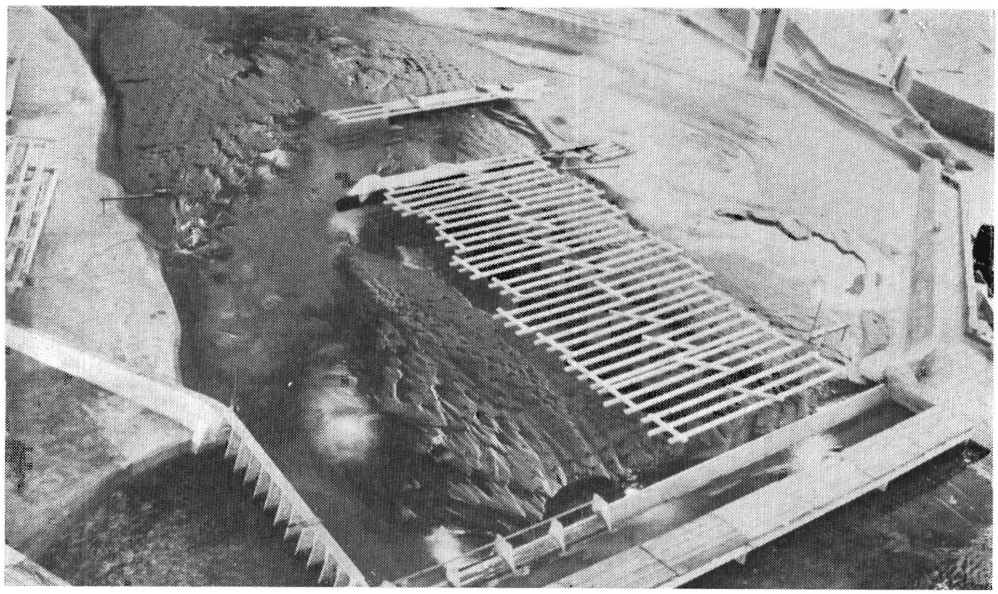
SIX LOWER
ORIGINAL
INLET
PORTS

PHOTO 17



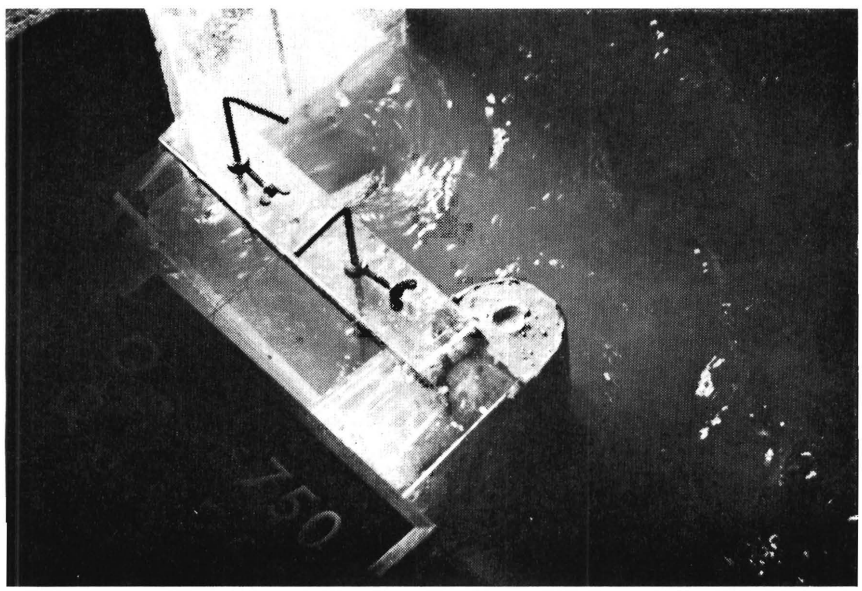
SAND DEPOSIT
IN DIVERSION
CANAL.
SIX LOWER
ORIGINAL INLET
PORTS
Q = 1000 m^3/s
Qst = 250 »

PHOTO 18



EFFECT OF
GROINS

PHOTO 19



VORTEX IN
CORNER
BETWEEN
INLET AND DAM

PHOTO 20

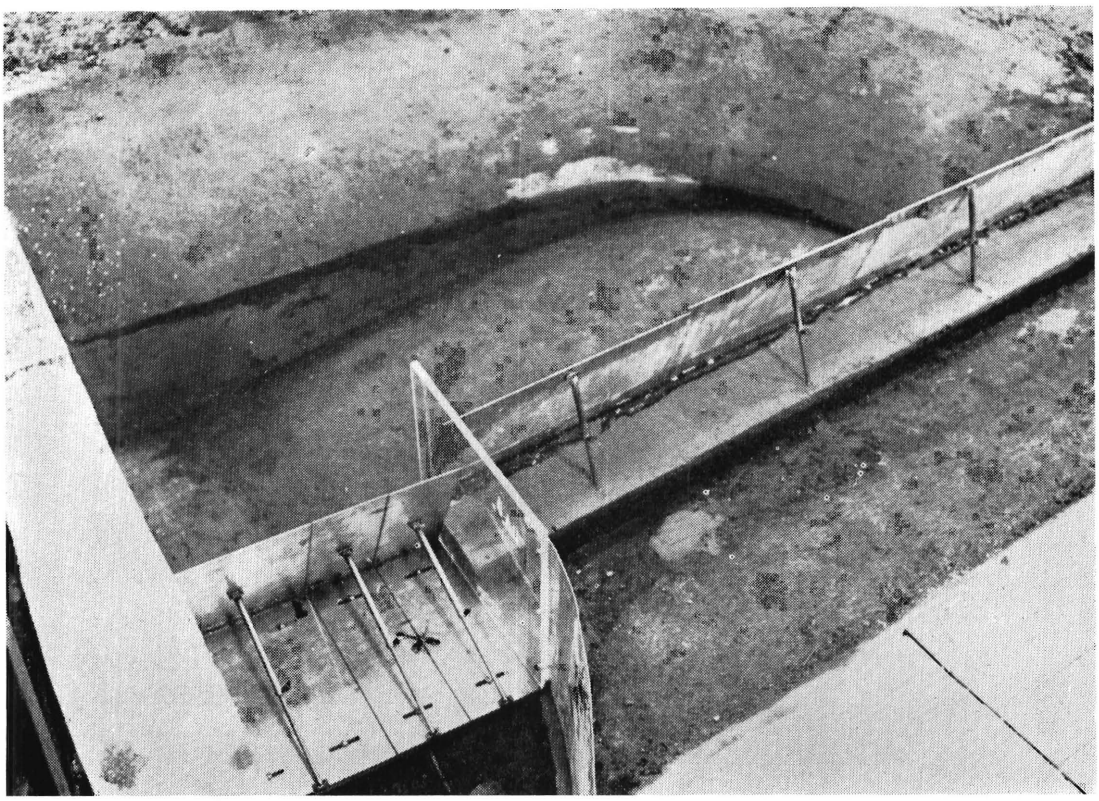


PHOTO 21

THE REVISED MODEL DAM DESIGN

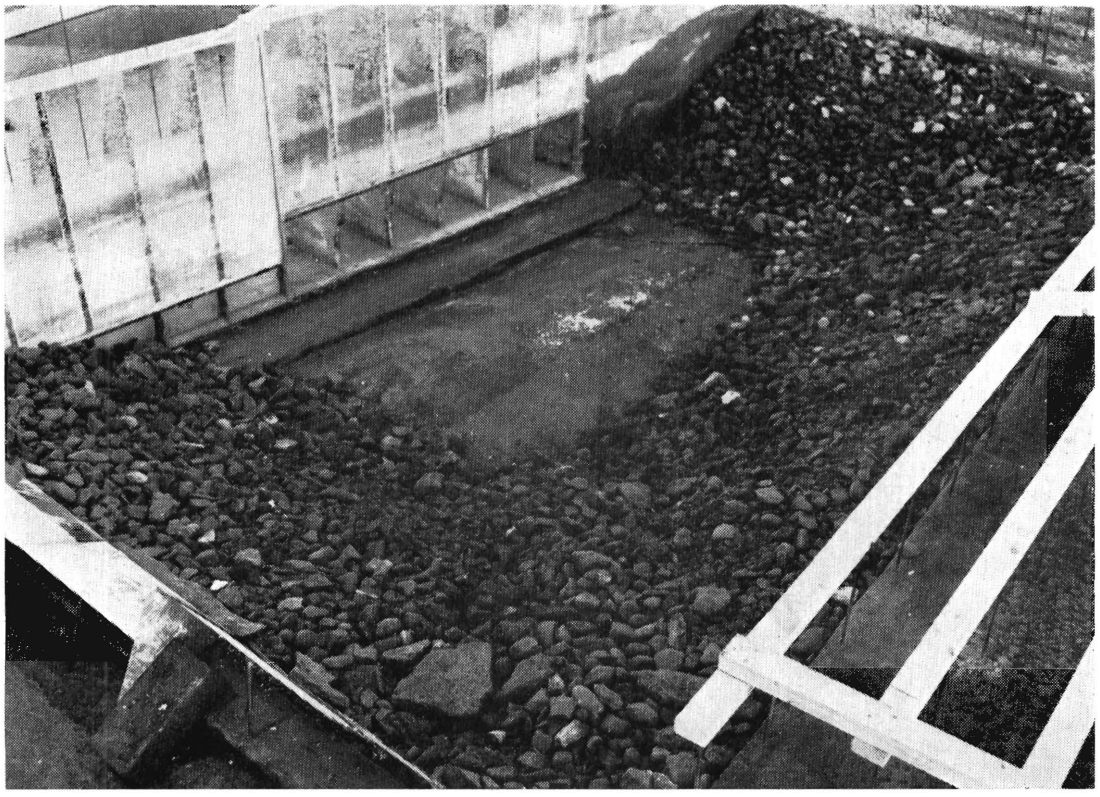


PHOTO 22

THE INLET IN UPPER POSITION

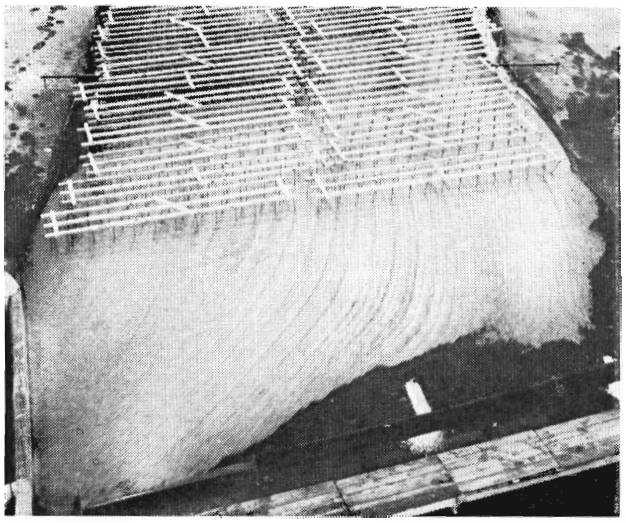


PHOTO 23

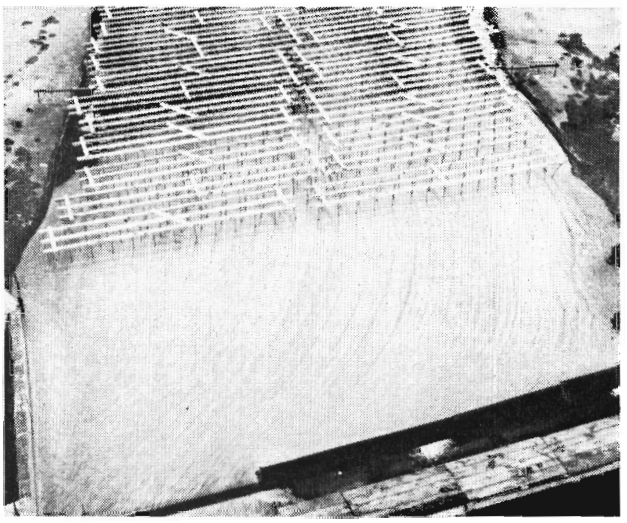


PHOTO 24

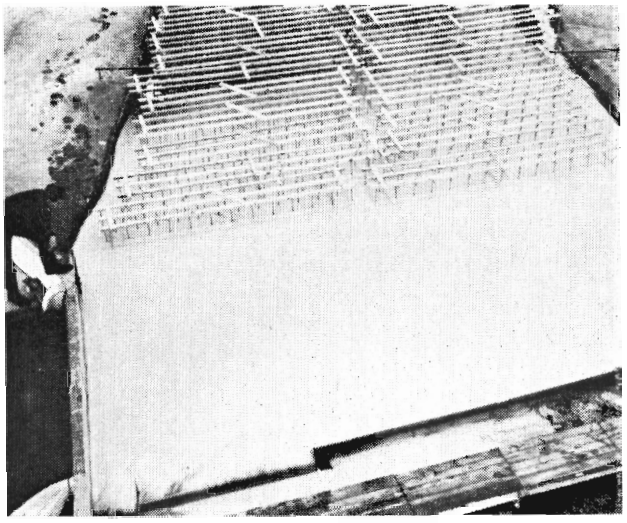


PHOTO 25

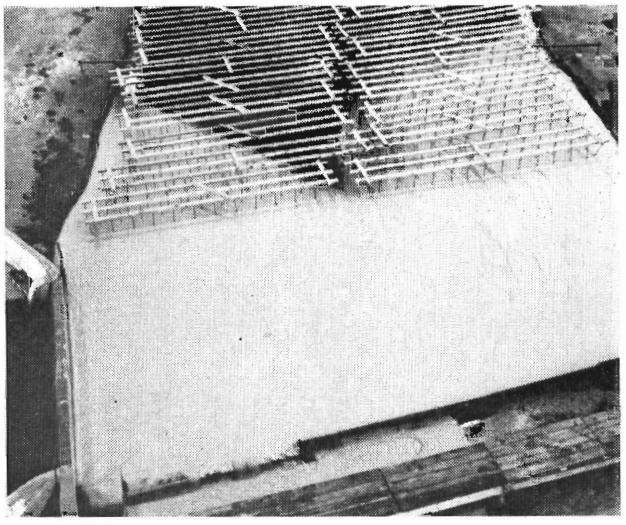


PHOTO 26

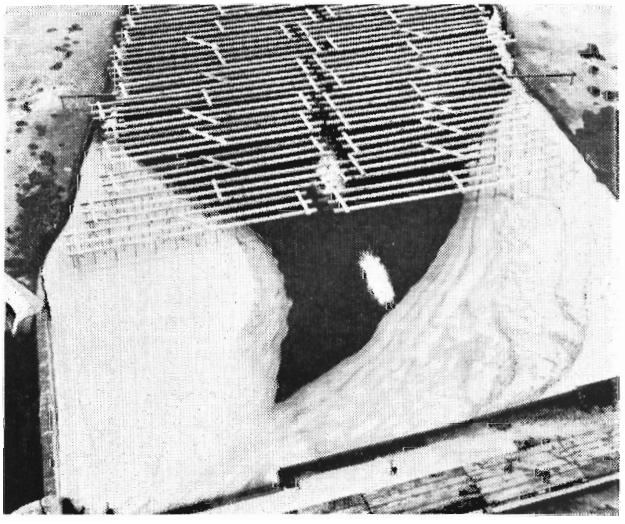


PHOTO 27

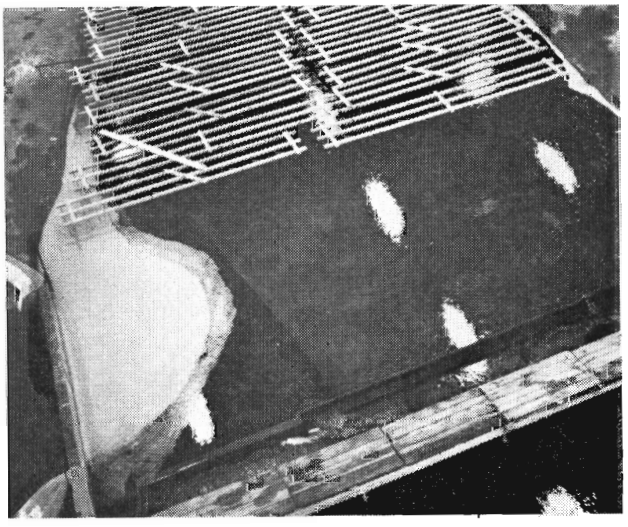


PHOTO 28

ICE RUN WITH THE DESIGN SHOWN ON PHOTO 22.

Q = 150 m^3/s Qst = 100 m^3/s Ice = 28 m^3/s

PHOTO 29

PHOTO 30

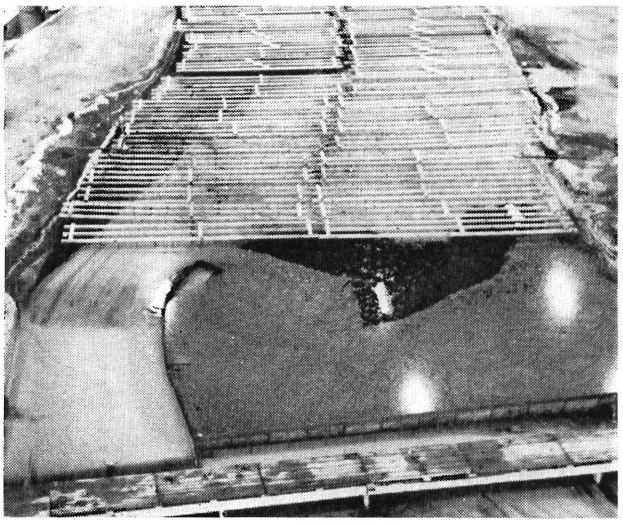


PHOTO 31

PHOTO 32

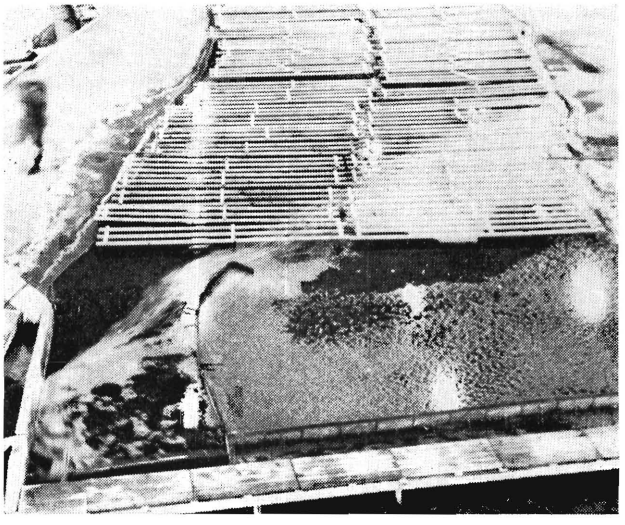
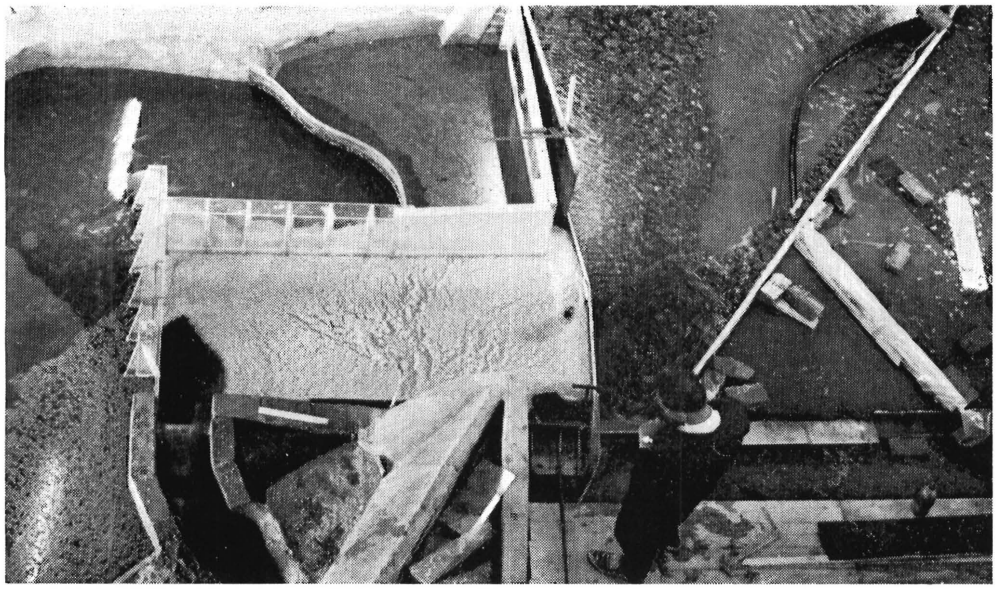


PHOTO 33

ICE RUN WITH THE JETTY DESIGN, SEE FIG. 19

Q = 150 m^3/s Qst = 75 m^3/s Ice = 48 m^3/s



RUN WITH THE INLET STRUCTURES WITHDRAWN INTO THE DIVERSION CANAL.
Q = 150 m^3/s
Qst = 100 »
Ice = 28 »

PHOTO 34

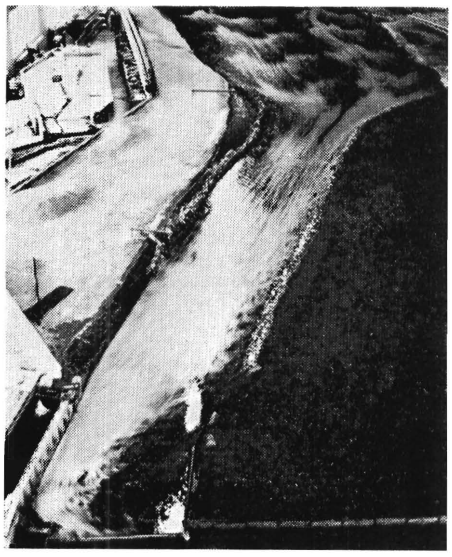


PHOTO 35

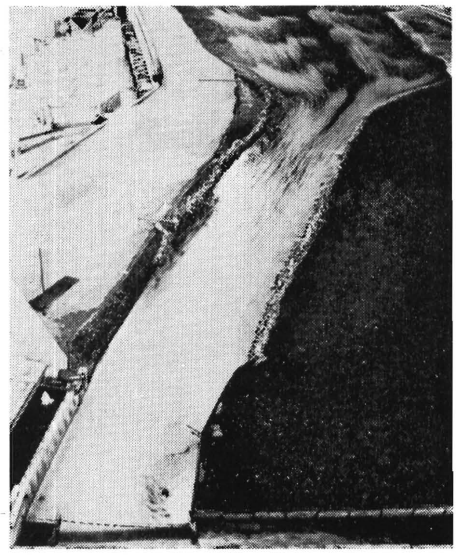


PHOTO 36

PHOTOS 35—37:

INFLUENCE OF STRAIGHT CHANNEL FORMATION

PHOTO 37

PHOTO 38

PHOTO 39

PHOTOS 38—40:
INFLUENCE OF
CURVED
CHANNEL FORMATION

PHOTO 40

FIRST RUN
WITH THE
TROUGH
Q = 250 m³/s
Qst = 200 »
Ice = 28 »

PHOTO 41

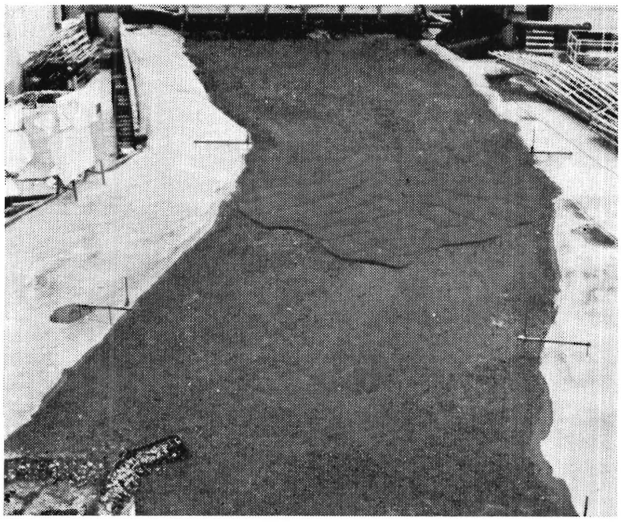


PHOTO 42

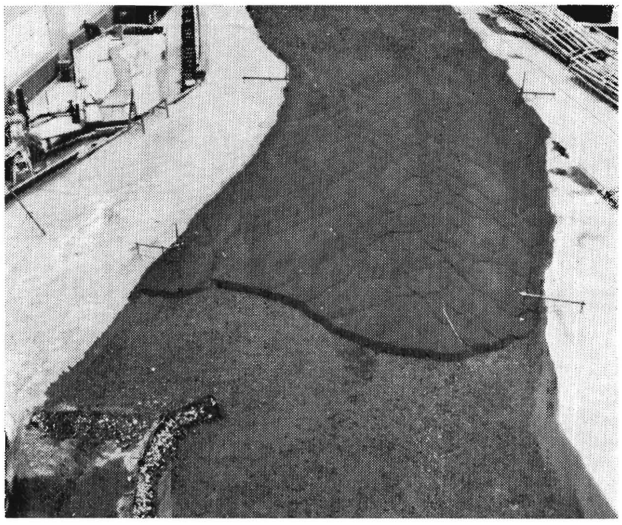


PHOTO 43

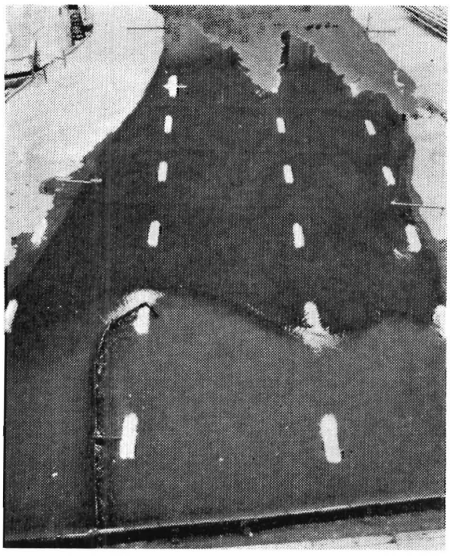


PHOTO 44

EL.244.9

RESULTS OF SAND RUN:
SAND 15 LITRES/MIN.
Q = 600 m^3/s
Qst = 250 »
Qse = 95 »
AFTER FIRST DAY: PHOTO 42
AFTER SECOND DAY: PHOTO 43
AFTER THIRD DAY: PHOTOS 44—46

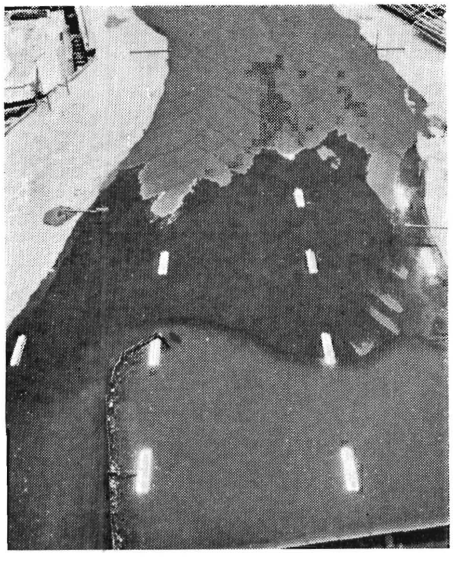


PHOTO 45

EL.244.5

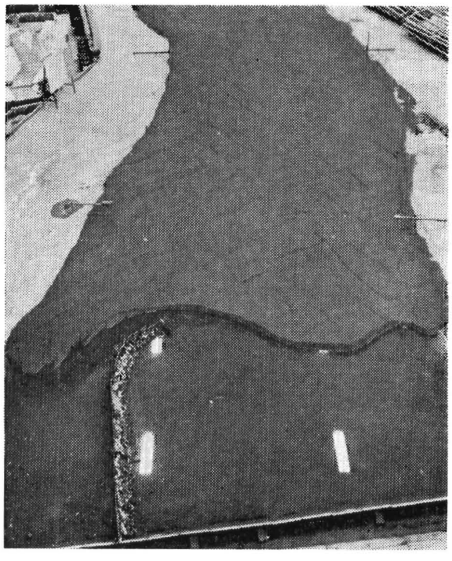
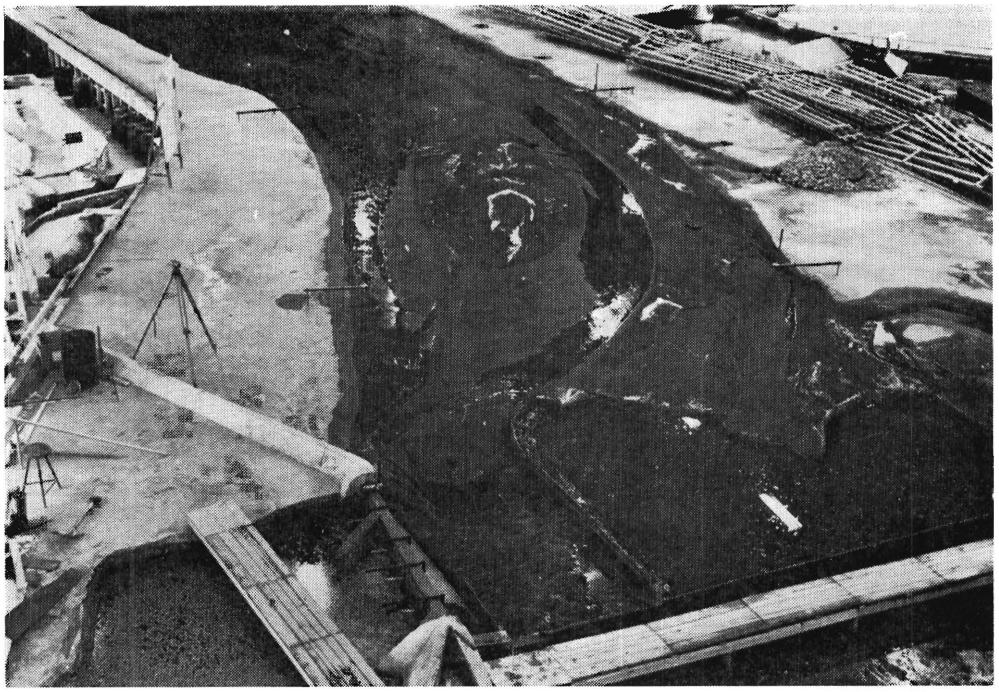


PHOTO 46

EL.243.0

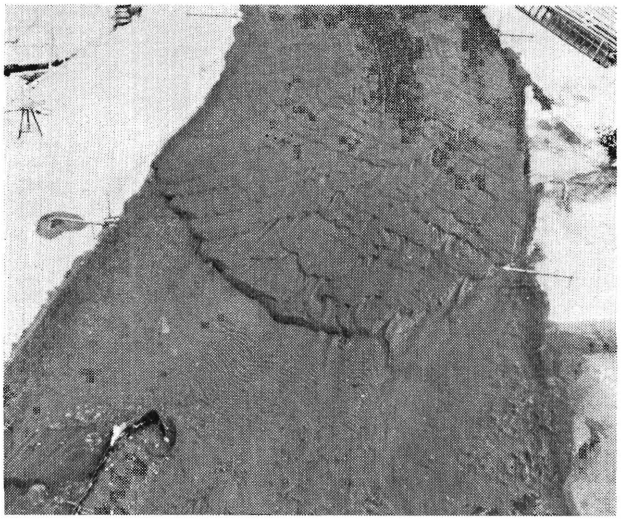


SITUATION ON
PHOTO 46
ERODED

Q = 200 m³/s
Qst = 120 »
Qse = 80 »

RUN TIME
3.5 HOURS
+ STEP DOWN
PERIOD

PHOTO 47



RESULT OF SAND RUN:

SAND: 15 LITRES/MIN
Q = 1000 m³/s
Qst = 250 »
Qse = 95 »

AFTER FIRST DAY: PHOTO 48
AFTER THIRD DAY: PHOTO 49-50

PHOTO 48

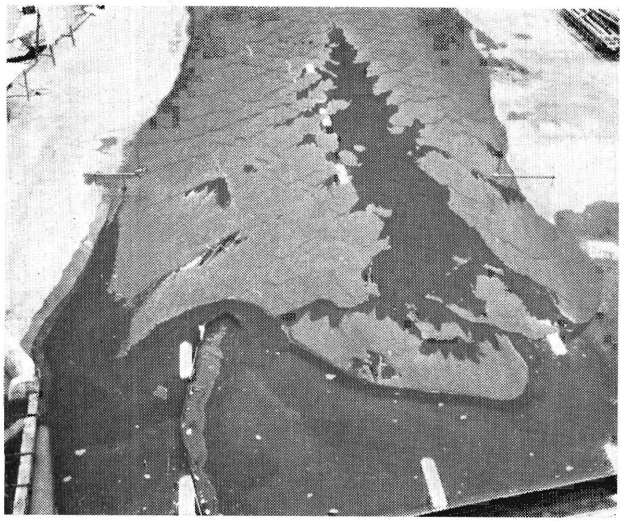


PHOTO 49

PHOTO 50

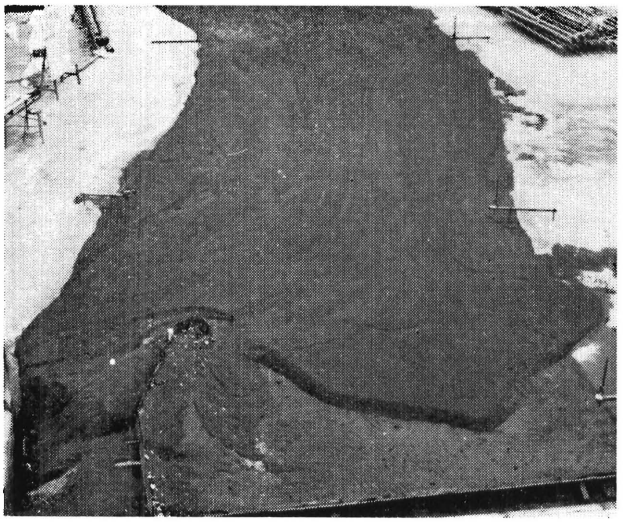
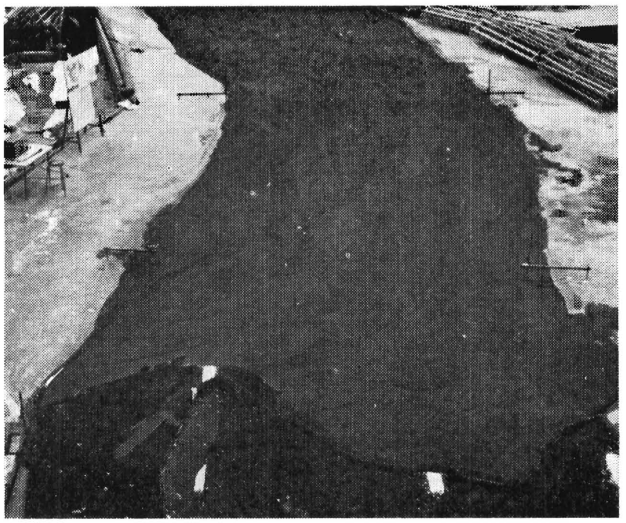
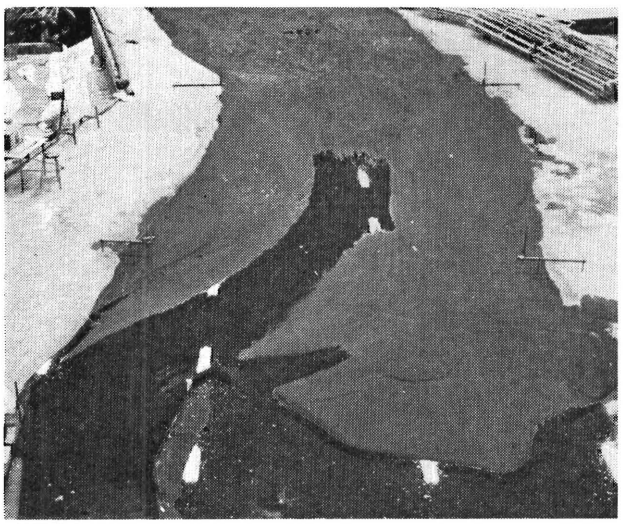


PHOTO 51



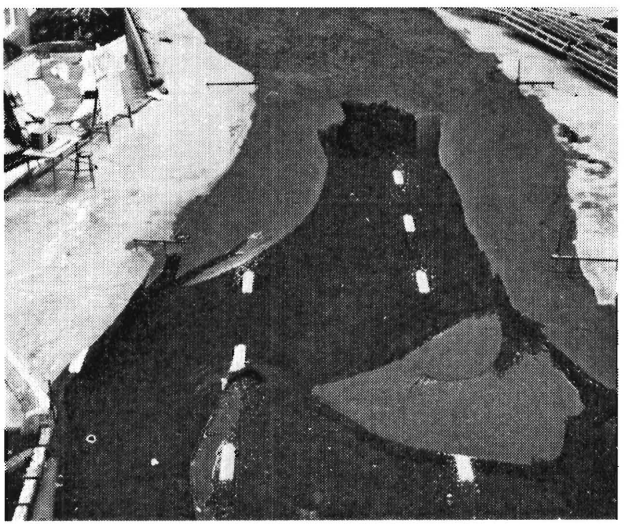
EL. 243.0

PHOTO 52



EL. 243.5

PHOTO 53



EL. 244.0

PHOTO 54

SITUATION ON PHOTOS 49-50 ERODED. Q = 250 m³/s, Qst = 150 m³/s, RUN TIME: 9.5 HOURS

SITUATION ON
PHOTO 51 AFTER
0.5 HOURS
ADDITIONAL RUN
Q = 250 m³/s
Qst = 200 »

PHOTO 55

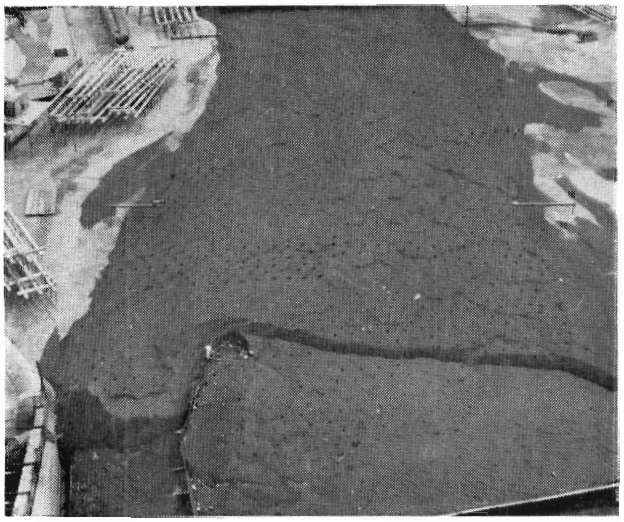
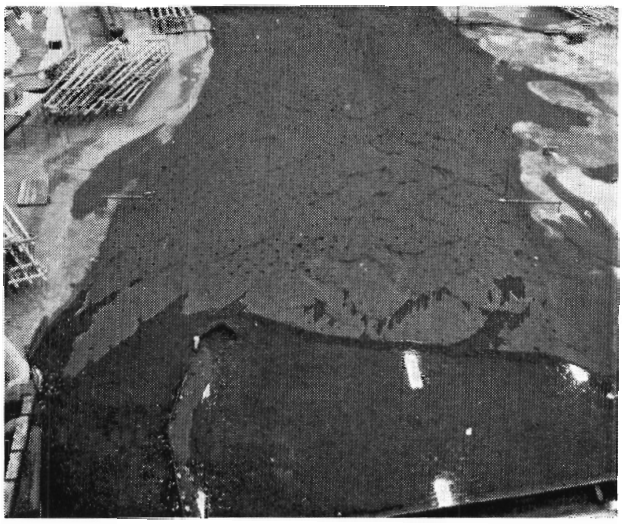
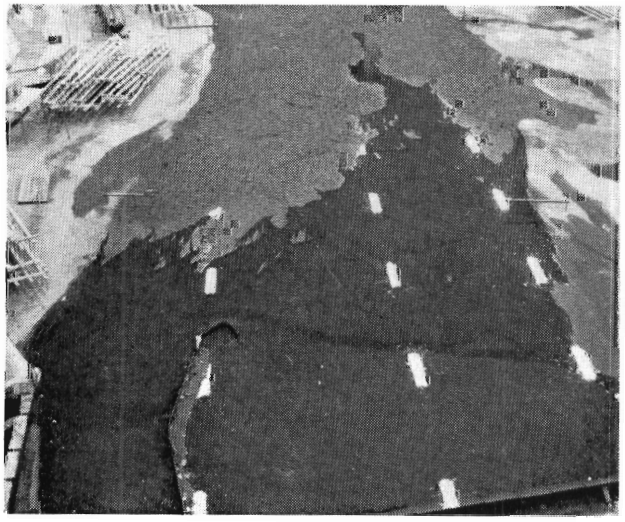


PHOTO 56



EL. 244.0

PHOTO 57



EL. 244.5

PHOTO 58

PHOTOS 56—58:
RESULT OF THREE DAYS
SAND RUN WITH PEGS
SAND: 15 LITRES/MIN
Q = 1000 m^3/s
Qst = 250 »
Qse = 95 »

PHOTOS 59—60:
SITUATION AFTER 2.5 HOURS
STEP DOWN PERIOD TO
Q = 250 m^3/s Qst = 150 m^3/s

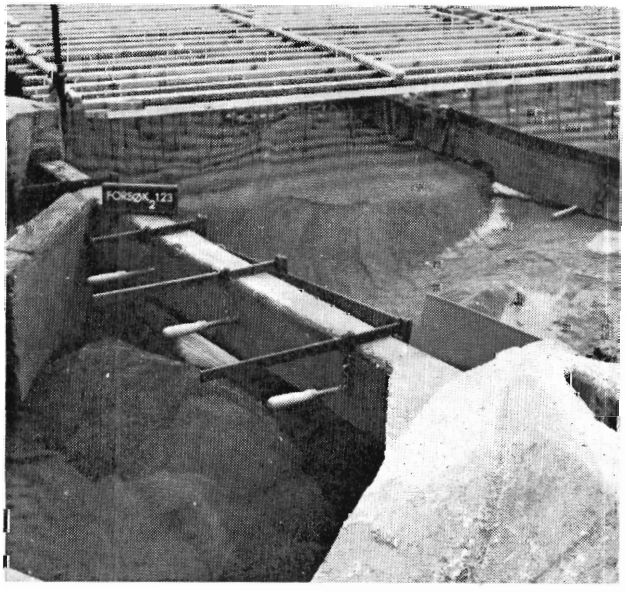


PHOTO 59

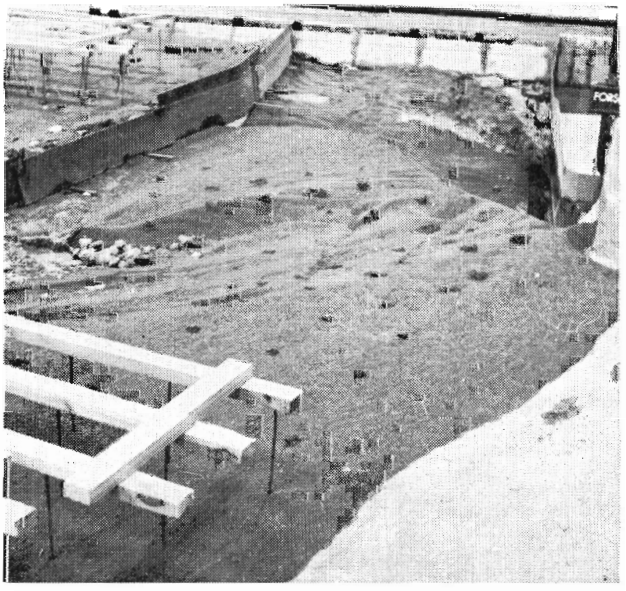
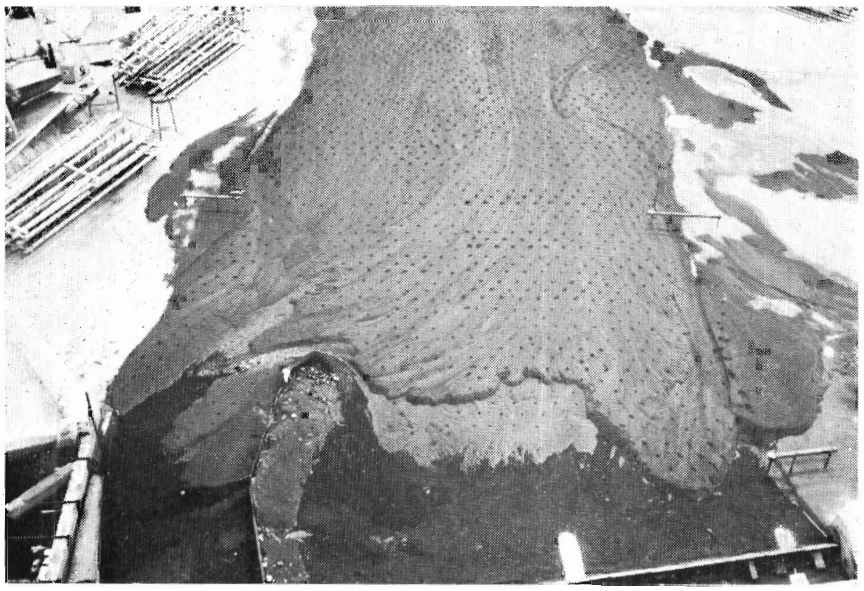


