HYD 20



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

- - -

HYDRAULIC LABORATORY REPORT NO. 20

- - -

# PREVENTION OF SCOUR AND ENERGY DISSIPATION.

by

A. Schoklitsch

Translated from the German by

E. F. Wilsey

- - -

Denver, Colorado
May 15, 1937

# PREVENTION OF SCOUR AND ENERGY DISSIPATION

by

A. Schoklitsch

Translated
by

Edward F. Wilsey, Ass't Engineer
U.S.B.R.

# PREVENTION OF SCOUR AND ENERGY DISSIPATION

A Translation of Pages 78 to 173

of

Kolkabwehr u n d Stauraumverlandung

by A. Schoklitsch

Published by Julius Springer, 1935.

------

Translated by

Edward F. Wilsey, Ass't. Eng'r.

UNITED STATES BUREAU OF RECLAMATION

## Translator's Preface

Prevention of scour and energy dissipation are two of the most important problems confronting the engineers engaged in hydraulic research at the Bureau of Reclamation. Schoklitsch's monograph is probably the most complete written to date from an engineering standpoint on these subjects.

This translation is neither complete nor final but practically all of the ideas and data presented by Schoklitsch can be obtained from it in its present form. All equations, graphs, and tables have been converted to English units wherever feasible. All photographs have been omitted and all references to them deleted. The grouping of several figures on one page for reasons of economy has introduced a certain amount of confusion in locating individual figures. For this reason a table of figures has been added to the translation. The numbers of the figures are identical with those in the German text.

The translator is indebted to Professor Samuel Shulits of the Colorado School of Mines for his generous loan of his personal copy of Schoklitsch's book and to D. P. Barnes, Associate Engineer, U. S. Bureau of Reclamation, for his frequent advice on the translation.

Edward F. Wilsey.

Denver,
May 15, 1937.

CONTENTS

CONTENTS (Continued)

FIGURES

FIGURES (Continued)

FIGURES (Continued)

<u>Figure No.</u>

FIGURES (Continued)

FIGURES (Continued)

TABLES

| No. | | Page No. |
|---|---|---|
| 15 | Observations on the flow in chutes ................ | 75 |
| 16 | Observations on the flow in pipe chutes ........... | 76 |

## C. PREVENTION OF SCOUR AND ENERGY DISSIPATION

Water released at dams or outlet works arrives at the foot of the structure with such a high kinetic energy that without particular precaution, deep scouring at the foundation is to be feared, which may endanger the safety of the entire structure. In order to forestall such dangers, this kinetic energy is transformed, as much as possible, into heat energy which is not readily reconvertible into mechanical energy. From the standpoint of the hydraulic engineer this destructive energy of motion is rendered harmless and is also dissipated at the same time. The auxiliary structure which produces this transformation of energy is called, in short, an energy destroyer, or energy dissipator.

In order to describe the action of energy dissipators it is proper to divide them into two groups, according to the arrangement to be used for the prevention of scour when; first, the flow through the structure carries silt; and second, when it is more or less silt-free. However, it is not possible to draw a sharp line between these two groups, because several devices are just as effective for clear as for silt-laden water.

### 1. ENERGY DISSIPATION

At first, energy dissipation was achieved by allowing the free impact of water against certain parts of the structure which produced the largest possible difference in velocity between adjacent stream lines and thus a large fluid friction, and in addition to this, it sprayed part of the water into the air. Parts of the structure suffered an intense shock from the impinging water, and the spray, frozen in the winter, created difficulties and impaired the effectiveness of the device.

At present, energy dissipation is effected without this free impact by designing the dissipator so that efficient rollers are produced as extensively as possible, in which, as a result of the high fluid friction, the kinetic energy is converted into heat without shock to the structure. A splashing at the water surface is not completely eliminated but this does not create any great difficulty.

Rollers occurring in connection with these energy dissipators are classified according to their position relative to the main stream (figure 55) thus: they are called surface rollers if they lie above the jet; ground rollers, if they lie between the jet and the river bed; and side rollers, if they form at the sides of the jet.

To propogate rollers, the jet is deflected upward in the energy dissipator and, at best, away from all solid walls. Devices such as baffle piers need not penetrate into the jet itself; they should only be used to produce rollers and to give them an efficient form. Energy dissipation occurs only in these parts of the roller where there are large differences of velocity between adjacent stream lines. L. Prandtl[1] has shown that if a difference in velocity occurs between two layers of water flowing past one another, the boundary surface does not remain smooth but first assumes a wave form, then curls back on itself and finally disappears completely in vortices (see figure 56). Between the individual vortices, all of which have the same direction of rotation similar to those formed between two moving plates, large differences in velocity occur and these produce considerable fluid friction which transforms the kinetic energy into heat. Rather than so simple a vortex, probably a large number of vortices of a higher order are formed, although their existence has not yet been fully established by experiments.

Energy dissipation by surface rollers has already been studied by numerous investigators. Theodor Rehbock[2] was probably the first to demonstrate the high degree of energy dissipation occurring in rollers, and he suggested that the larger the surface roller covering a hydraulic jump the greater the energy dissipation; at the same time, however, he pointed out that two rollers of the same size may have different efficiencies. Later K. Safranez[3], as a result of measurements on fourteen different hydraulic jumps, found that besides the size of the surface roller, the so-called dimensionless number, A, is also a factor.

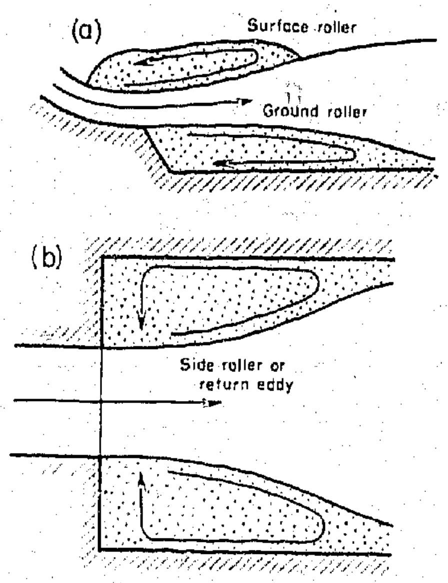



FIGURE 55 - CLASSIFICATION OF ROLLERS:
(a) VERTICAL SECTION,
(b) HORIZONTAL SECTION.

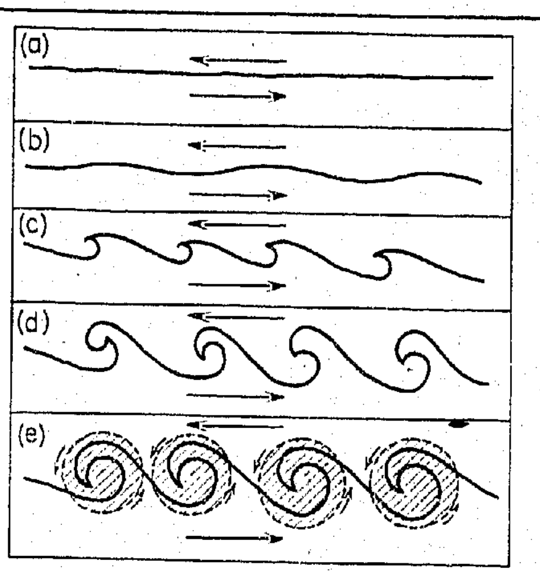



FIGURE 56 - EVOLUTION OF VORTICES AT A BOUNDARY SURFACE (ACCORDING TO L. PRANTL).

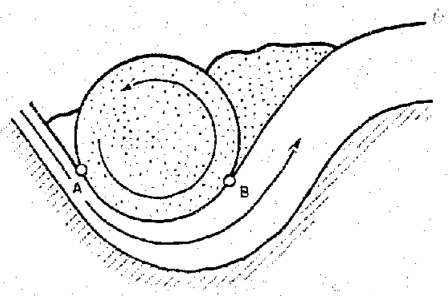



FIGURE 57 - CIRCULAR SURFACE ROLLER.

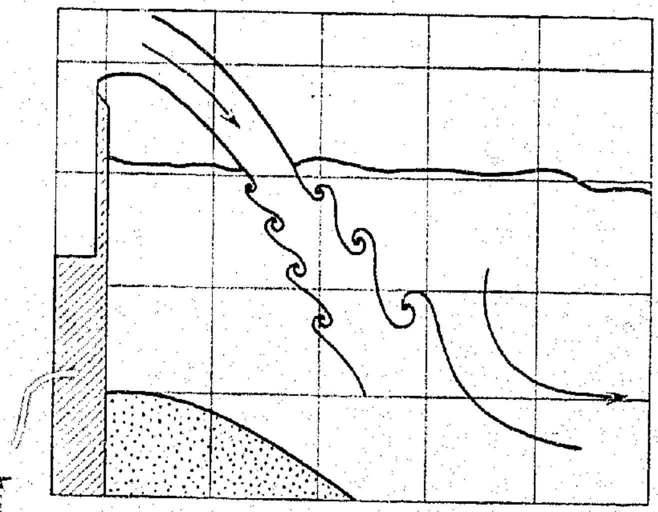


FIGURE 58(b)-CURLING OF THE BOUNDARY SURFACE OF A WEIR NAPPE AND THE DEVELOPMENT OF VORTICES.

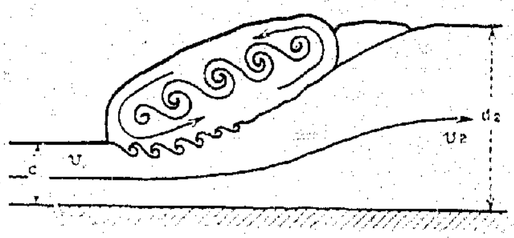



FIGURE 59 - THE STRUCTURE OF AN ELONGATED SURFACE ROLLER.

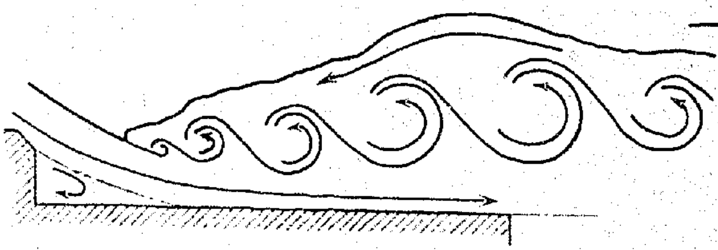


FIGURE 60 - VORTICES IN A SURFACE ROLLER.

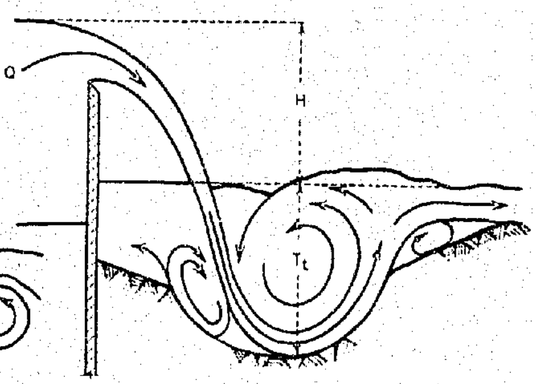



FIGURE 62 - SCOUR PRODUCED BY A WEIR NAPPE.

A is defined as the ratio of the velocity of the shooting flow, $v_1$, to the wave velocity, $\sqrt{gd_1}$, $d_1$ being the depth of the flow. He arrived at the following empirical equation:

$$N = c\ V\ (A^{3/7} - 1) \quad \ldots \quad (61)$$

in which N is the power dissipated in horsepower; V, the volume of the surface roller in cubic foot; and c, a coefficient of the dimension $\frac{t}{\text{sec.-feet}}$, for which he gives the value 0.209.

In spite of the high quality of these experiments, J. Kozeny[4] has disputed the efficiency of surface rollers as a means of energy dissipation. Kozeny's results have been refuted by P. Nemenyi[5].

According to model tests, the efficiency of a roller depends principally on its form. The more it approaches a circular form, the less is its effectiveness in dissipating energy. For example, the jet in figure 57 is directed in such a way that a large roller of approximately circular form is produced. If the roller rotates so that its peripheral velocity is approximately equal to the velocity of the boundary layer of the jet, although in the interior of the roller large differences of velocity exist, the dissipation of energy is nevertheless small. If it rotates so that its peripheral velocity is somewhat less than the velocity of the jet, a difference of velocity exists along the boundary surface between A and B in figure 57. A series of vortices is formed, as explained in the foregoing, in which energy is transformed; the interior of the roller, however, does not assist to any large degree in the transformation of energy.

If the nappe of a weir impinges on an unprotected river bed, a roller is formed which is also nearly circular. Figure 58 shows a series of vortices along the surface of contact between the nappe and the tail water, the interior of the roller, however, being free from vortices. In spite of the large total volume occupied by these vortices, the energy dissipation is small, and the nappe erodes the bed to a great depth.

If the jet is directed along a flat slope or horizontally into the tail water, a hydraulic jump is formed providing the velocity of the jet is greater than the depth, $d_1$, corresponding to the wave velocity, $\sqrt{gd_1}$; in other words, it is shooting, and in the region where the velocity changes from shooting to streaming, a flat surface roller is formed. While a series of vortices can develop in the limiting case of a circular roller only along the boundary surface between the jet and roller, in this case a flat roller is formed (figure 59), and in which vortices are formed not only along the surface of contact between the roller and the jet, but also in the interior of the roller. These internal vortices, because the flow in the upper part of the roller is opposite in direction to the flow in the lower part, are more prominent than those between the surface roller and the jet, where the difference of velocity is relatively small. Because of the severe disturbances occurring within the hydraulic jump, those internal vortices are difficult to investigate experimentally. Large internal vortices as well as those indicated by arrows, which occur along the surface of contact between the roller and the jet, are shown in figure 60.

The flow in ground rollers and in side rollers is similar to that in surface rollers; their efficiencies likewise depend largely on their shape.

## II. PREVENTION OF SCOUR

Special structures for scour prevention are used if the discharge through the structure ordinarily carries silt. Recently they have come into use in the prevailing number of projects. The energy dissipation is carried out, for the most part, in subsidiary structures which may be but a small part of the entire project. Since the largest part of the energy is dissipated above the unprotected river bed itself, it is by designing such structures so as to deflect the jet upward that the river bed in this region of the dam is least eroded. A complete prevention of scour, as will be shown later in detail, is for all practical purposes not only not possible but also not necessary.

Before the different kinds of stilling pools are discussed, the types of flow existing downstream from the dam and the corresponding scour will be defined.

### a. The Relation of the Form of the Jet to the Production of Scour

Whenever a dam is built, the reservoir formed silts up, and as long as this continues, a deepening of the river bed downstream from the dam ensues. Later, if the silting is completed and silt is once again transported through the structure, the river bed downstream from the dam is raised, and it may fill up to the same slope existing before the erection of the dam. At least during the first ten years in the life of a dam, on account of this silting of the reservoir, a continually changing elevation of the river bed will have to be reckoned with. In addition to this, the digging of cut-offs downstream from the dam may cause a permanent lowering of the bed. The influence of the elevation of the river bed and the corresponding tail-water elevations on the type of flow and formation of scour will be investigated next.

Figure 61 shows the different forms of discharge and scour occurring at a sluiceway with a horizontal apron without a sill, under constant head and discharge but with different elevations of the river bed downstream from the dam. If the river bed is appreciably higher than the apron, a surface roller is formed covering the jet and extending upstream to the apron; if it reaches the gate and if, as it were the upstream part of the roller is dammed up by the gate, the surface roller is said to be drowned (figure 61a). The discharge jet, as a rule, assumes a wave form. If the crest of the wave occurs just below the end of the apron, the jet is called a positive wave jet (figures 61b and 61c); if a wave trough occurs there, it is called a negative wave jet (figure 61e). The deeper the bed of the river downstream from the dam below the elevation of the apron, the farther downstream the surface roller is displaced by the jet, the smaller it is and the more the wave is flattened out. If the elevation of the river bed is above the apron, a positive wave jet occurs which, if it bears a surface roller in front