PHOTO 60



PHOTOS 61—63:
SITUATION ON
PHOTOS 59—60
AFTER SIX
HOURS FURTHER
RUN
Q = 250 m³/s
Qst = 150 »

PHOTO 61

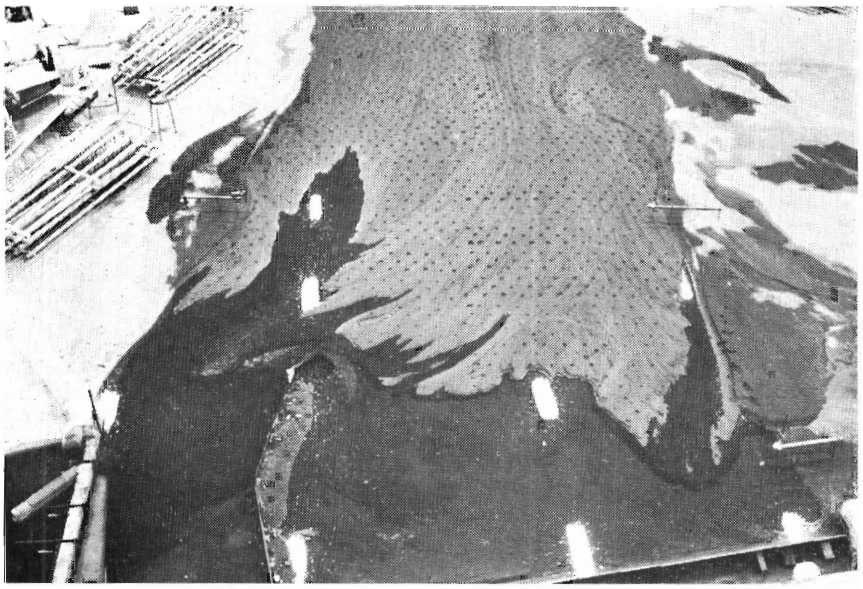


PHOTO 62

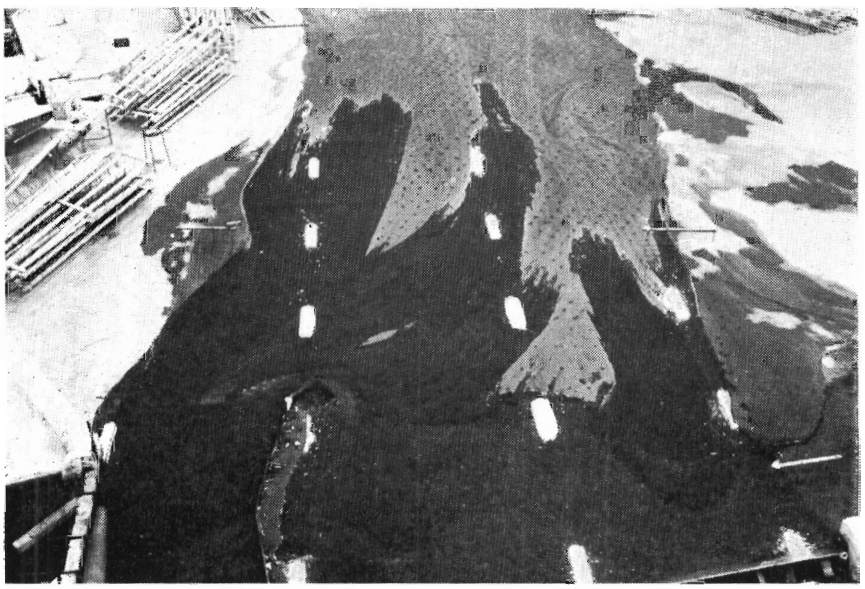


PHOTO 63

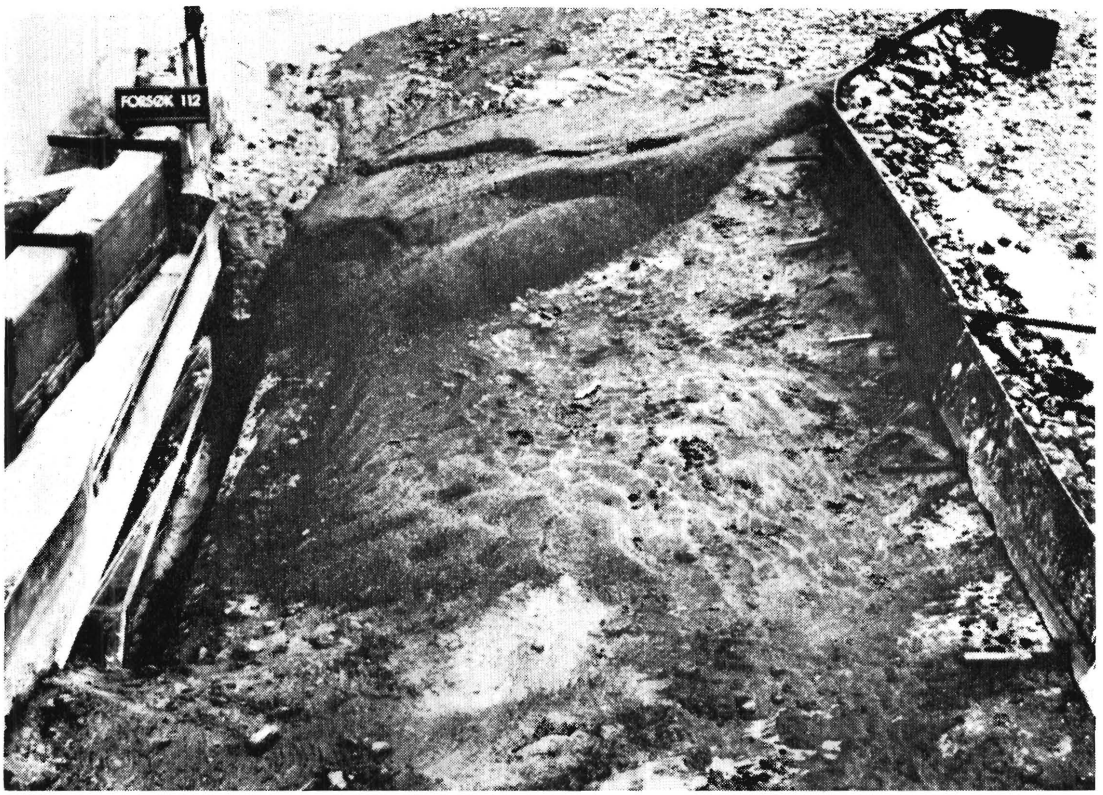


PHOTO 64

RESULT OF 2.5 HOURS LOCAL SAND STUDY. EL. = 244.6
SAND: 2 LITRES/MIN, Q = 600 m^3/s, Qst = 250 m^3/s, Qse = 95 m^3/s

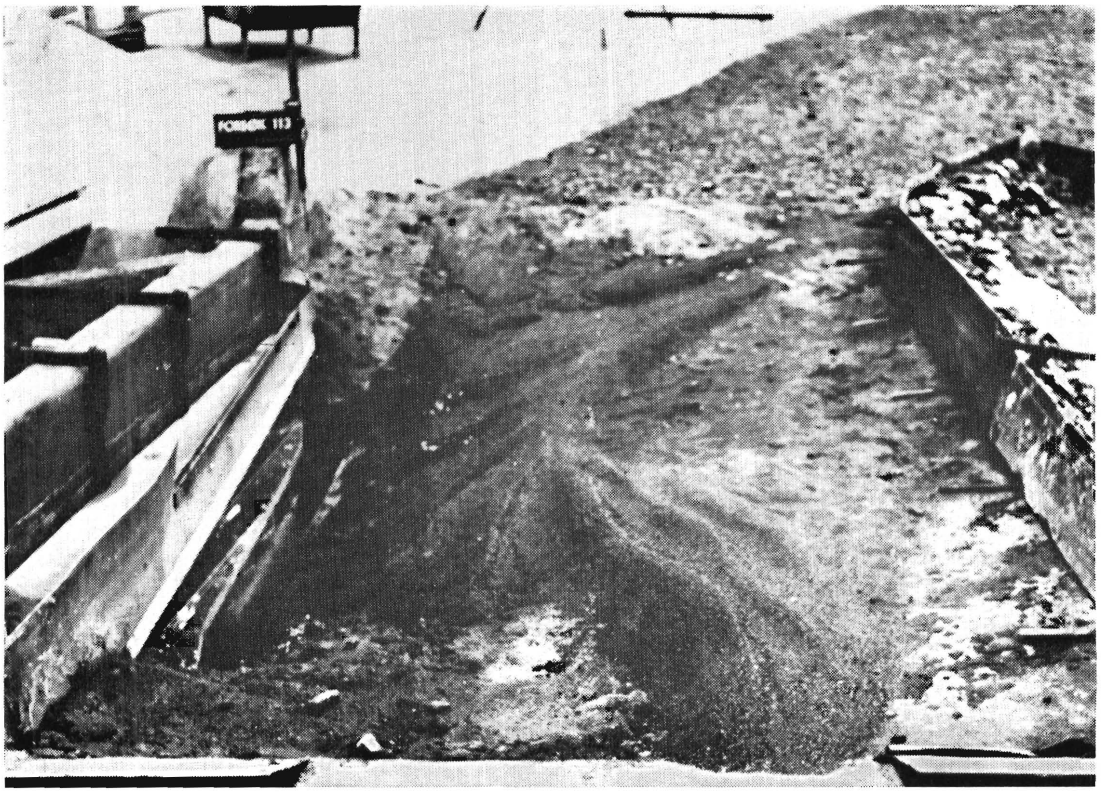
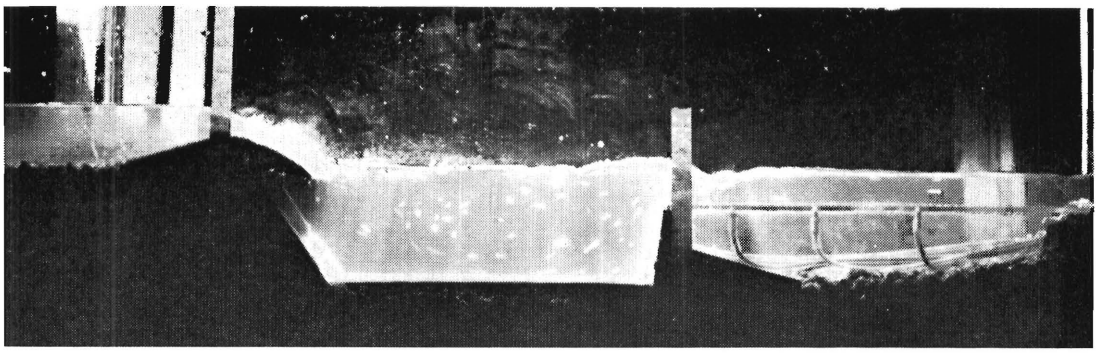


PHOTO 65

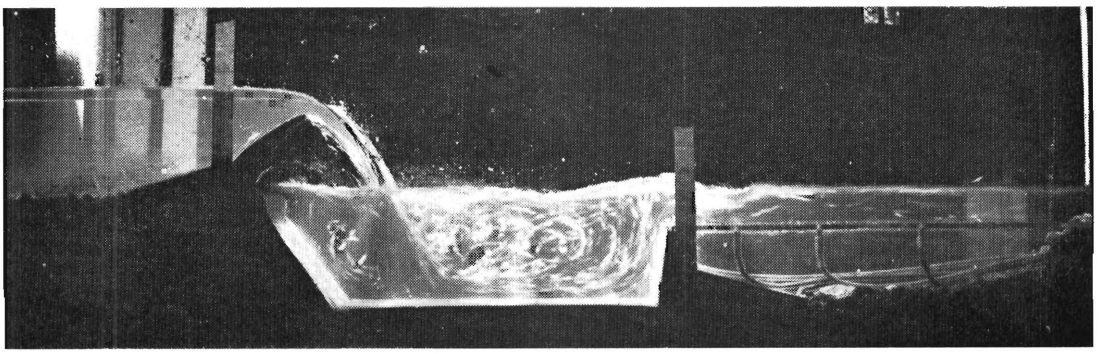
RESULT OF 3 HOURS LOCAL SAND STUDY. EL. = 244.6
SAND: 2 LITRES/MIN, Q = 900 m^3/s, Qst = 250 m^3/s, Qse = 95 m^3/s



50 m^3/s
PER
GATE

PHOTO 66

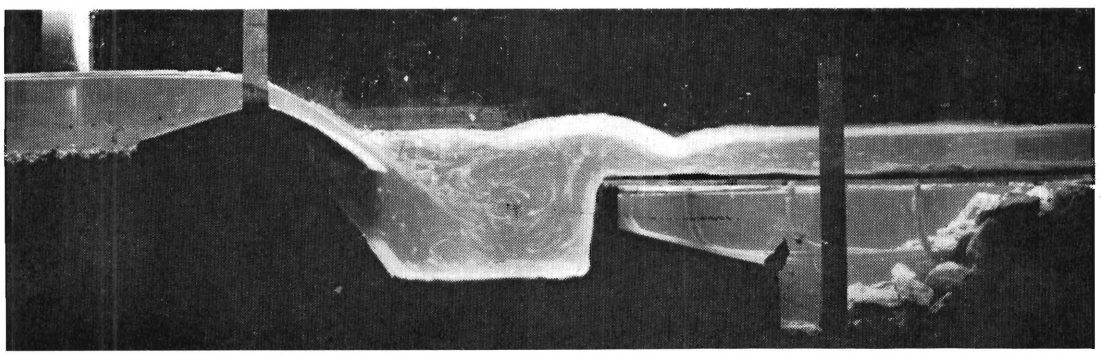
DAM CREST EL. 242.5. CHANNEL WIDTH = 12 m
DOWNSTREAM SILL AND SLAB EL. 240 0. TAILWATTR EL. 241.5



50 m^3/s
PER
GATE

PHOTO 67

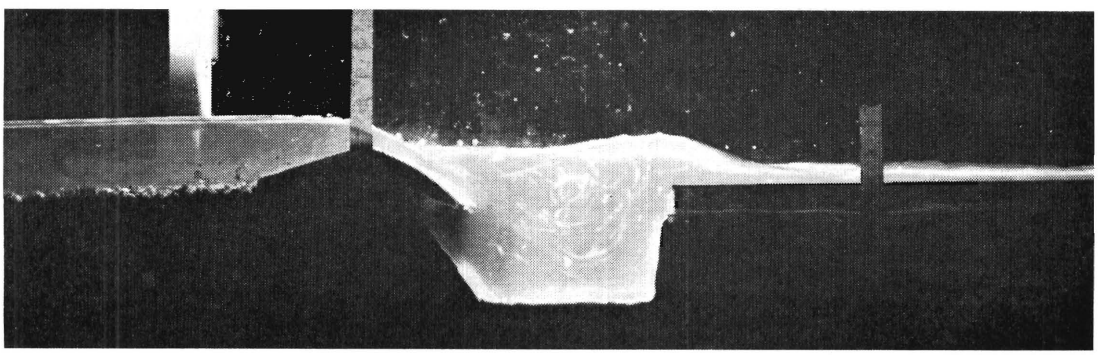
GATE CREST EL. 244.5
ELSE AS PHOTO 66



50 m^3/s
PER
GATE

PHOTO 68

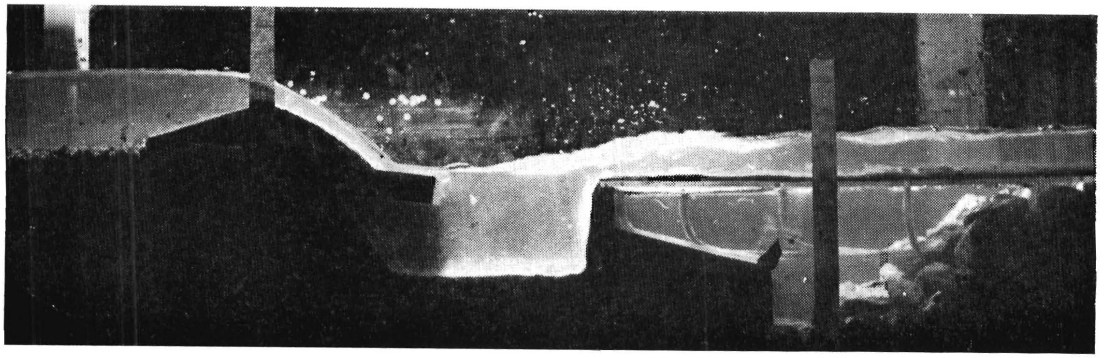
WIDTH OF CHANNEL = 6 m
ELSE AS PHOTO 66



50 m^3/s
PER
GATE

PHOTO 69

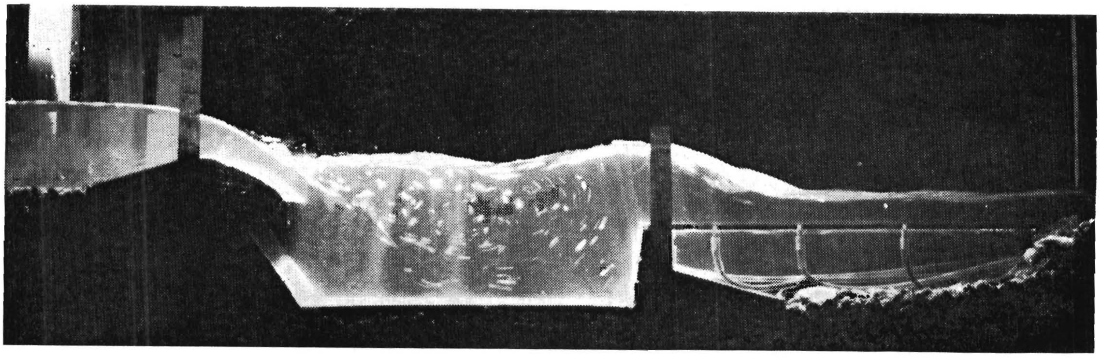
DOWNSTREAM SILL AND SLAB EL. 241.0
ELSE AS PHOTO 68



50 m^3/s
PER
GATE

PHOTO 70

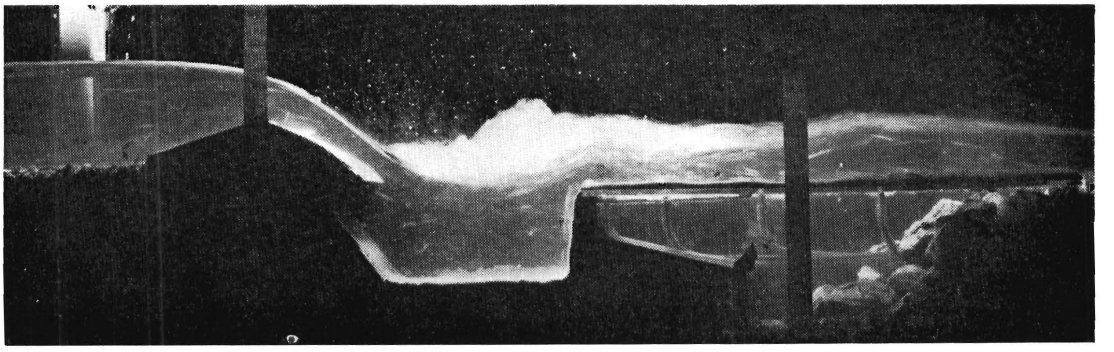
AS PHOTO 68, BUT BAFFLE INSERTED IN THE CHANNEL



150 m^3/s
PER
GATE

PHOTO 71

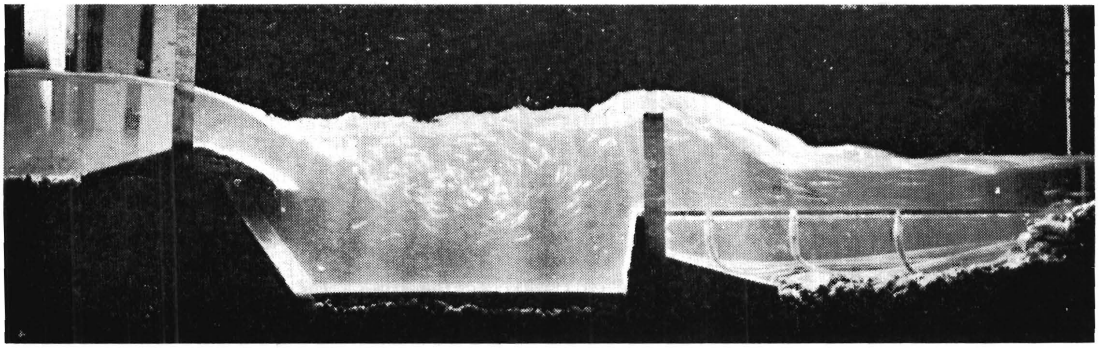
DAM CREST EL. 242.5. WIDTH OF CHANNEL = 12 m
DOWNSTREAM SILL EL. 240.0. TAILWATER EL. 241.50



150 m^3/s
PER
GATE

PHOTO 72

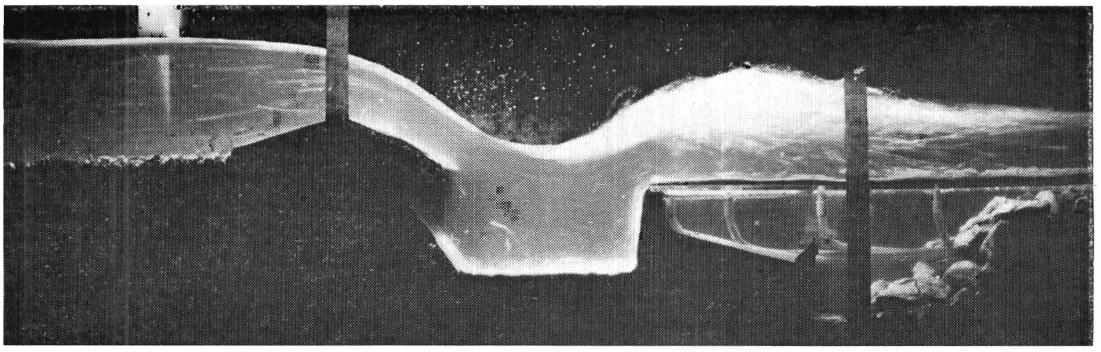
WIDTH OF CHANNEL = 6 m. TAILWATER EL. 242. (APPROX)
ELSE AS PHOTO 71



250 m^3/s
PER
GATE

PHOTO 73

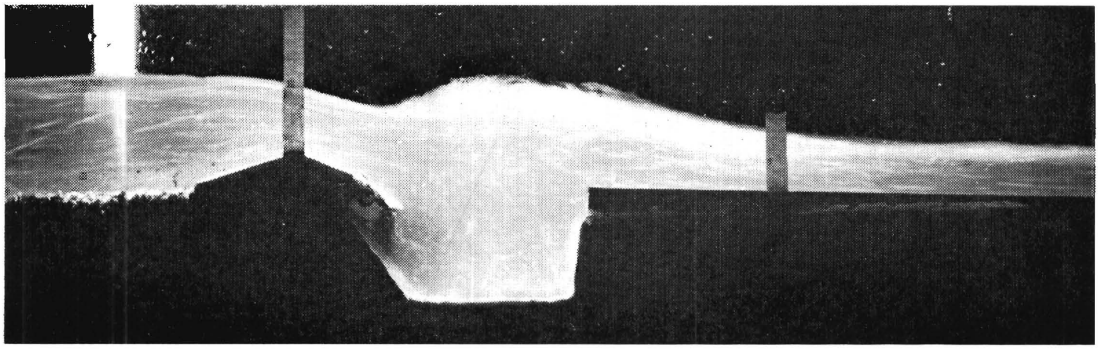
DAM CREST EL. 242.5. WIDTH OF CHANNEL = 12 m
DOWNSTREAM SILL EL. 240.0. TAILWATER EL. 242.0



250 m^3/s
PER
GATE

PHOTO 74

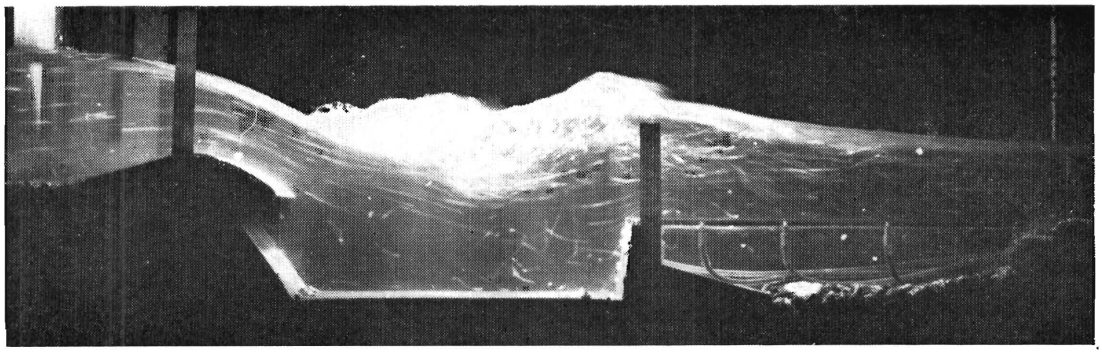
WIDTH OF CHANNEL = 6 m. TAILWATER EL. 242.2
ELSE AS PHOTO 73



250 m^3/s
PER
GATE

PHOTO 75

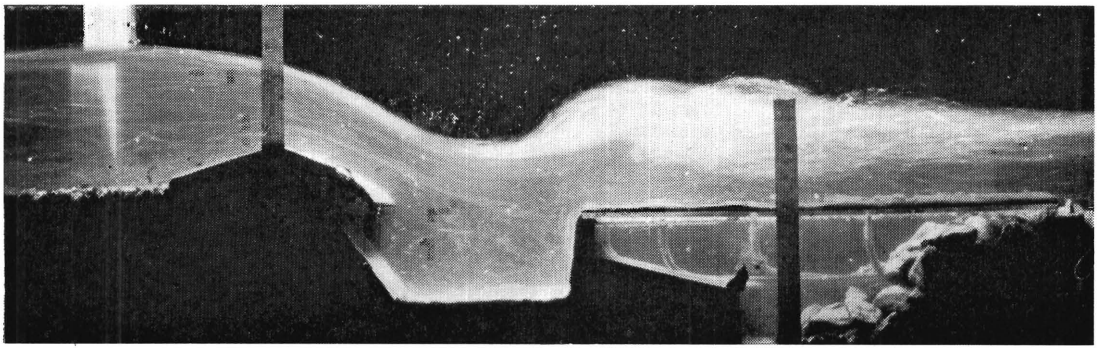
DAM CREST EL. 242.5. WIDTH OF CHANNEL = 6 m
DOWNSTREAM SILL EL. 241.0. TAILWATER EL. 242.2



400 m^3/s
PER
GATE

PHOTO 76

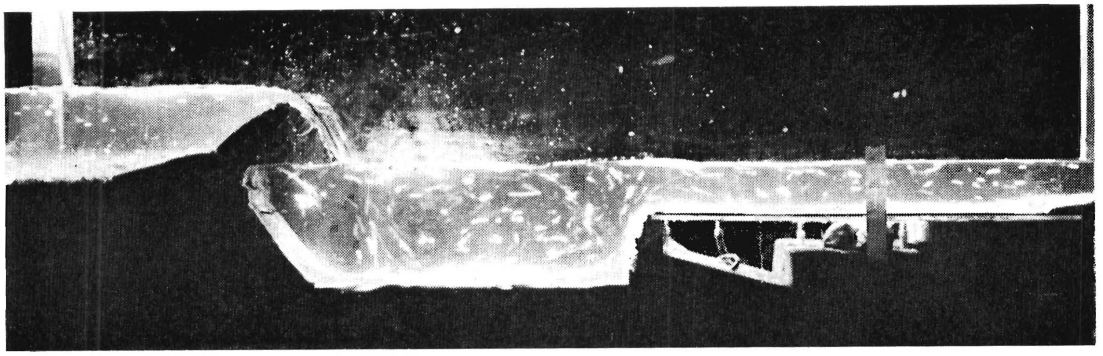
DAM CREST EL. 242.5. WIDTH OF CHANNEL = 12 m
DOWNSTREAM SILL AND SLAB EL. 240.0. TAILWATER EL. 242.5.



400 m^3/s
PER
GATE

PHOTO 77

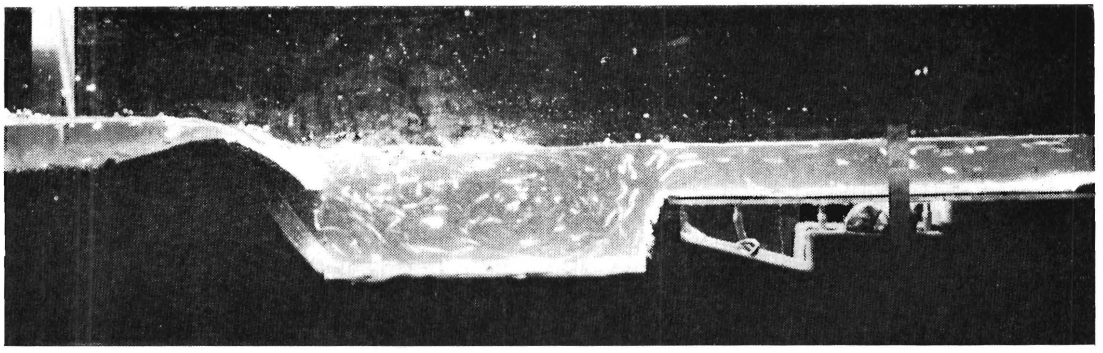
WIDTH OF CHANNEL = 6 m. TAILWATER EL. 242.8
ELSE AS PHOTO 76



25 m^3/s
PER
GATE

PHOTO 78

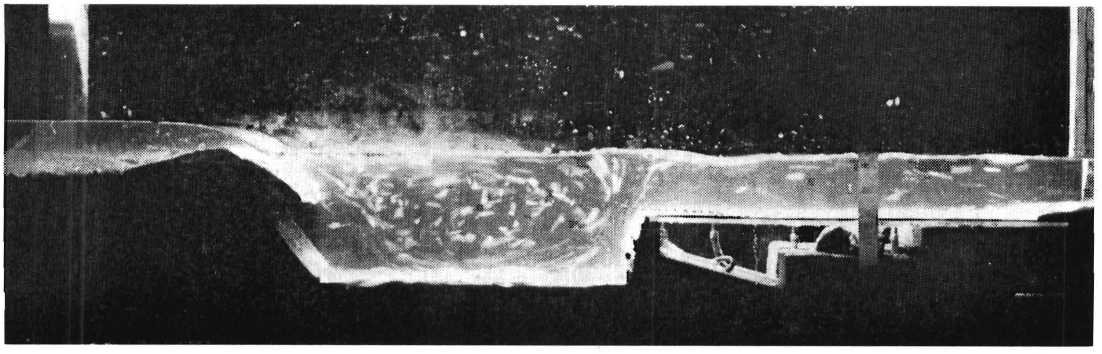
GATE CREST EL. 244.5. SLAB EL. 240.0.
DOWNSTREAM BOTTOM EL. 241. TAILWATER EL. 241.9.



25 m^3/s
PER
GATE

PHOTO 79

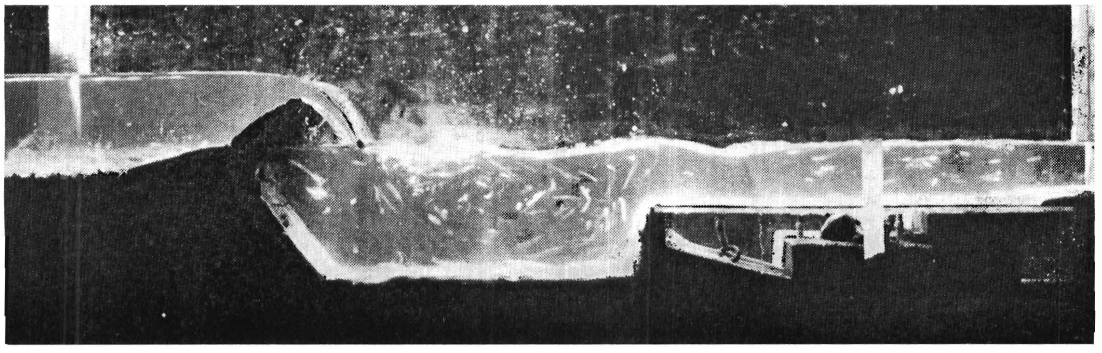
DAM CREST EL. 242.5
ELSE AS PHOTO 78



50 m^3/s
PER
GATE

PHOTO 80

DAM CREST EL. 242.5. TAILWATER EL. 242.3
ELSE AS PHOTO 78



50 m^3/s
PER
GATE

PHOTO 81

TAILWATER EL. 242.3
ELSE AS PHOTO 78

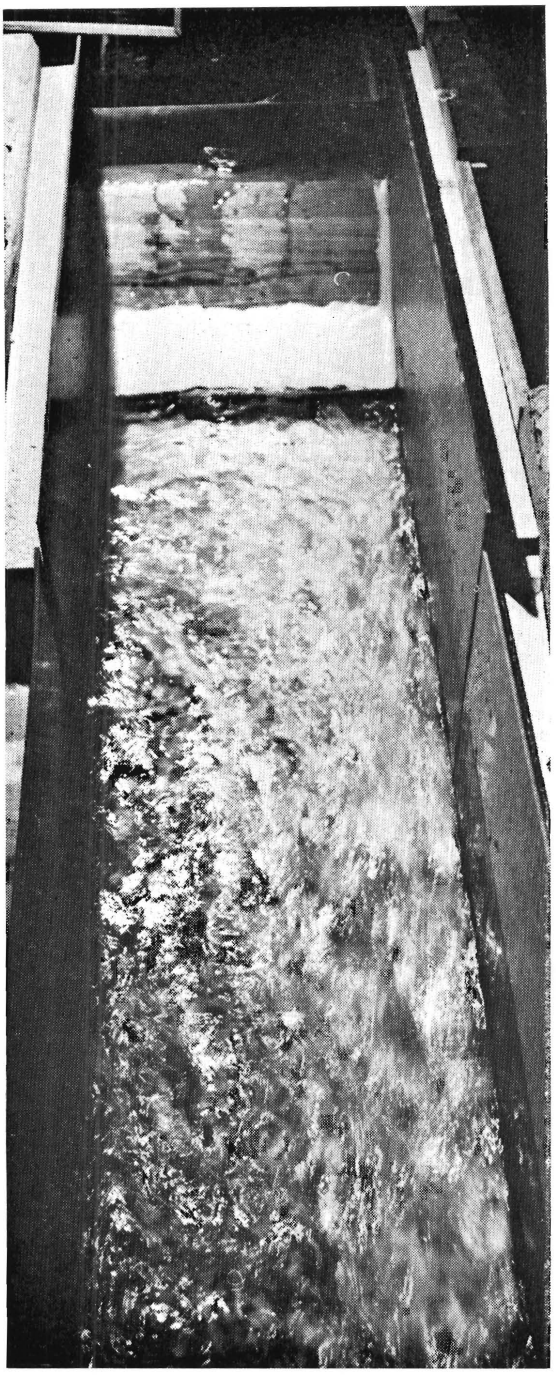


PHOTO 82

EASTERN SLUICE OPEN
Qse = 42 m^3/s
EL. 243.90

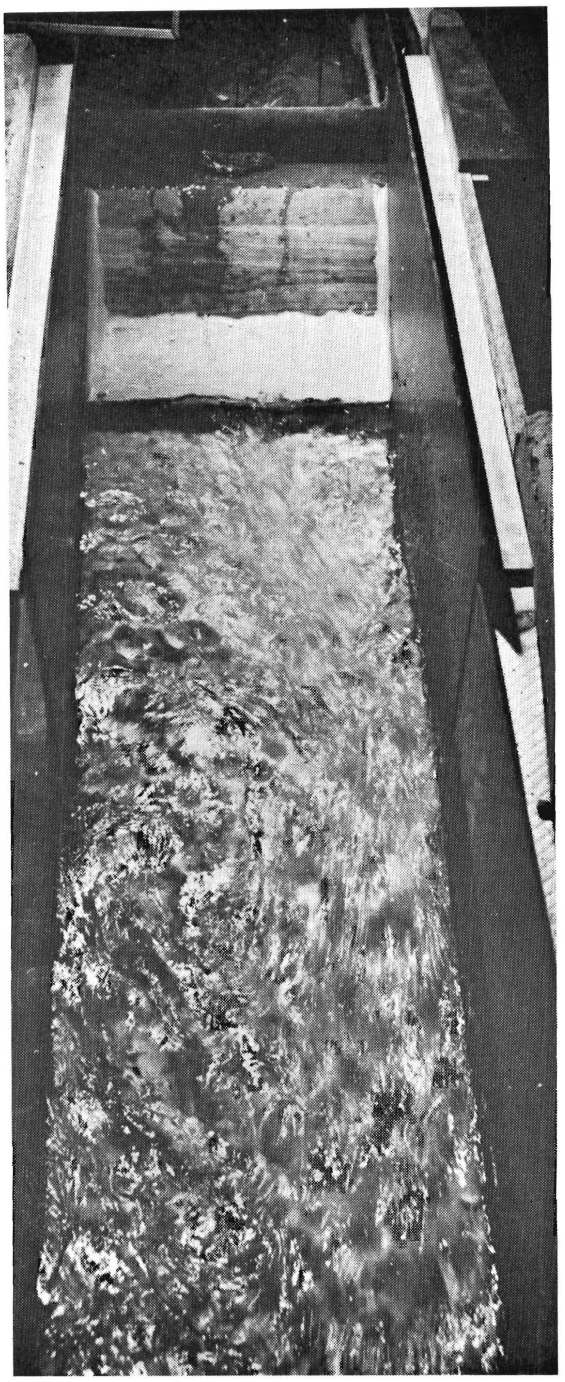


PHOTO 83

MIDDLE SLUICE OPEN
Qse = 42 m^3/s
EL. 243.65

BJARNALÆKUR CANAL. DISCHARGE THROUGH BOTTOM SLUICES
PLUNGE POOL 14 m WIDE. SYMMETRICAL SLUICE OUTLETS
INTAKE POOL LEVEL AS NOTED

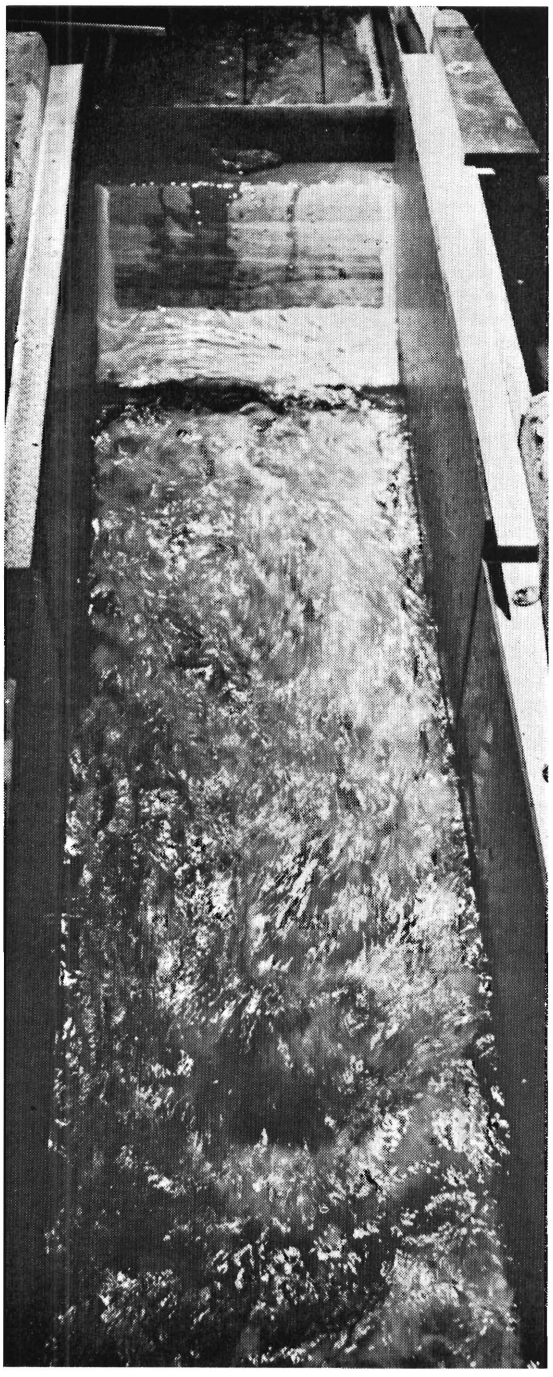


PHOTO 84

BOTH SIDE SLUICES OPEN
Qse = 62
EL. 243.7

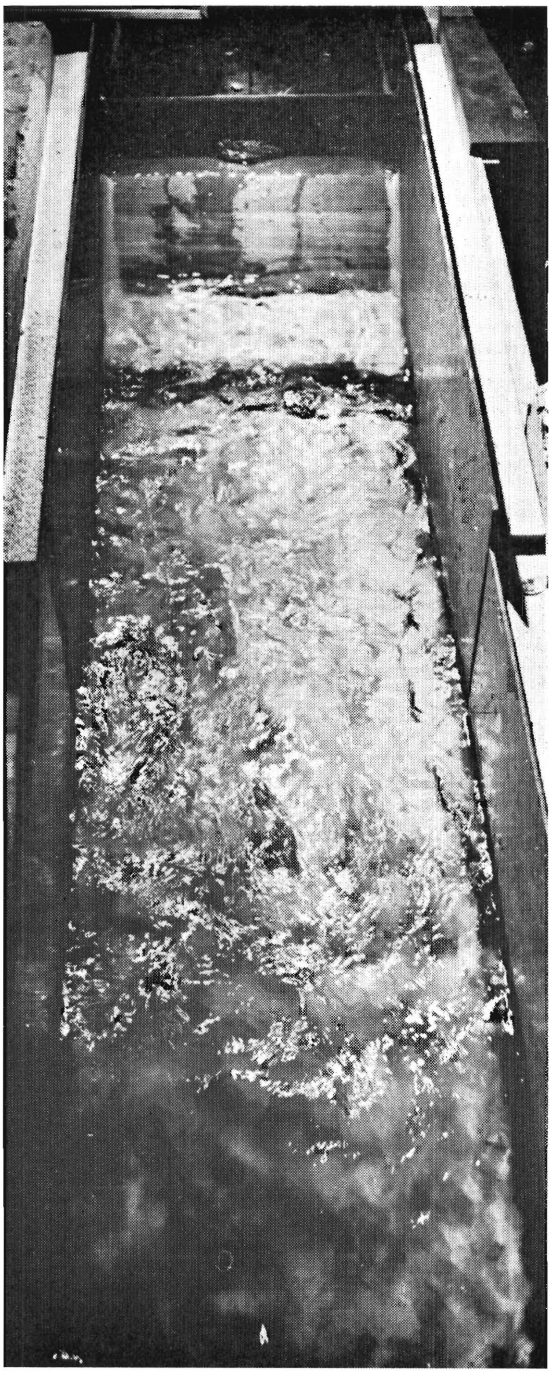


PHOTO 85

ALL SLUICES OPEN
Qse = 83
EL. 243.45

BJARNALÆKUR CANAL. DISCHARGE THROUGH BOTTOM SLUICES
PLUNGE POOL 14 m WIDE. SYMMETRICAL SLUICE OUTLETS
INTAKE POOL LEVEL AS NOTED