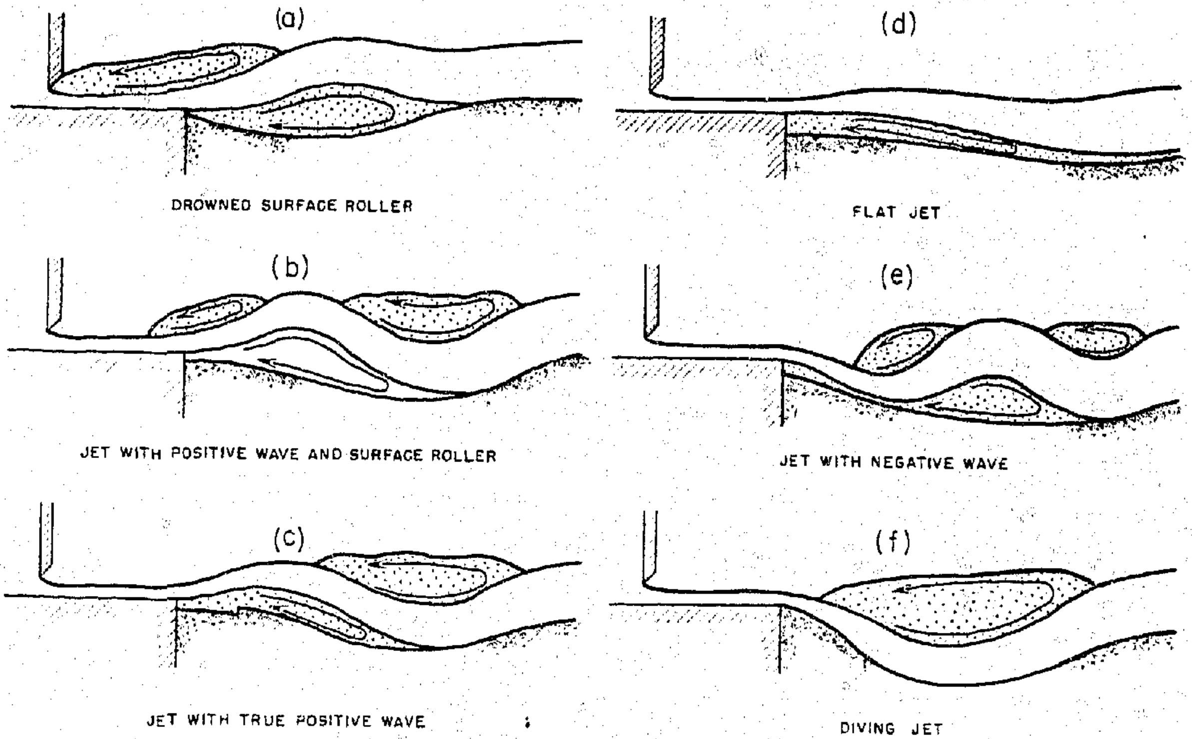



FIGURE 61 – FORMS OF DISCHARGE FROM A SLUICE WITH A HORIZONTAL APRON FOR VARIOUS ELEVATIONS OF THE RIVER BED DOWNSTREAM FROM THE APRON.

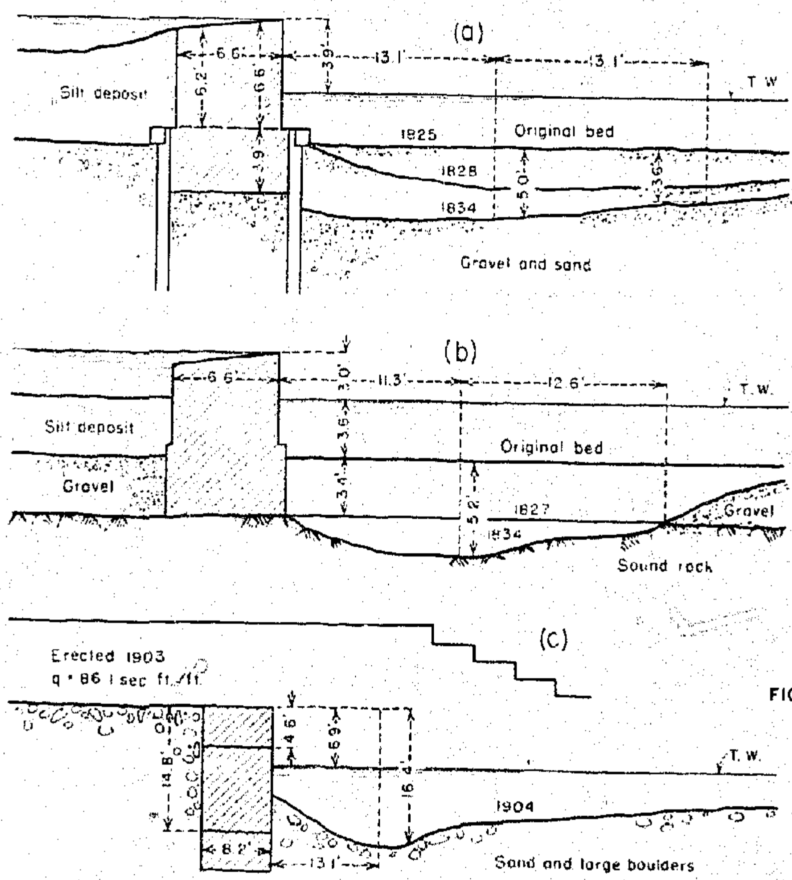



FIGURE 64 – SCOUR CAUSED BY FREE IMPACT AGAINST THE UNPROTECTED RIVER BED. (C) DURING A FLOOD THE DAM SANK VERTICALLY 4.6 FEET AND WAS LATER RAISED (ACCORDING TO H. ROTH).

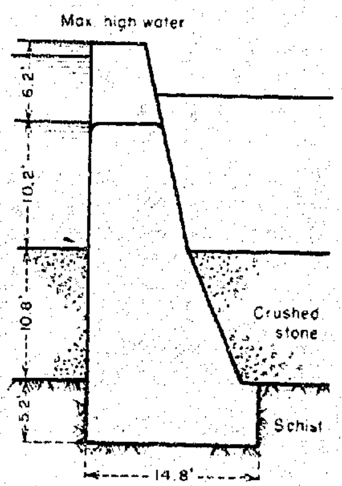



FIGURE 65 – THE DAM AT NEUHOFEN ON THE KREMS RIVER, AUSTRIA, "SAFE" AGAINST SCOUR.

of the first wave crest (figure 61b), is called a positive wave jet with a surface roller. The more the wave is depressed, the deeper will be the scour and the farther downstream the place of greatest depth of scour. If the bed is approximately at the same elevation as the apron, the surface roller lying in front of the first wave crest vanishes completely and we have a true positive wave jet. The greatest depth of scour is produced by this kind of flow.

Scour data for a positive wave jet and also a true positive jet varies greatly for fine bed material, even uder identical conditions. In the results of the experiments shown in the graphs of the following section, the depths of scour produced by the various jet forms are distinguished by different symbols, and the curves drawn through the points are likewise differentiated. The larger the material of the bed is and the greater the head, the smaller the surface scoured by a wave jet, until, with a bed grain size of 10 millimeters and a head of 2.3 feet, depth of scour which had increased with the appearance of a true wave jet by leaps and bounds showed no further development.

The more the river bed was lowered below the apron the more the jet streched out, until the surface roller lying in the first wave valley suddenly disappeared, and the jet became flat (figure 61d), attacking the bed for a considerable distance. The smaller sizes of bed material scoured out by the jet were returned upstream by the ground roller and deposited to a negligible depth. The surface of this flat jet created an impression of instability It was wildly agitated, and it fluctuated wildly in comparison with the surface previously observed at a large depth of scour.

Even within the range of the true positive wave, the discharge jet shows signs of instability. A well-developed ground roller forms under the first wave crest and carries a considerable amount of the scoured material back upstream against the apron . These deposits decrease the volume occupied by the ground roller more and more until finally the jet strikes against the bed at the end of the apron and the space formerly occupied by the ground roller is now taken up by a so-called diving jet (figure 61f). After that, the jet rises again, becoming a positive wave jet. Once a positive wave jet is formed, a diving jet rarely occurs. When it does form, it lasts only a short time and produces a scour whose greatest depth is essentially smaller than that lying farther downstream produced by the wave jet. However, the lower the river bed below the apron, the more pronounced, more persistent, and more destructive the diving jet will be. If a diving jet is replaced by a flat jet, it will appear again provided that by a further lowering of the river bed a wave trough appears downstream from the apron; in other words, a negative wave jet appears (figure 61e). Finally, by depressing the river bed still farther, the diving jet persists so long that the scour produced by the negative wave jet is not fully developed because the interval before the reappearance of the diving jet is too short. Under this circumstance, the depth of scour produced by the diving jet is somewhat greater than that developed by a negative wave jet.

If a sill is placed at the end of the horizontal apron, the type of flow does not change providing the continuous alternation between wave jet and diving jet and, above all, the appearing of a negative wave jet can be prevented. The type of flow depends greatly on the form and the dimensions of the sill and cannot be described in more general terms. However, the influence of various kinds of sills on the depth of scour and type of flow will be briefly discussed.

b. The Free Development of Scour at Weirs and Dams

Formerly dams, and in particular fixed dams, were constructed without any provision for a stilling pool, thus allowing the overfall jet to impinge against the unprotected bed. The unusually deep scour of even rocky river beds occurring under such circumstances was combatted by the costly dumping of stones on the downstream side of the dam. Such structures are now rarely built, but some still exist and must be preserved. Now it is quite possible to design dams without an objectionable increase in cost, which can be easily maintained and the scour holes filled by deposits, in part, at least, by silt. But before considering the feasibility of such measures, the process by which scour is produced at these structures must be clarified. Dams with no adequate protection against scour can be classified in three groups; namely, those with a free overfall, those with aprons sloping downstream, and those with horizontal aprons.