PHOTO 86

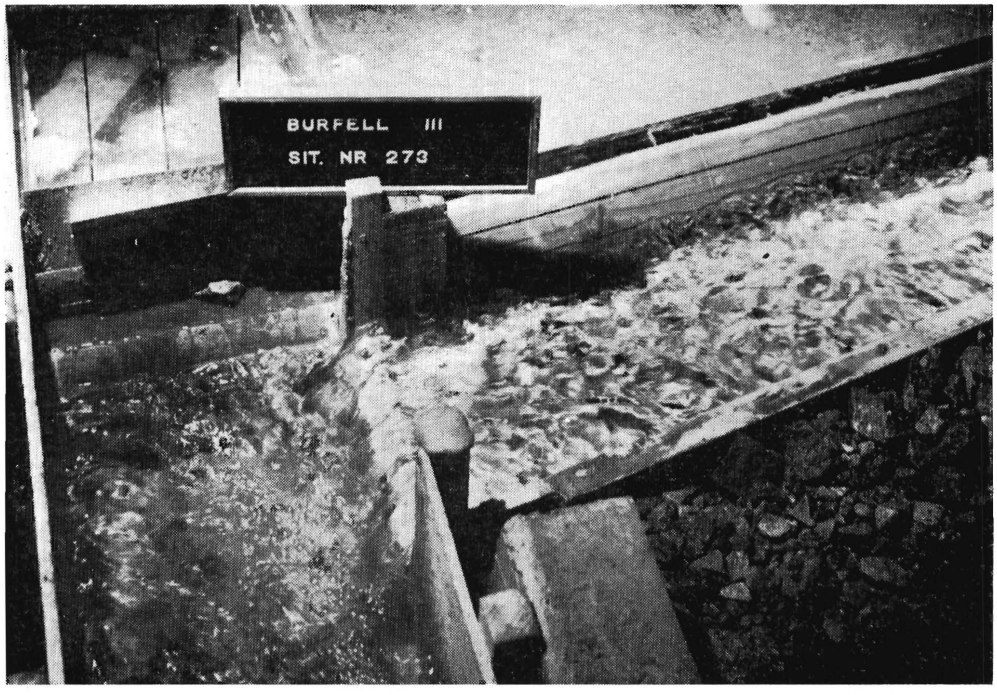


PHOTO 87

DISCHARGE THROUGH SIDE CHANNEL GATE
INTO BJARNALÆKUR CANAL
Q = 100 m^3/s

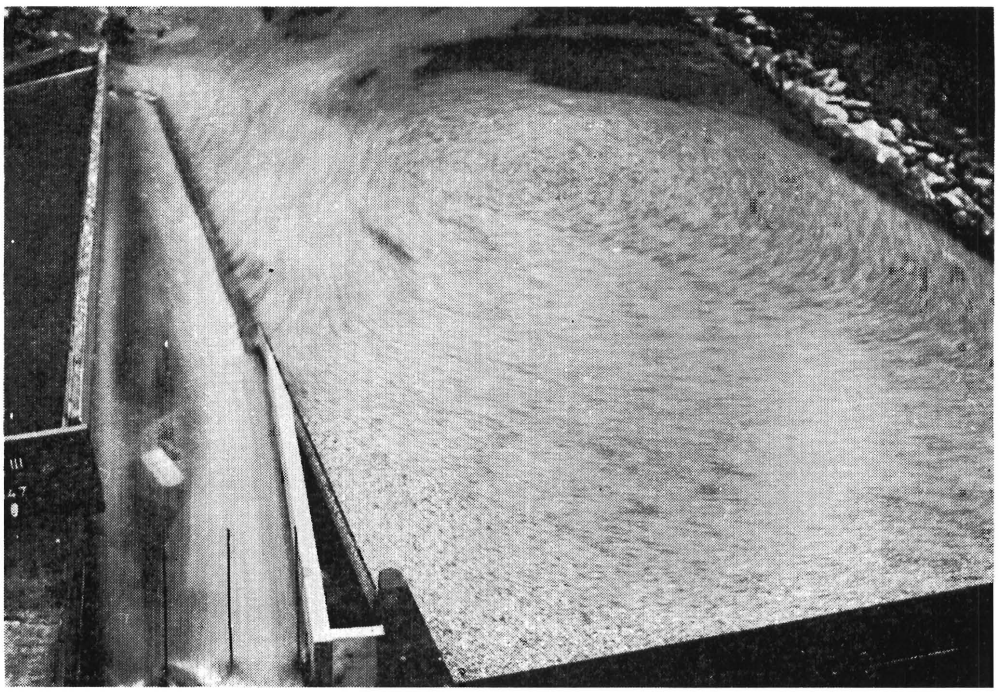
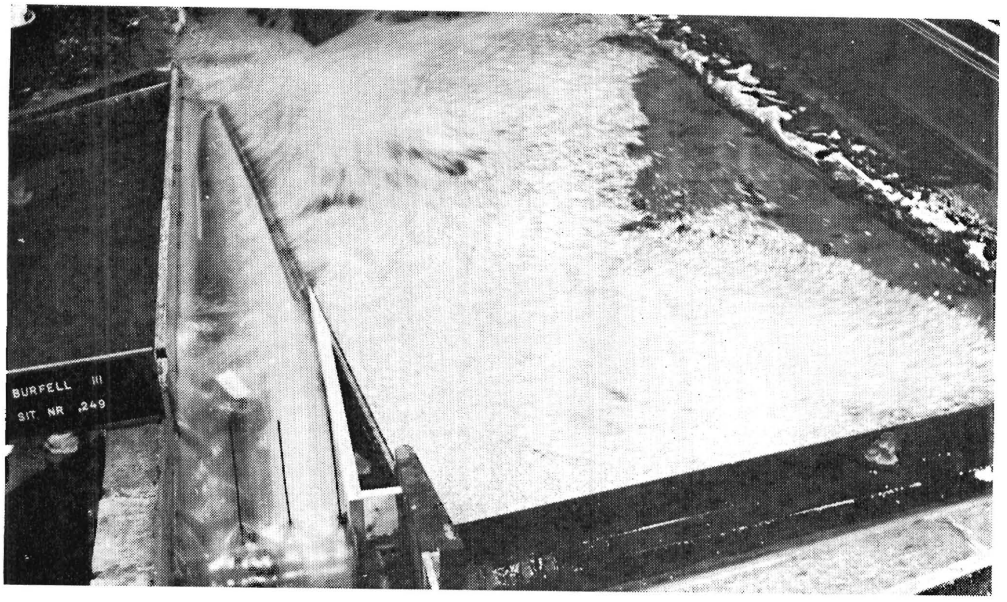


PHOTO 88

TROUGH ALTERNATIVE FIG. 26 c
CREST OF BJARNALÆKUR GATE EL. 240.50
Q = 150 m^3/s, Qst = 100 m^3/s Qbc = 50 m^3/s

PHOTO 89

TROUGH ALTERNATIVE FIG. 26 d
CREST OF BJARNALÆKUR GATE EL. 240.30
Q = 150 m^3/s, Qst = 100 m^3/s Qbc = 50 m^3/s

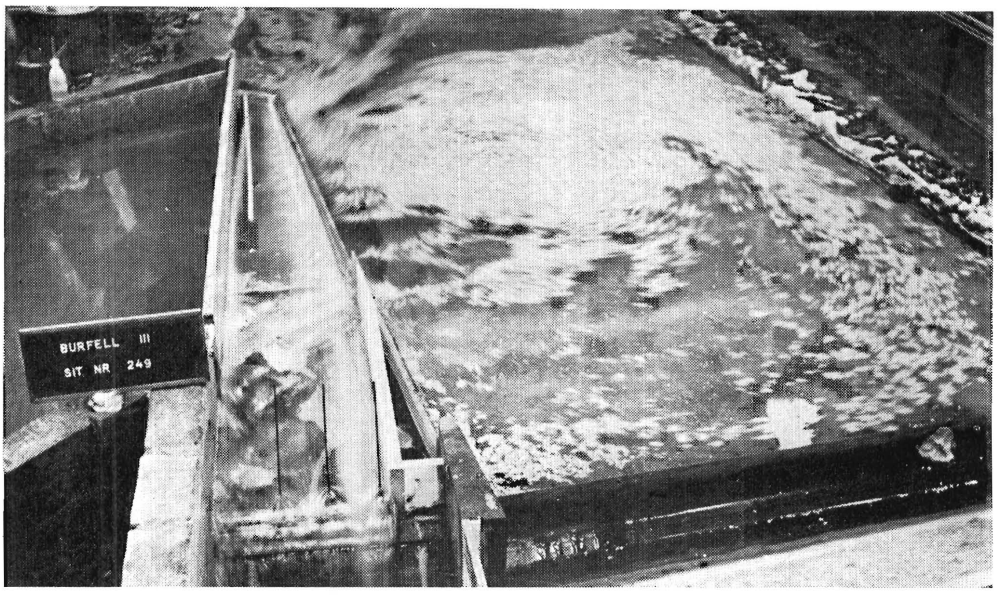


TROUGH
ALTERNATIVE
FIG. 26 F

CREST OF GATE
EL. 240.5

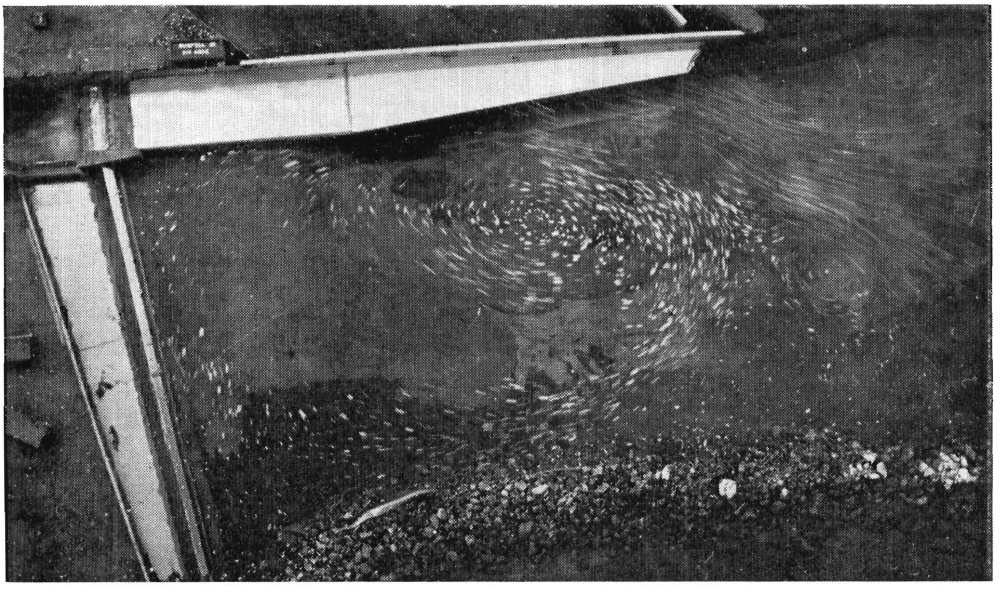
Q = 150 m³/s
Qst = 100 »

PHOTO 90



TROUGH
AS PHOTO 90

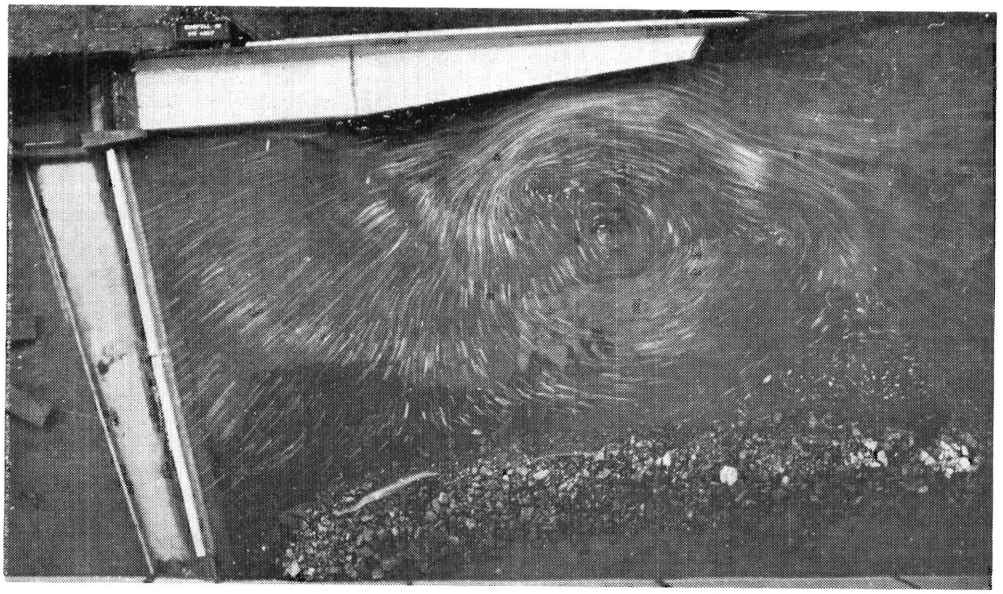
PHOTO 91



FLOW
LINES

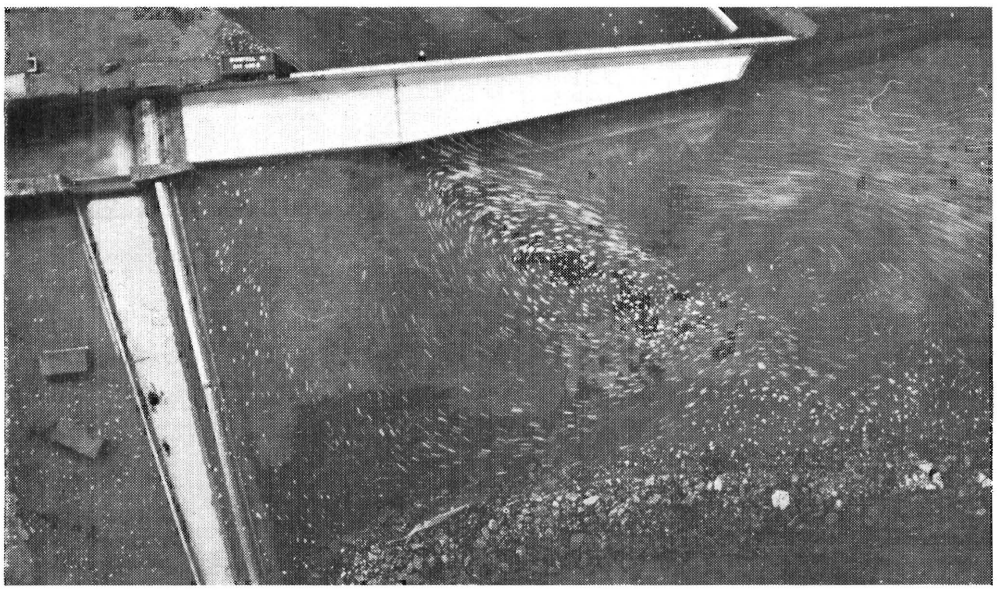
Q = 150 m³/s
Qst = 100 »
Qtr = 50 »

PHOTO 92



Q = 150 m³/s
Qst = 100 »
Qd = 50 »

PHOTO 93

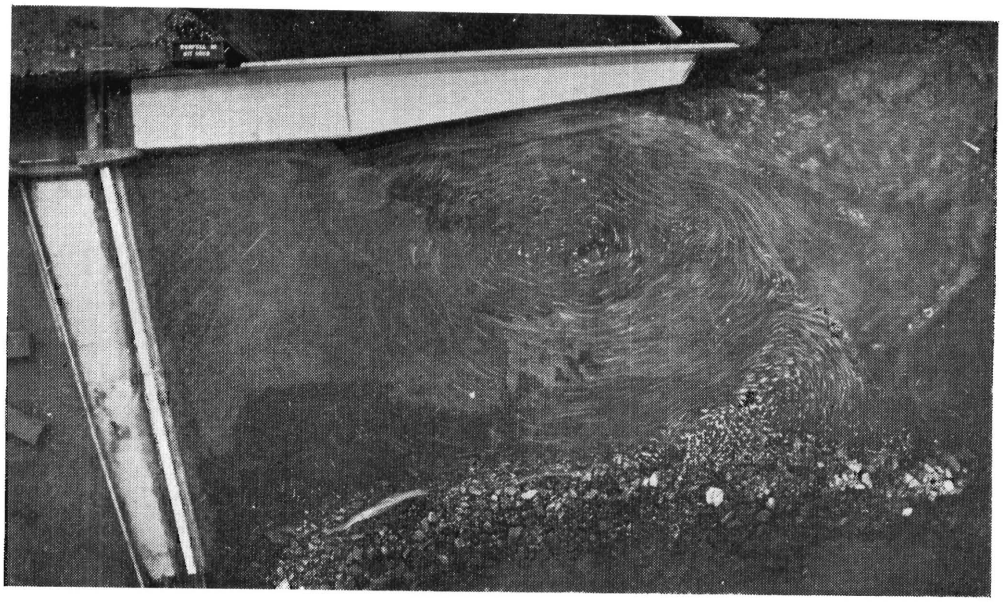


Q = 150 m³/s
Qst = 50 »
Qtr = 100 »

PHOTO 94

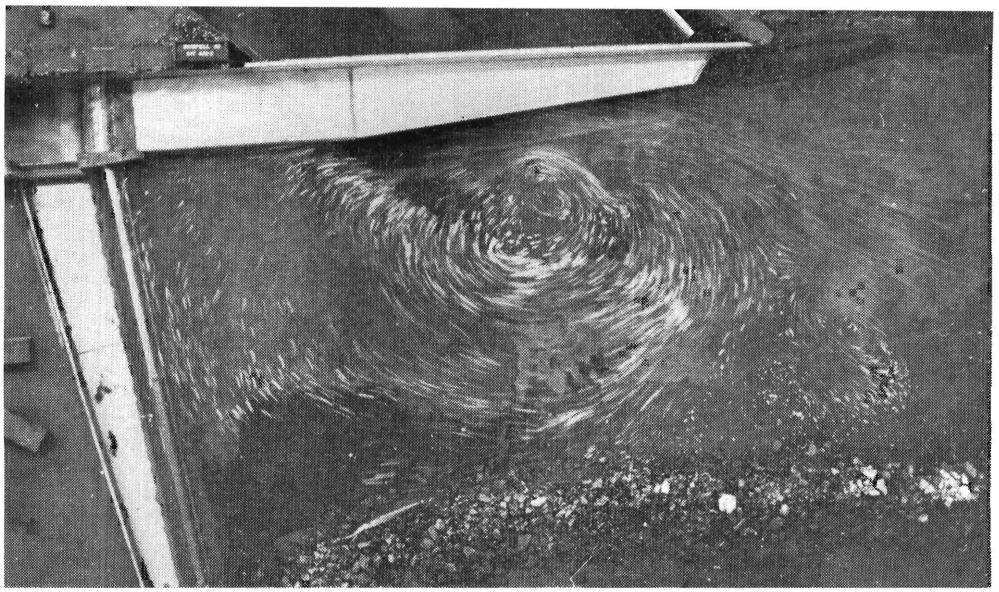
Q = 300 m³/s
Qst = 250 »
Qtr = 50 »

PHOTO 95



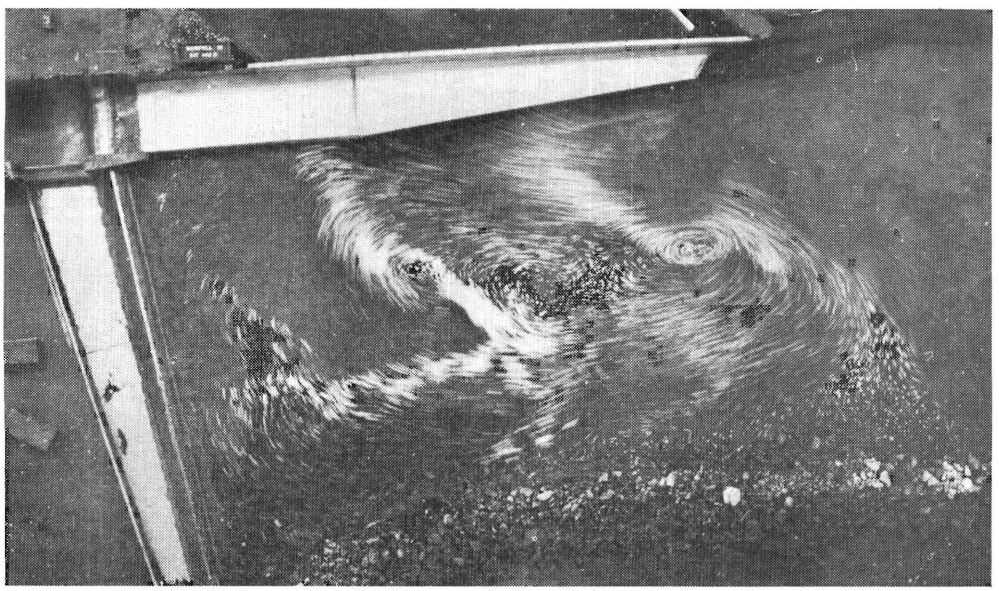
Q = 300 m^3/s
Qst = 250 »
Qd = 50 »

PHOTO 96



Q = 300 m^3/s
Qst = 200 »
Qtr = 100 »

PHOTO 97

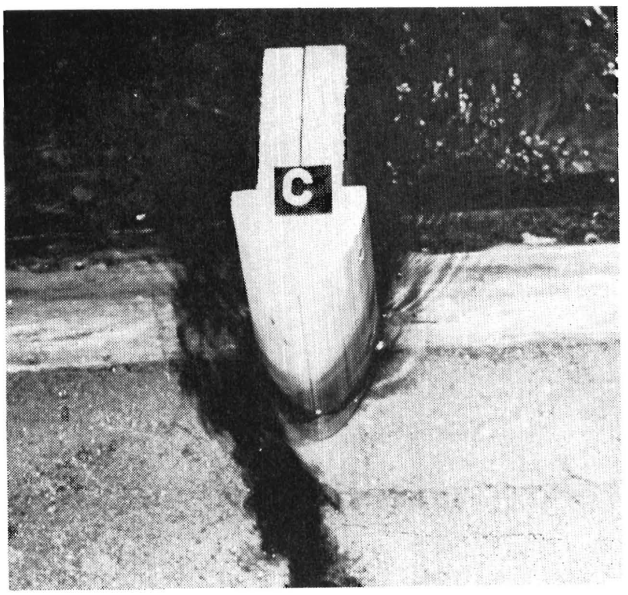


Q = 300 m^3/s
Qst = 150 »
Qtr = 50 »

PHOTO 98

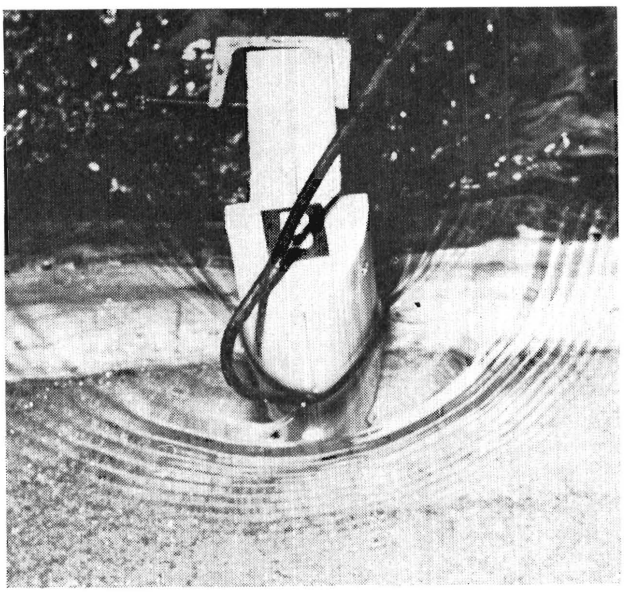
Qd = 150 m^3/s

PHOTO 99



Qd = 150 m^3/s

PHOTO 100



Qd = 600 m^3/s

PHOTO 101

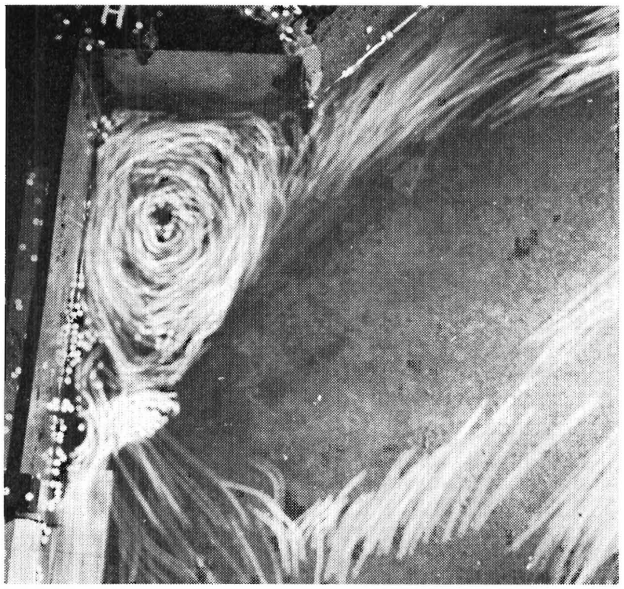


PHOTO 102

ORIGINAL DESIGN (DWG 290 S 35 R1)
Q = 250 m³/s Qst = 150 m³/s

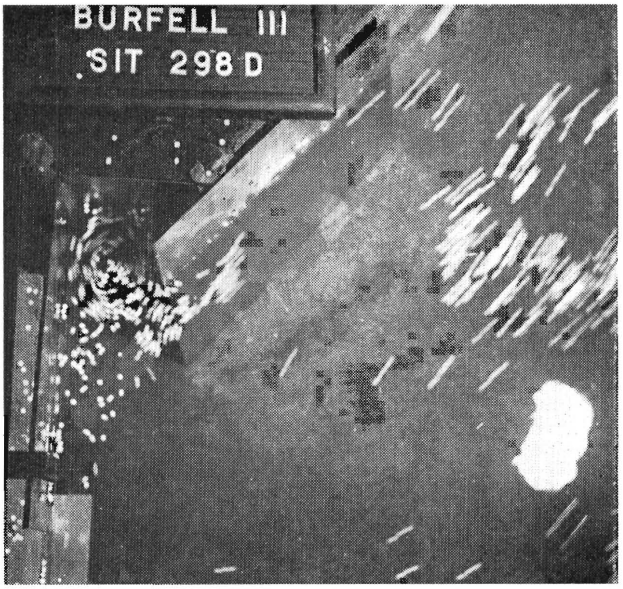


PHOTO 103

EFFECT OF SHELTER WALL
Q = 300 m³/s Qst = 250 m³/s

PHOTO 104

STRAIGHT WALL. BOTTOM OF
INTAKE EL. 234.50
Q = 300 m³/s Qst = 250 m³/s

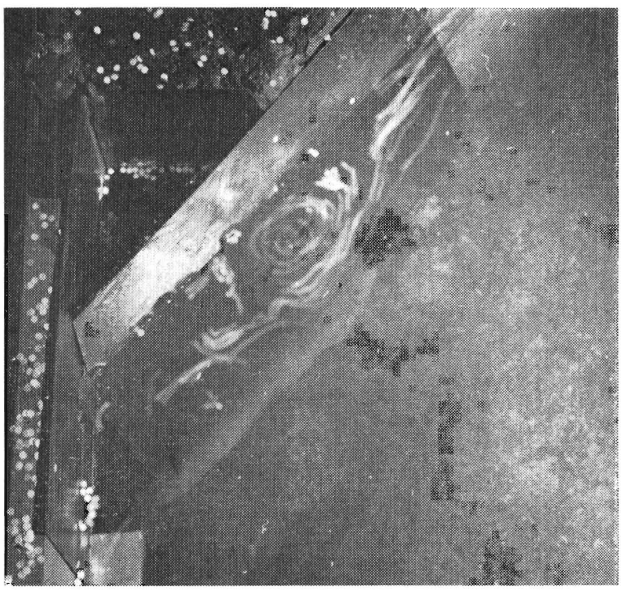
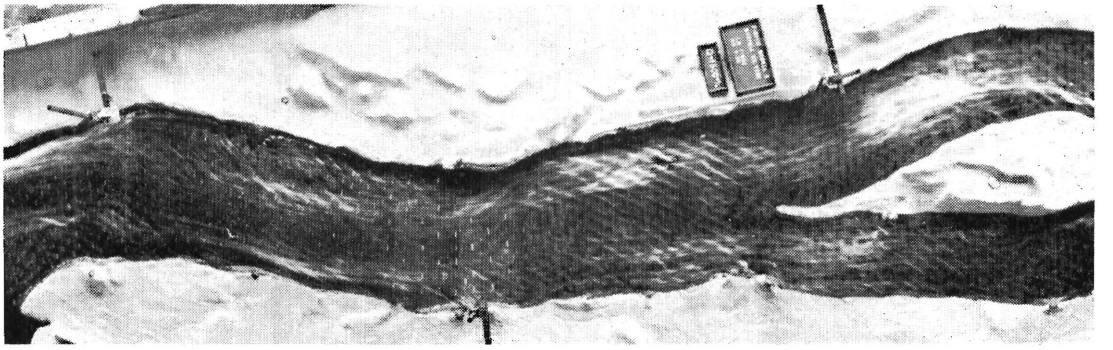


PHOTO 105

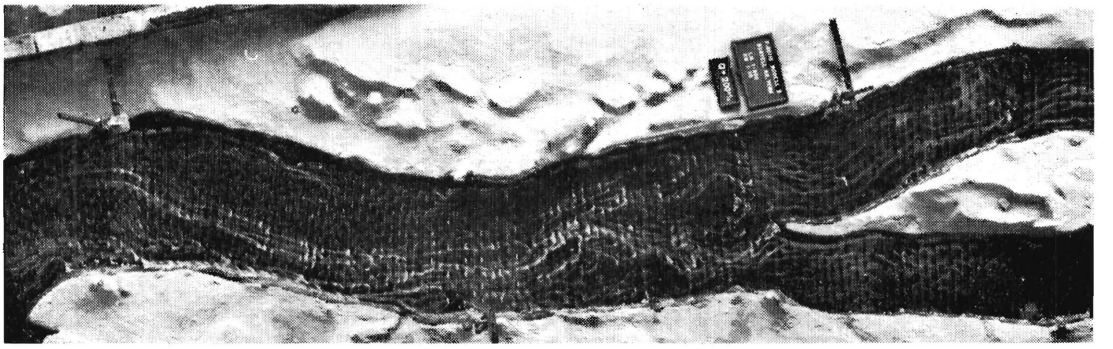
STRAIGHT WALL. BOTTOM OF
INTAKE EL. 240.0
Q = 300 m³/s Qst = 250 m³/s

INTAKE TO AUXILIARY WATER SUPPLY

ICE RUN WITH SAND DEPOSITS PHOTO 106
Q = m³/s, Qst = 100 m³/s, Ise = 28 m³/s

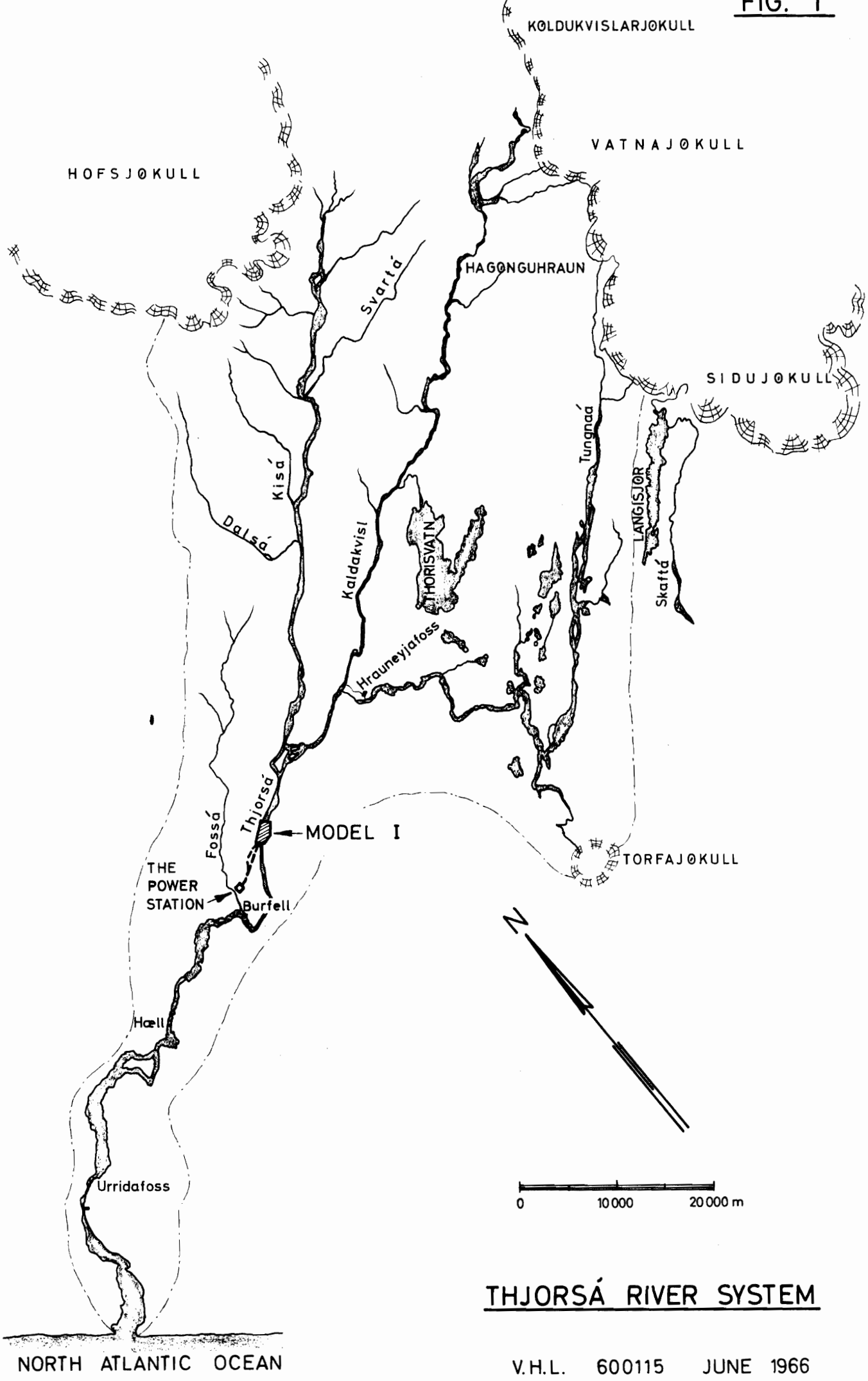


MODEL II Q = 1500 m³/s PHOTO 107



MODEL II Q = 200 m³/s PHOTO 108

FIG. 1



THJORSÁ RIVER SYSTEM

V.H.L. 600115 JUNE 1966

FIG. 2

PROJECT PLAN

SCALE RATIO 1:50 000

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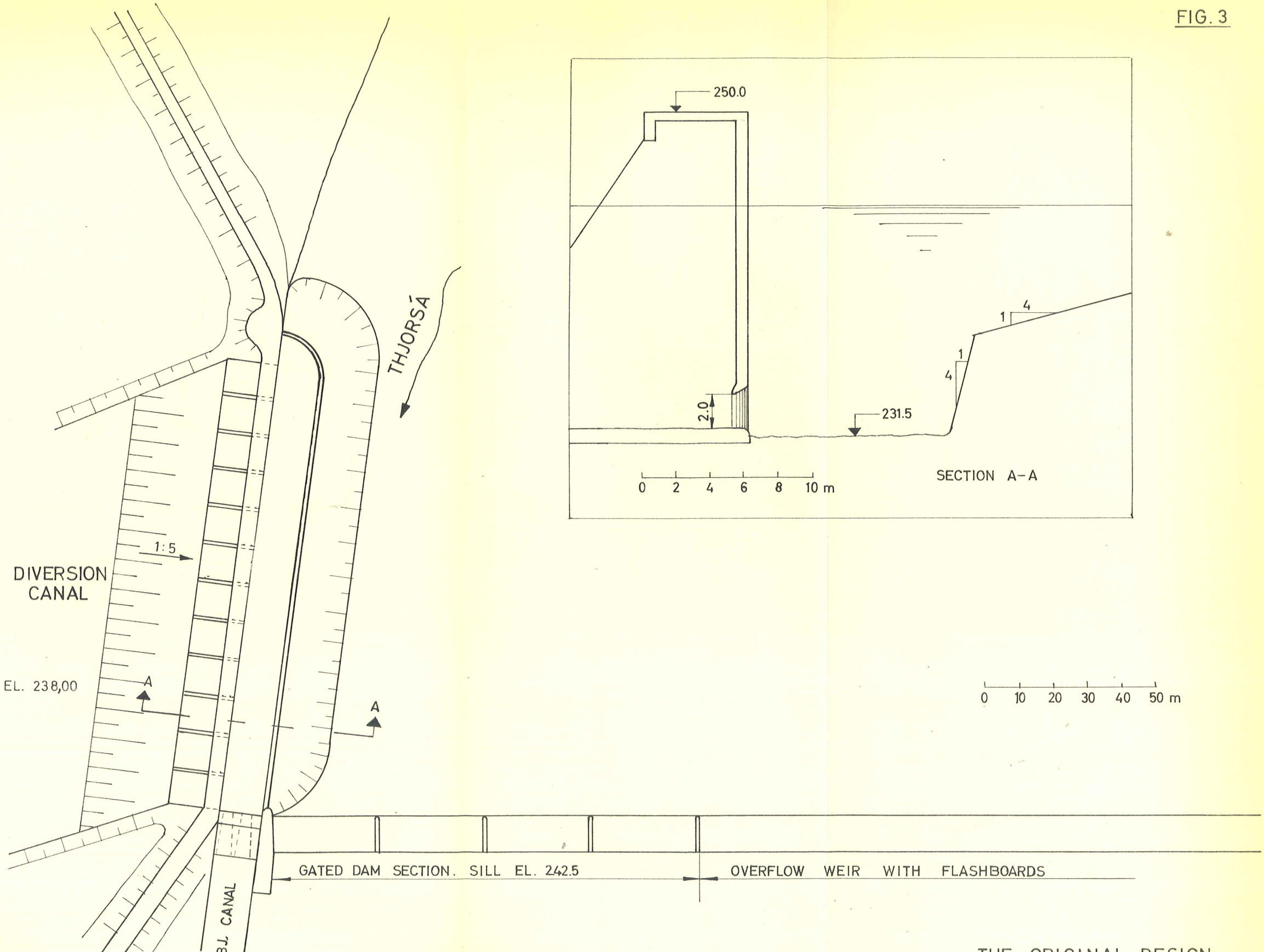
FIG. 3



THE ORIGINAL DESIGN.