### 1. THE PRODUCTION OF SCOUR AT DAMS WITH A FREE OVERFALL TO THE RIVER BED

The author has already investigated[6] the scouring process at a free overfall of water on an unprotected river bed. Let q be the discharge per unit length of dam in second-feet per foot; H, the head in feet measured from the headwater elevation to the tail-water elevation, as shown in figure 62; and $T_t$, the maximum depth of water above the scoured region in feet; then

$$T_t = \frac{0.30}{d_m^{0.32}} H^{0.2} q^{0.57} \qquad (62)$$

in which $d_m$ is the effective diameter of the bed material in millimeters. This average diameter, $d_m$, is determined from a mechanical sieve analysis and is defined as follows: It is that diameter such that 10 percent by weight of a sample is coarser. The depth of scour, calculated from this formula, is the maximum occurring over the entire width of the stream after a prolonged impact of the discharge; it is this amount only if this discharge is maintained. As soon as the unit discharge, q, slackens, the steep sides of the scour holes cave in, and hence the depth of scour measured after a flood is always smaller than that occurring during the flood. If silt is carried over the dam, the depth of scour after a flood is less, for the receding flood deposits sufficient material in the scour holes so that when the normal discharge is reached again only a negligible increase in scour will be found.

Scour can become particularly dangerous when the nappe clings to the downstream vertical face of a weir, for the maximum depth of scour is, as a rule, greater than for a freely falling nappe and, too, it occurs directly at the weir.

The redepositing of material in the scoured region during a receding flood is exemplified at the Faal Dam on the Drau River. This dam is built with an apron whose effective length (measured from the gate downstream) is not even equal to the head, H. When the gates are opened, a part of the nappe jumps completely over the apron and erodes the river bed to a considerable depth. During grouting operations, occasioned by an urgent strengthening of the dam on the downstream side, iron parts were found many feet below the eroded river bed. These had fallen into the water during construction and had settled lower and lower as the erosion became deeper, later being covered by depositions of silt.

In figure 64 are shown three examples of scour produced by a free overfall of water, reported by H. Roth[7]. The scour shown in b and c of this figure are of particular interest; scour in b eroded solid rock in less than a year to a depth of 1.80 feet, and in c, during a flood, the dam itself sank 4.6 feet into the scour which had extended under the foundation. Subsequently the dam was raised. This disaster also confirmed the observations made in the experimental channel to the effect that depths of scour during floods are always greater than after a flood.

In order to reduce the depth of a scour in such cases, the so-called stepped dam has been proposed in which the total overfall height is divided into a series of smaller drops by a number of steps in cascade. At small discharges the depth of scour can be reduced by this method, but at greater discharges, silt carried over the dam is deposited on the steps, transforming the jagged surface into a comparatively smooth one, which conducts the discharge at a steep angle onto the unprotected river bed. Therefore, during floods, the depth of scour is not reduced by subdividing the overfall height. As the flood recedes, the material deposited on the step is scoured away and the dam functions again as a stepped dam.

Dams with a free overfall are now seldom built; they are still in use in river construction and on mountain streams for bed sills and check dams, and also at a few projects where it is desirable to dispose of the discharge in a simple way. Even without an apron to deflect the overfalling jet and although a great depth of scour was to be expected, the dam[8] in figure 65 had such a deep foundation that it was not endangered by scour.

## 2. FORMATION OF SCOUR AT DAMS WITH APRONS SLOPING DOWNSTREAM

The undesirable effects at a free overfall have been known for a long time and an attempt has been made to reduce the depth of scour produced thereby by directing the flow along an apron sloping downward at a small angle, and thence into the tail-water. A small decrease in the depth of scour, compared to that produced by a free overfall has been achieved by this means, but the depth of scour was still extraordinarily high.

In order to determine the effect of the slope of the apron on the depth of scour, several models of dams were constructed with aprons having the following slopes: 1:00, 1:10, 1:4, 1:3.3, and 1:1. In those experiments the water was discharged from under a sluice gate and the apron had an effective length equal to 1.5 times the head. The bed downstream from the apron consisted of gravel with a diameter, $d$, equal to 6.25 millimeters and the unit discharge, $q$, was held constant at 0.60 second-feet per foot.

With these sloping aprons, the flow was first in the form of a wave jet (figure 66a), then a diving jet (figure 66b), regularly alternating from one to the other. It was thus necessary to consider the composite scour produced by both types of jets taken together in order to obtain a true concept of the influence of the slope of the apron. In all of the experiments, trough-shaped scour holes were formed whose greatest depth occurred at some distance downstream from the apron; the maximum depth of scour, T, measured from the elevation of the downstream end of the apron was sufficient for describing the scour.

A freely falling nappe from a weir produces a diving jet, exclusively; likewise only a diving jet appears with an apron sloping

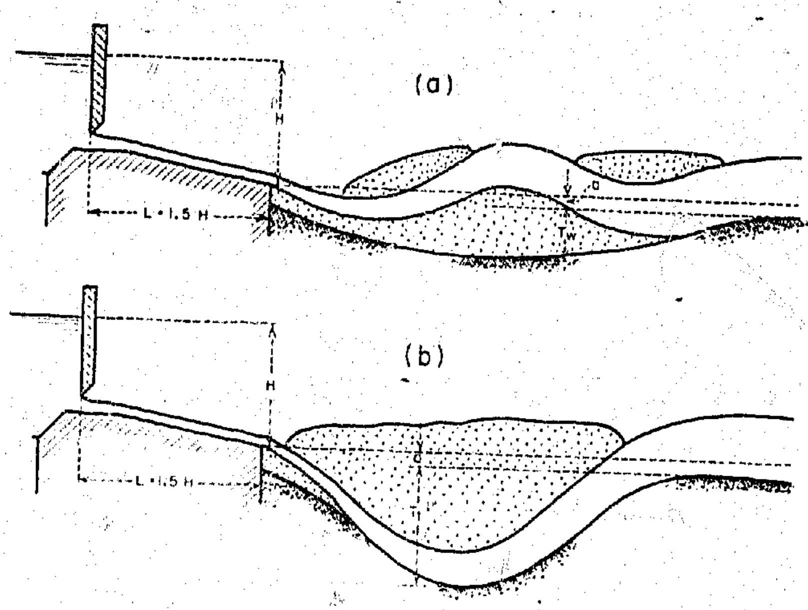


FIGURE 66 - SLUICEWAY APRON WITH DOWNWARD SLOPE. (a) WAVE JET. (b) DIVING JET.

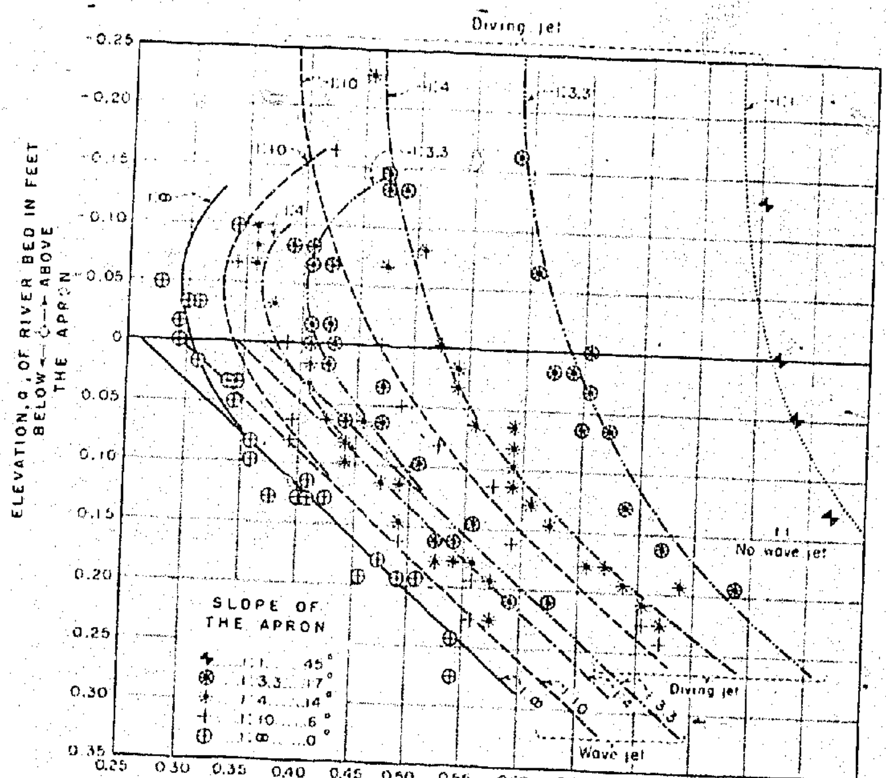



FIGURE 67 - MAXIMUM DEPTH OF SCOUR BELOW A DOWNWARD SLOPING APRON OF A SLUICEWAY. DIAMETER OF BED MATERIAL, d = 6.2 mm.; HEAD, H = 1.08 FEET; DISCHARGE, q = 0.6 SEC. FT. / FT.