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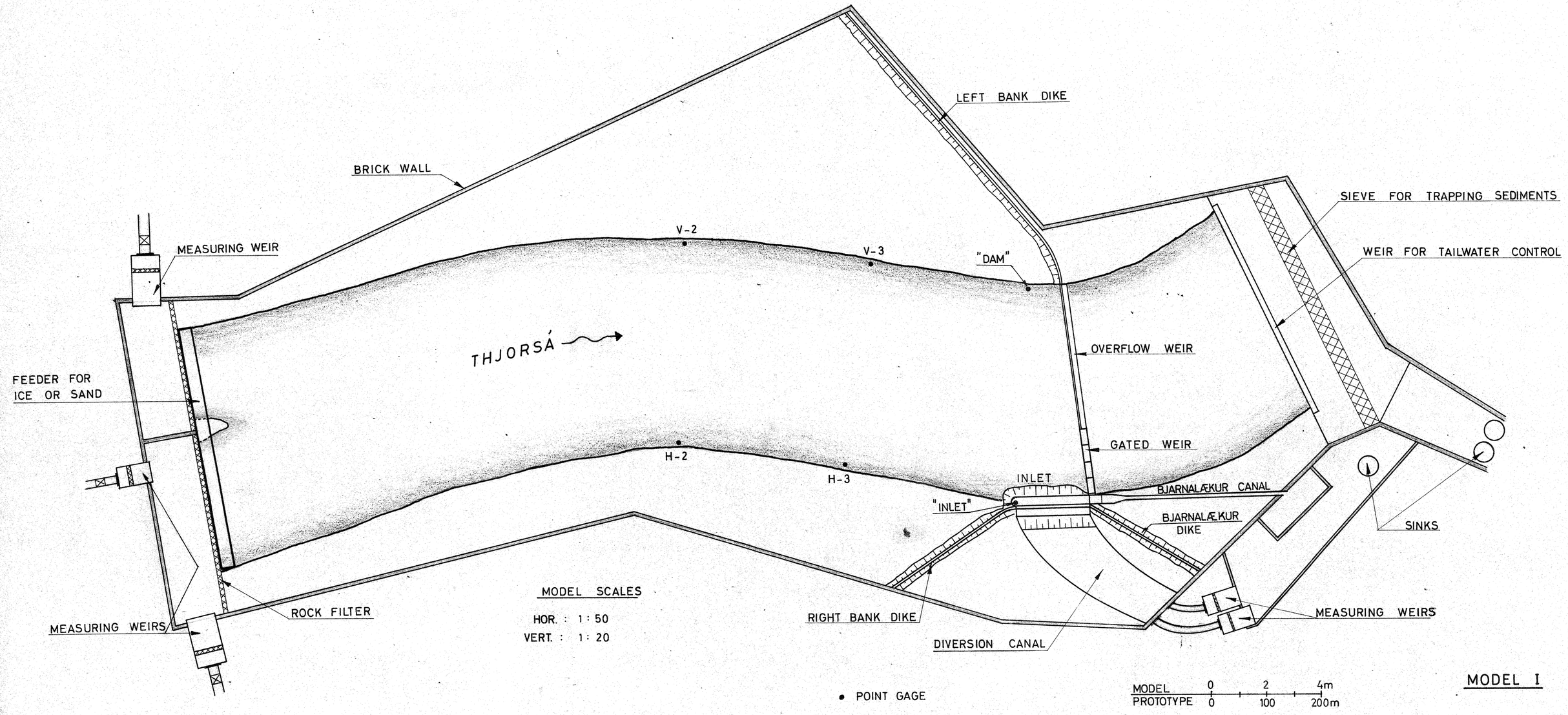
FIG. 4



MODEL I

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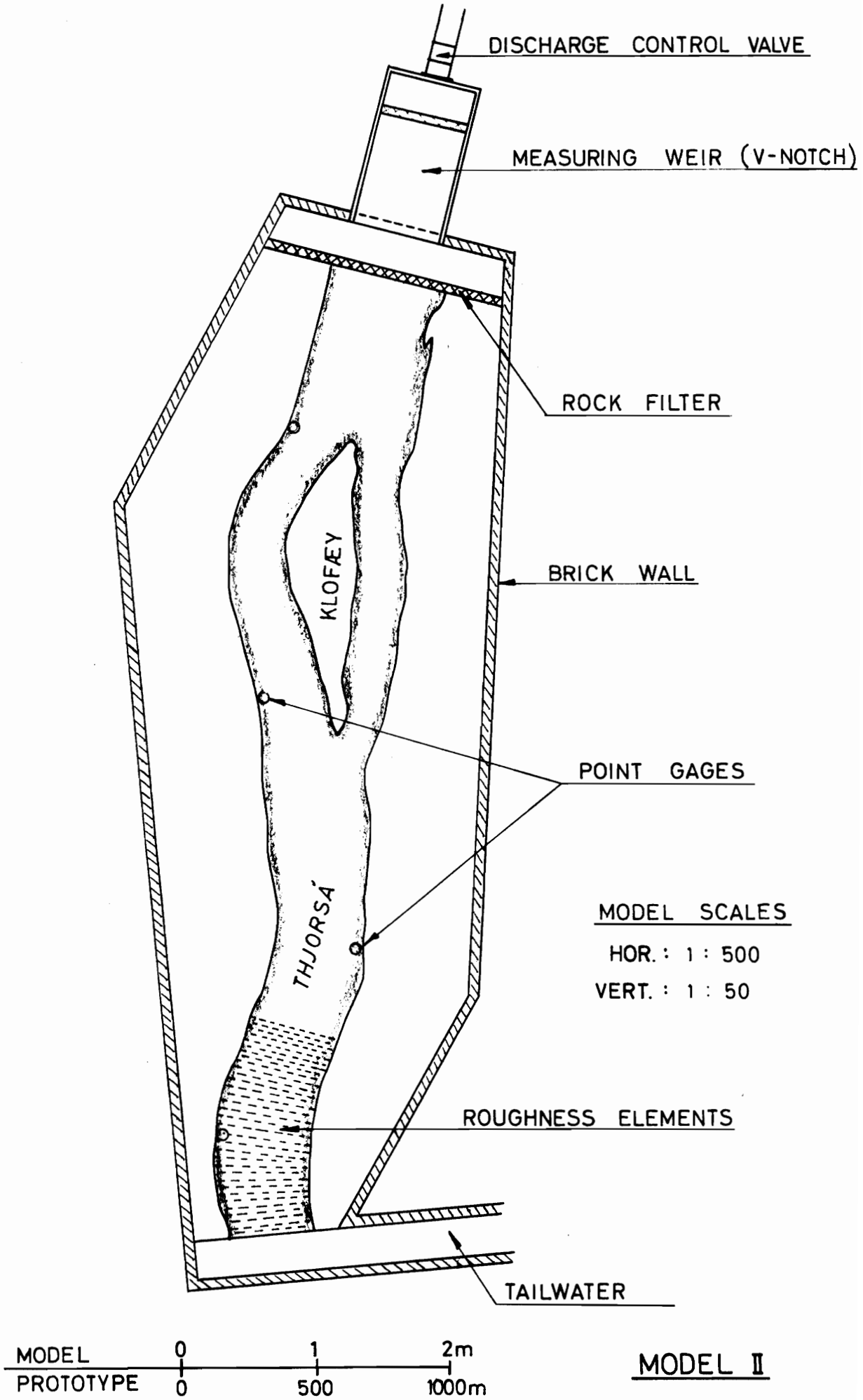
FIG. 5



MODEL II

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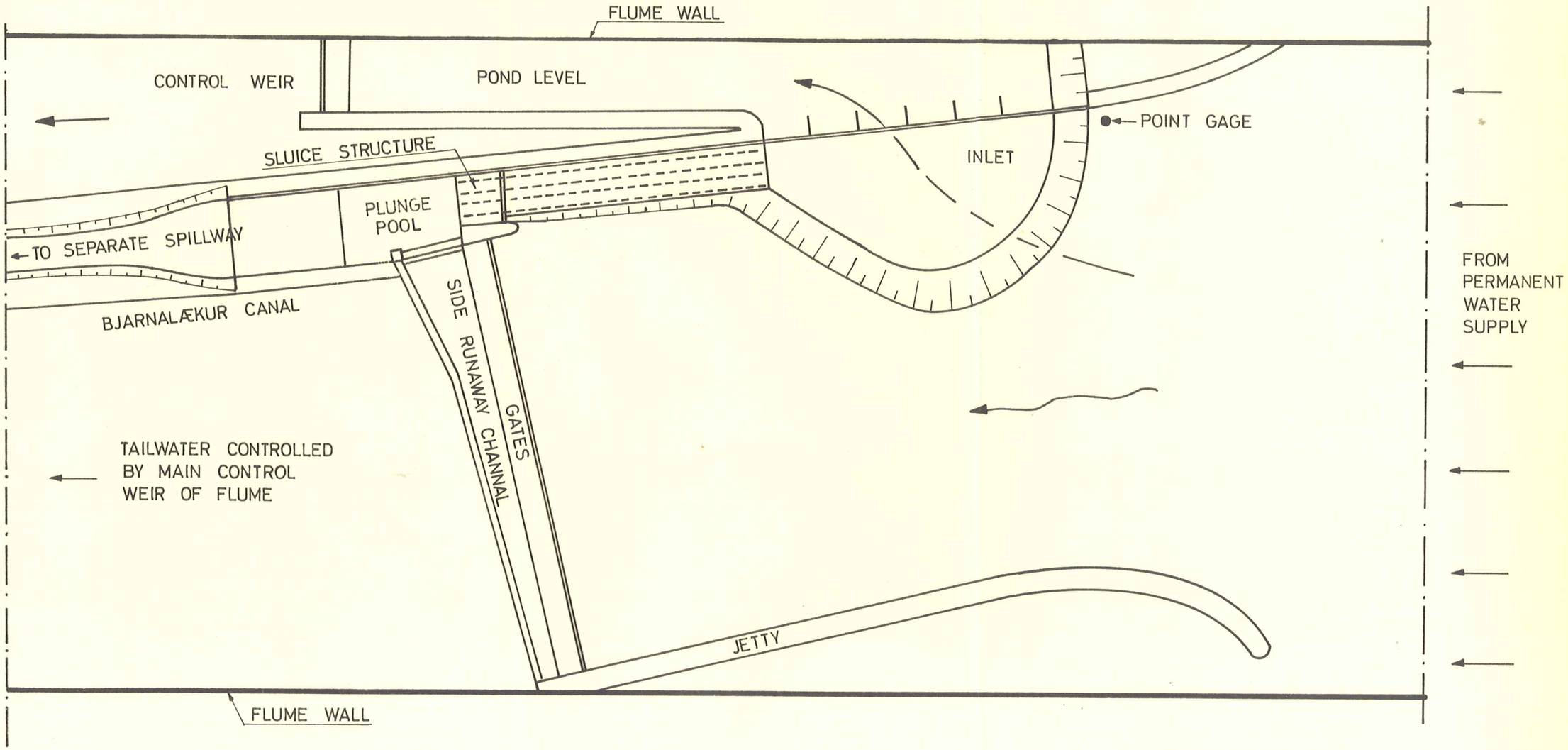

MODEL SCALE RATIO 1:40

0 1 2m MODEL
0 20 40 60 80m PROTOTYPE

THE MODEL WAS LATER COMPLETELY REVISED, COMPARE PHOTO 3

MODEL III
ORIGINAL LAY OUT

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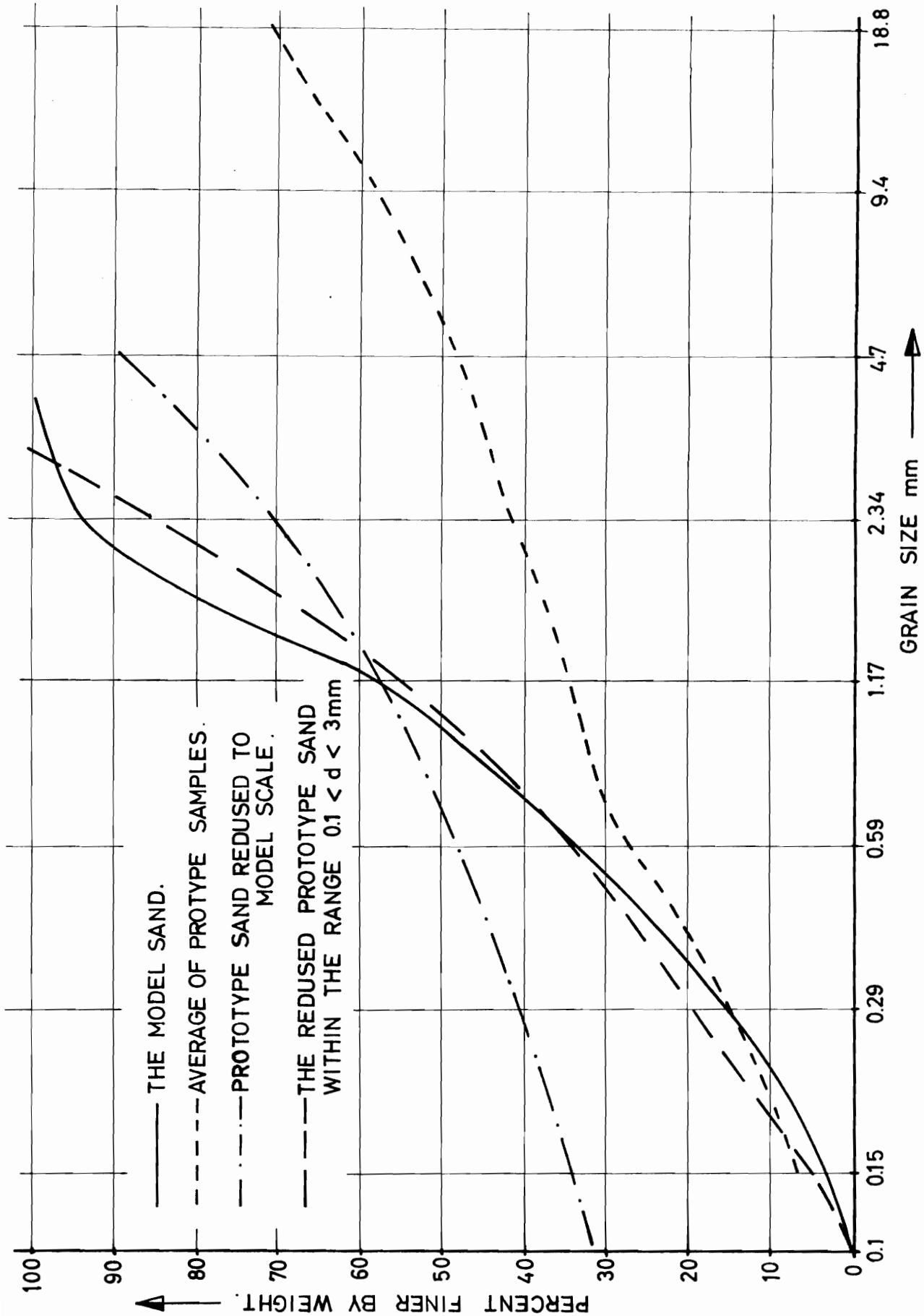
FIG. 7



GRAIN SIZE CURVES
FOR MODEL AND PROTOTYPE

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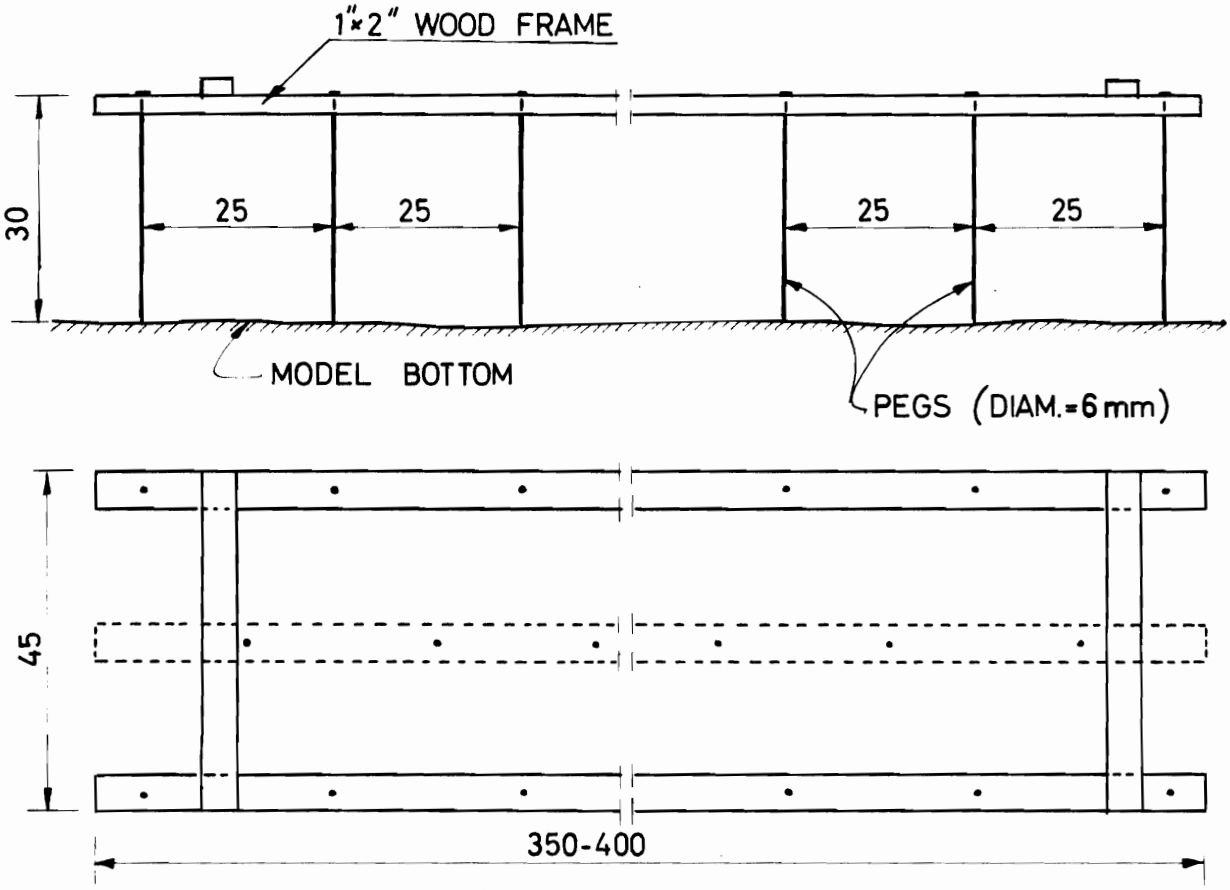
FIG. 8

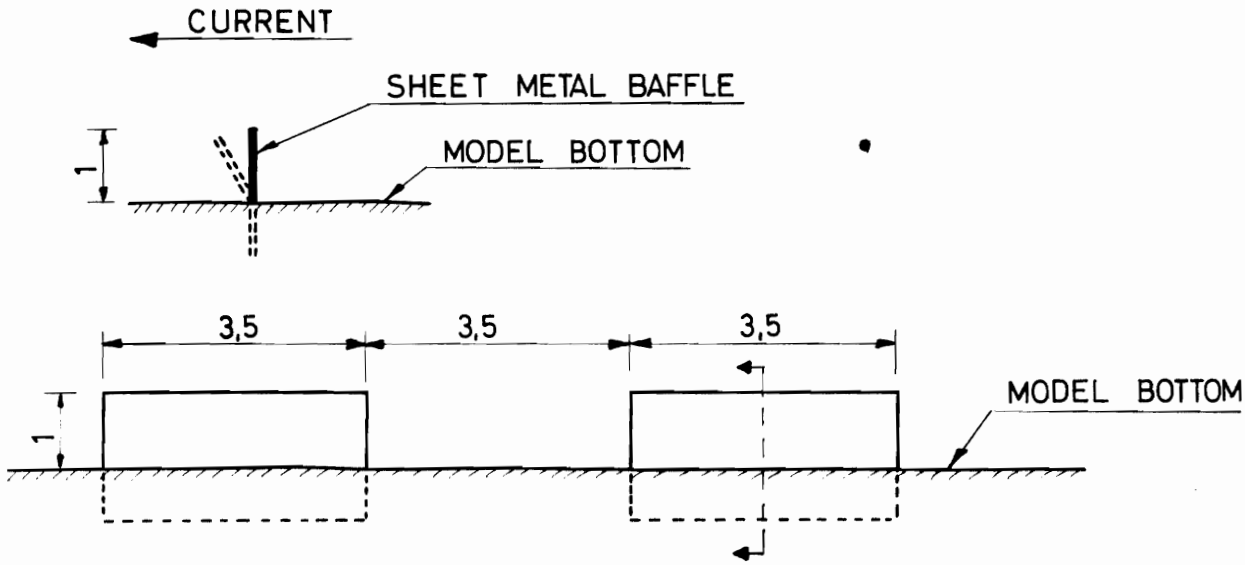


ROUGHNESS ELEMENTS. MODEL I

SCALE RATIO 1:10

CENTIMETRES

FIG. 9

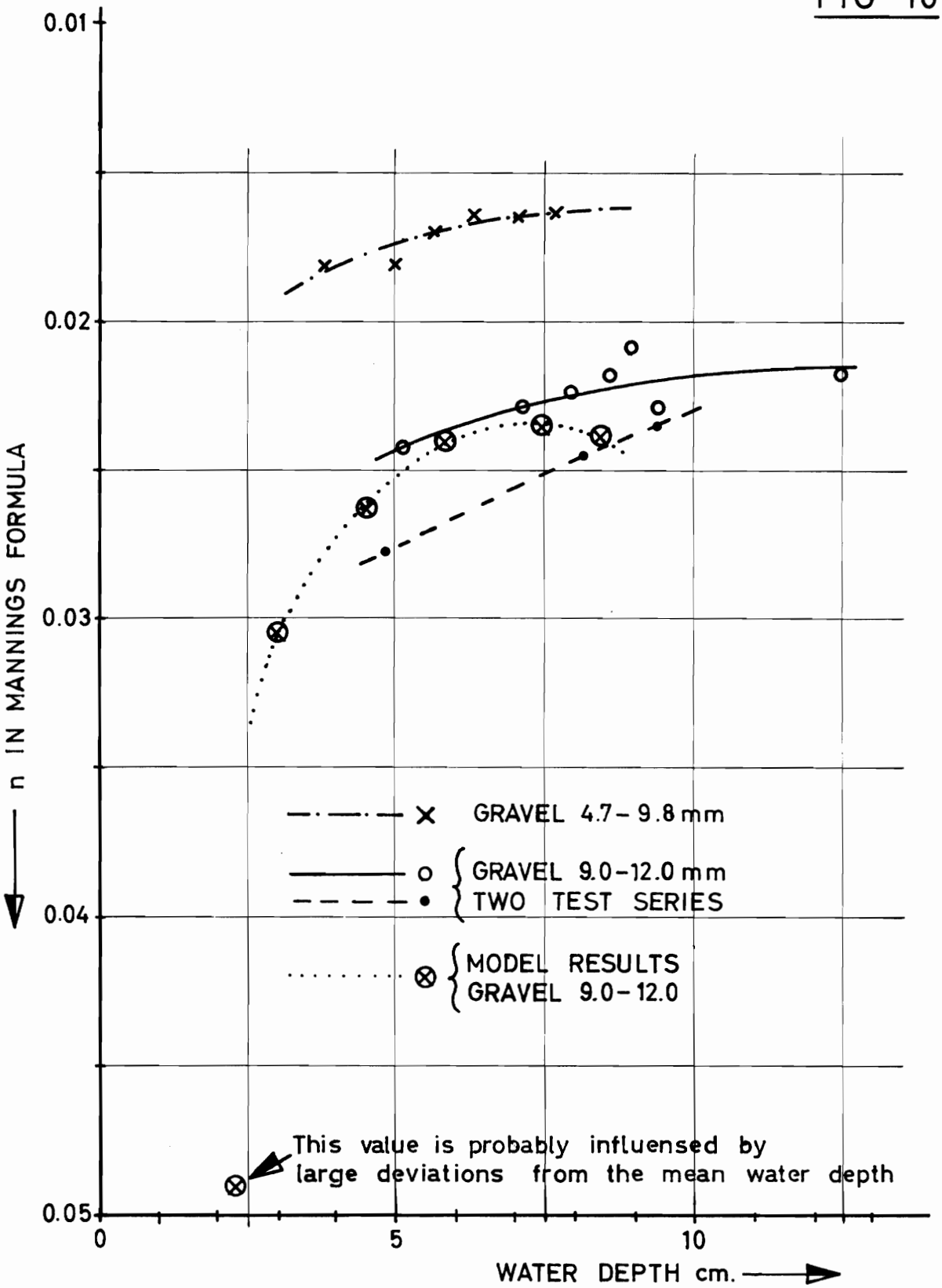


ROUGHNESS ELEMENTS. MODEL II

SCALE RATIO 1:1

CENTIMETRES

FIG 10.



FLUME STUDY OF GRAVEL BED ROUGHNESS COMPARED WITH DATA FROM MODEL I.

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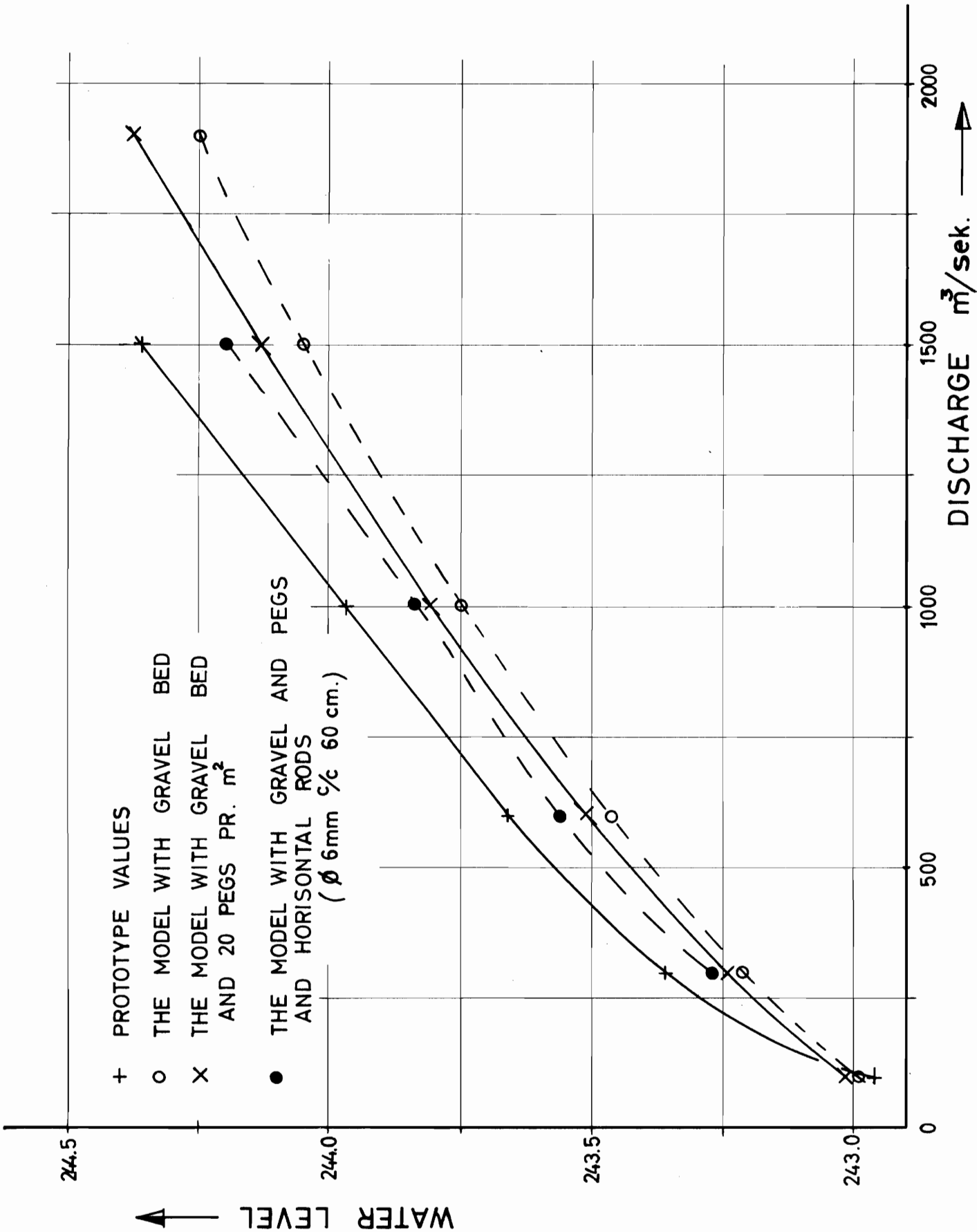
FIG. 11.



CALIBRATION OF MODEL I.

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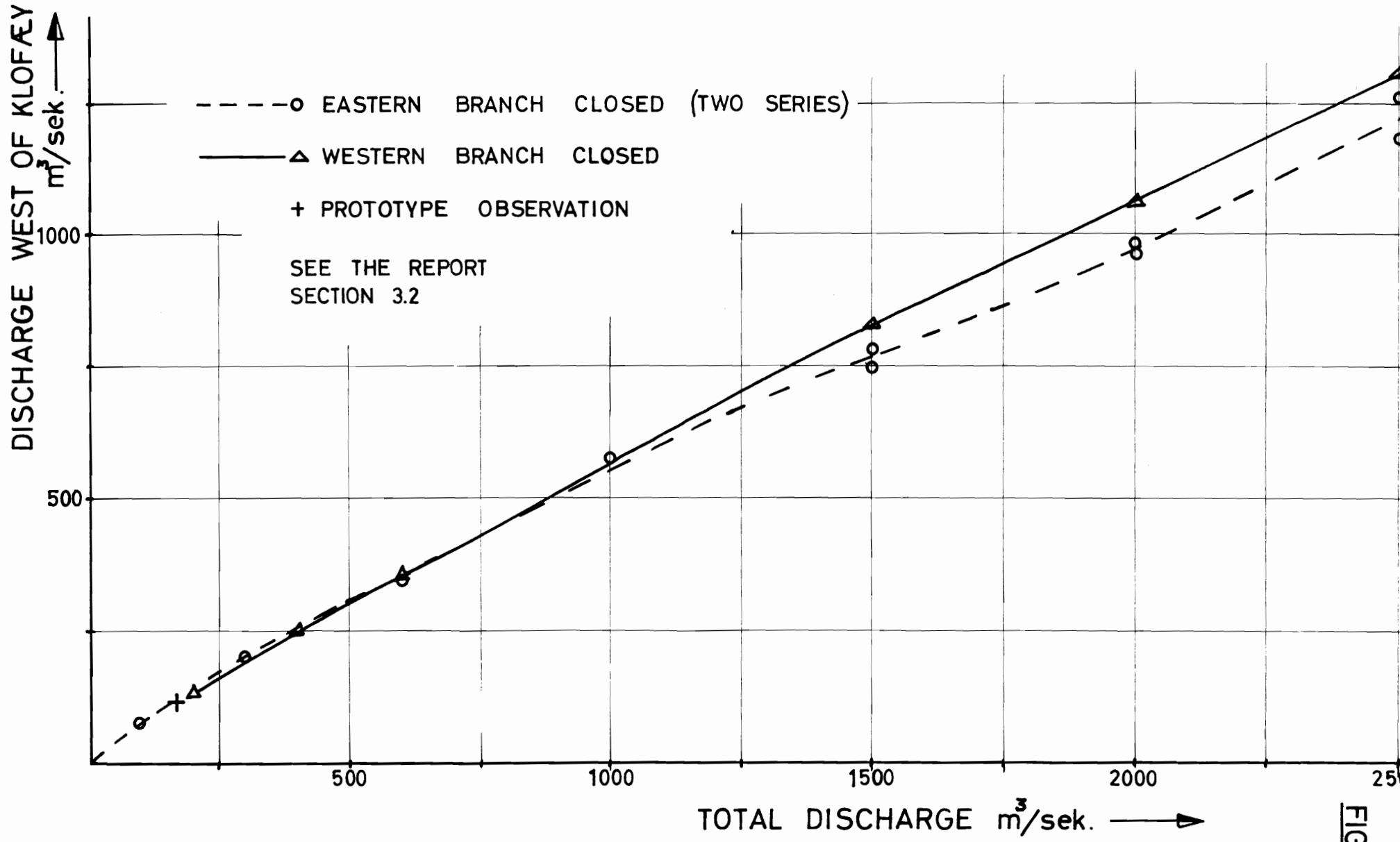

FIG 12.

DISCHARGE EAST AND WEST OF KLOFÆY.

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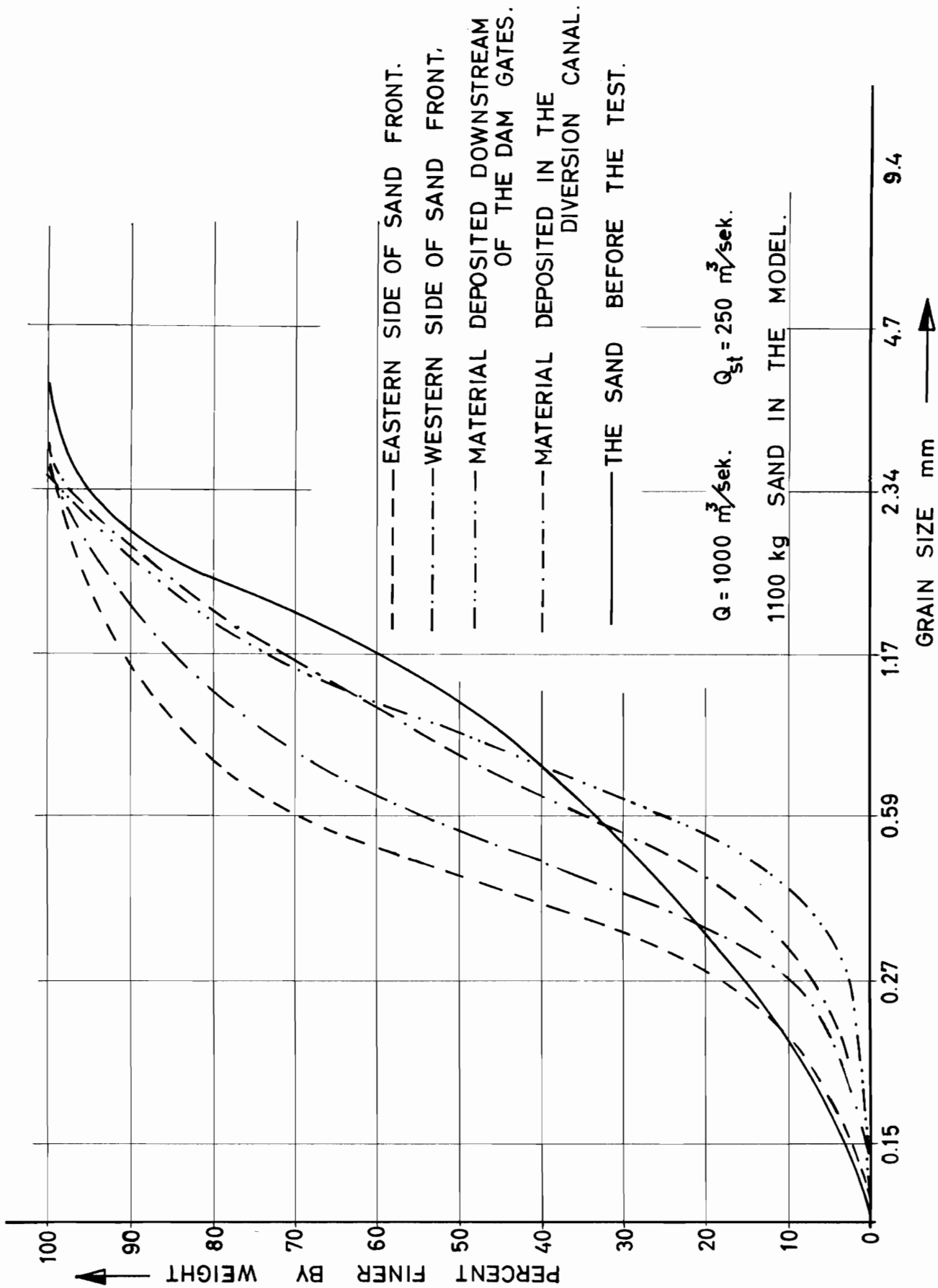
FIG. 13.



SAND SAMPLES AFTER 23 HOURS INTERRUPTED SAND RUN.

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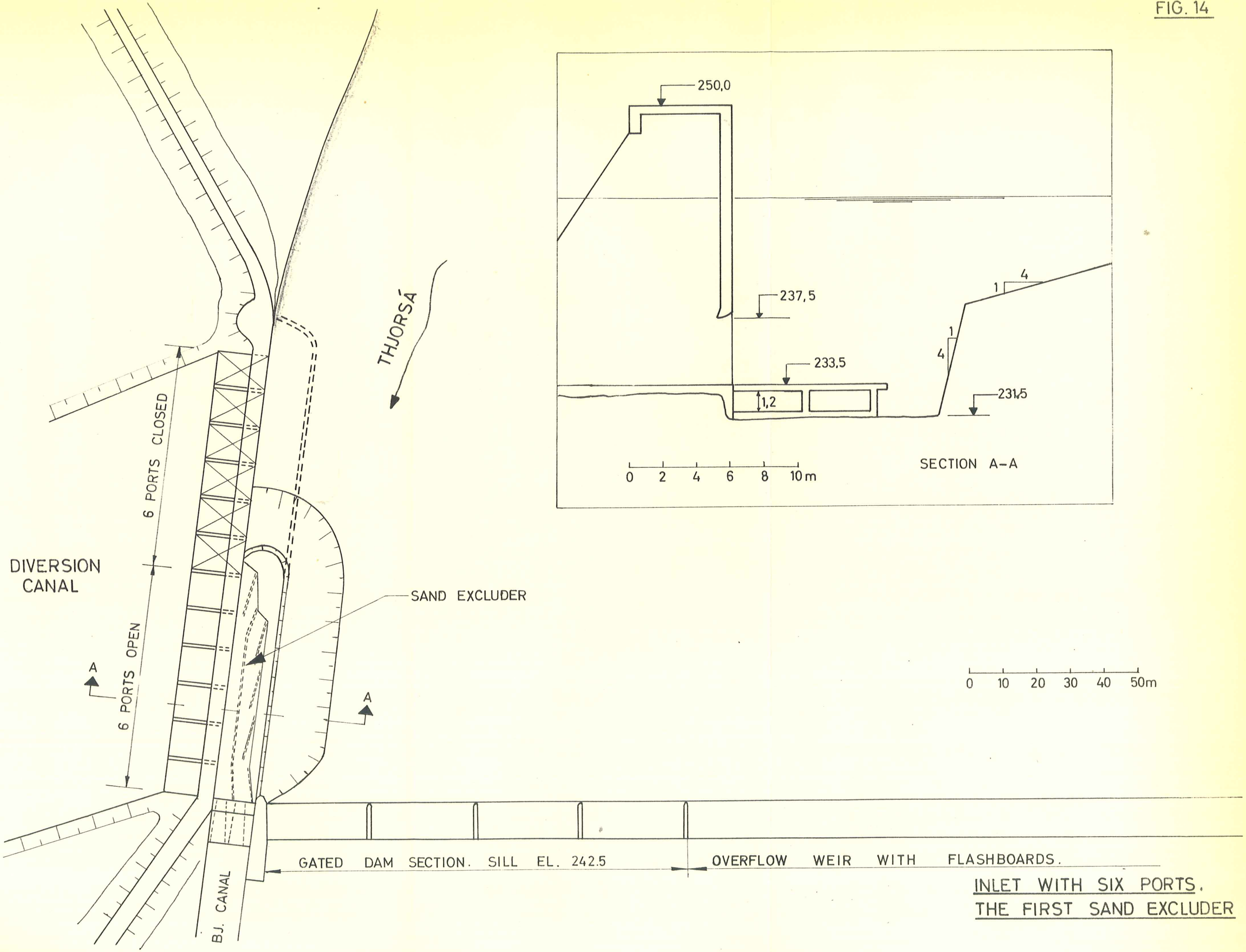

INLET WITH SIX PORTS.
THE FIRST SAND EXCLUDER

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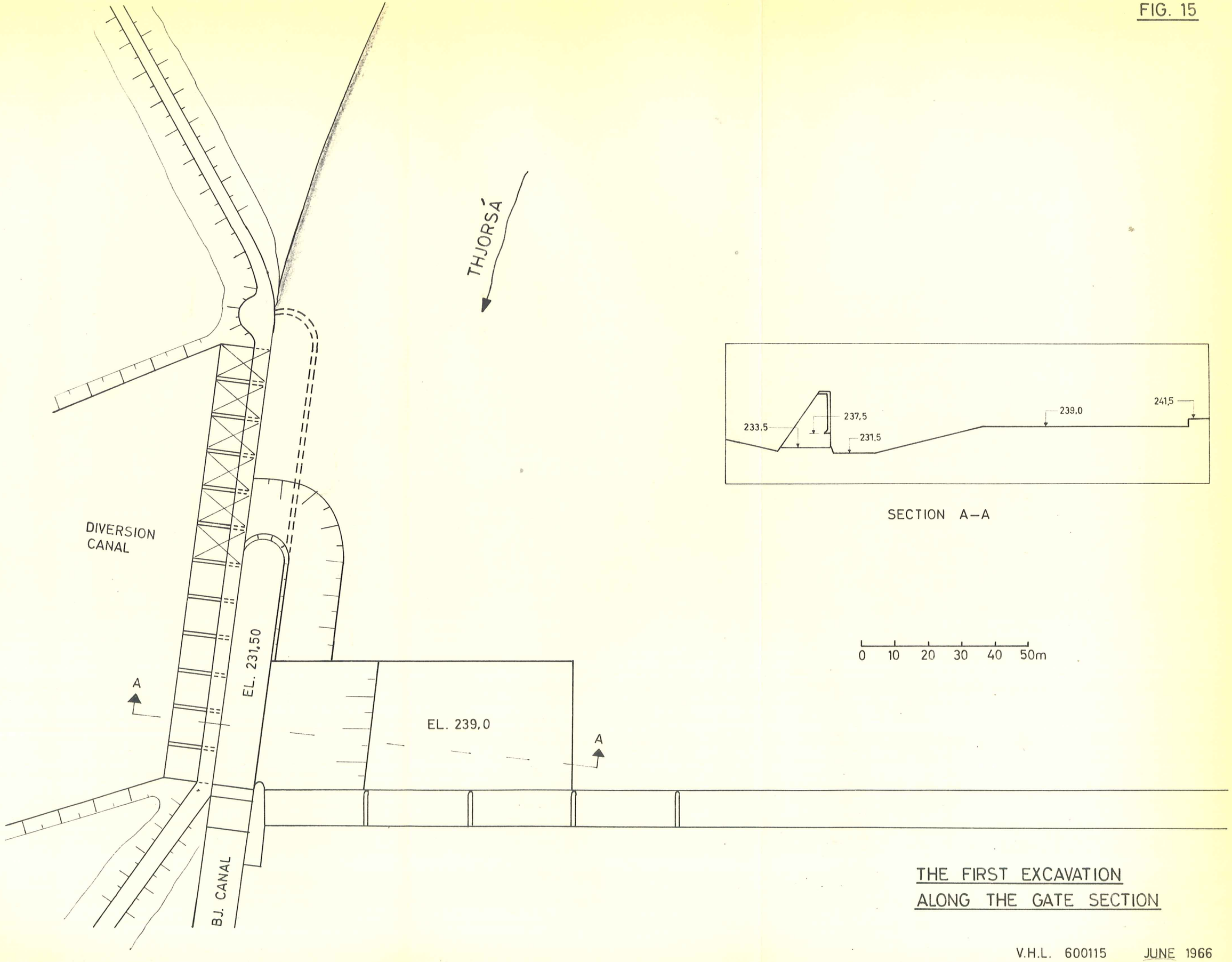
FIG. 15