X-D-2012

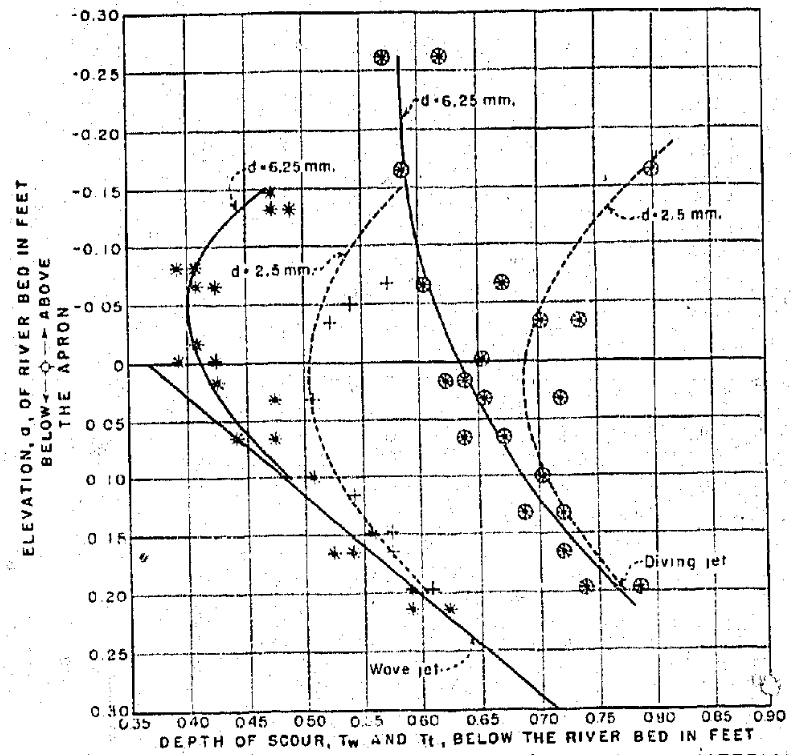



FIGURE 68 — INFLUENCE OF THE GRAIN SIZE, d, OF THE BED MATERIAL ON THE DEPTH OF SCOUR AT A SLUICEWAY WITH AN APRON SLOPING DOWNWARD 1:3.3. HEAD, H = 1.08 FEET; DISCHARGE, q = 0.60 SEC.FT./FT.

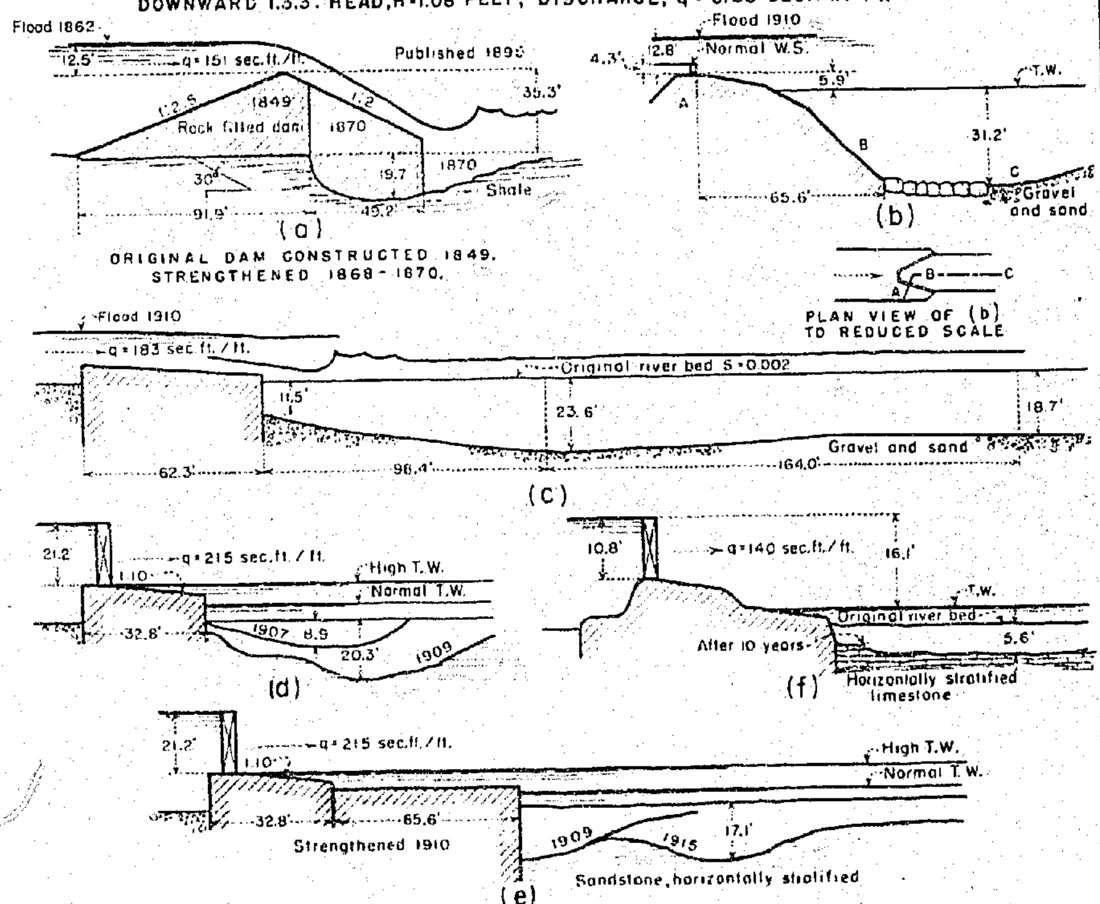



FIGURE 69 — SCOUR AT DAMS WITH DOWNWARD SLOPING APRONS (ACCORDING TO H. ROTH).

downward at 1:1. With smaller apron slopes, a wave jet alternates regularly with a diving jet, as has been previously described. The results of the tests are shown in figure 67. Since the elevation, a, of the river bed downstream from the apron exercises an important influence on the erosion, model tests on aprons had to be carried out with different elevations of the river bed. The less the slope of the apron the less was the depth of scour, and the less the diffenernce between $T_w$ and $T_t$, the depths of scour produced by the wave jet and diving jet, respectively. The important influence exerted by the position of the bed downstream from the apron on the depth of scour was thus clearly established.

As in the case of a freely falling jet, figure 68 shows that the depth of scour is dependent on the size of the bed material. The less the slope of the apron the less the influence of the size of the bed material. For a horizontal apron and for the lower elevations of the river bed, the size of the bed material as far as the maximum depth of scour is concerned, is wholly unimportant. However, the finer the material the more elongated will be the scour.

Scour measurements at several dams with sloping aprons are shown in figure 69.

In view of the fact that the scour with sloping aprons is so decidedly unfavorable, further experiments are contemplated. However, the experiments already completed are comprehensive enough to show that downward sloping aprons should not be used under any circumstances.

## 3. DEVELOPMENT OF SCOUR AT DAMS WITH HORIZONTAL APRONS WITHOUT SILLS

Dams having horizontal aprons without sills are frequently built, and they are popular today even though it is well known how to reduce the depth of scour and fix the type of flow by means of sills. The author has carried out numerous series of experiments not only to determine the scour at structures having horizontal aprons without sills, but also to clarify the circumstances which govern its formation.

Preliminary experiments had shown that the development of scour, other things being equal, was influenced by the means of releasing the discharge. As a rule, it is preferred to discharge the water over spillways at power plants, apparently because the impact of the jet striking the apron helps to dissipate more energy than when the water is discharged from under a sluice gate. Several series of experiments were undertaken to establish conclusively the proper method of discharging water at a dam. The first series of experiments was performed on the sluiceway model shown in figure 70. Throughout these tests, the average diameter, d, of the bed material was 1.5 millimeters, and the discharge, q, 0.60 second-feet per foot. In the second series of tests, water was discharged over a weir, and in the third, half of the discharge was through the sluiceway, the other half over the weir. The elevation of the river bed downstream from the dam was the same in all of the experiments. However, the effective length of the apron, L, (see figure 70) was varied.

The maximum depth of scour, T', below the elevation of the apron was taken as the measure of the scour. The results of these three groups of tests are given in figures 71 to 73. It is seen that with a free discharge over the weir, the depth of scour is greater than in the other cases. In this case, the shorter the effective length of the apron, the deeper is the scour, and with an effective length of apron, L, equal to 0.9 of the head, H, part of the discharge overleaps the apron, producing a depth of scour almost equal to that produced by free impact on an unprotected bed. An example[7] of the deep scour caused by this overleaping of the jet is shown in figure 74.