THE FIRST EXCAVATION
ALONG THE GATE SECTION

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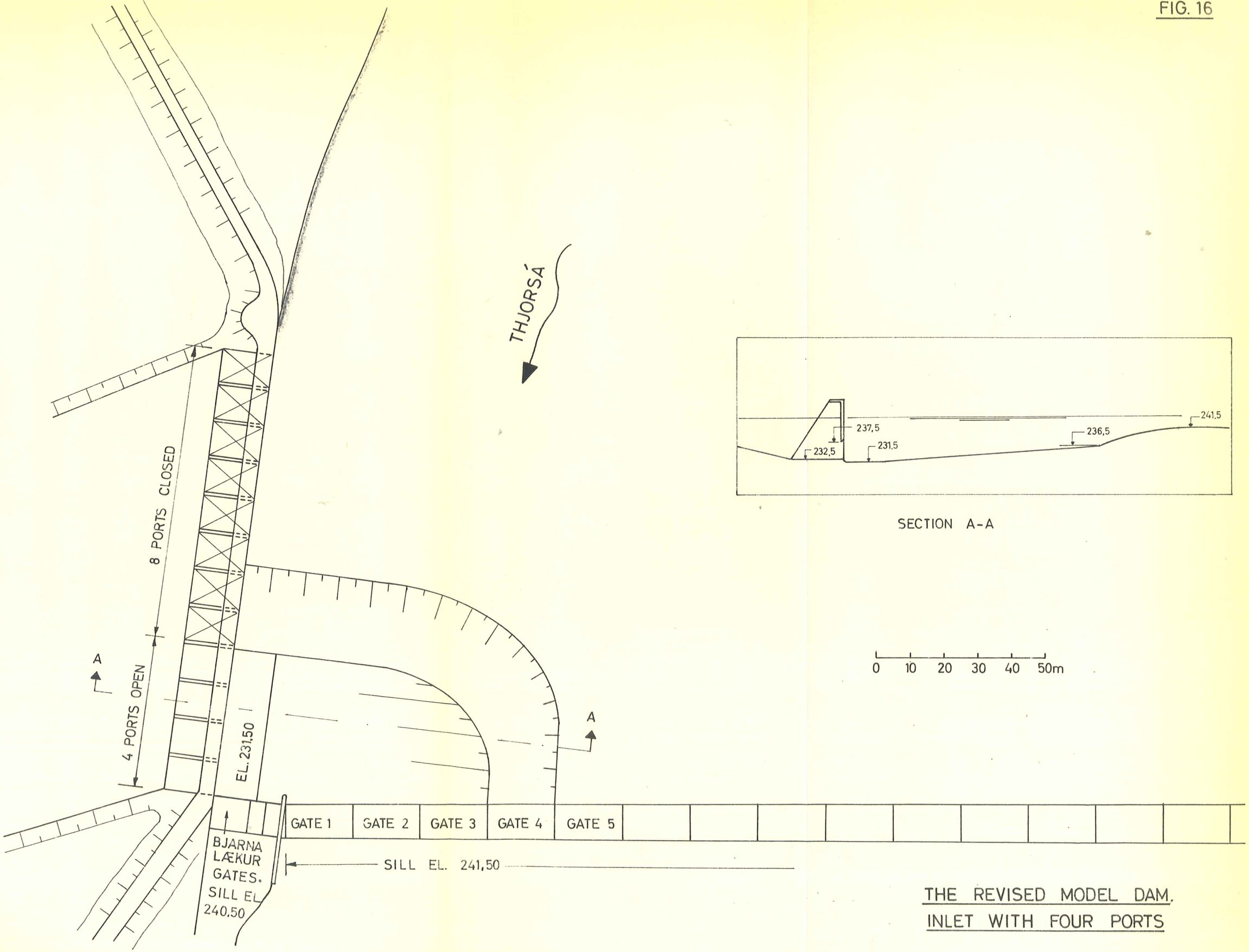
FIG. 16



THE REVISED MODEL DAM.
INLET WITH FOUR PORTS

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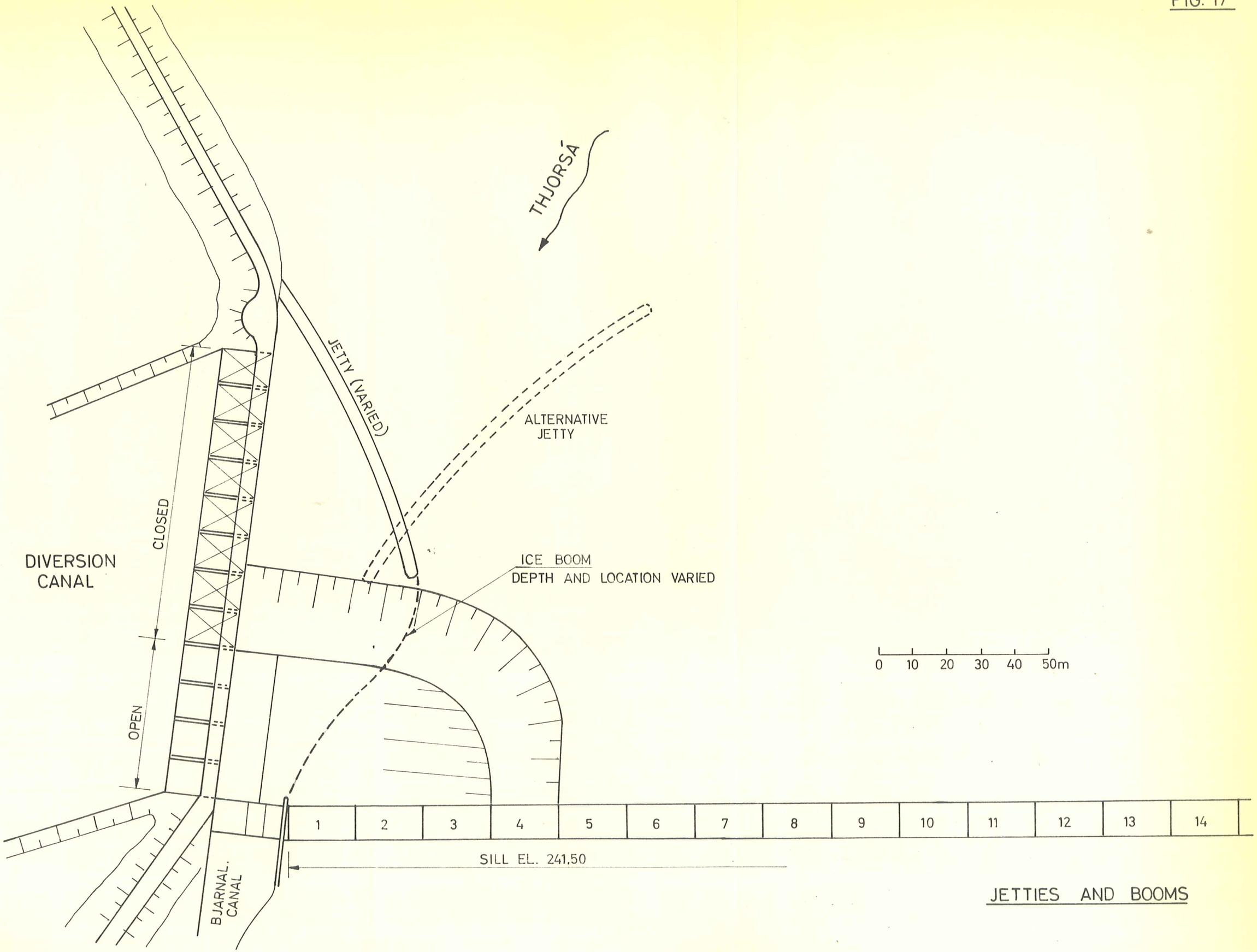

JETTIES AND BOOMS

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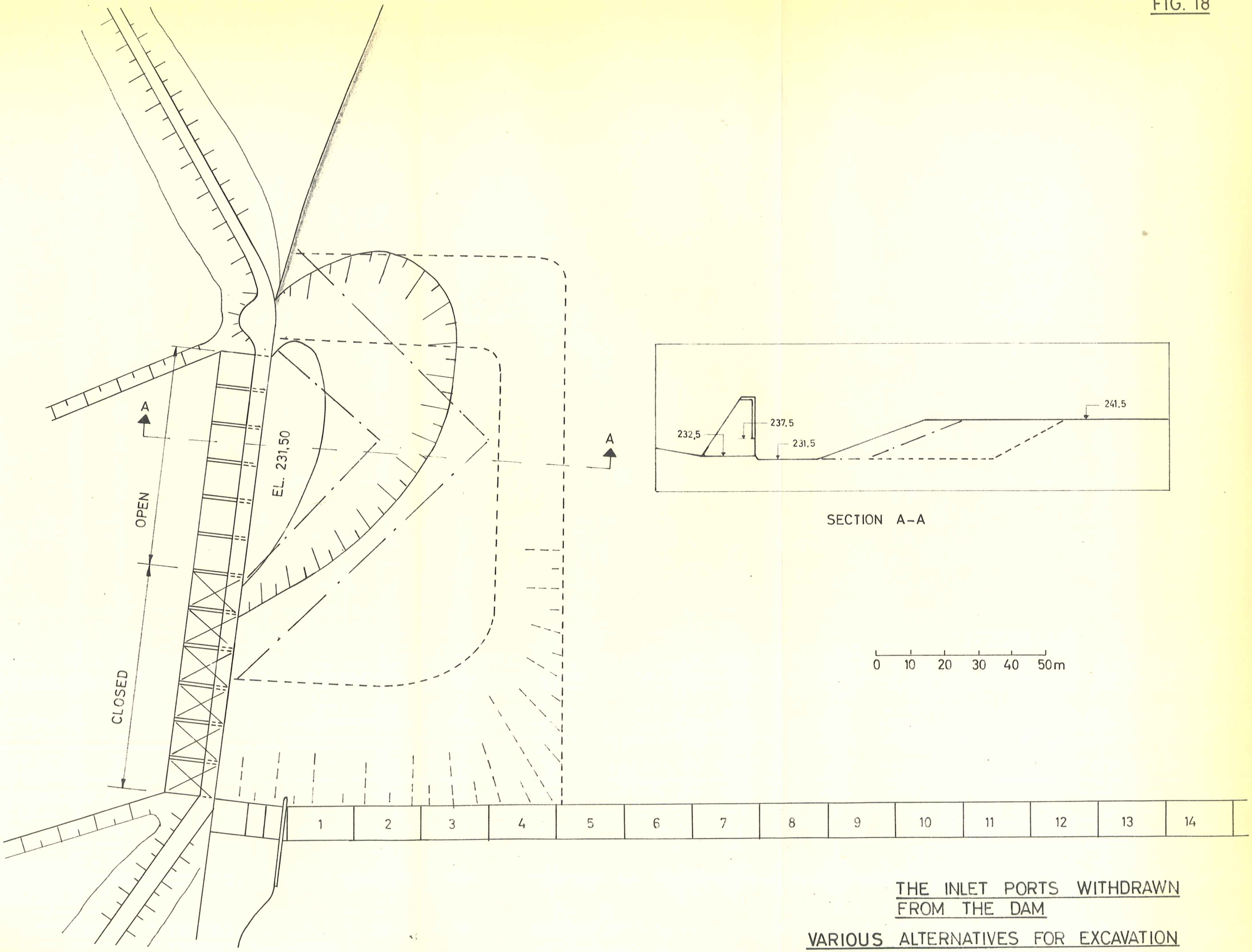

THE INLET PORTS WITHDRAWN FROM THE DAM

VARIOUS ALTERNATIVES FOR EXCAVATION

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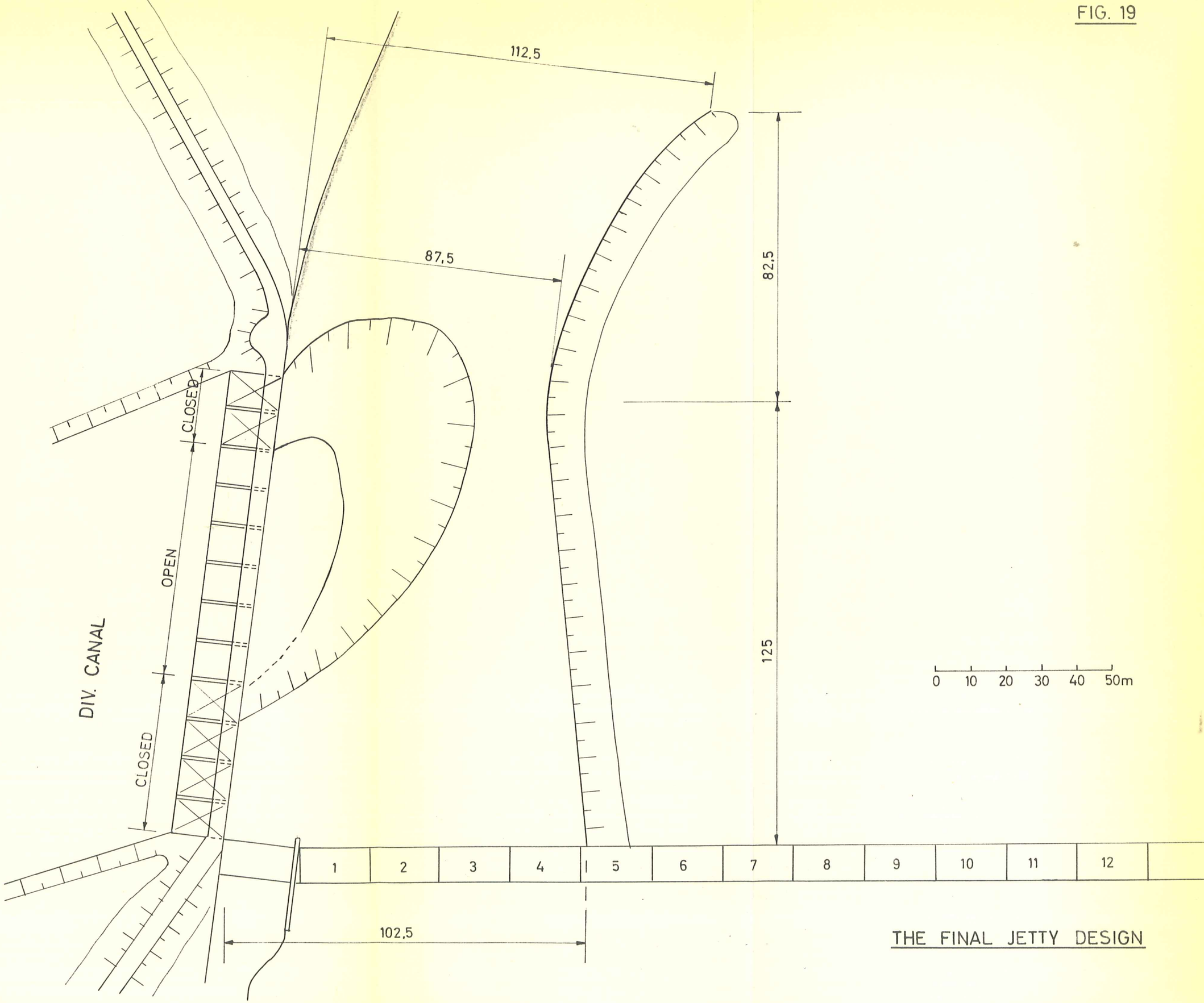
FIG. 19



THE FINAL JETTY DESIGN

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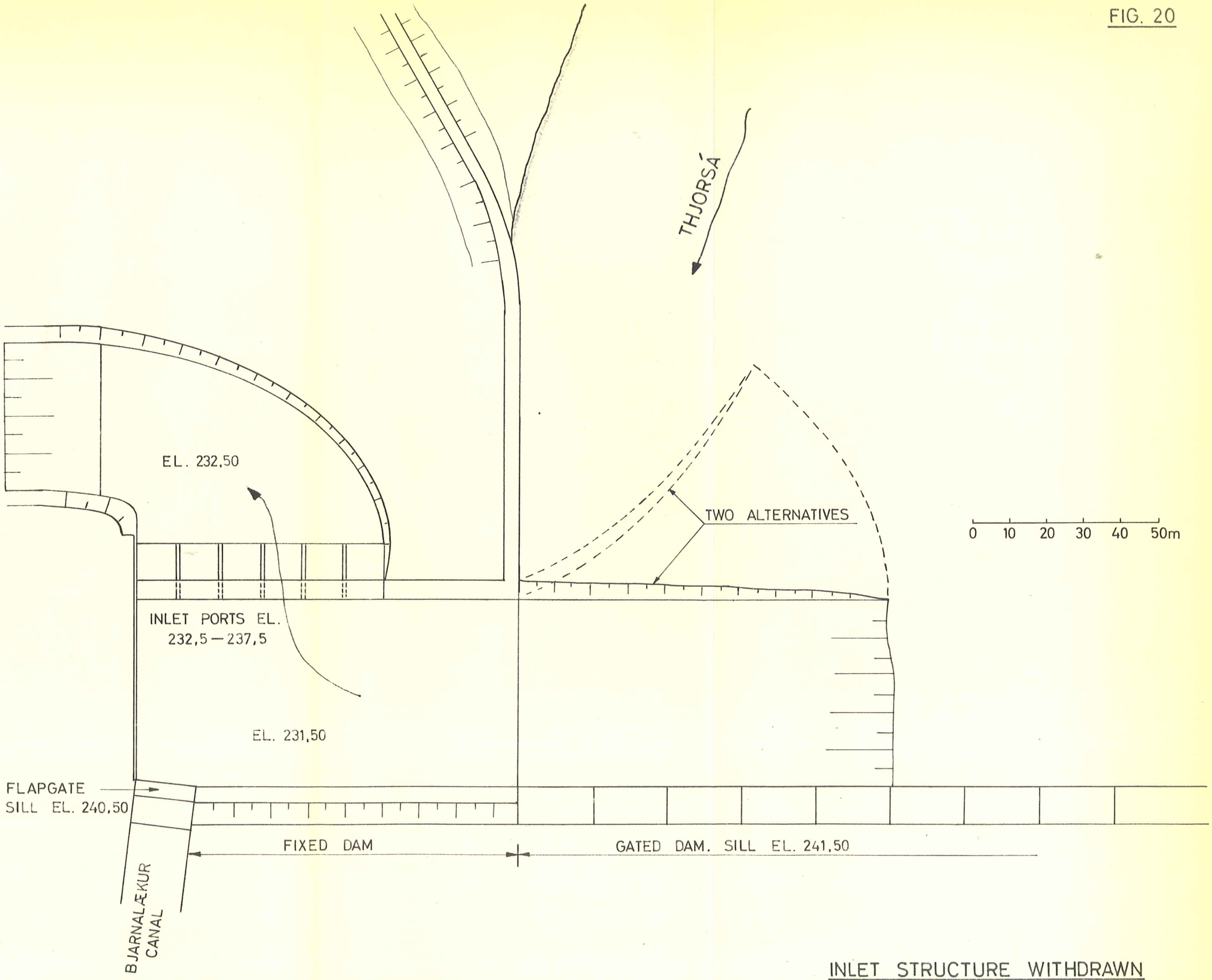
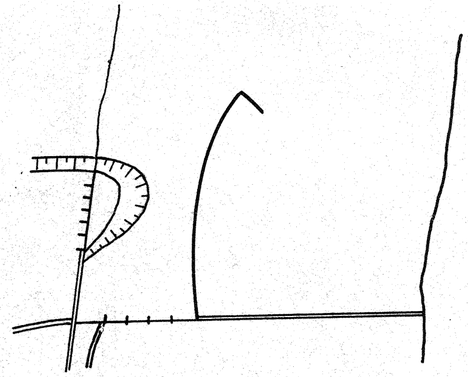
FIG. 20



INLET STRUCTURE WITHDRAWN
FROM THE RIVER BANK

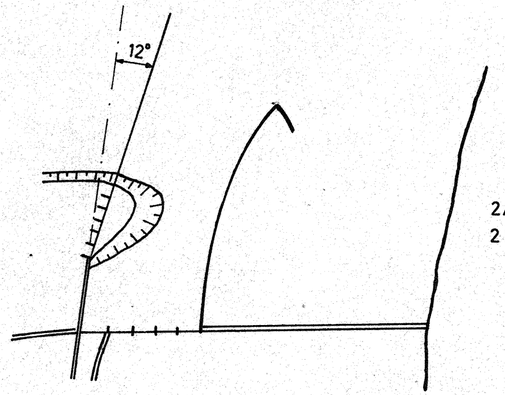
V.H.L. 600115 JUNE 1966



SCHEME 1

PEGS IN THE MODEL

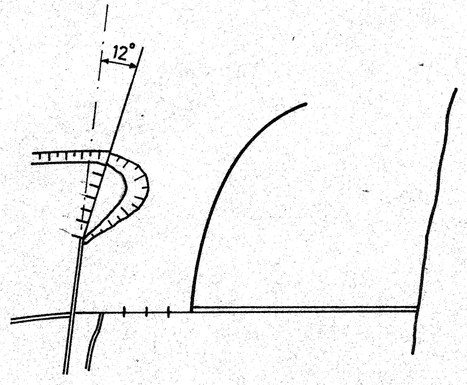
INLET PORTS EL.
232,5 – 237,5



SCHEME 2

2A – WITH PEGS
2 B – WITHOUT PEGS

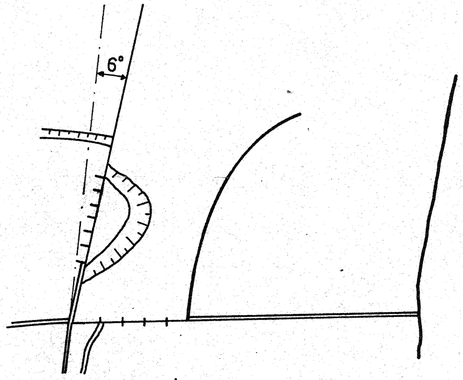
INLET PORTS EL.
232,5 – 237,5



SCHEME 3

WITHOUT PEGS

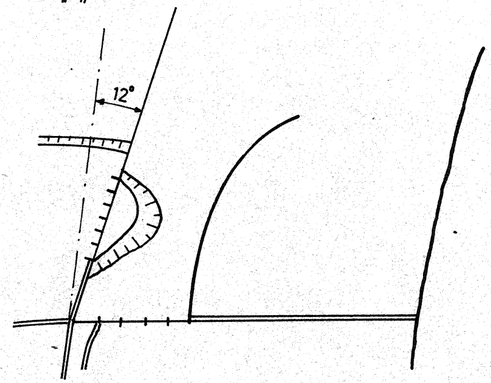
INLET PORTS EL.
232,5 – 237,5



SCHEME 4

WITHOUT PEGS

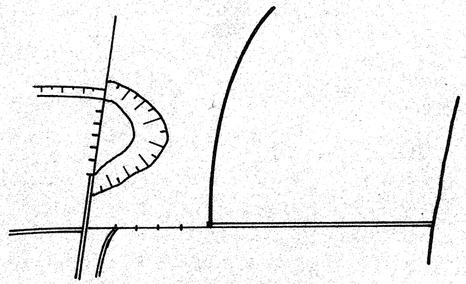
INLET PORTS EL.
232,5 – 237,5



SCHEME 5

WITHOUT PEGS

INLET PORTS EL.
232,5 – 237,5



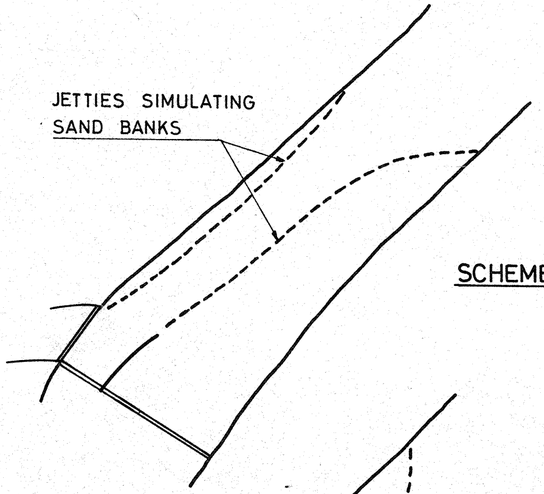
SCHEME 6

INLET PORTS EL.
235,0 – 240.0

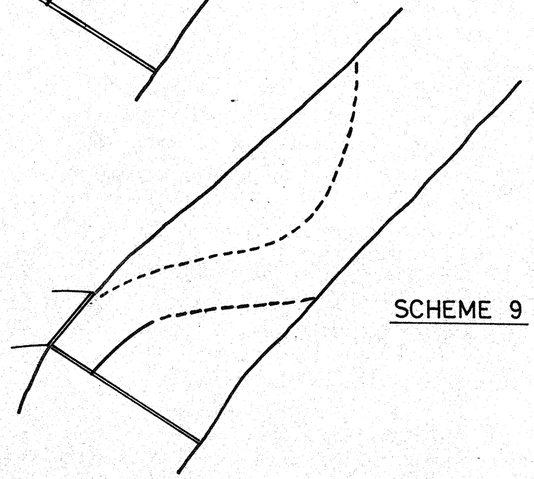
NO PEGS

SCHEME 7

JETTY AND INLET AS SCHEME 1,
BUT INLET PORTS EL. 235,0 – 240,0
EXCAVATON AS ON SCHEME 6
NO PEGS



SCHEME 8



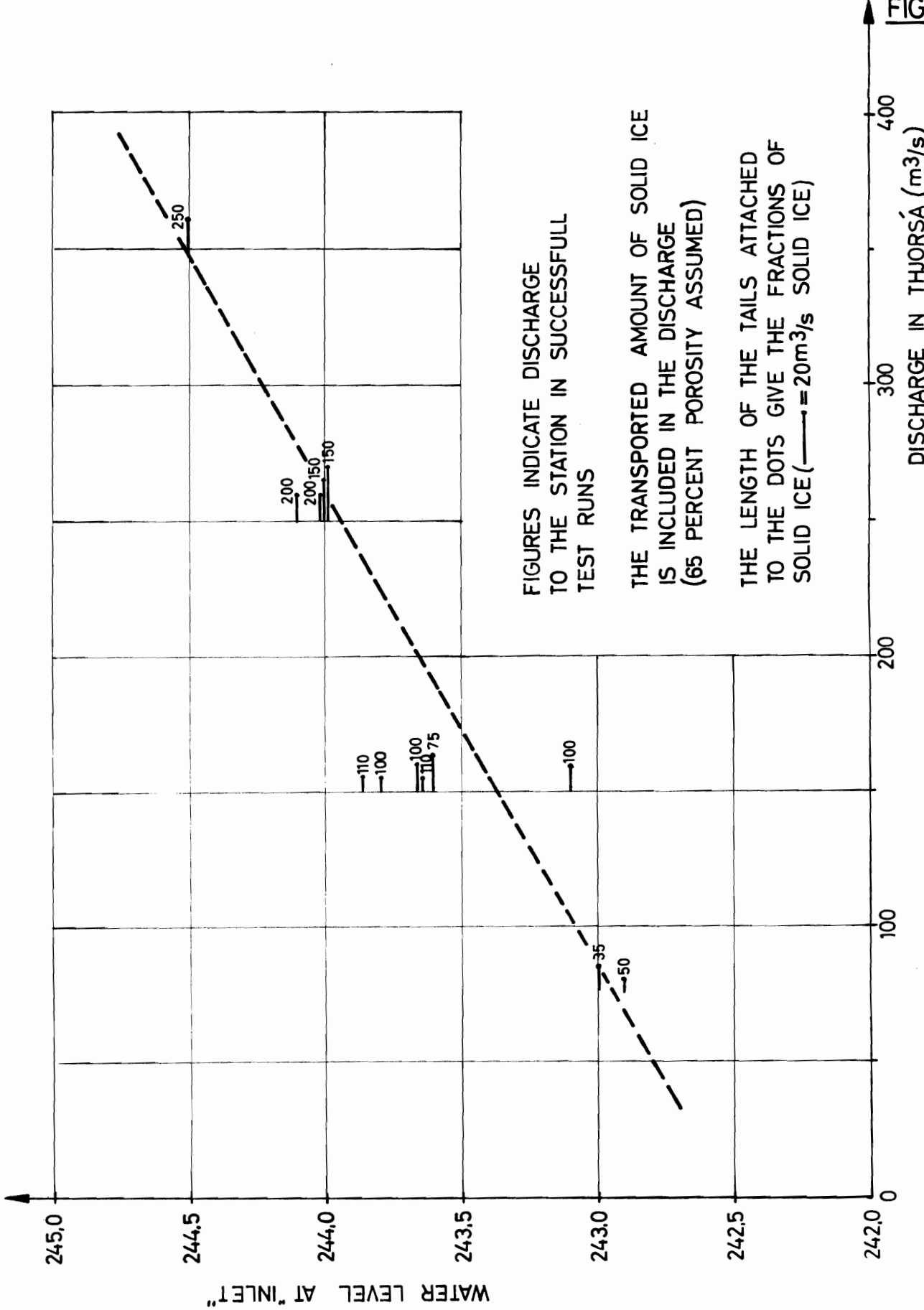
SCHEME 9

SCHEME 1–7 : 0 100 200m

SCHEME 8–9 : 0 100 200 300 400 500m

SCHEMES 1–9 TO ICE RUNS REFERRED IN TABLES 1–6

FIG. 22



ICE AND WATER LEVEL IN THE INLET AREA
RESULTS FROM JETTY DESIGNS WITHOUT THE TROUGH

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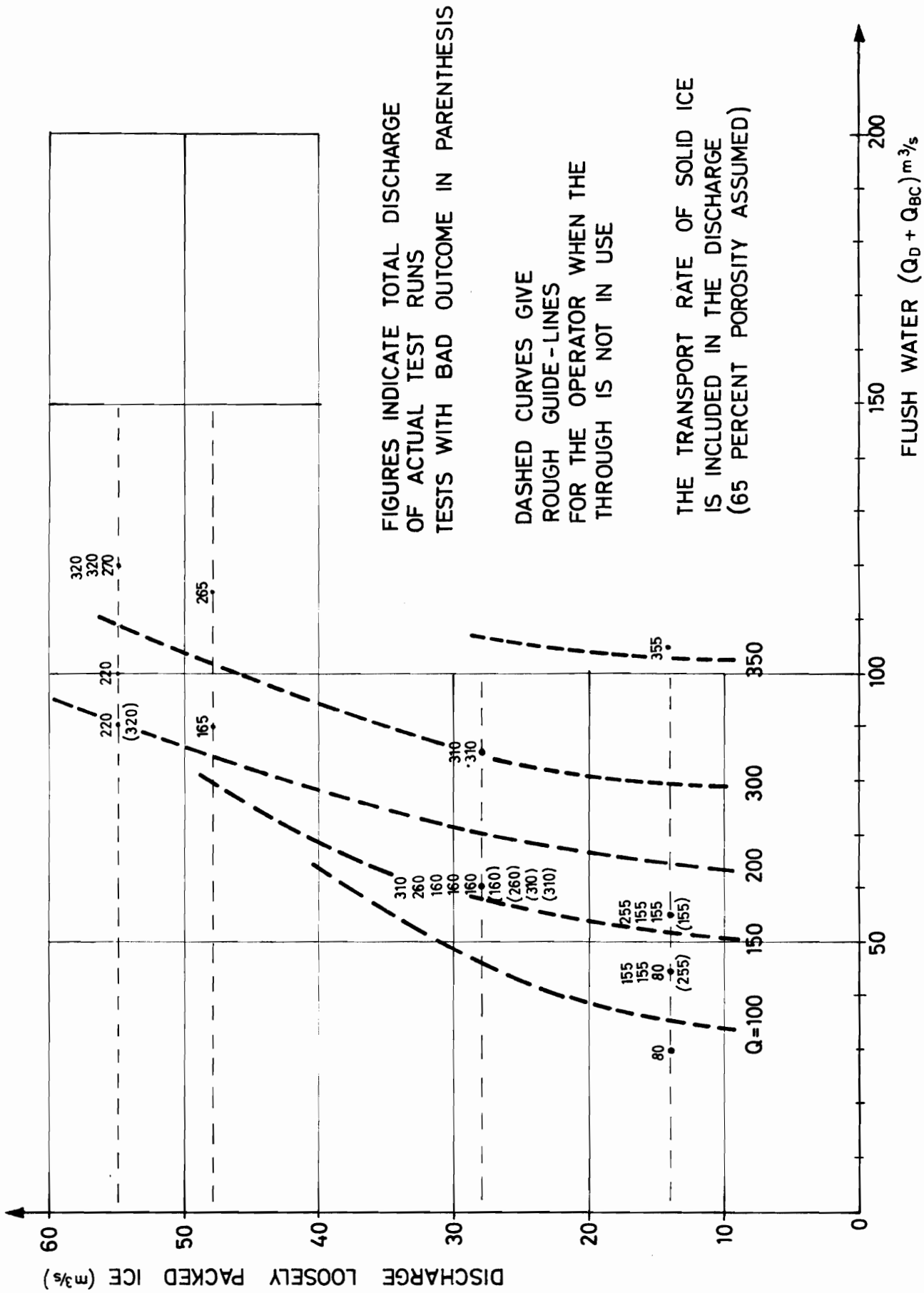

ICE AND FLUSH WATER
RESULTS FROM JETTY DESIGNS
WITHOUT THE TROUGH

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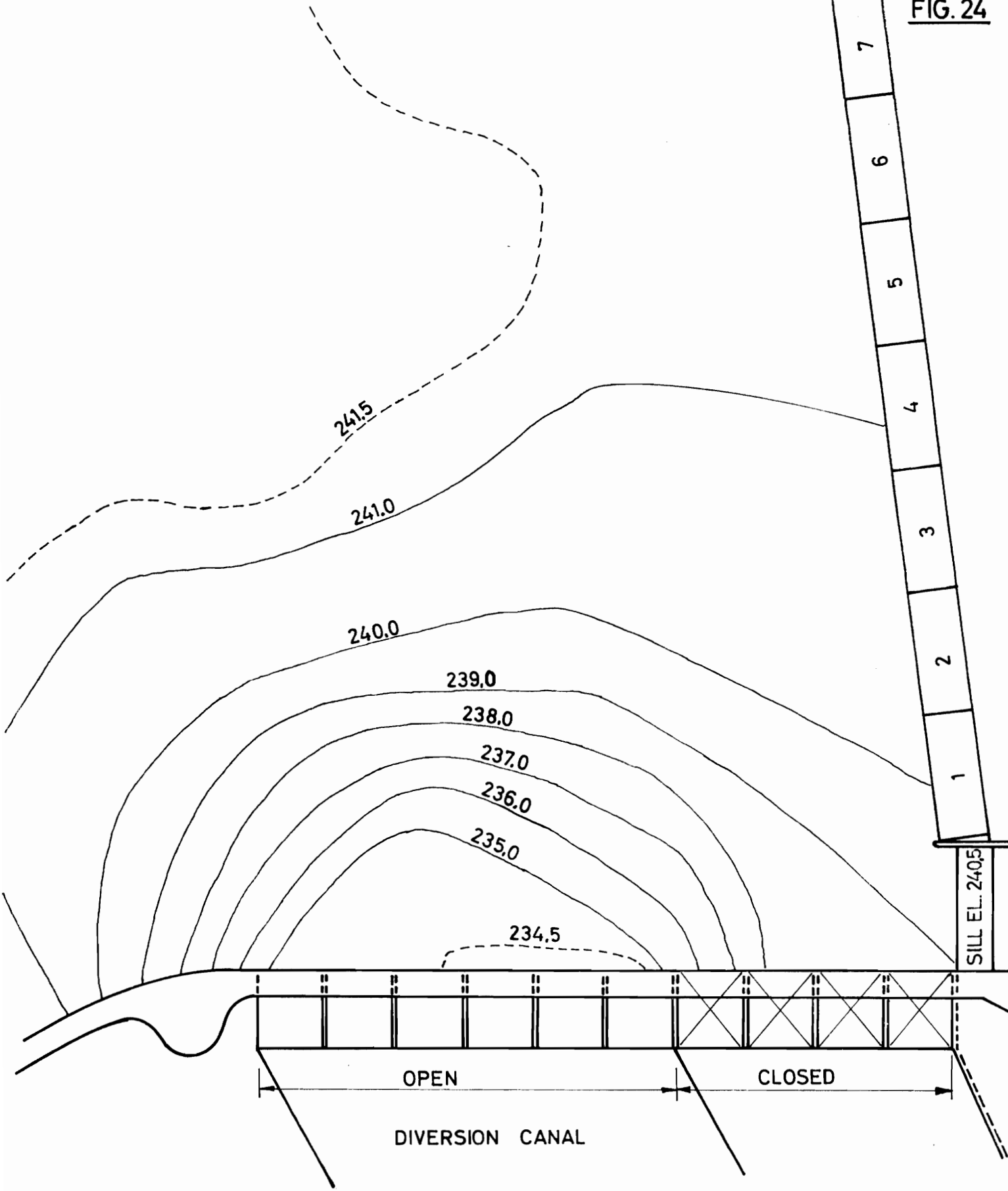
FIG. 24



INLET PORTS EL. 235,0 – 240.0

0 10 20 30 40 50m

INLET AREA DURING INITIAL CONSTRUTION STUDY

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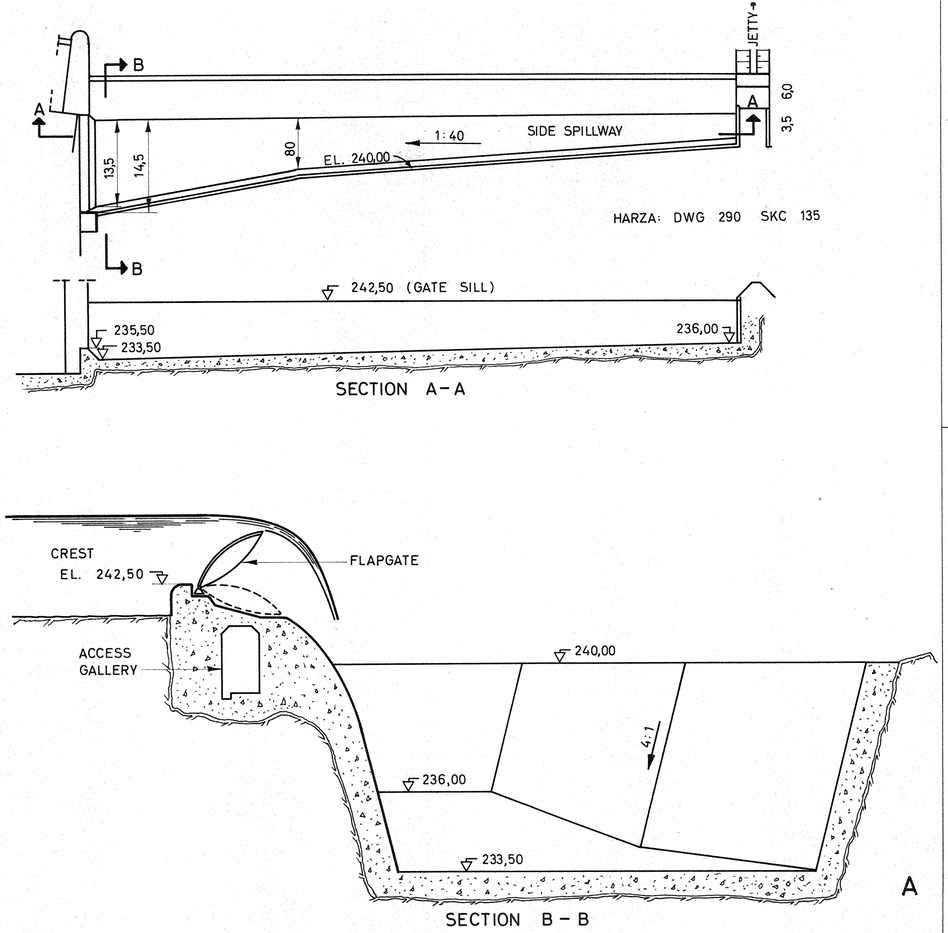
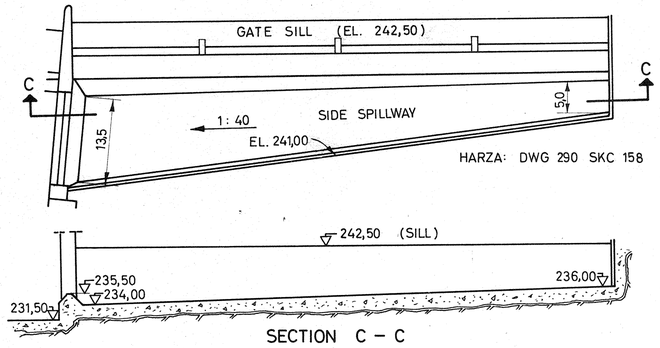
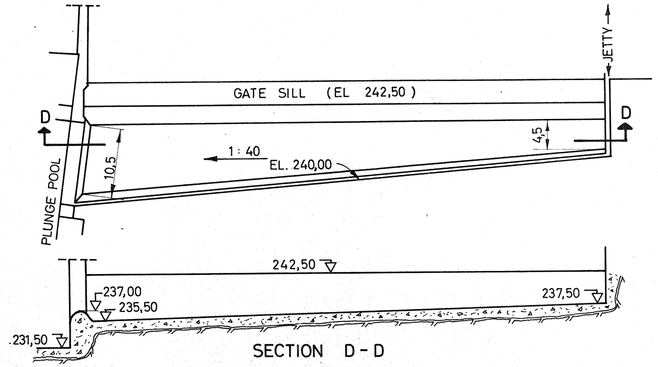
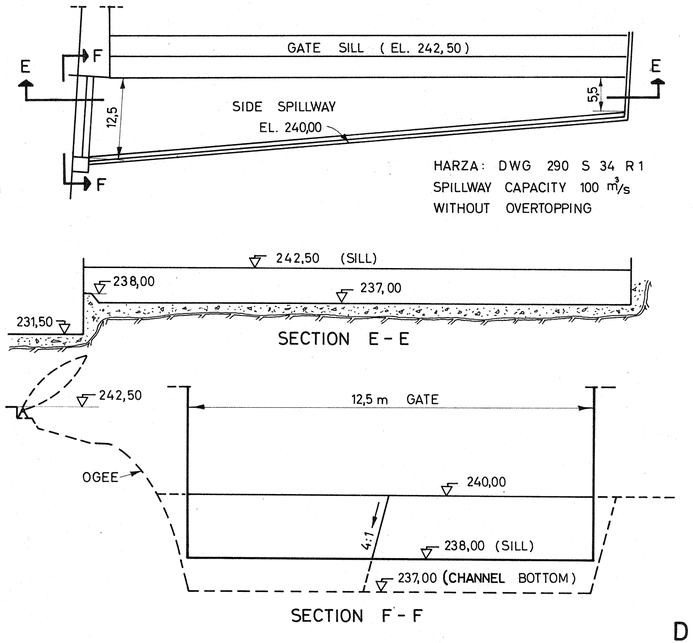
FIG. 25









CROSS SECTIONS 0 1 2 3 4 5m

OTHER 0 5 10 15 20 25m

THE SIDE RUNAWAY CHANNEL

VARIOUS DESIGNS

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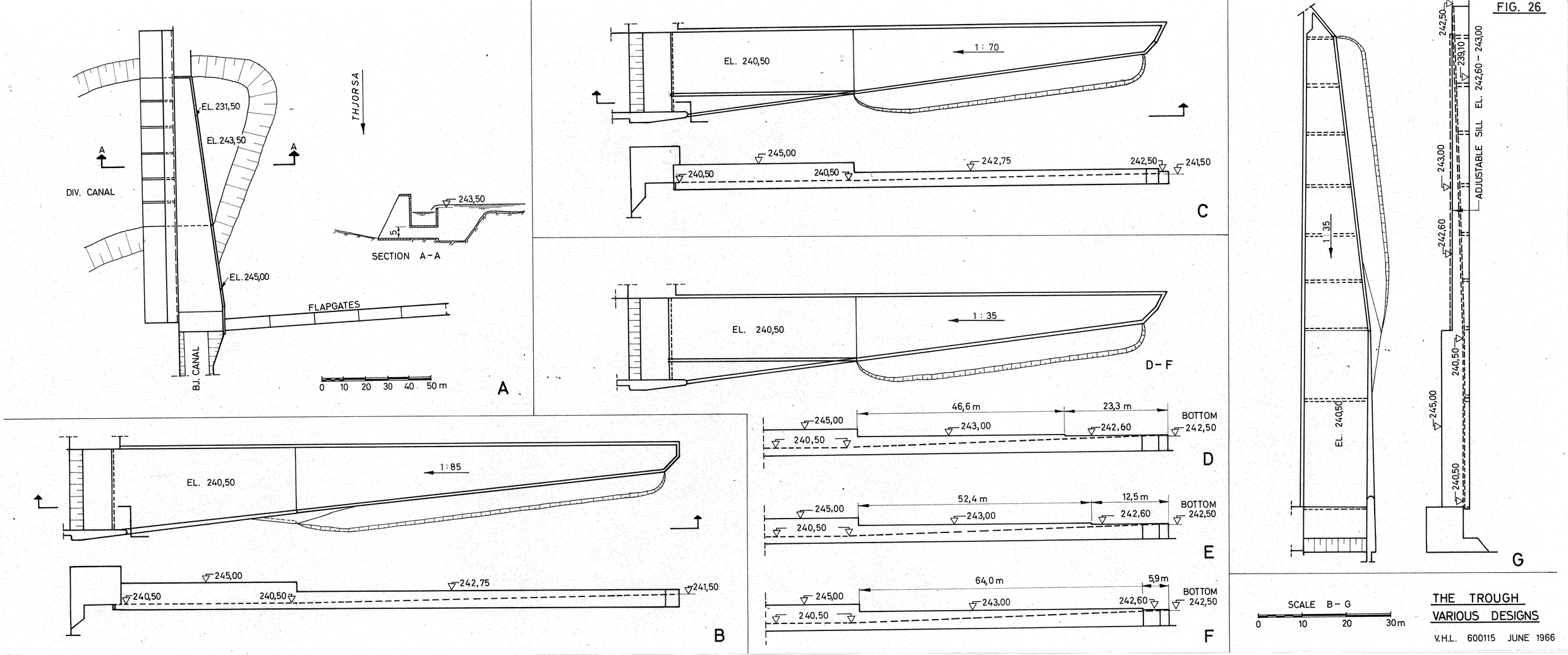
FIG. 26