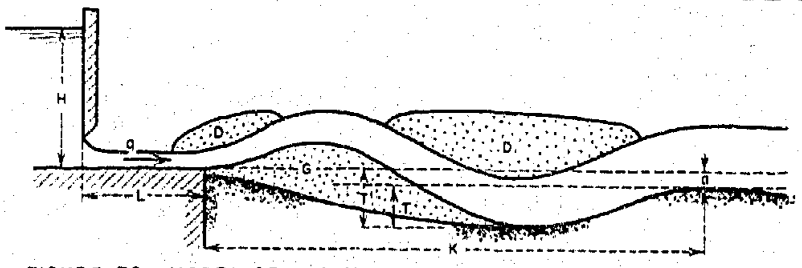



FIGURE 70 — MODEL OF A SLUICEWAY SHOWING THE SYMBOLS USED.
D = SURFACE ROLLER, G = GROUND ROLLER, K = LENGTH OF SCOUR,
AND L = EFFECTIVE LENGTH OF THE APRON.

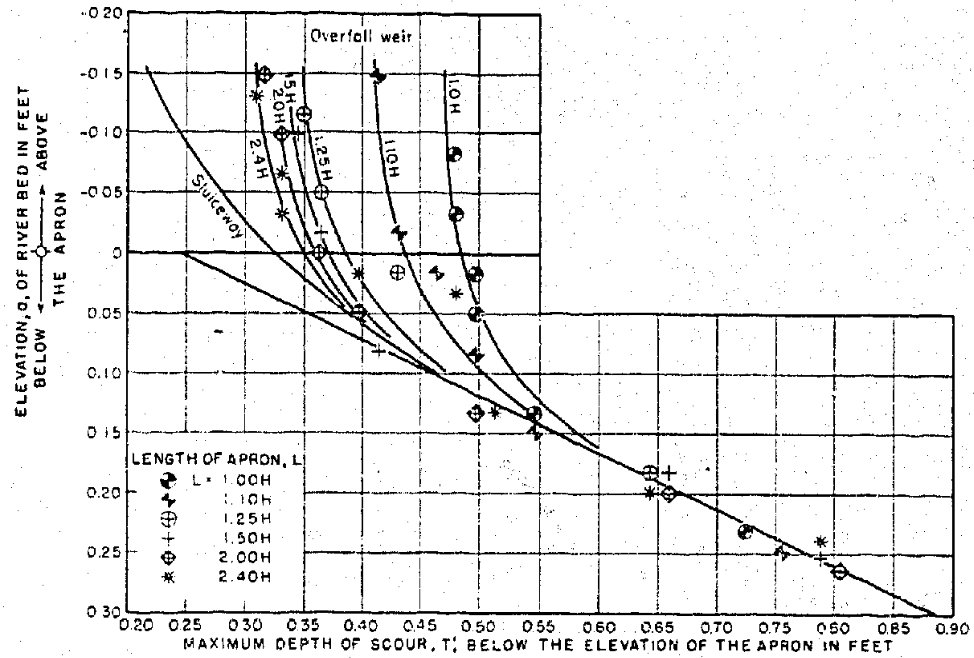



FIGURE 71 — MAXIMUM DEPTH OF SCOUR, T', AT AN OVERFALL WEIR WITH VARIOUS LENGTHS OF APRONS, L.
HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.; DIAMETER OF BED MATERIAL, d = 1.5 mm.

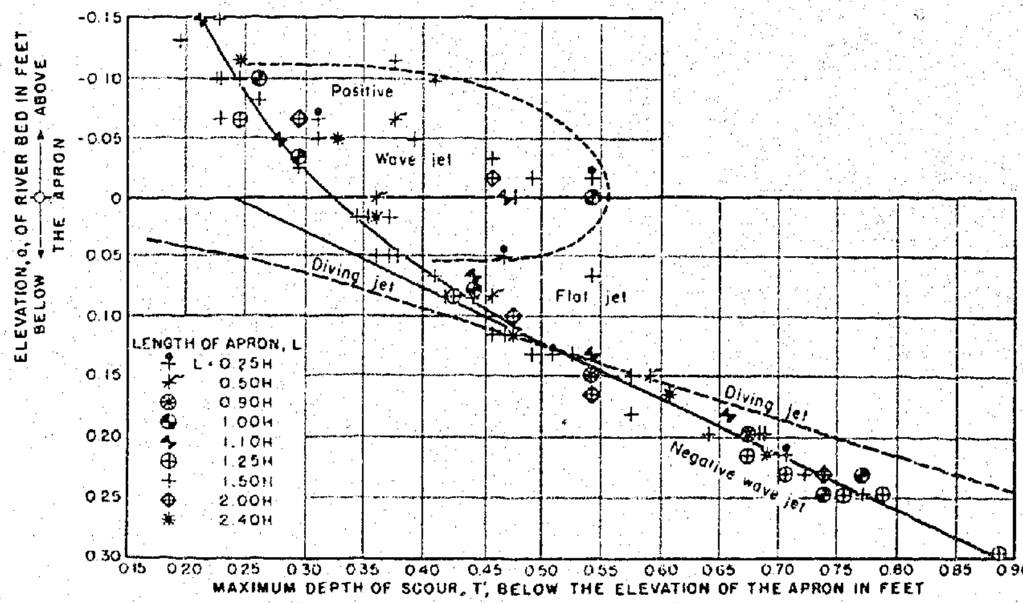



FIGURE 72 — MAXIMUM DEPTH OF SCOUR, T', AT A SLUICEWAY WITH APRONS OF VARIOUS LENGTHS, L.
HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.; DIAMETER OF BED MATERIAL, d = 1.5 mm.

HYD-20



X-D-2014

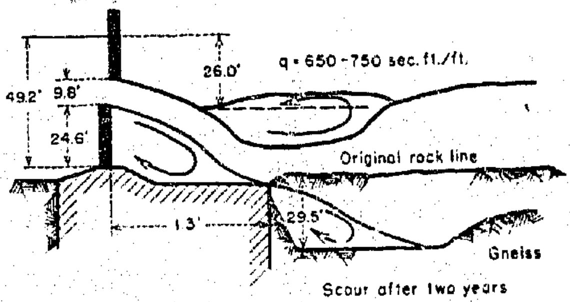



FIGURE 74 - FORMATION OF SCOUR WITH AN INSUFFICIENT LENGTH OF APRON (ACCORDING TO H. ROTH).

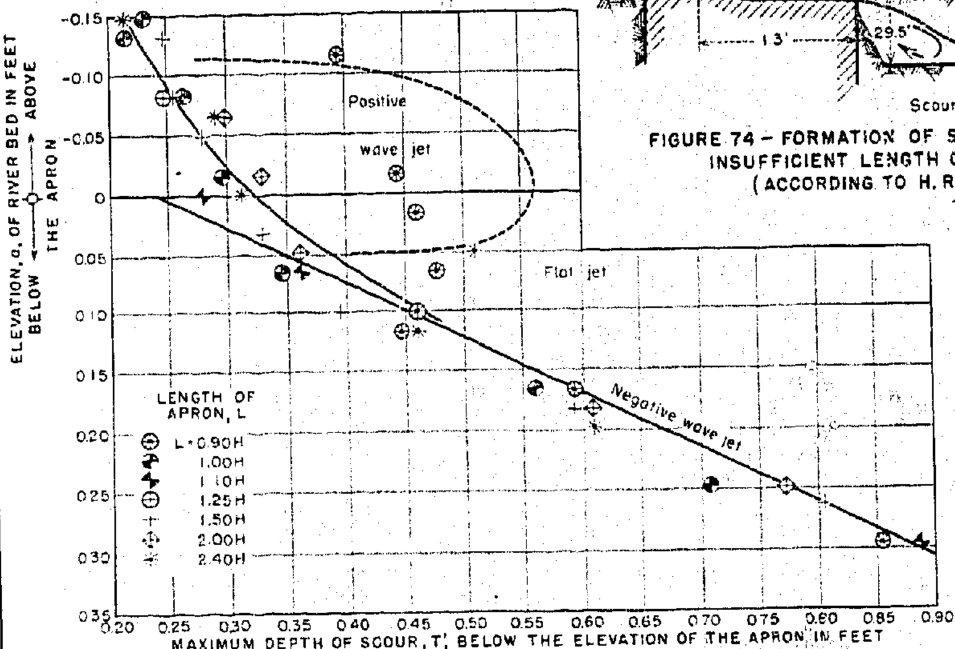



FIGURE 73 - MAXIMUM DEPTH OF SCOUR, T', FOR VARIOUS APRON LENGTHS, HALF THE DISCHARGE (0.30 SEC. FT./FT.) BEING OVER THE WEIR, THE OTHER HALF THROUGH THE SLUICE GATE. HEAD, H = 0.92 FT.; DIAMETER OF THE BED MATERIAL, d = 1.5 mm.