THE TROUGH
VARIOUS DESIGNS

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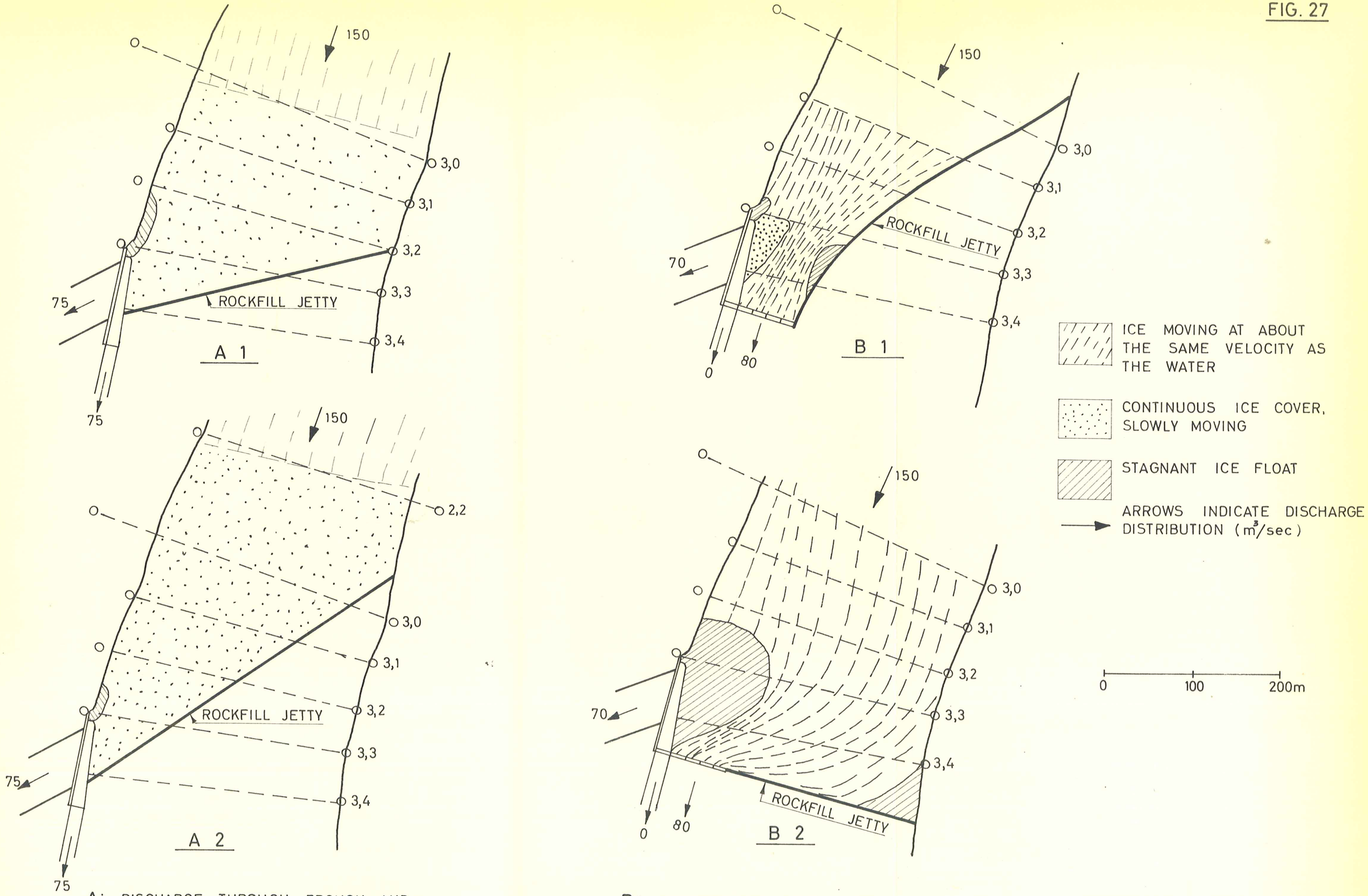
FIG. 27



A: DISCHARGE THROUGH TROUGH AND BJARNALÆKUR CANAL ONLY. CREST OF TROUGH EL 243,0

B: TROUGH AND CANAL NOT IN USE. DISCHARGE OVER WEIRS AT EL 242

ALTERNATIVES FOR INITIAL CONSTRUCTION

SCALE RATIO 1:5000

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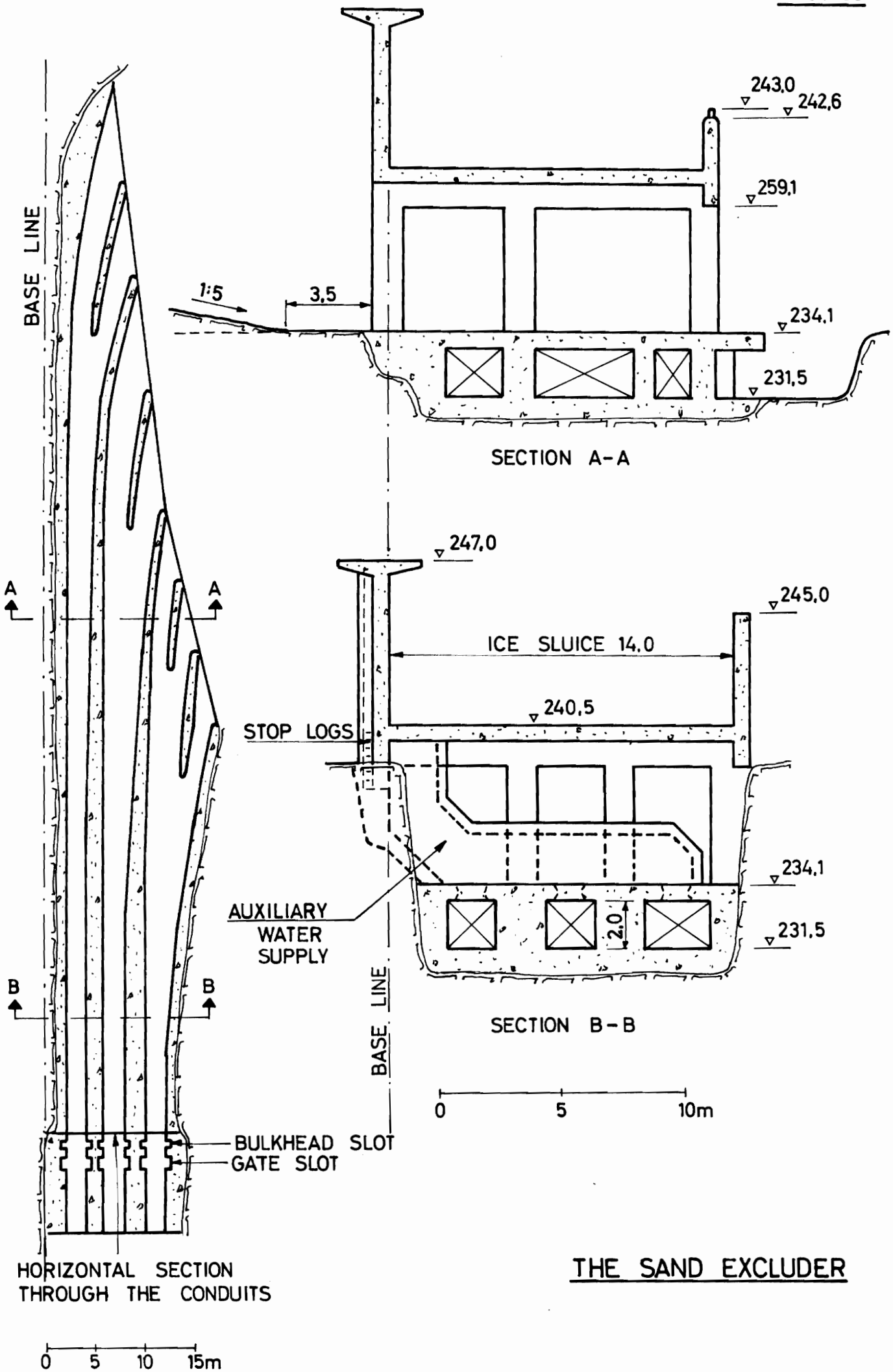
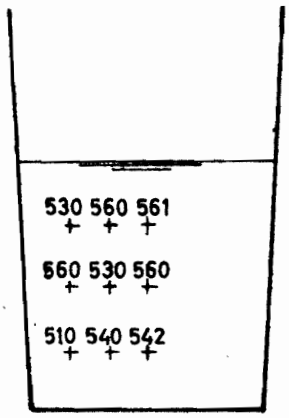
FIG. 28



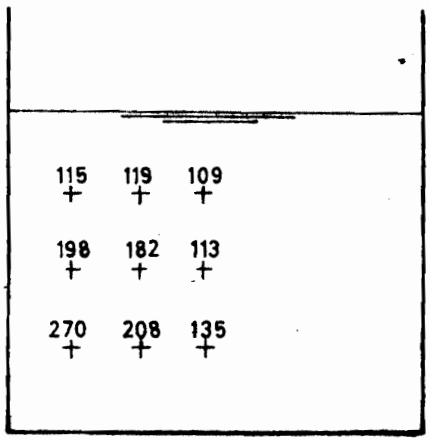
THE SAND EXCLUDER

V.H.L. 600115 JUNE 1966

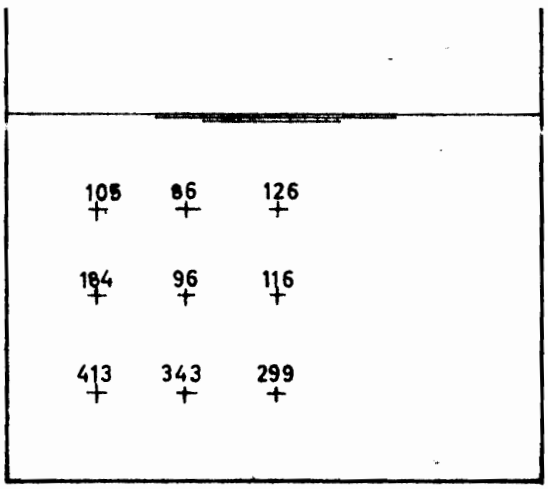
FIG. 29



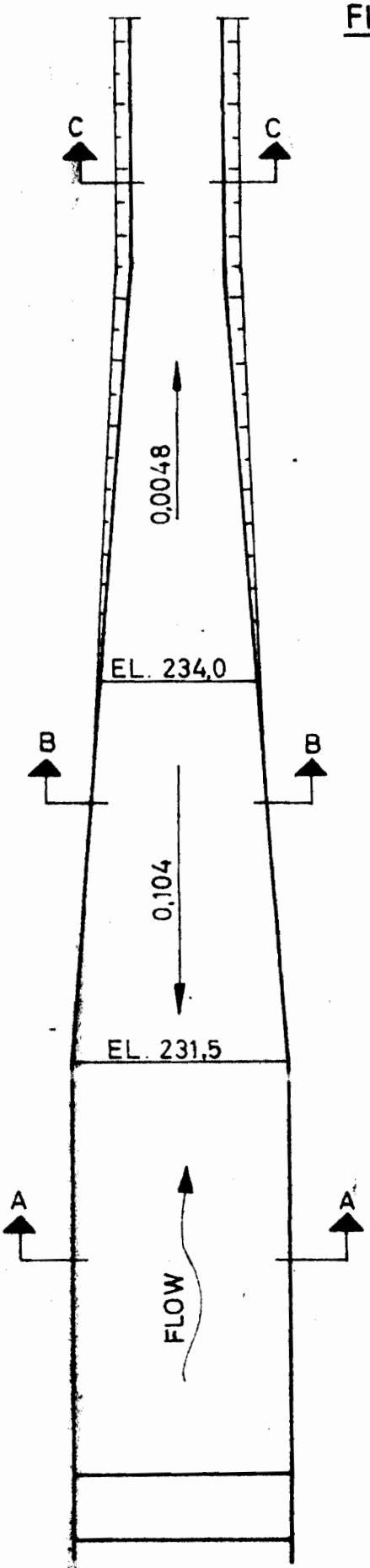
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 200\ m^3/s$

$Q_{SE} = 0$

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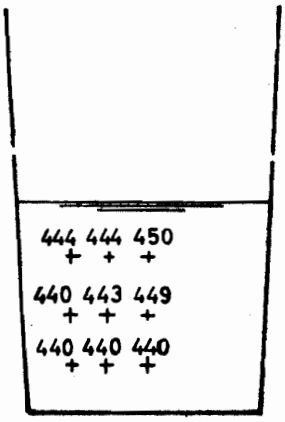
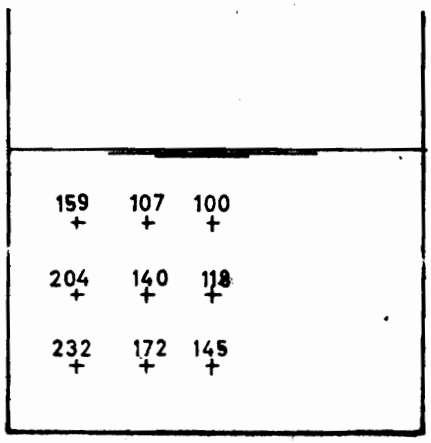
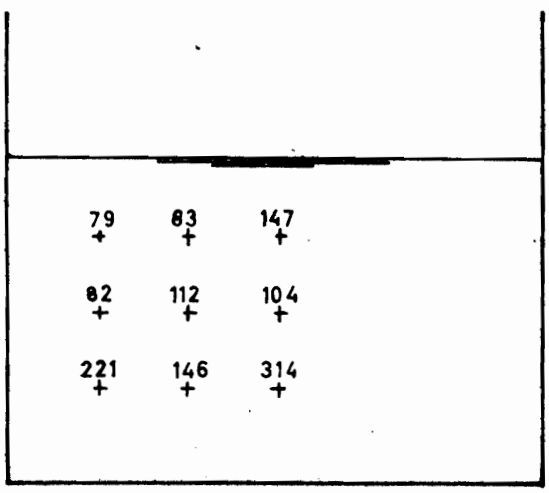
FIG. 30



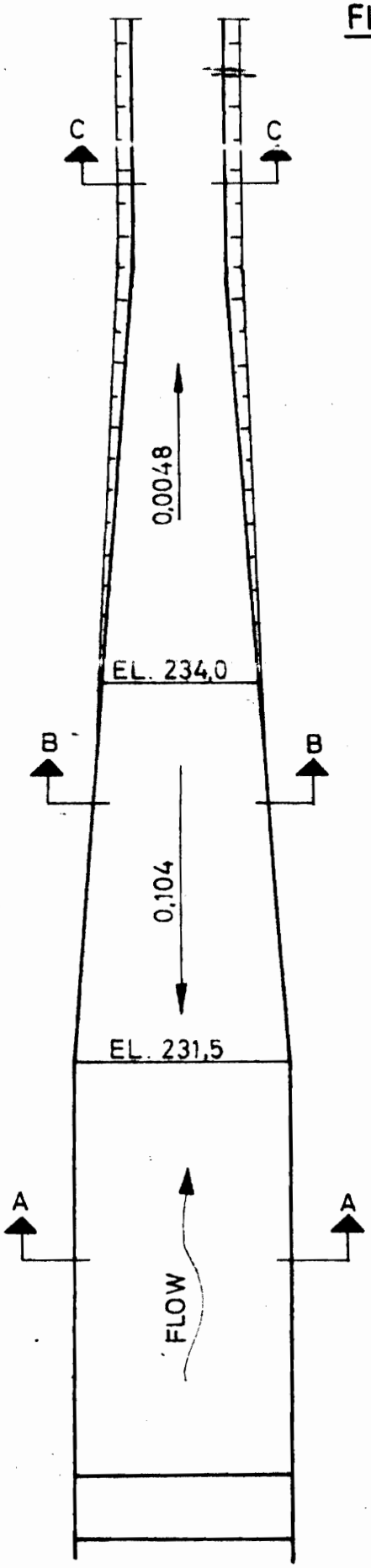
C-C
1:200



B-B 1:200



A-A 1:200



1 : 400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 150\ m^3/s$

$Q_{SE} = 0$

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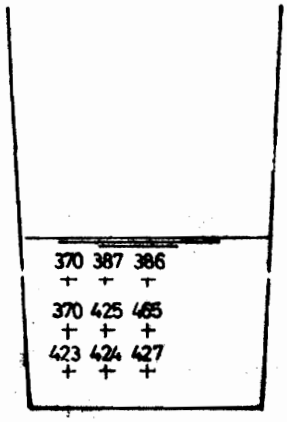
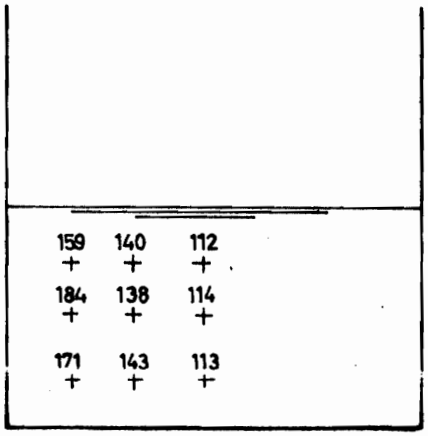
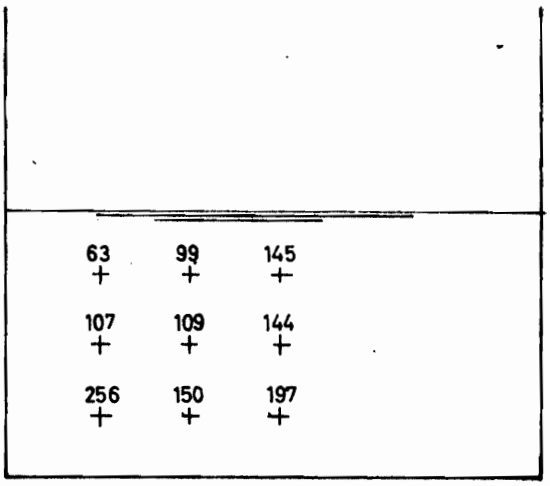
FIG. 31



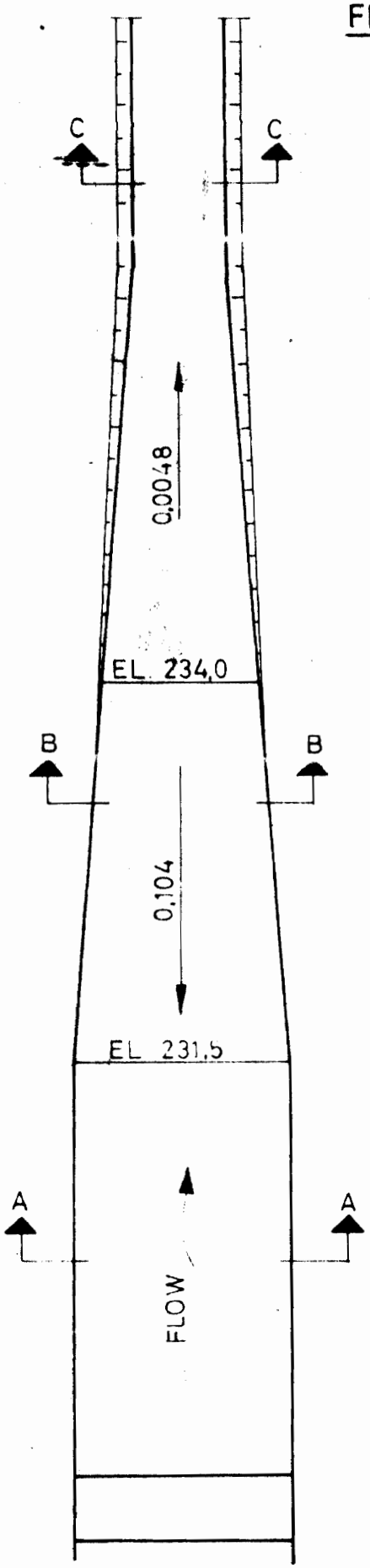
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 100\ m^3/s$

$Q_{SE} = 0$

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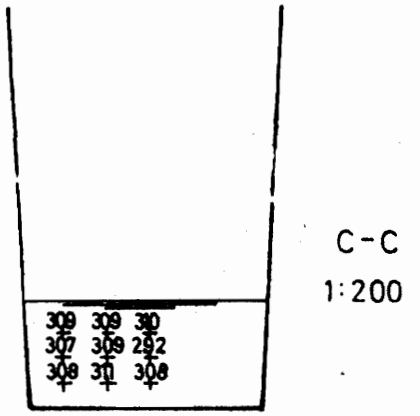
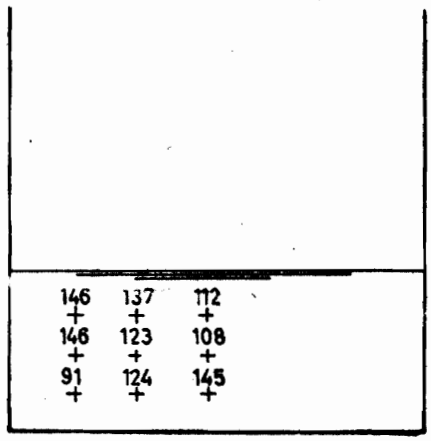
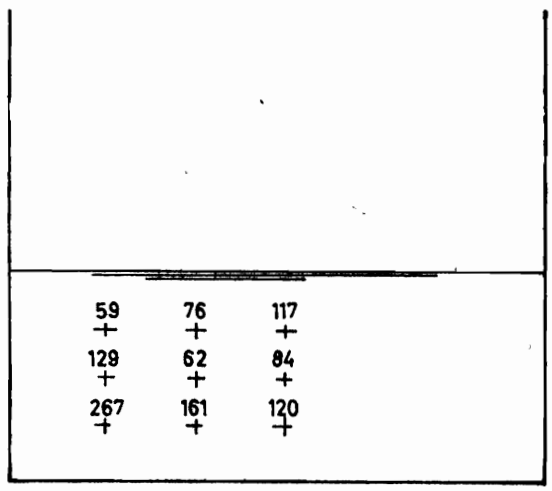
FIG. 32



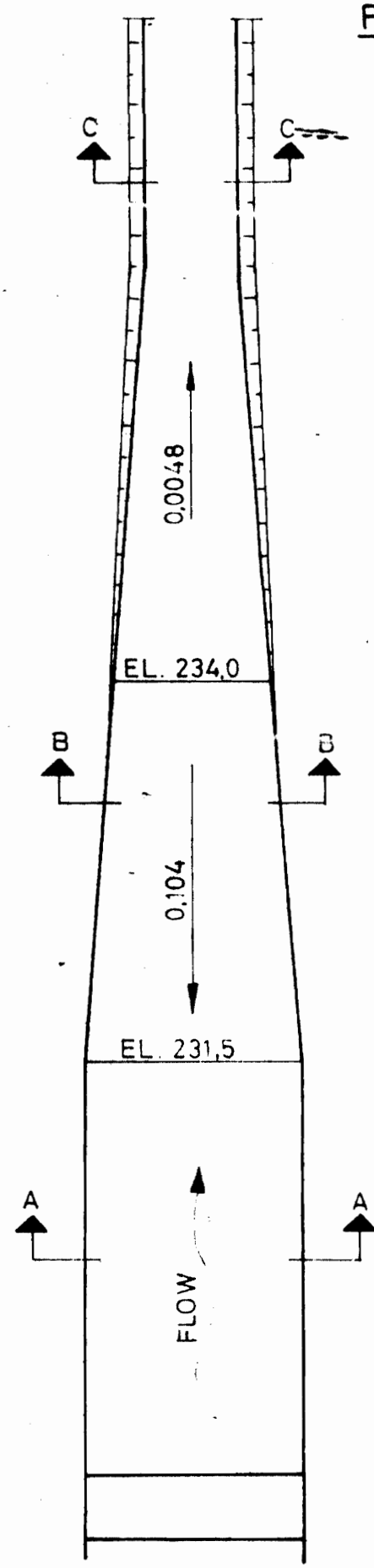
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 50\ m^3/s$

$Q_{SE} = 0$

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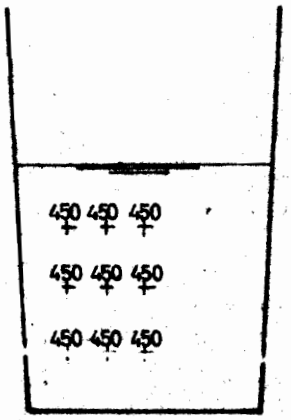
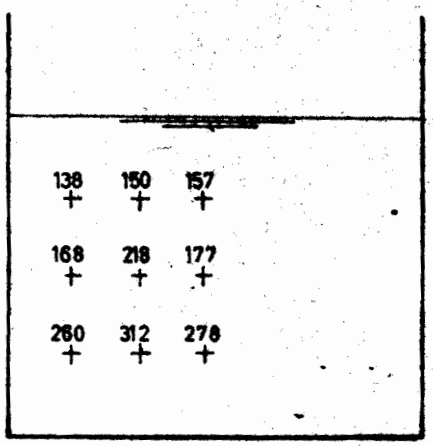
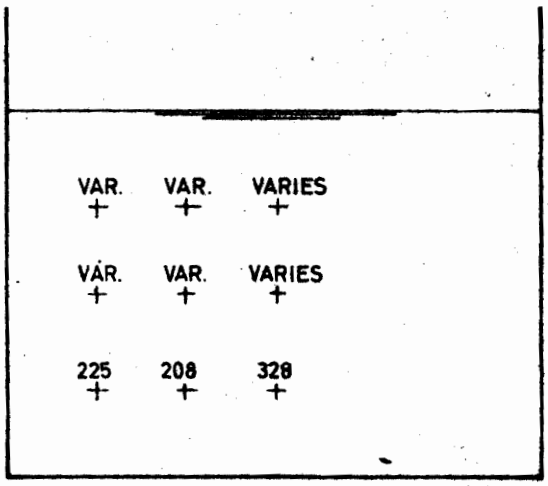
FIG. 33



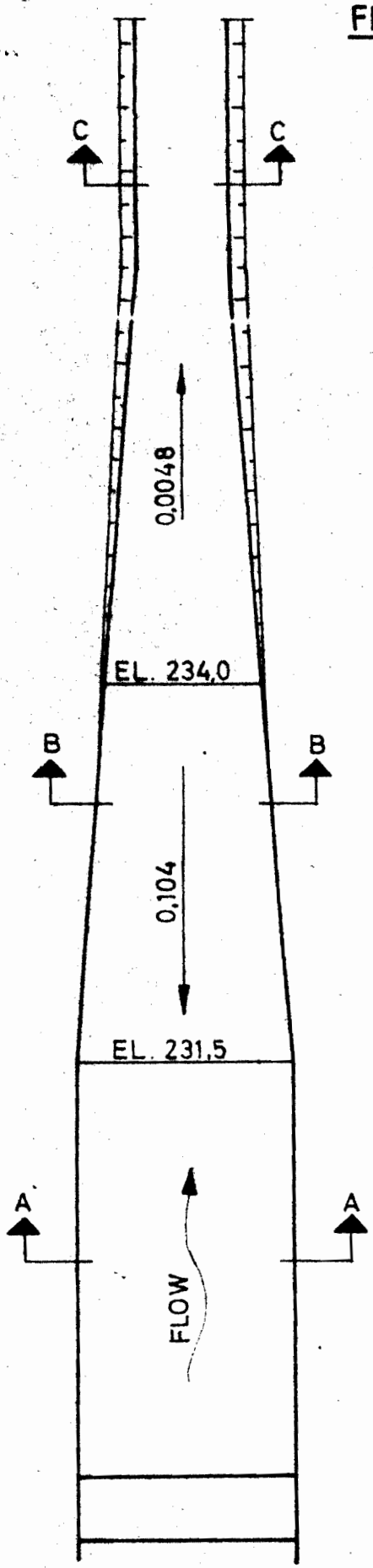
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES
IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 200\ m^3/s$
$Q_{SE} = 83\ m^3/s$ (ALL SLUICES OPEN)

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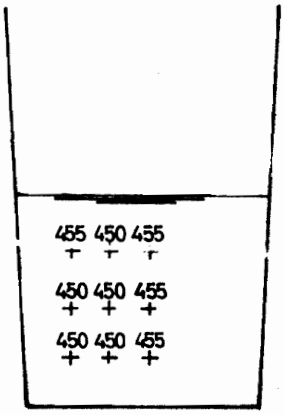
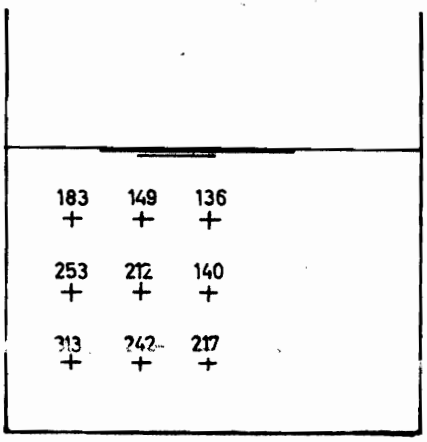
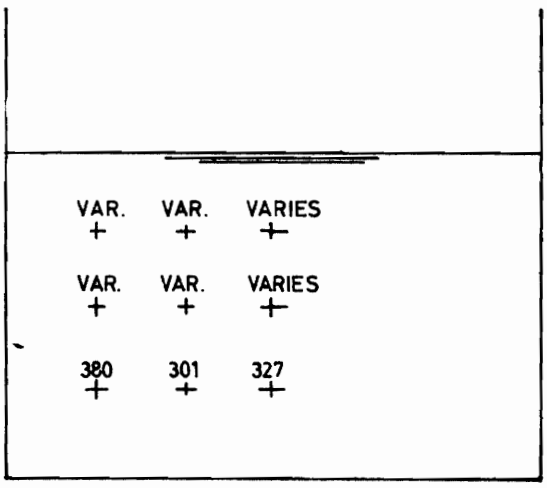
FIG. 34



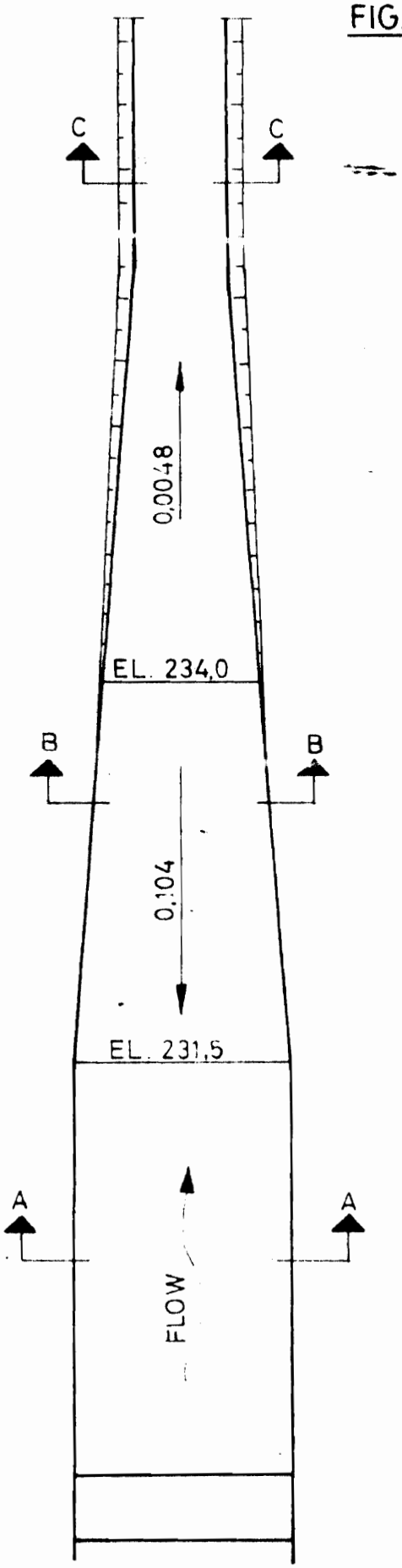
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 150\ m^3/s$

$Q_{SE} = 83$ -"- (ALL SLUICES OPEN)

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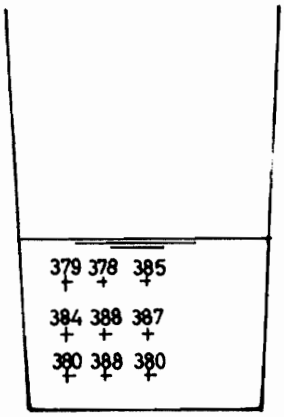
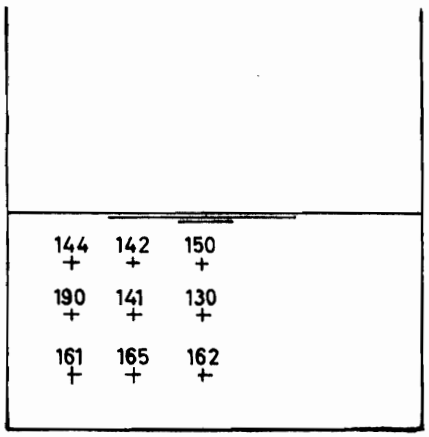
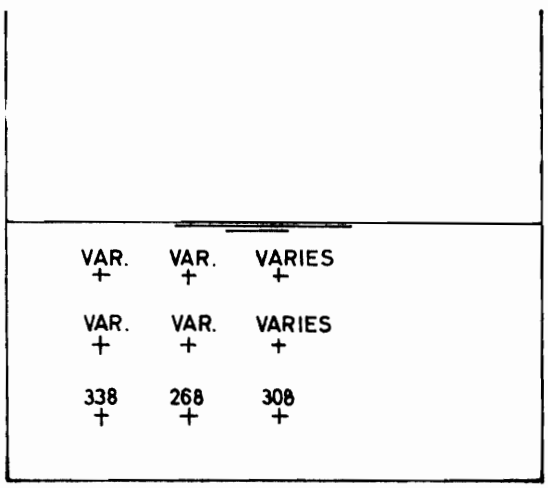
FIG. 35



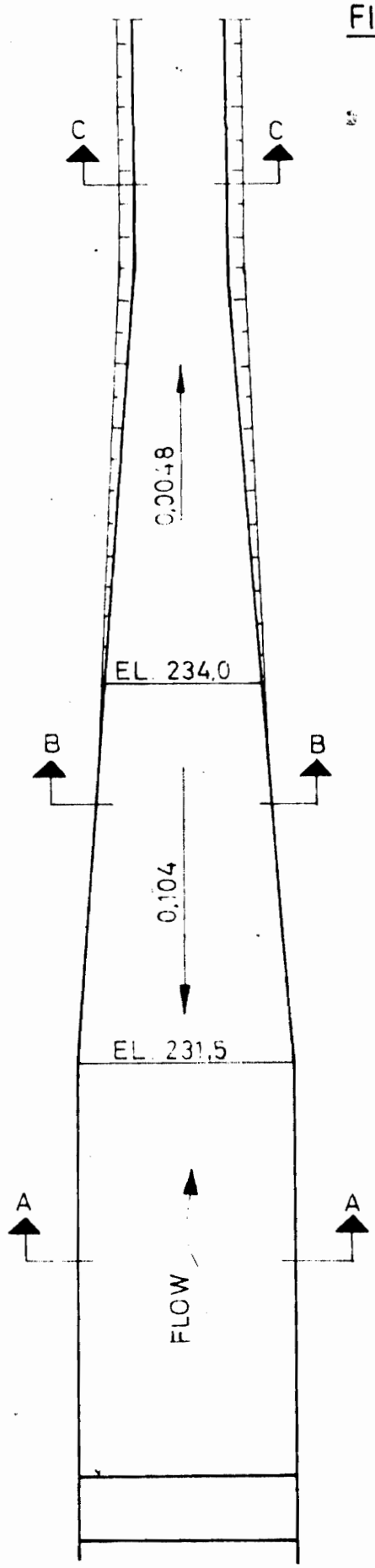
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES
IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 100 m^3/s$

Q_{SE} = 83—"— (ALL SLUICES OPEN)

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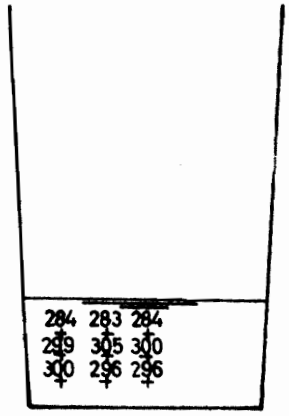
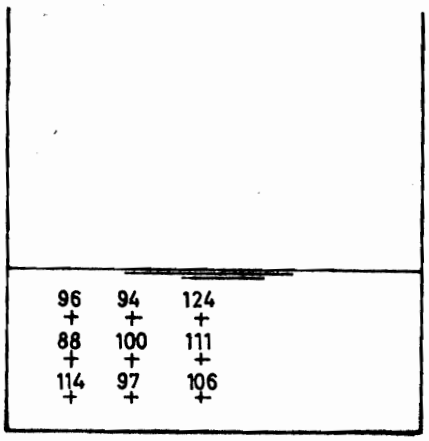
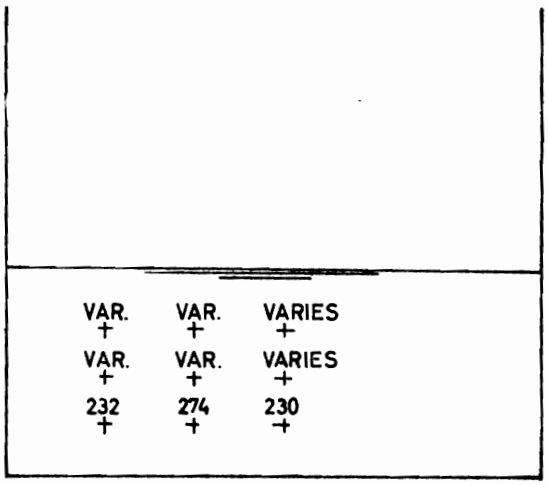
FIG. 36



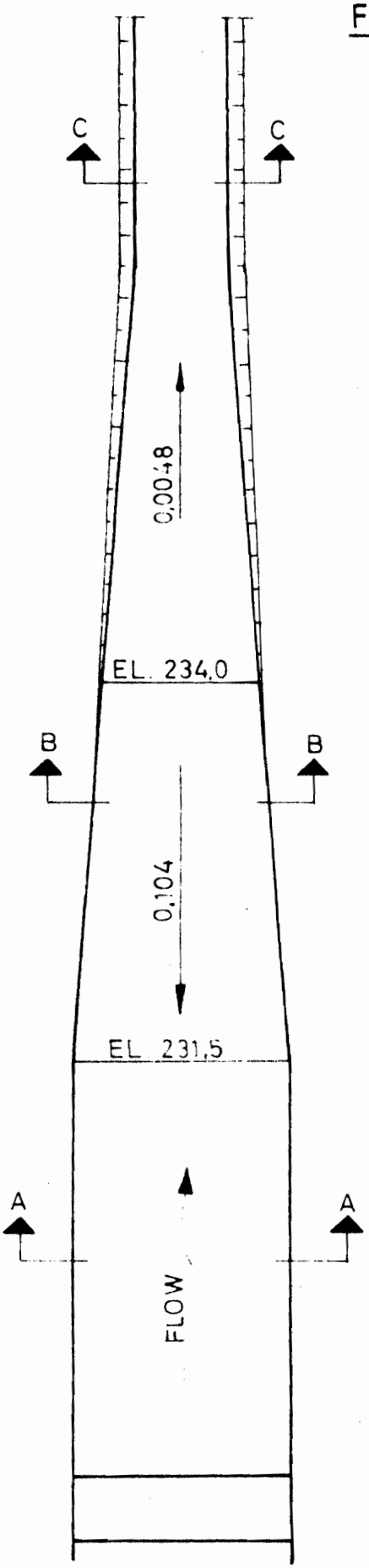
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 50 m^3/s$

$Q_{SE} = 50$ –"– (ALL SLUICES PARTIALLY OPEN)

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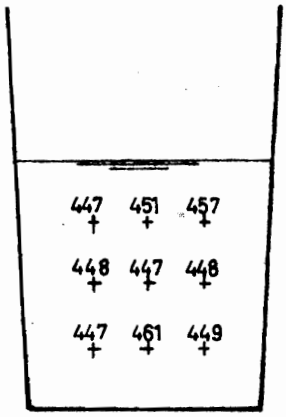
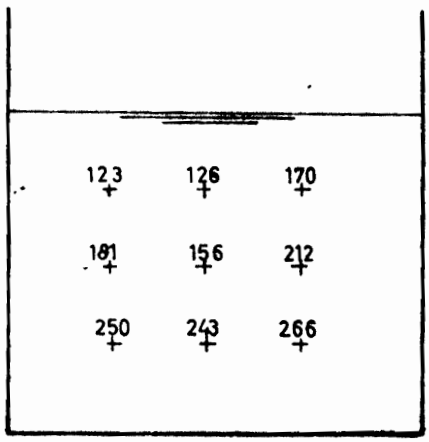
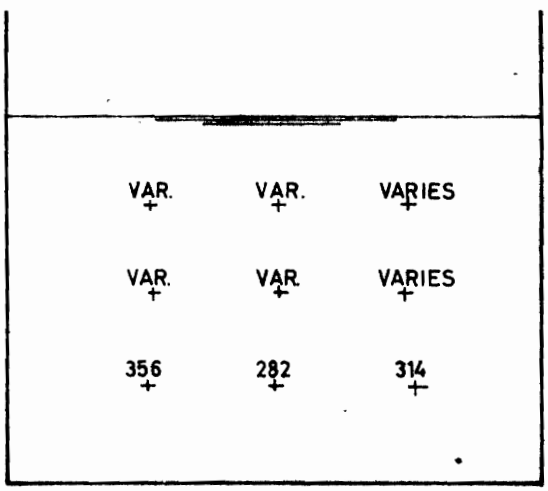
FIG. 37



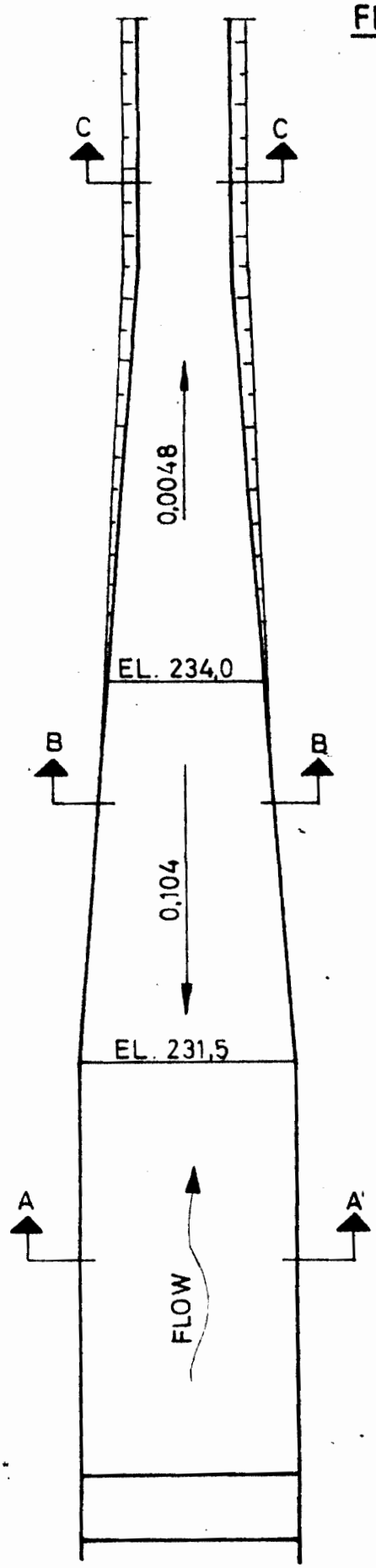
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 200 m^3/s$

$Q_{SE} = 42 m^3/s$ (EASTERN SLUICE OPEN)

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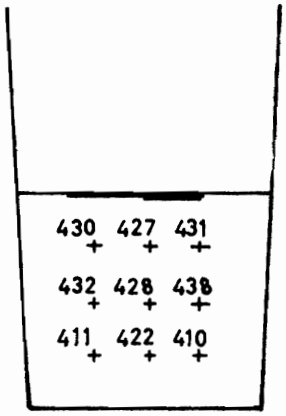
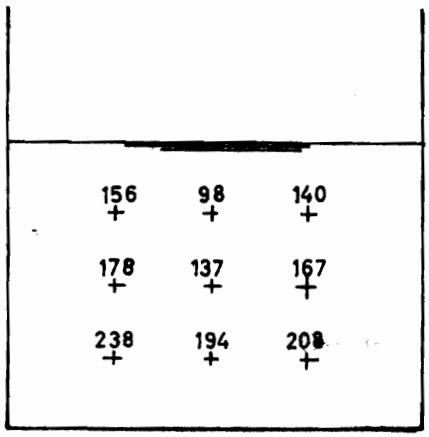
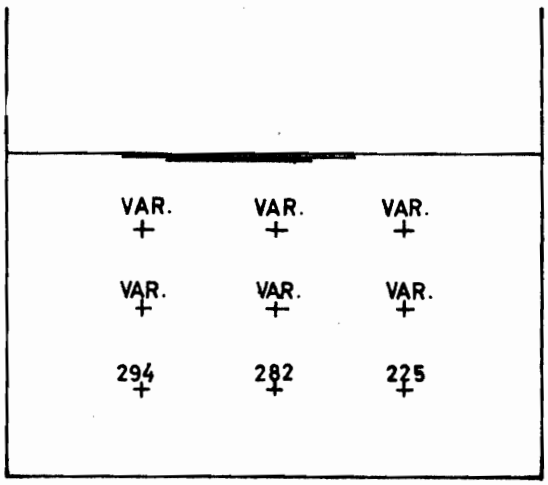
FIG. 38



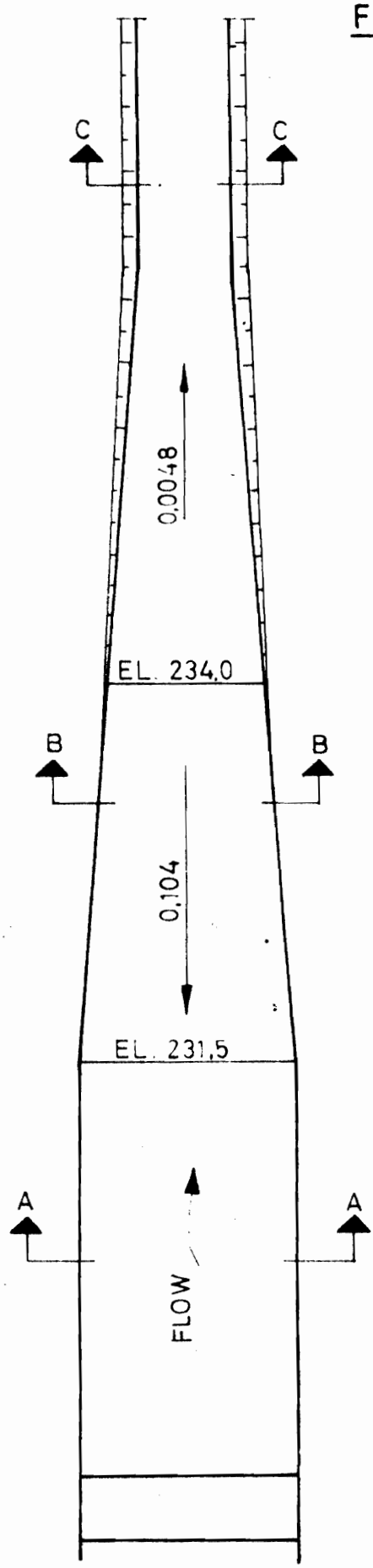
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 150\ m^3/s$

$Q_{SE} = 42\ m^3/s$ (EASTERN SLUICE OPEN)

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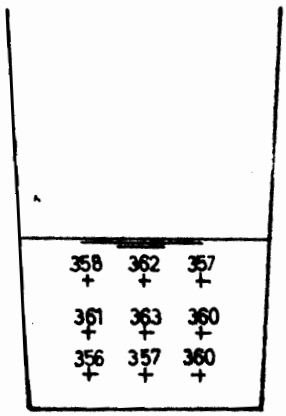
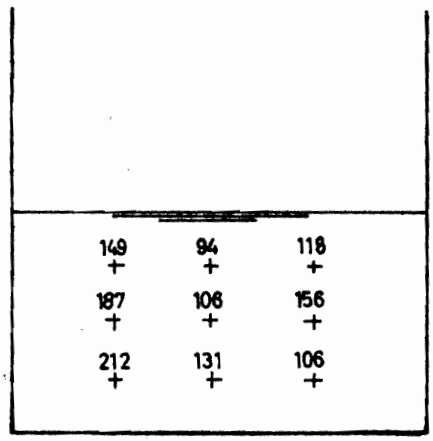
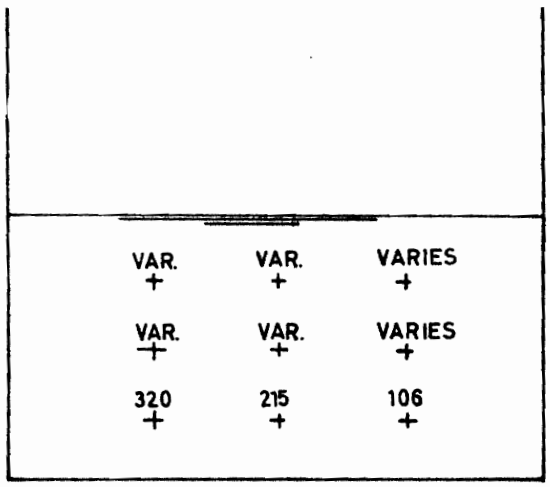
FIG. 39



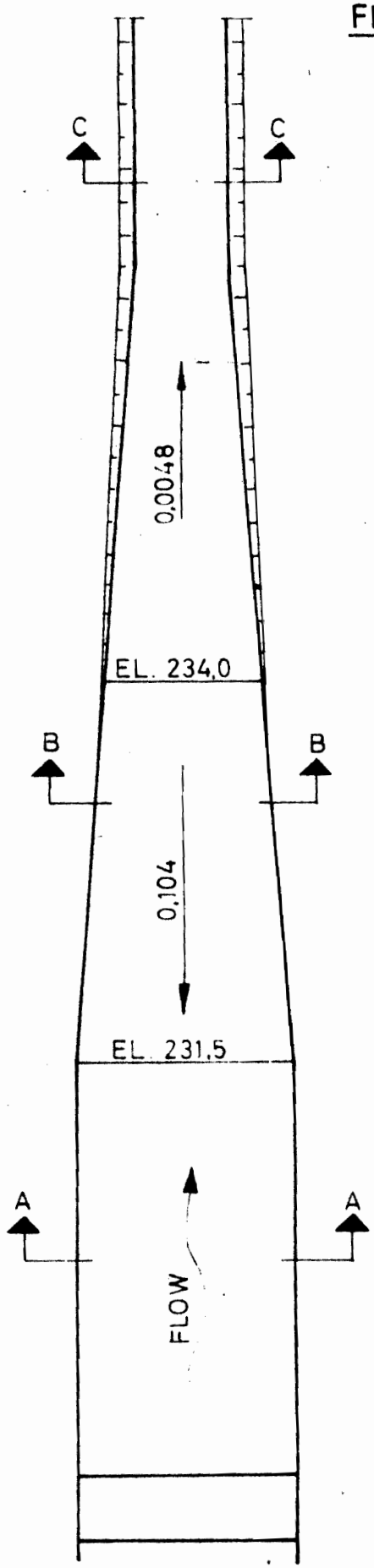
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES
IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 100\,m^3/s$

$Q_{SE} = 42$ –"– (EASTERN SLUICE OPEN)

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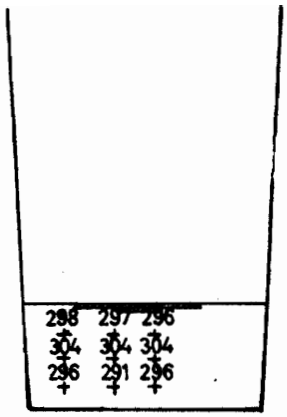
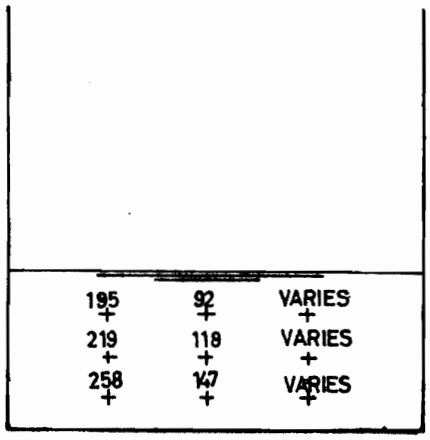
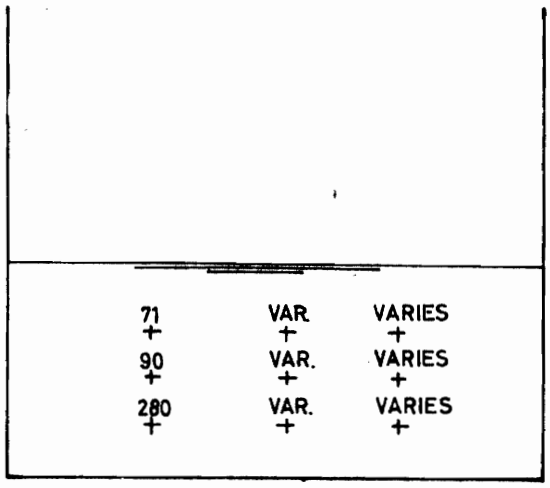
FIG. 40



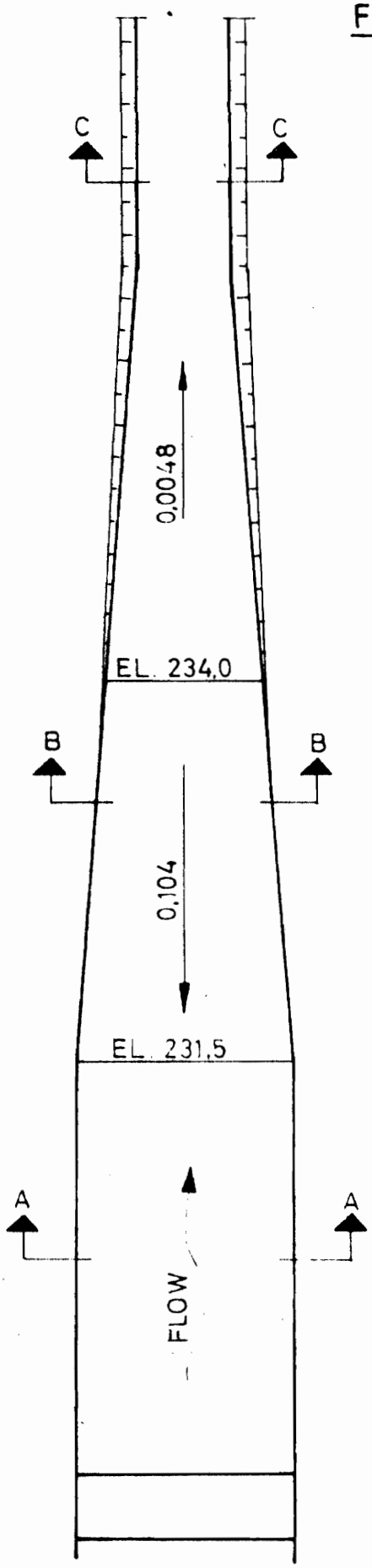
C-C
1:200



B-B 1:200



A-A 1:200



1:400

HORIZONTAL VELOCITIES
IN BJARNALÆKUR CANAL

PROTOTYPE VALUES cm/sec

$Q_{BC} = 50\ m^3/s$

$Q_{SE} = 42$ — (EASTERN SLUICE OPEN)

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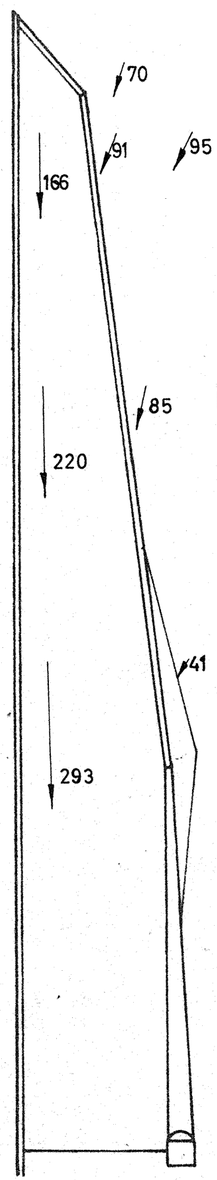
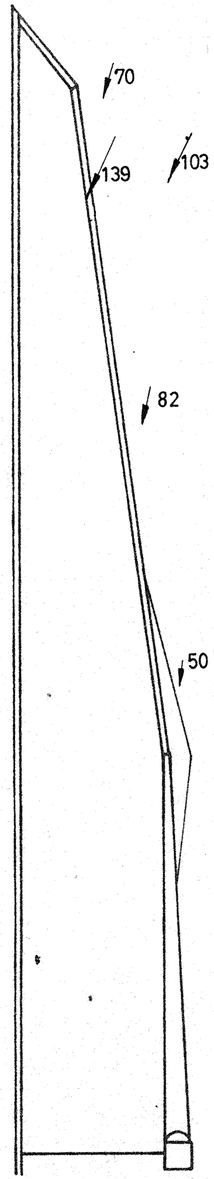
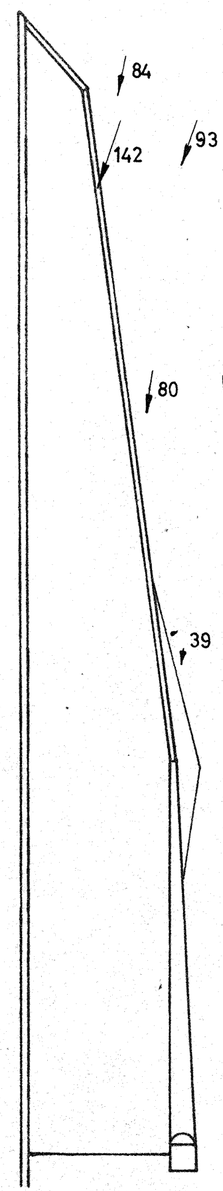
FIG.41



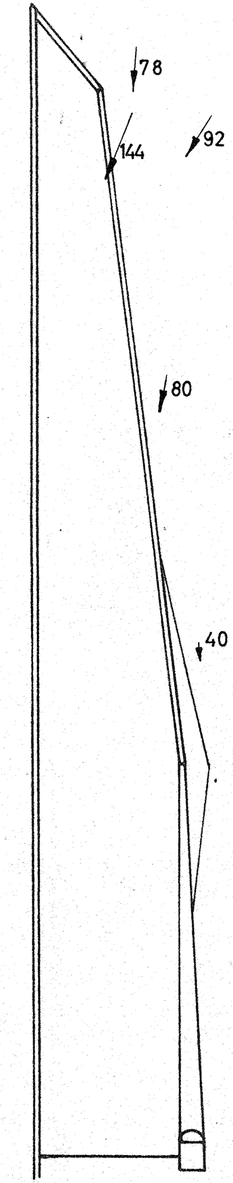
EL. 242,0



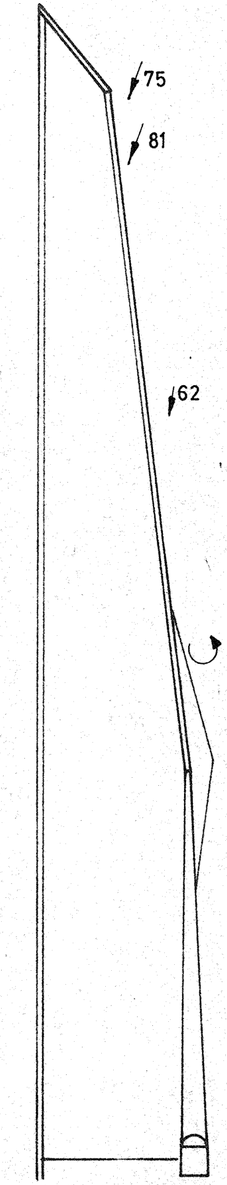
EL. 241,0



EL. 239,0



EL. 237,0



EL. 235,0

HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

$Q = 150$ m³/sek.
$Q_{ST} = 100$ —"—
$Q_{TR} = 50$ —"—
$Q_{SE} = 0$ —"—
$Q_D = 0$ —"—

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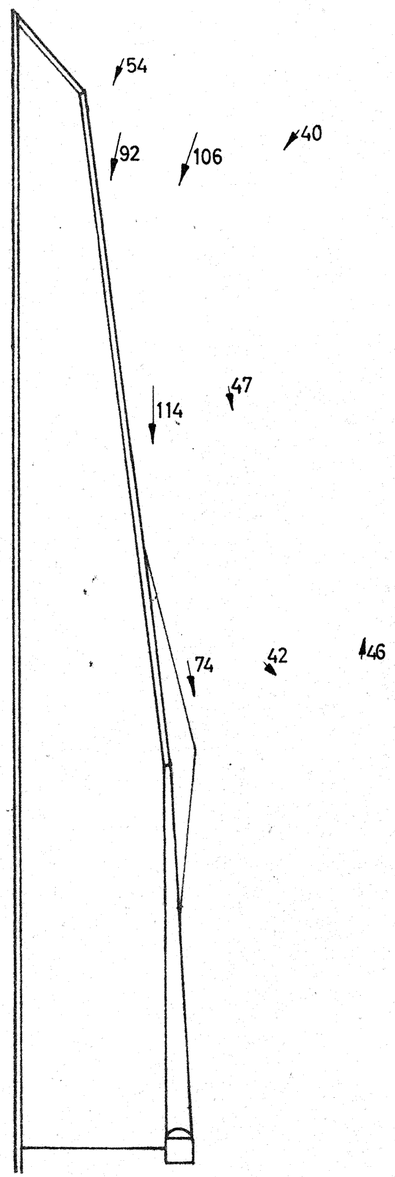
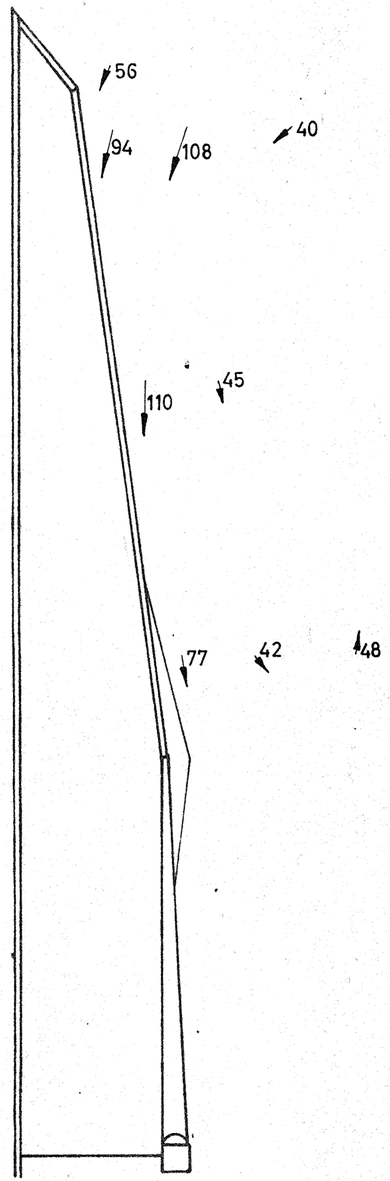
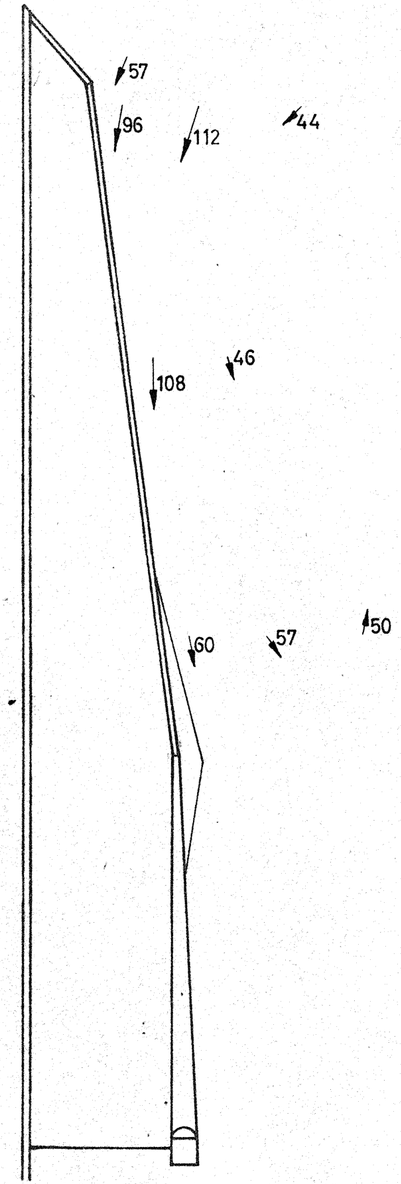
FIG. 42



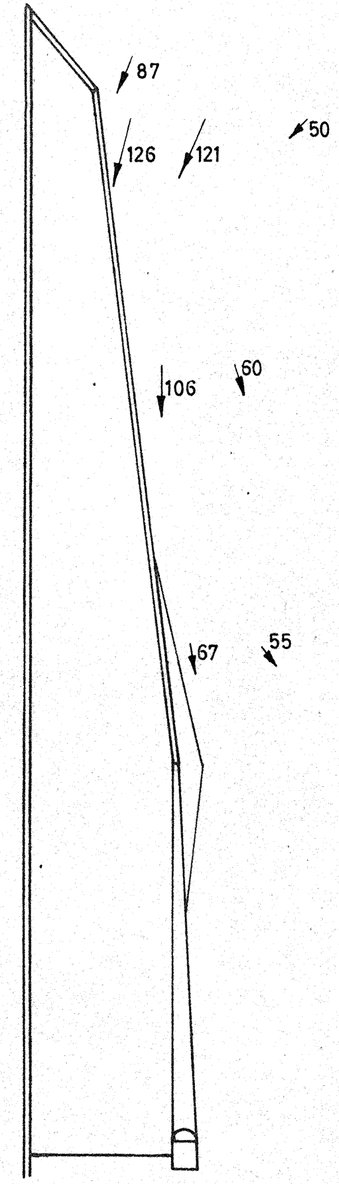
EL. 242.0



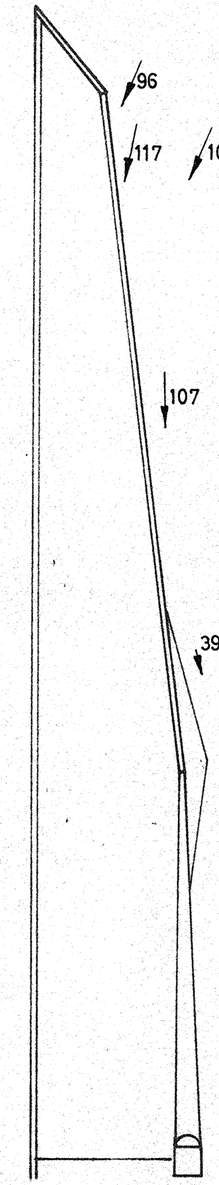
EL. 241.0



EL. 239.0



EL. 237.0



EL. 235.0

HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

Q = 150 m^3/sek.
Q_{ST} = 100 —"—
Q_{TR} = 0 —"—
Q_{SE} = 0 —"—
Q_D = 50 —"—

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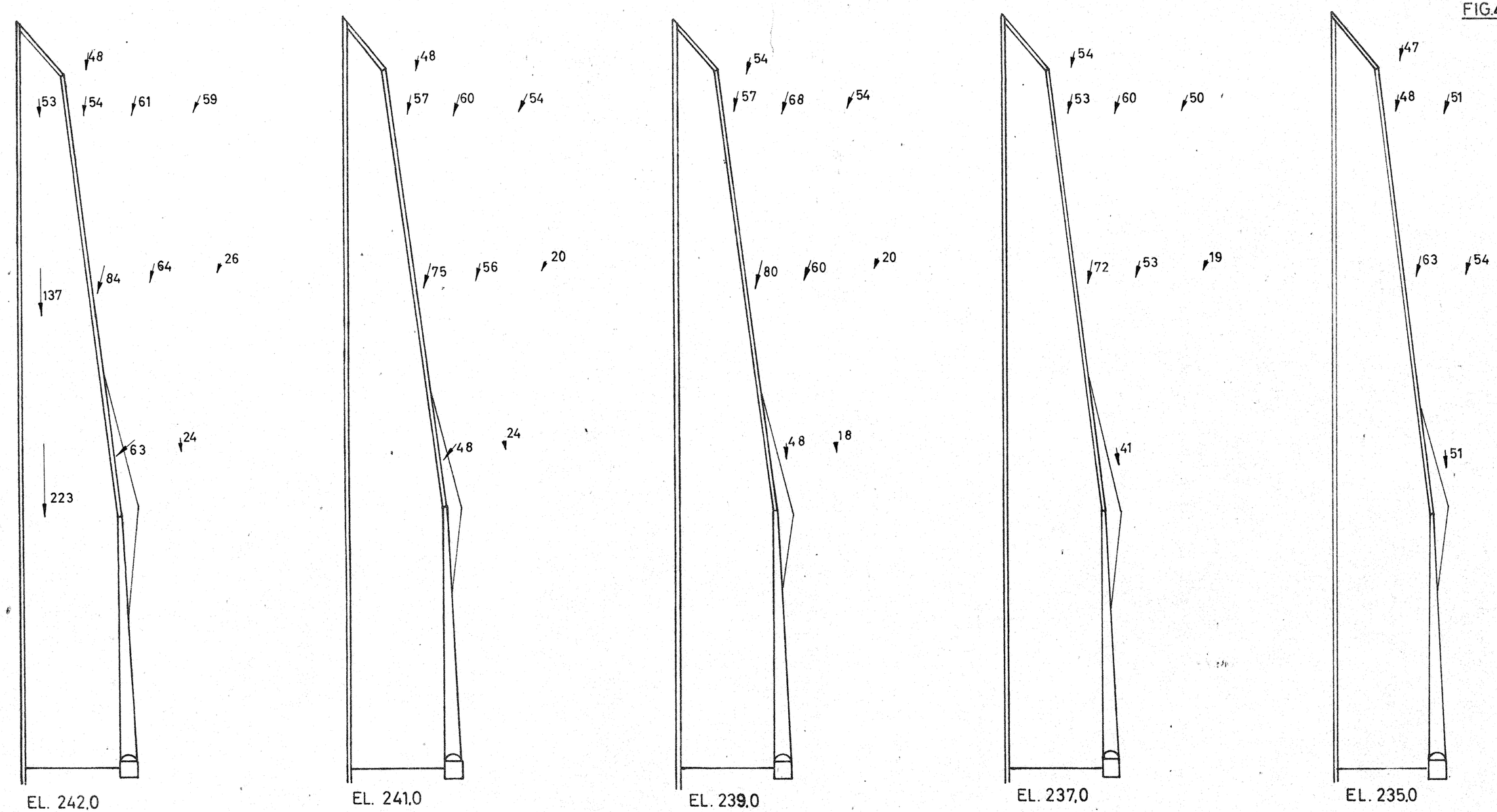
FIG.43



HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

$Q = 150\ m^3/sek.$
$Q_{ST} = 50$ —"—
$Q_{TR} = 100$ —"—
$Q_{SE} = 0$ —"—
$Q_D = 0$ —"—

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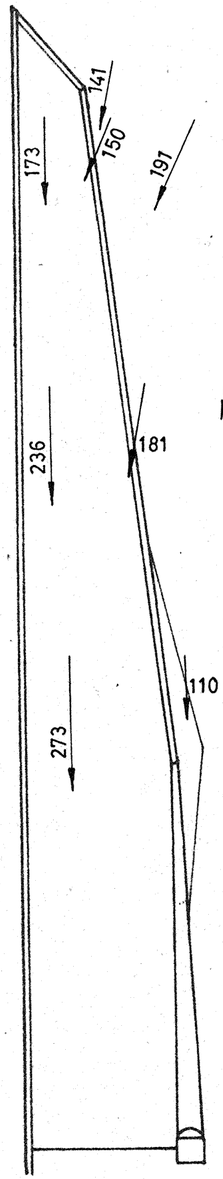
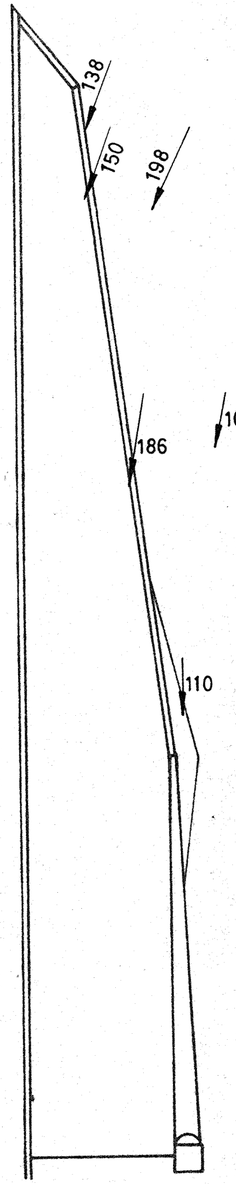
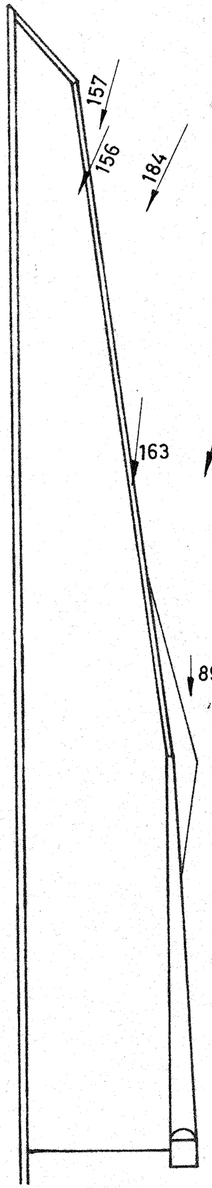
FIG.44



EL. 242.0



EL. 241.0



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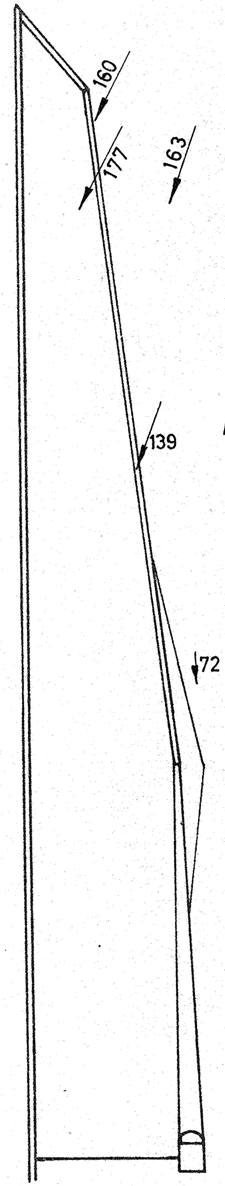
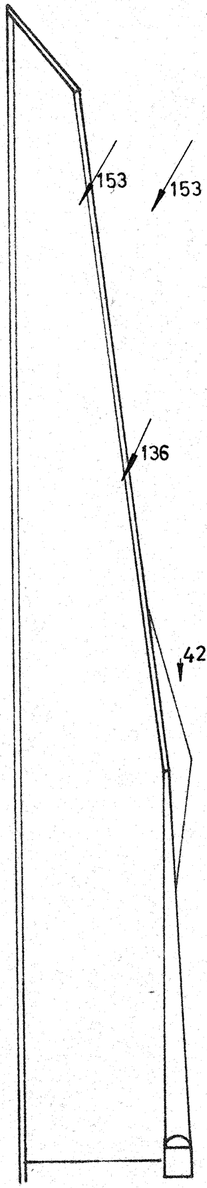


EL. 237.0



EL. 235.0

HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

Q = 300 m³/sek.
Q_{ST} = 250 —"—
Q_{TR} = 50 —"—
Q_{SE} = 0 —"—
Q_D = 0 —"—

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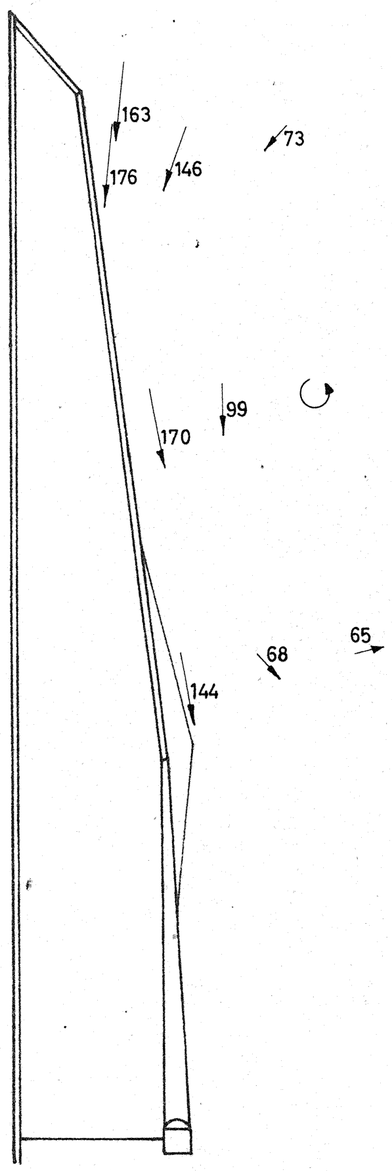
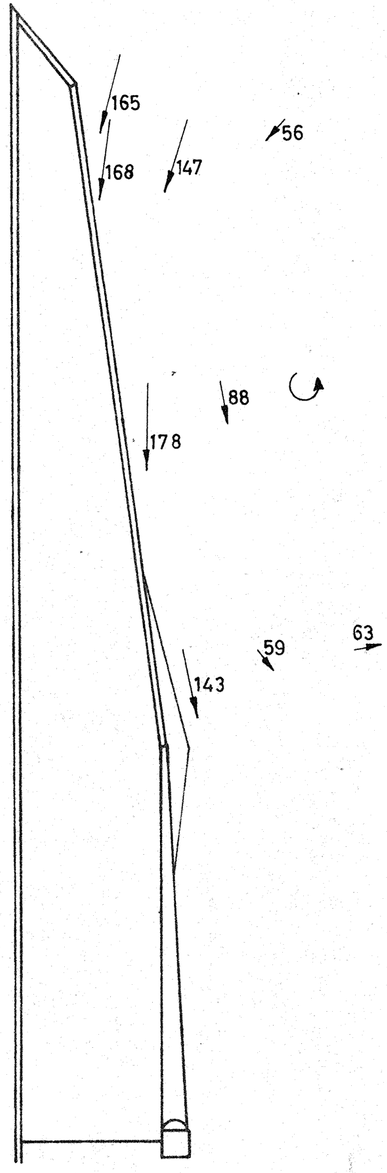
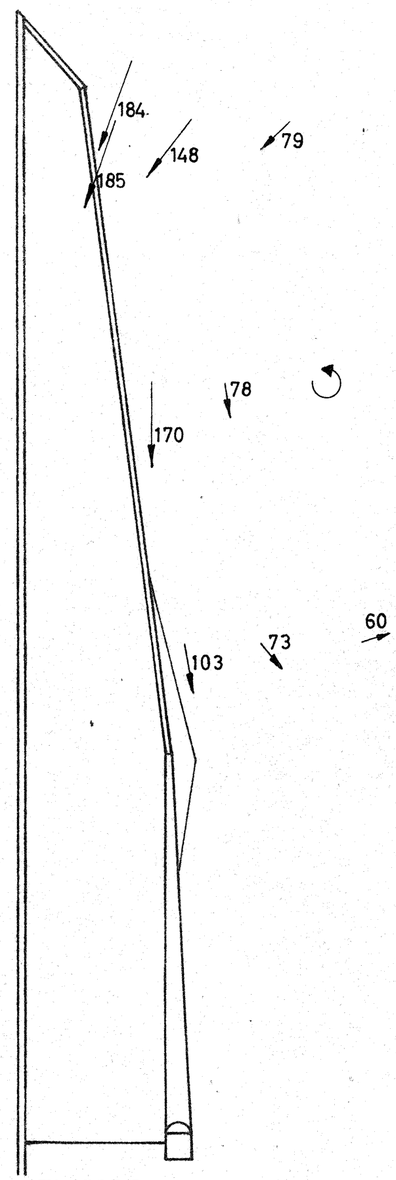
FIG. 45



EL. 242.0



EL. 241.0



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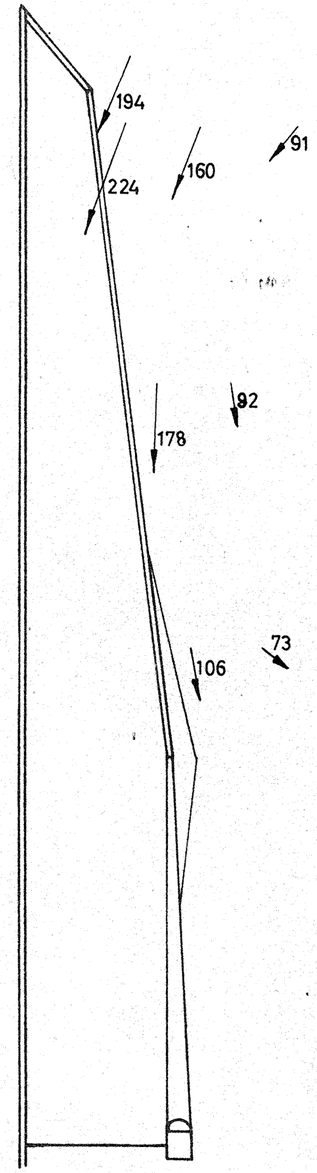
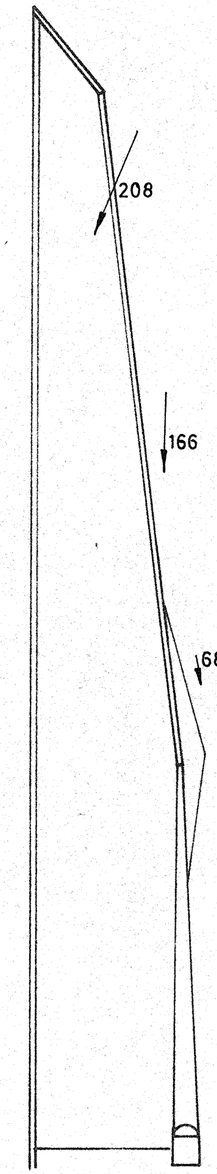


EL. 237.0



EL. 235.0

HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

Q = 300 m³/sek.
Q_{ST} = 250 —"—
Q_{TR} = 0 —"—
Q_{SE} = 0 —"—
Q_D = 50 —"—

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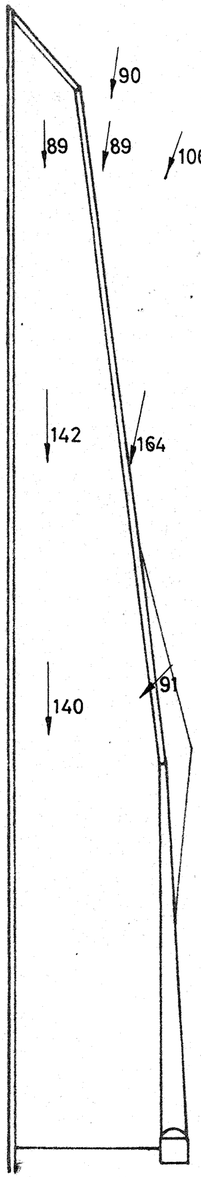
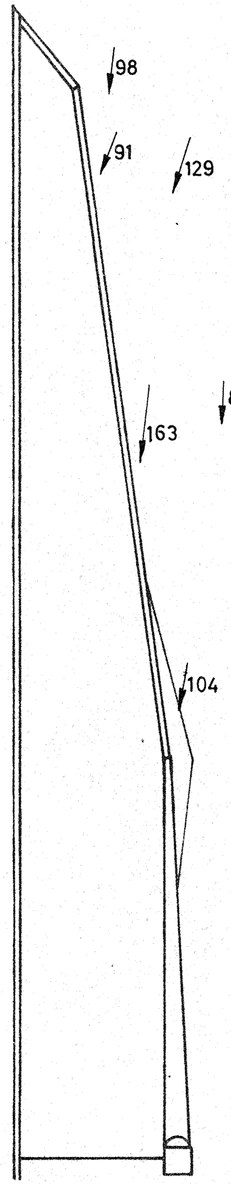
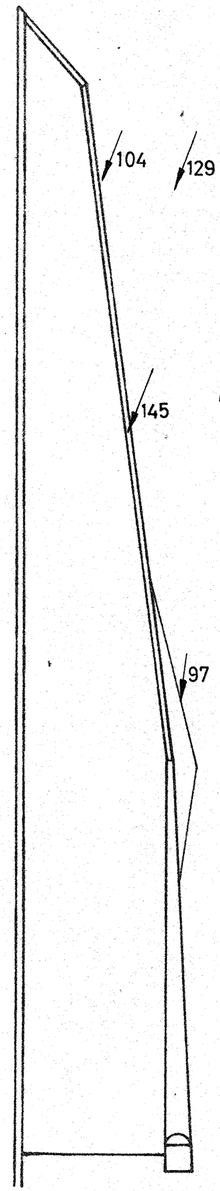
FIG.46