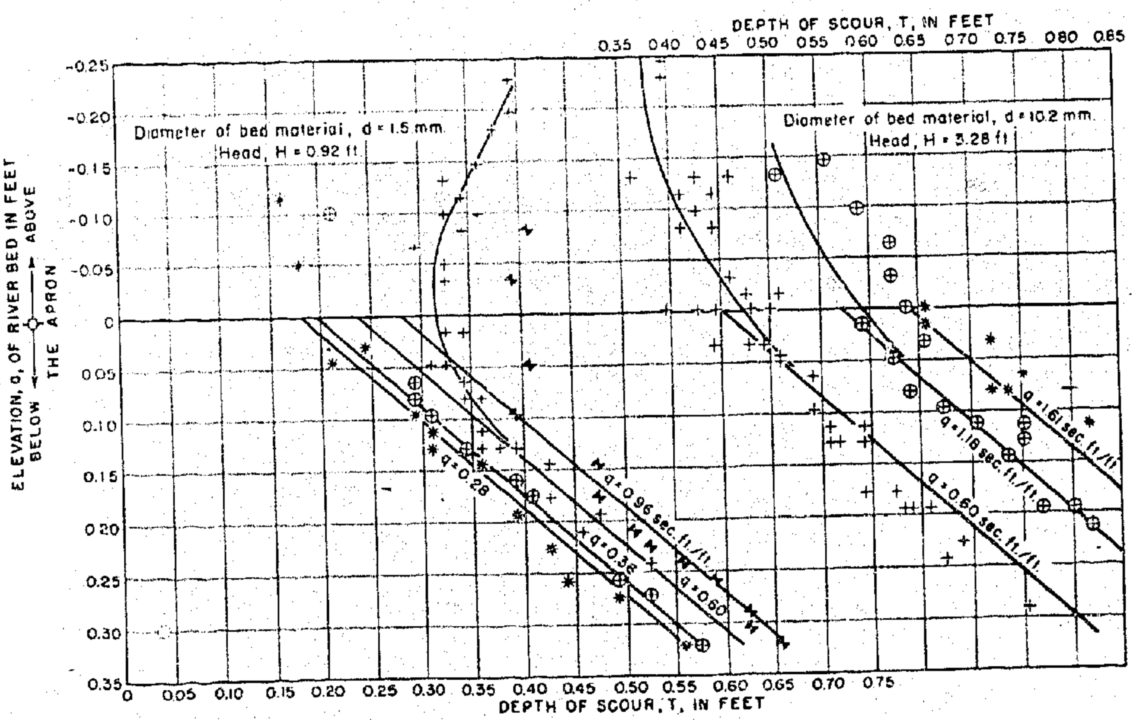



FIGURE 76 - MAXIMUM DEPTH OF SCOUR AT A SLUICEWAY WITH HORIZONTAL APRON WITHOUT A SILL FOR VARIOUS DISCHARGES, q, AND TWO DIAMETERS OF BED MATERIAL, d.

HYD-20



X-D-2015

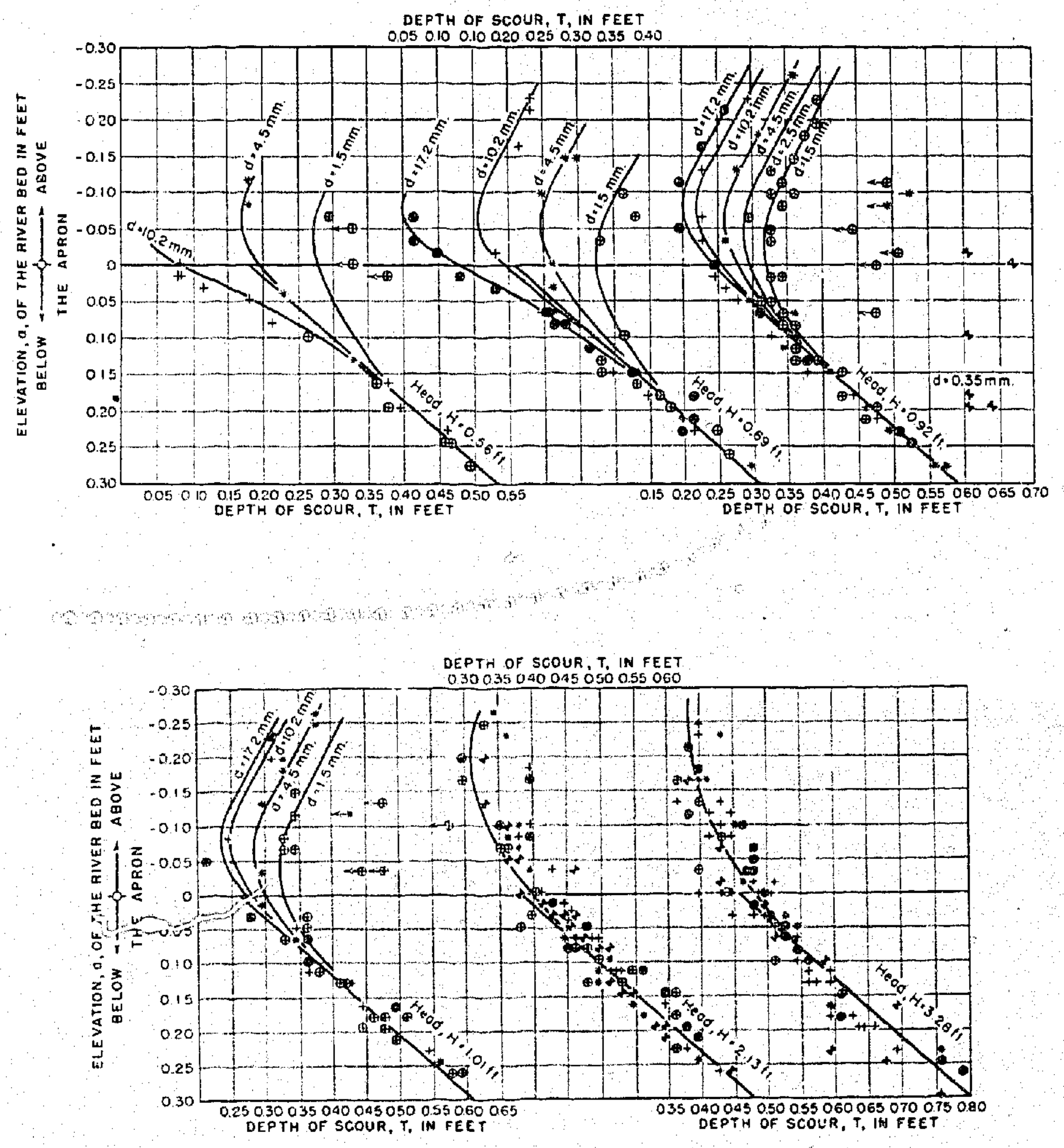



FIGURE 75 – SCOUR AT A SLUICEWAY WITH HORIZONTAL APRON.
EFFECTIVE LENGTH OF APRON, L = 1.5H;
DISCHARGE, q = 0.60 SEC. FT./FT.

In view of the fact that this may also occur for the composite discharge of sluiceway and weir, the effective apron length, L, should not be chosen less than 1.5 times the head, H. The jet flows over a horizontal apron and into the tail-water, almost without exception, as a shooting jet and is not covered by a surface roller. As a result, there is no significant dissipation of energy on the apron. A lengthening of the apron above 1.5 times the head, H, has no great effect on the depth of scour. Only if a properly designed sill at the end of the apron causes the formation of a surface roller on the apron itself and holds it there, is the lengthening of the apron to more than 1.5H effective.

It is seen that the scour produced by a free-falling nappe from the weir with a short apron is the most unfavorable arrangement. The longer the apron and the lower the river bed below the elevation of the apron, the less the directional effect of the jet on the scour. Releasing the water from under a sluice gate is to be preferred as far as deposition of bed material at the end of the apron is concerned.

The conclusions that an effective length of apron, L, equal to 1.5 times the head, H, is sufficient for deflecting the nappe of a weir and that a further lengthening of the apron will not reduce the depth of scour appreciably, were verified in all further experiments. Additional experiments were made with unigranular bed material, the average diameter, d, of which had the following values: 0.35, 1.5, 2.5, 4.5, 6.2, 10.2, and 17.2 millimeters. The head, H, was varied from 0.56 to 3.28 feet and the discharge, q, from 0.28 to 1.61 second-feet per foot. The elevation, a, of the river bed downstream from the apron was changed for each series of experiments, since the elevation of the river bed exerts a significant influence on the development of scour. Water was discharged from under a sluice gate in all further experiments and it carried no silt. The maximum depth of scour, T, was chosen as the measure for the scour because again all scour holes had a similar trough-shaped form. T is the maximum depth of scour below the elevation of the river bed downstream from the dam, although as before, T', measured below the elevation of the apron could have been used; the calculation of T' from T, or vice versa, is extremely simple (see figure 70).