EL. 242,0



EL. 241,0



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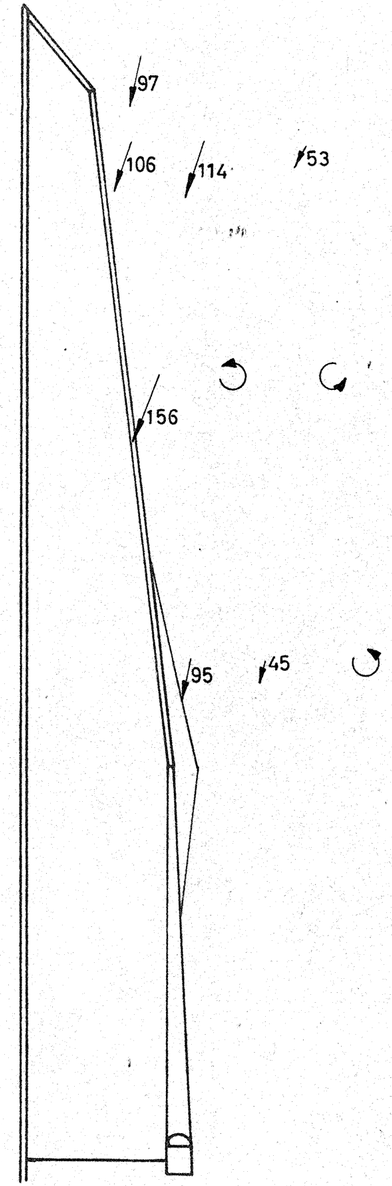
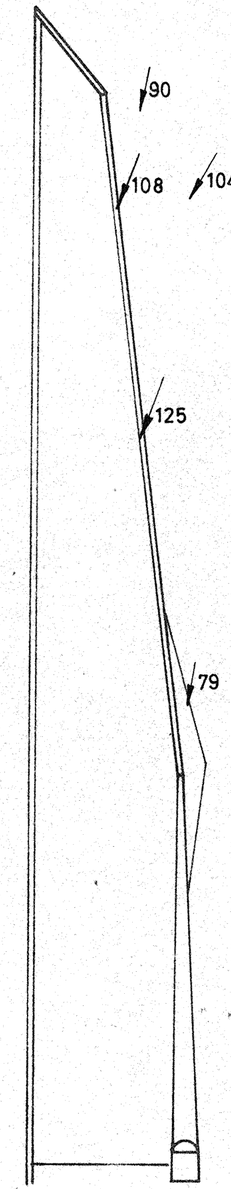


EL. 237,0



EL. 235,0

HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

Q = 300 m^3/sek.
Q_{ST} = 200 —"—
Q_{TR} = 100 —"—
Q_{SE} = 0 —"—
Q_D = 0 —"—

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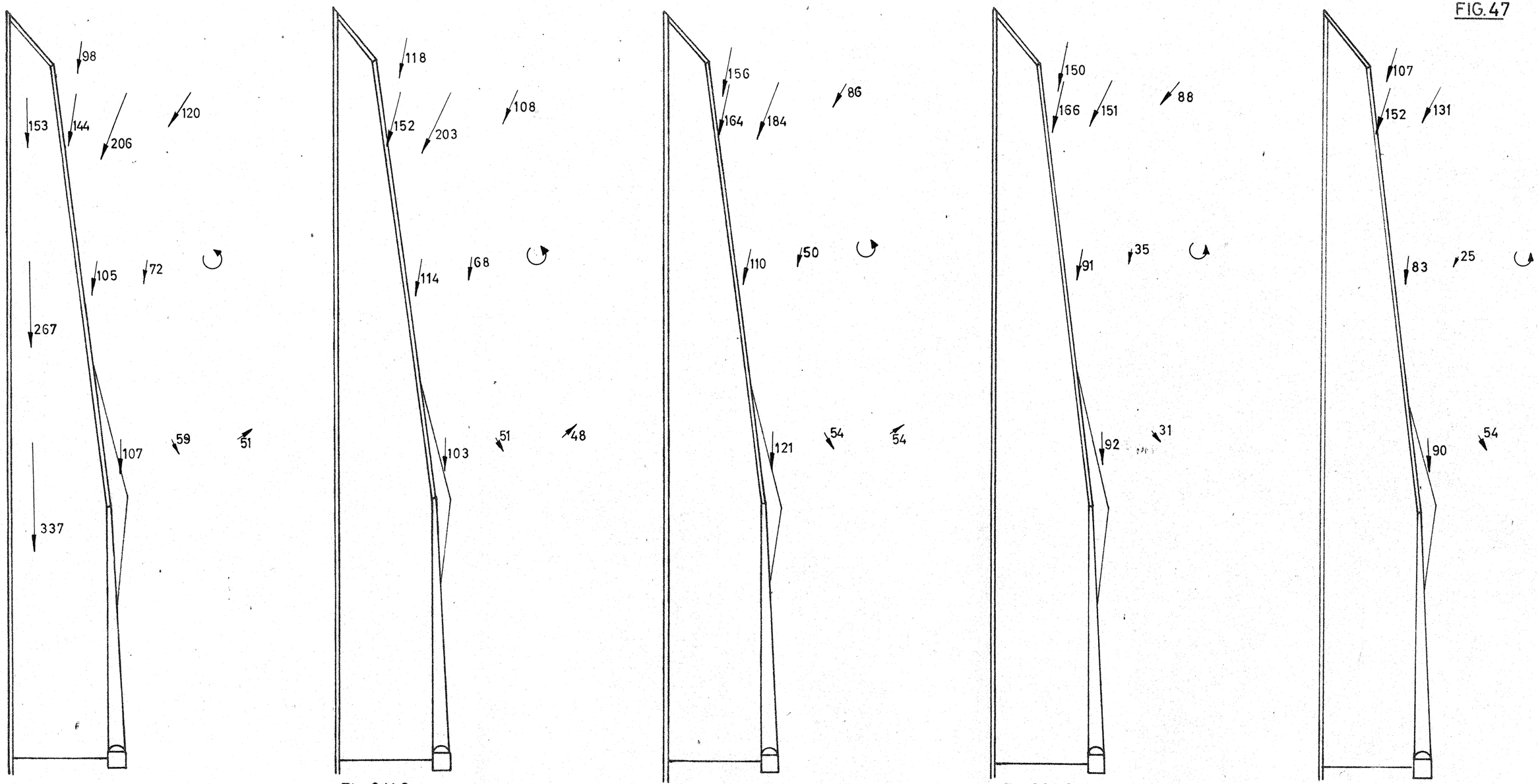
FIG. 47



HORIZONTAL VELOCITIES IN THE INLET AREA

PROTOTYPE VALUES (cm/sec)

0 10 20 30m

$Q = 300\ m^3/sek.$
$Q_{ST} = 150$ -"-
$Q_{TR} = 50$ -"-
$Q_{SE} = 100$ -"-
$Q_D = 0$ -"-

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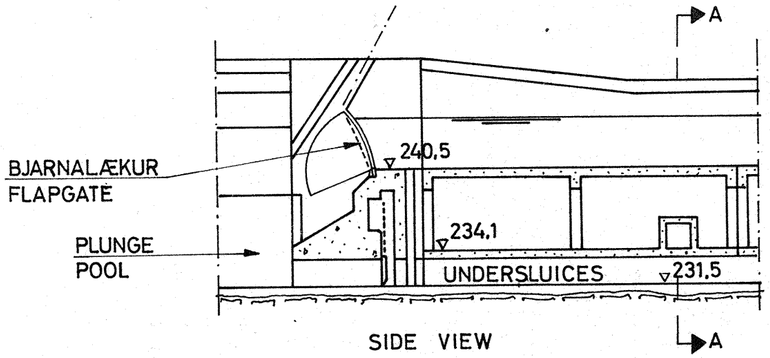
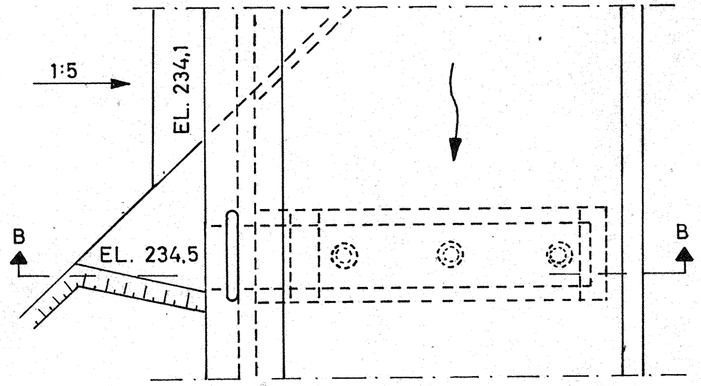
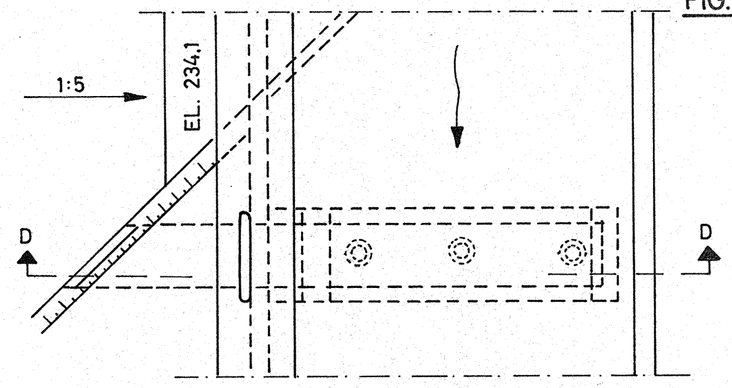
FIG. 48



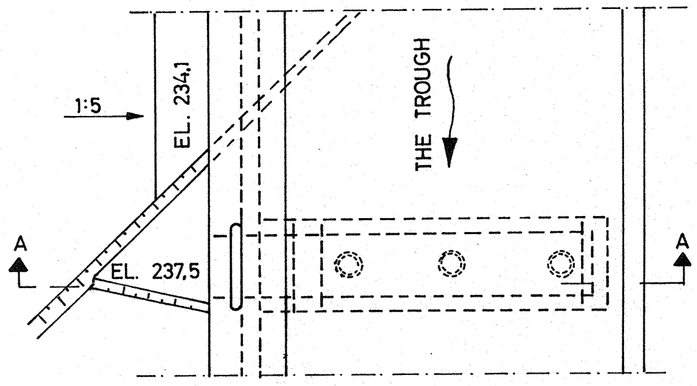
SIDE VIEW



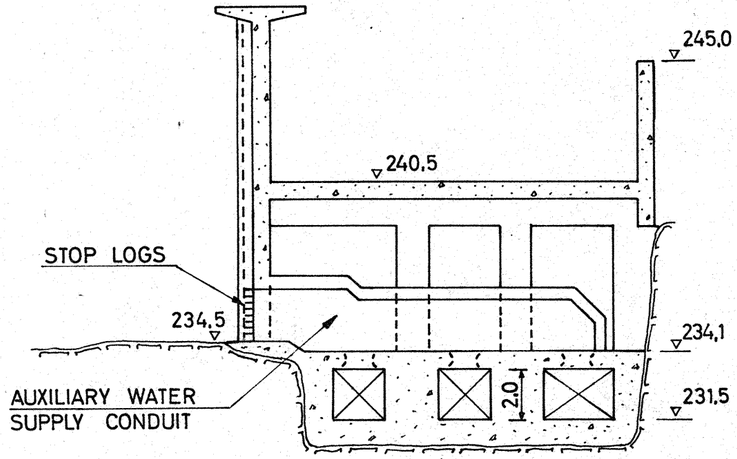
PLAN



PLAN



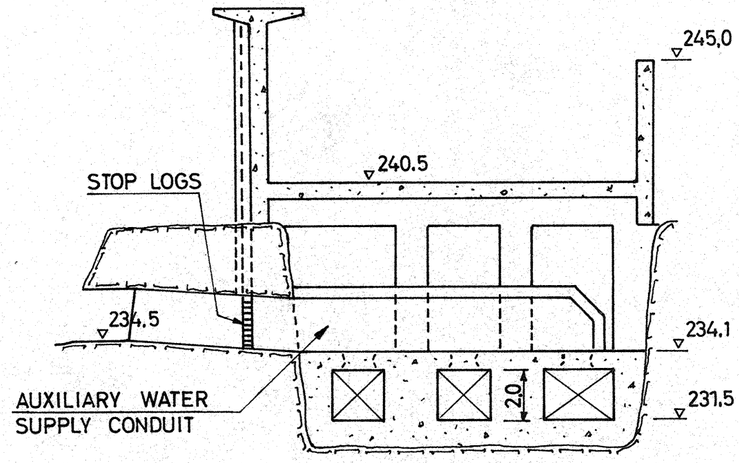
PLAN



SECTION B-B

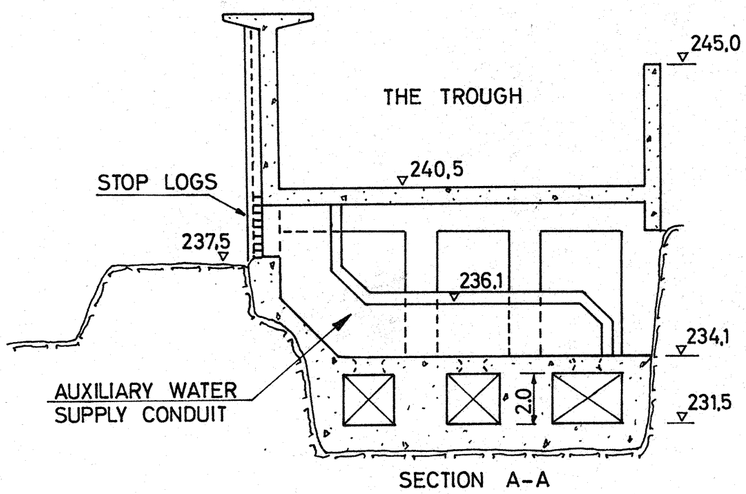
LOWER INTAKE TO THE CONDUIT

B



SECTION D-D

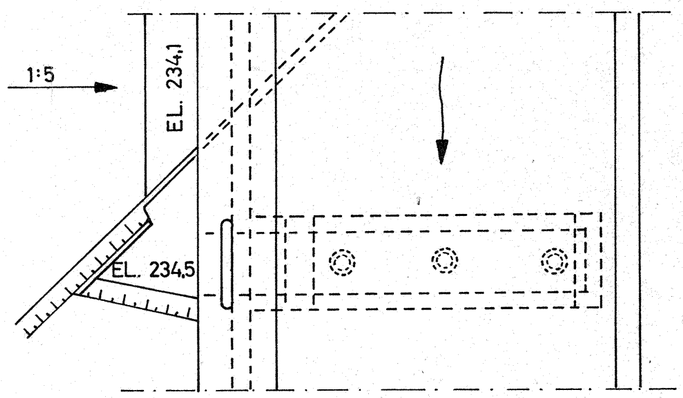
D



SECTION A-A

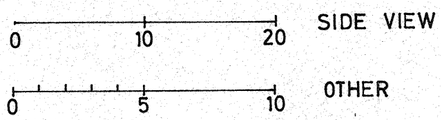
THE ORIGINAL DESIGN (HARZA DWG 290 S35RI)

A



SHELTER WALL ADDED

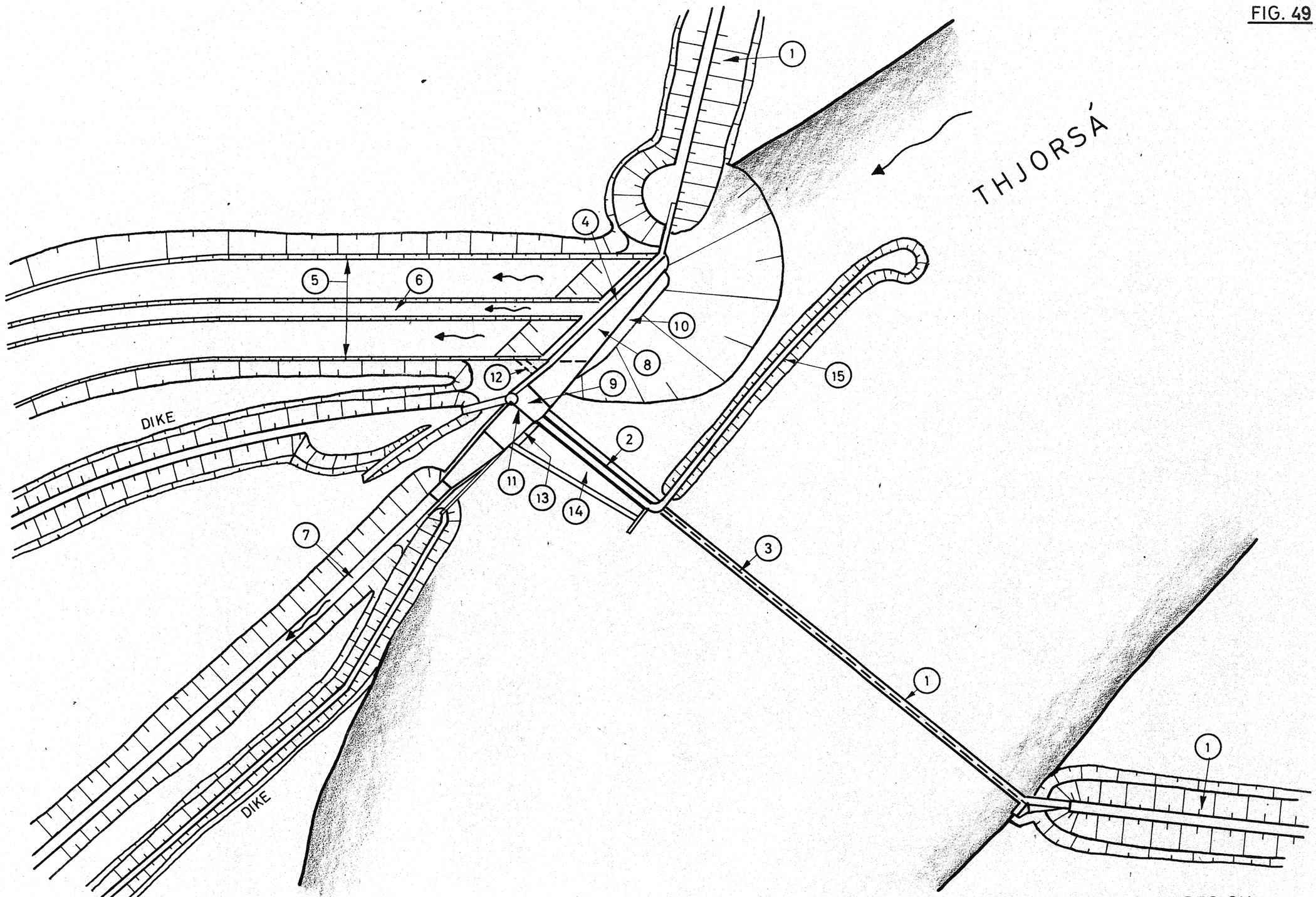
C



THE AUXILIARY
WATER SUPPLY

V.H.L. 60115 JUNE 1966

FIG. 49



HARZA DWG 290 S 34 R 1

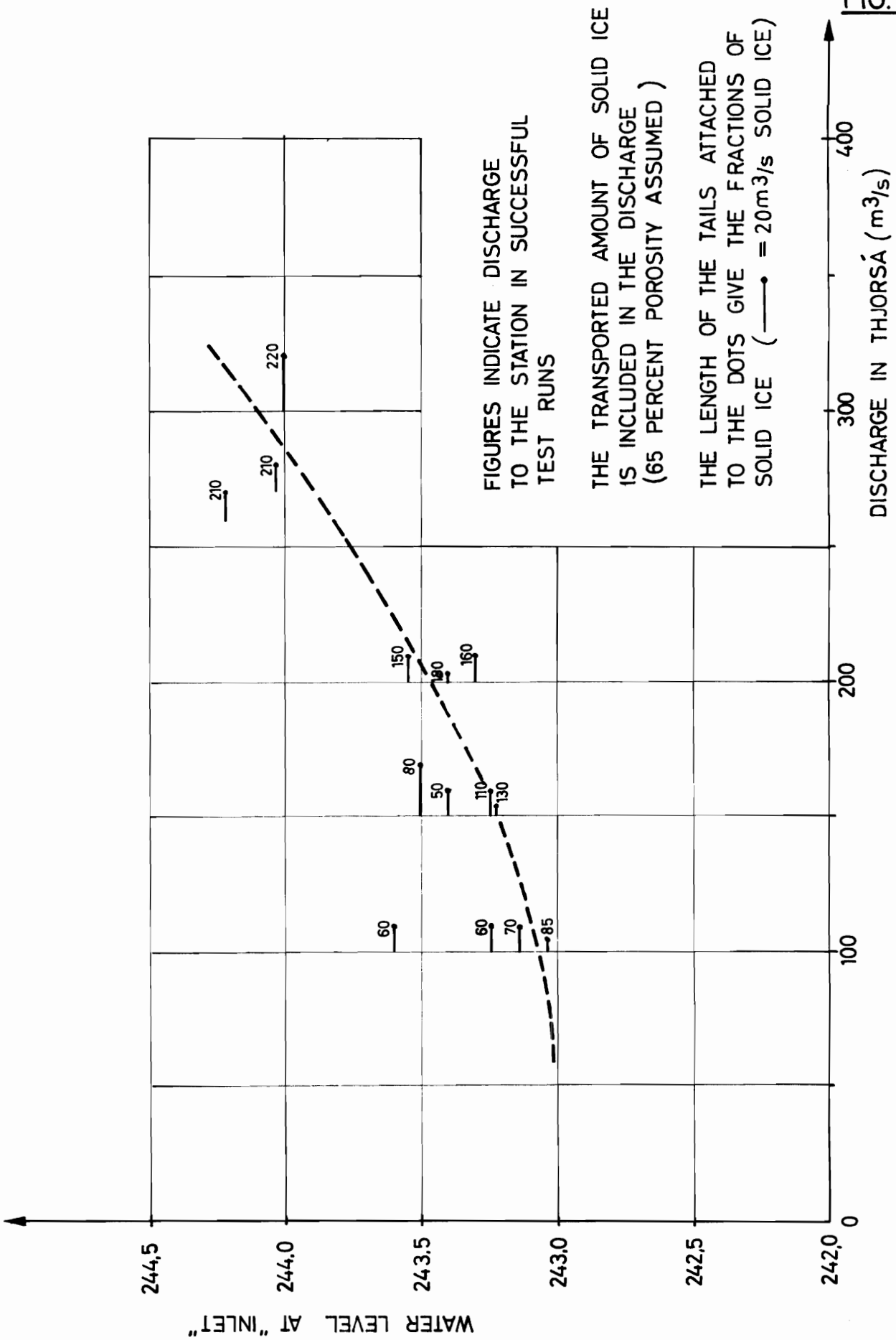
FIGURES CORRESPOND TO DESCRIPTION IN APPENDIX B

THE FINAL DESIGN

SCALE RATIO 1:2000

V.H.L. 600115 JUNE 1966

FIG. 50

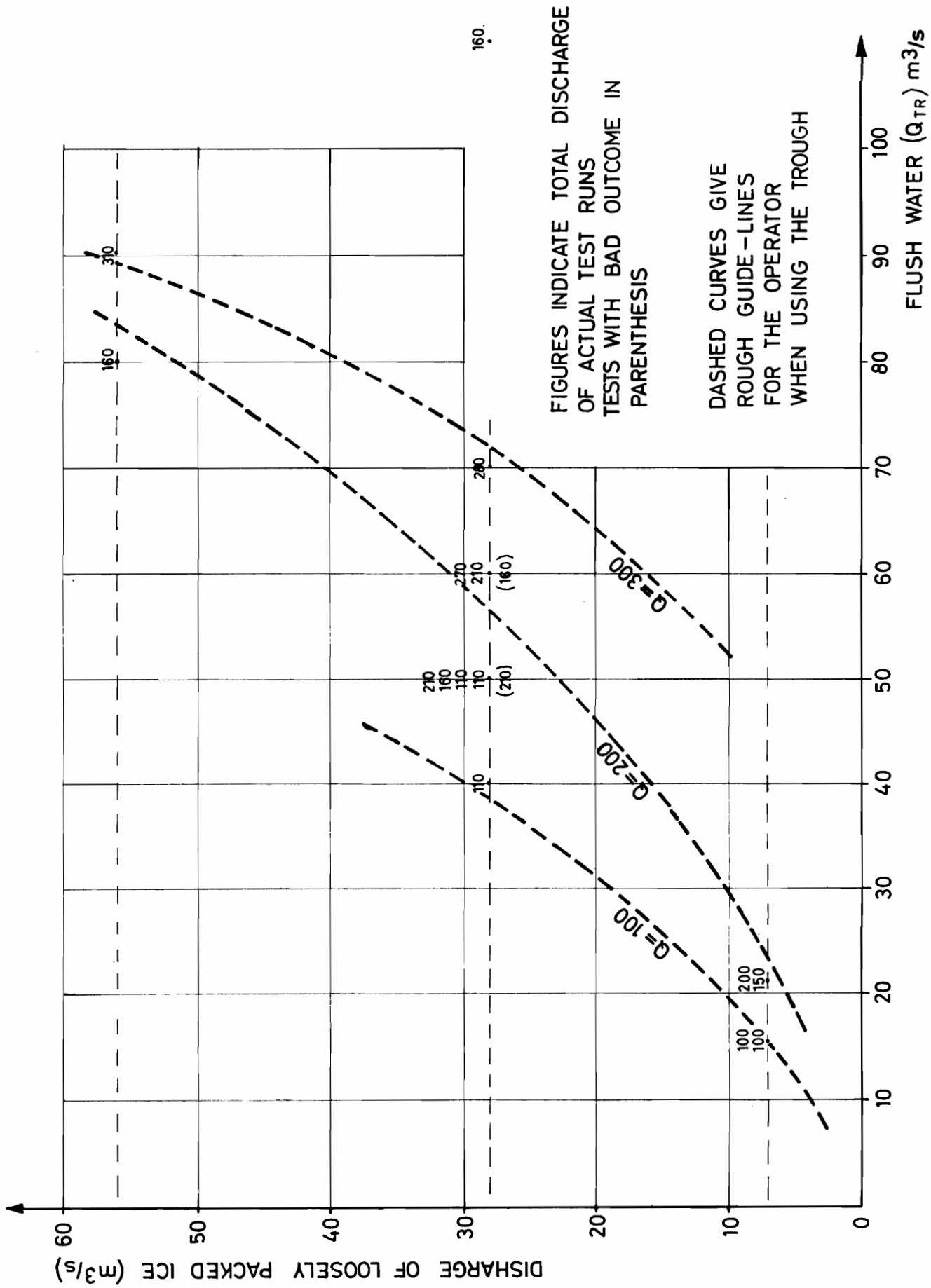


ICE AND WATER LEVEL IN THE INLET AREA

ICE FLUSHED THROUGH THE TROUGH

V.H.L. 600115 JUNE 1966

FIG. 51

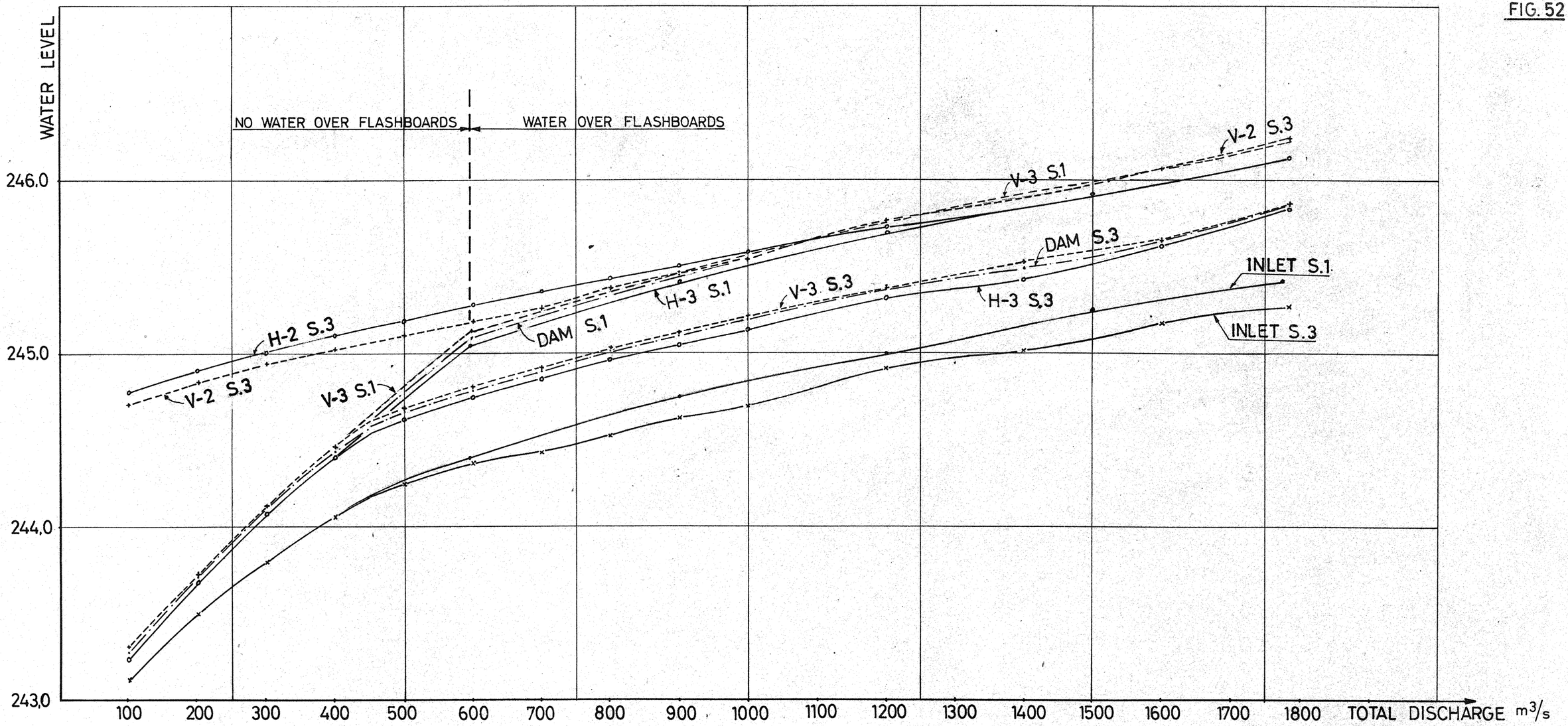


ICE AND FLUSH WATER

ICE FLUSHED TROUGH THE TROUGH

V.H.L. 600115 JUNE 1966

FIG. 52



SITUATION 1 (S.1)

ALL GATES AT EL. 242,50

FLASHBOARDS AT EL. 245,00

$Q_{station} = 0$

SITUATION 3 (S.3)

ALL GATES AT EL. 242.50

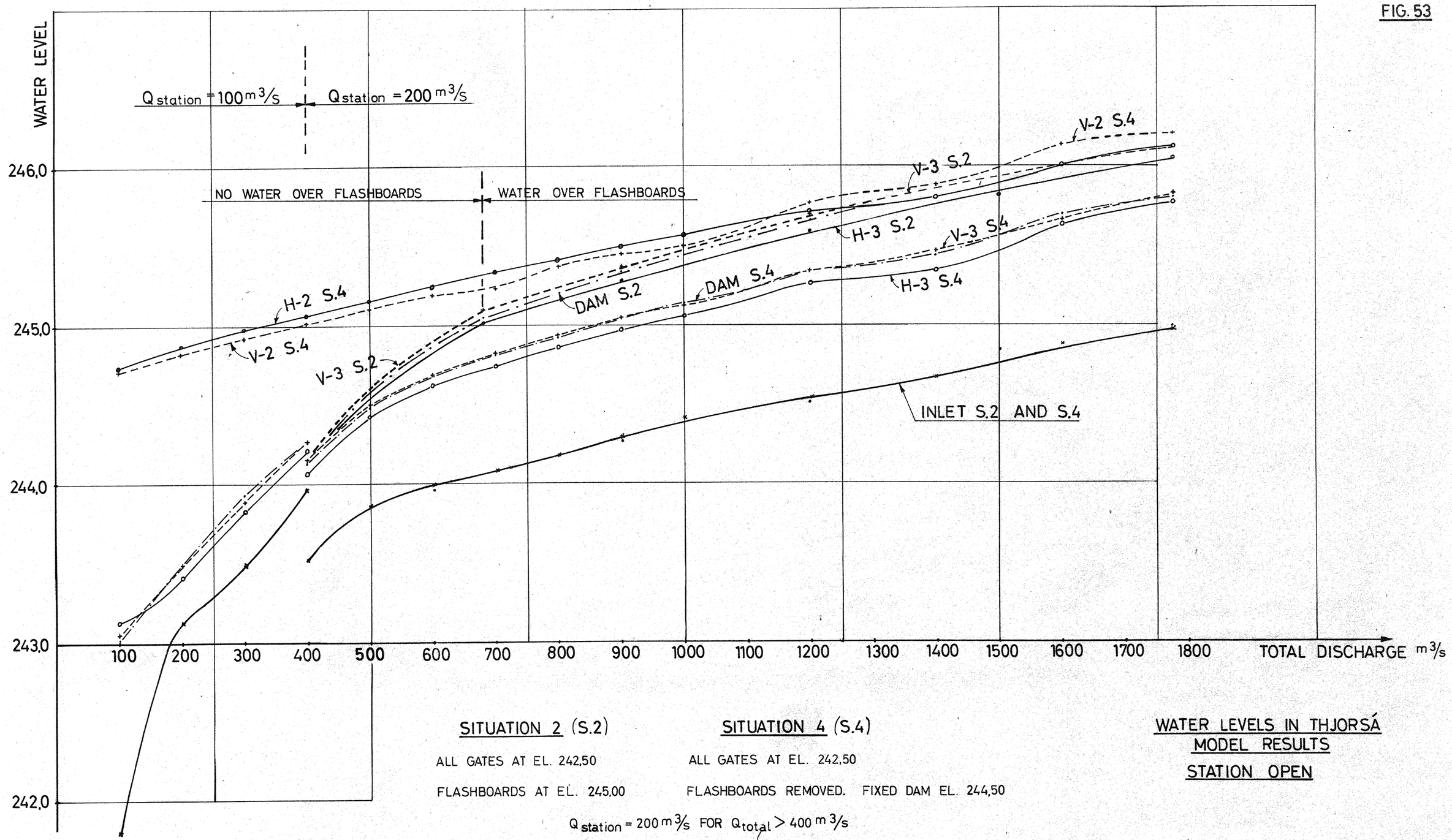
FLASHBOARDS REMOVED. FIXED DAM EL. 244,50

$Q_{station} = 0$

WATER LEVELS IN THJORSÁ
MODEL RESULTS
STATION CLOSED

V.H.L. 600115 JUNE 1966

FIG. 53



SITUATION 2 (S.2)

ALL GATES AT EL. 242,50

FLASHBOARDS AT EL. 245,00

SITUATION 4 (S.4)

ALL GATES AT EL. 242,50

FLASHBOARDS REMOVED. FIXED DAM EL. 244,50

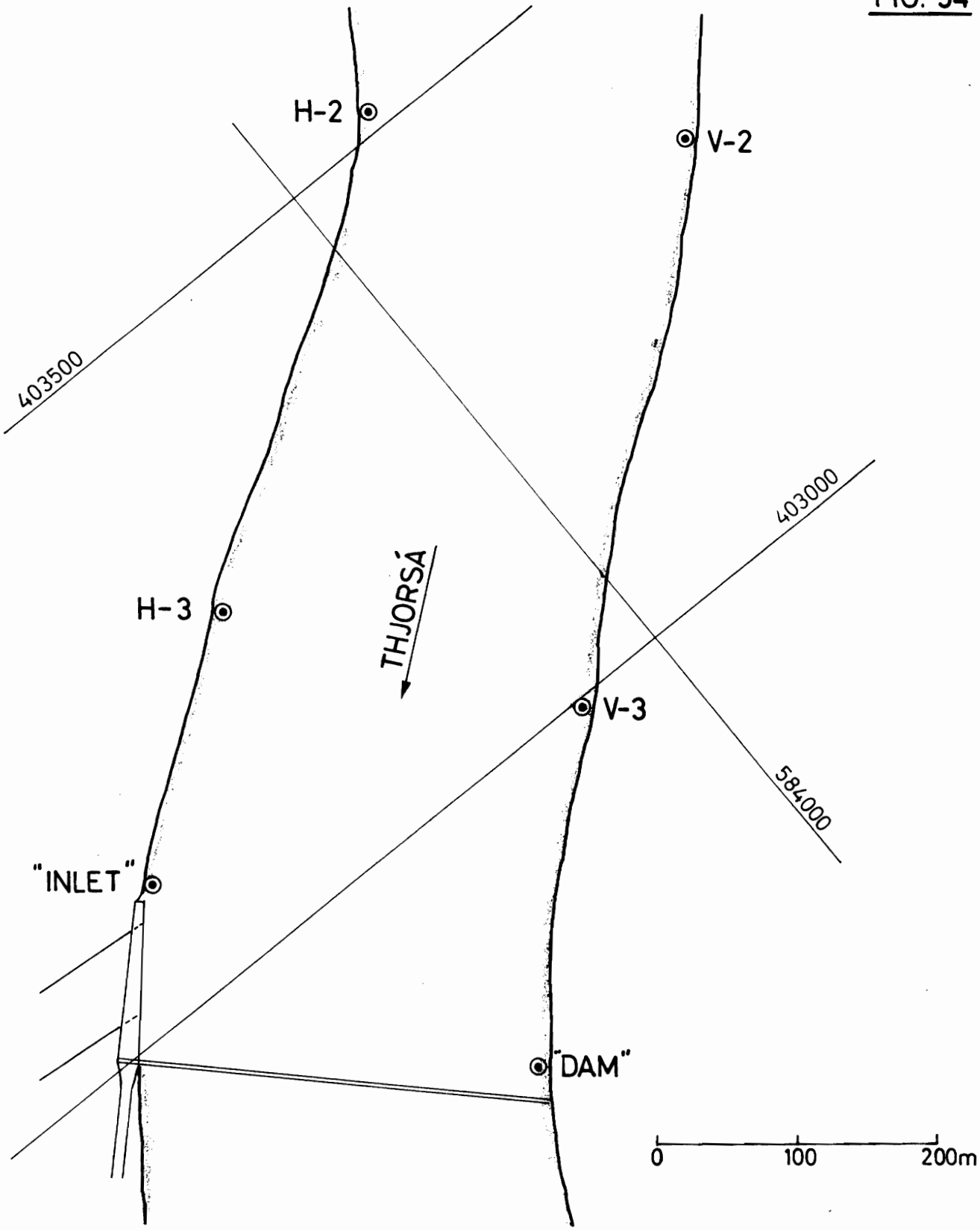
$Q_{station} = 200\ m^3/s$ FOR $Q_{total} > 400\ m^3/s$

— " — 100 m³/s — " — < — " —

WATER LEVELS IN THJORSÁ
MODEL RESULTS
STATION OPEN

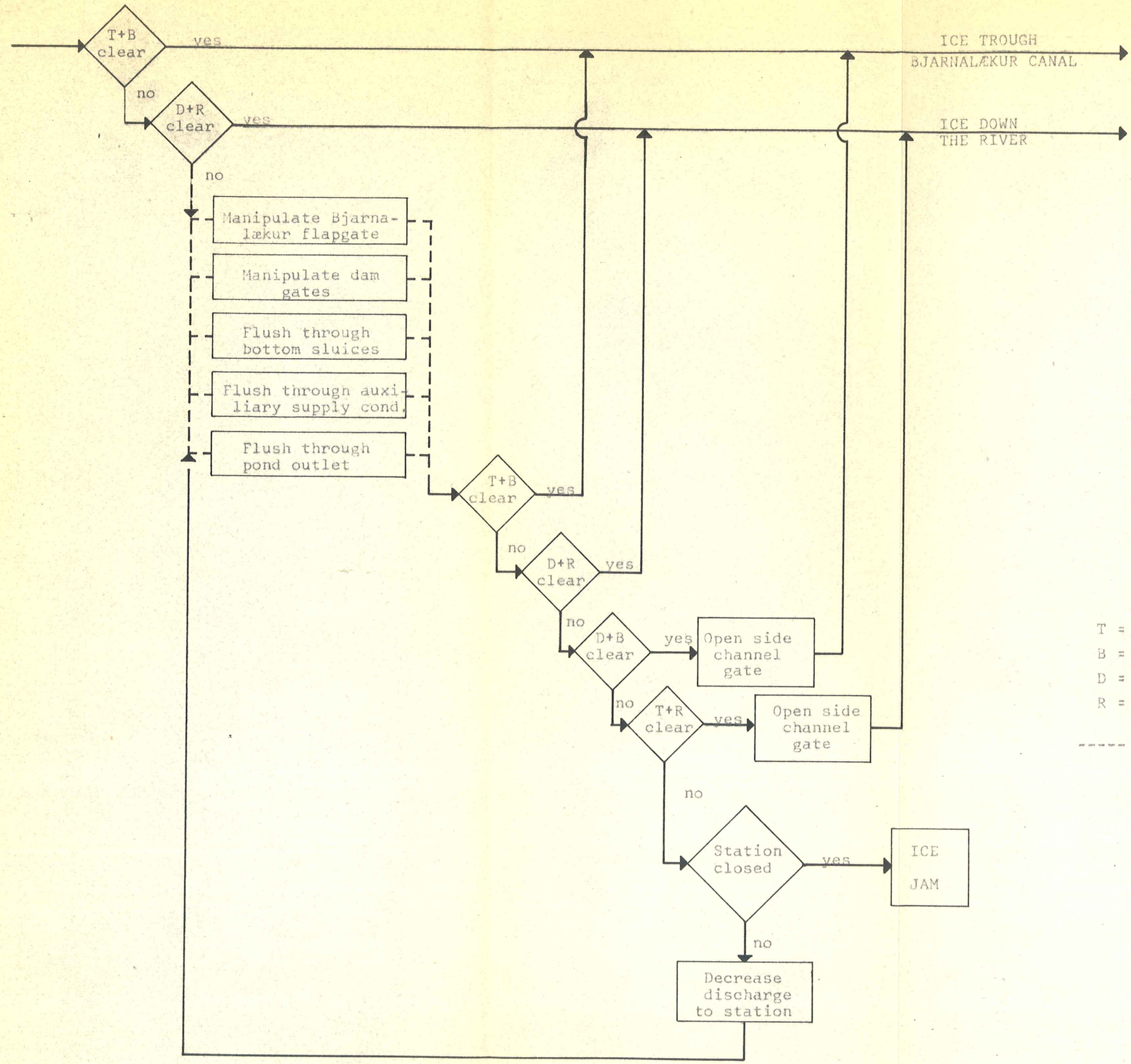
V.H.L. 600115 JUNE 1966

FIG. 54



LOCATION OF LEVEL GAGES

FIG. 55



T = trough
B = Bjarnalækur Canal
D = dam gates
R = river

----- alternative

FLOW SHEET
FOR ICE PASSING
OPERATIONS

V.H.L. 600115 JULY 1966