The curves showing the relation between the depth of scour, T, and the elevation, a, of the river bed in figures 75 and 76 are all asymptotic to straight lines of the same slope. The slope of these lines is independent of the size of the bed material, d; the head, H; and the discharge, q; their intercepts with the x- axis vary with the head, H, and the discharge, q. The lower the river bed below the elevation of the apron, the less important the size of the bed material is, as far as the maximum depth of scour is concerned. As the head increases, the curves for the different sizes of bed material approach each other and at a head, H, equal to 2.13 feet, the maximum depth of scour appears to be independent of the size of the bed material (see figure 75).

The length of scour is somewhat greater for fine bed material than for coarse (figure 77).

The equations for the straight lines to which the curves in figures 75 and 76 are asymptotic are:

$$T = 0.298H^{0.5}q^{0.35} + 1.15a \qquad (63)$$

or

$$T' = 0.298H^{0.5}q^{0.35} + 2.15a \qquad (64)$$

for which

$$T' = T + a \qquad (65)$$

T is the maximum depth of scour in feet measured from the elevation of the river bed downstream from the apron; T' the maximum depth of scour in feet measured from the elevation of the horizontal apron; a, the elevation of the river bed in feet measured from the elevation of the apron; H, the head in feet; and q, the discharge in second-feet per foot. The straight lines drawn in the figures are computed from equation 63. Equations for the curves in these figures have not been derived. Since at most dams either the river bed downstream from the apron lies lower than the apron, or, in the course of time becomes so, the curves are relatively unimportant.

With the sluice gate raised completely above the water level, the unobstructed flow is accelerated on the smooth apron. The resulting scour downstream from the apron is shown in figure 78.

During the last ten years, measurements of scour have been made at a number of existing dams having horizontal aprons without sills. These measurements present a picture of the development and the form of the scour at such dams and reveal that even rock can be eroded to a great depth provided the discharge has been maintained over a long enough period. The ultimate scour has not been attained yet at most dams because they are yet too new, or, in other words, because the work of the discharge has not covered a sufficiently long period.

The Beznau Dam on the Aare River resting on a limestone foundation is among the oldest sluiceway dams. Here, in a region of soft stone, the remarkable depth of scour equal to 41.3 feet has occurred. A cross section of the dam and the scour measurements made in 1913 and 1920 by H. Gruner in Basle, and similar measurements made by E. Meyer-Peter[9] in 1926 (apparently at a different gate) are shown in figure 79.

The dam on the Mur River in Lebring, Austria, is another example. The scour existing in 1927 is shown in figure 80 and the scour in 1915, 1923, and 1927 at sluiceway I is shown in figure 81.

Figure 82 shows the scour at the Bavarian Aluminum Company's dam on the Inn River at Jettenbach. This dam is similar to the Lebring Dam.

High dams in the United States of America were often built with spillways and horizontal aprons for deflecting the water. The problem of dissipating the kinetic energy of the water often very large due to the great height of the structure - was not considered. Although a simple horizontal apron may prove satisfactory for low-headed dams, the high dams noted in the two following examples incurred a large amount of damage due to scour, even though they were built on rock foundations.

A cross section of the Julian Griggs Dam[10] on the Scioto River in Ohio is shown in figure 83. A great flood of 160 second-feet per foot passed over the dam in March 1913. The scour does not seem to have increased since 1920. Under-scouring of the dam has not yet developed.

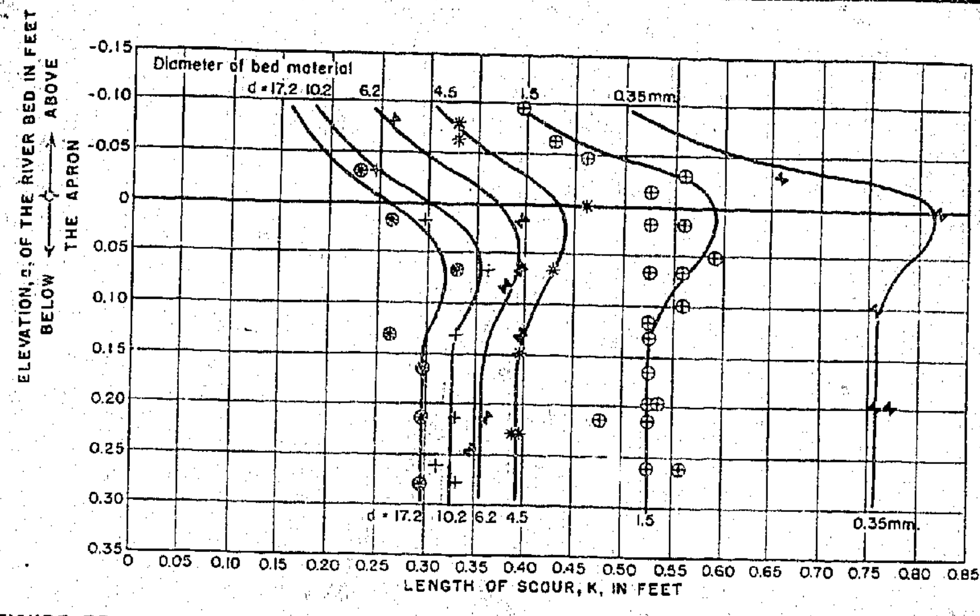



FIGURE 77 — LENGTH OF SCOUR, K, FOR VARIOUS SIZES OF BED MATERIAL AND HORIZONTAL APRON OF EFFECTIVE LENGTH, L = 1.5H. DISCHARGE, q = 0.60 SEC.FT./FT.; HEAD, H = 0.92 FT.

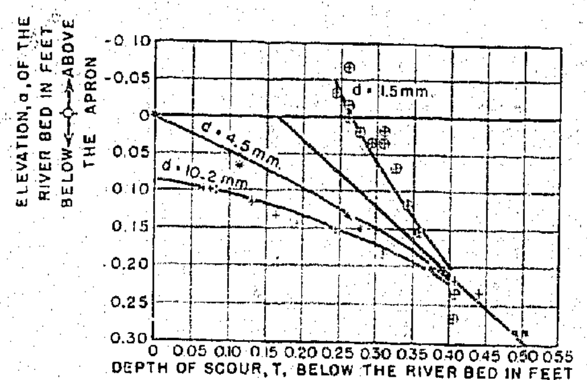



FIGURE 78 — MAXIMUM DEPTH OF SCOUR, T, FOR AN UNOBSTRUCTED DISCHARGE OVER A HORIZONTAL SLUICEWAY APRON AND FOR VARIOUS DIAMETERS OF BED MATERIAL, d. DISCHARGE, q = 0.60 SEC.FT./FT.

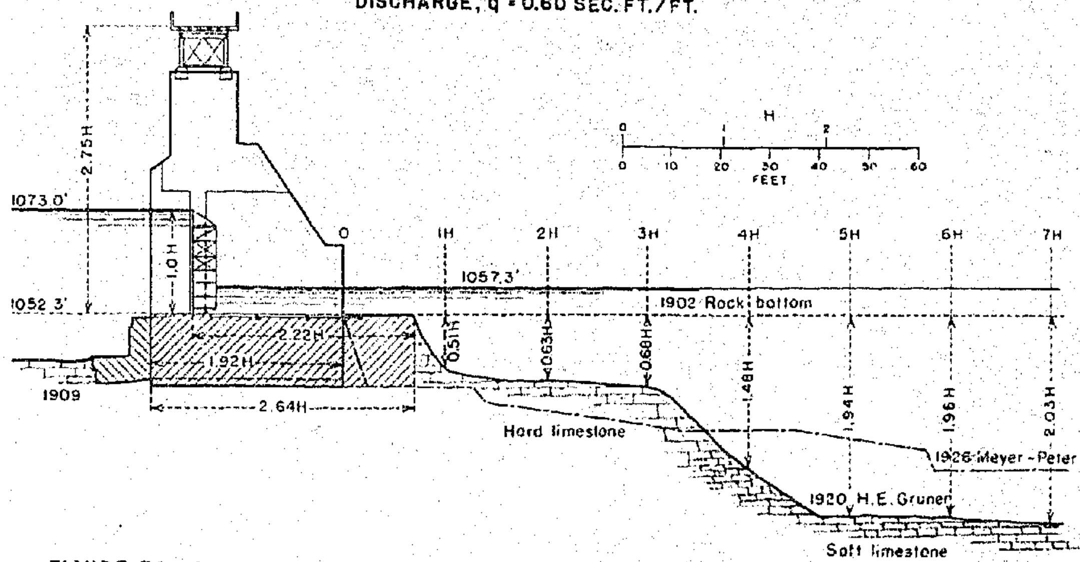



FIGURE 79 — SCOUR AT DAM AT BEZNAU. MAXIMUM DISCHARGE, q max = 189.4 SEC.FT./FT. DAM COMPLETED, 1902.

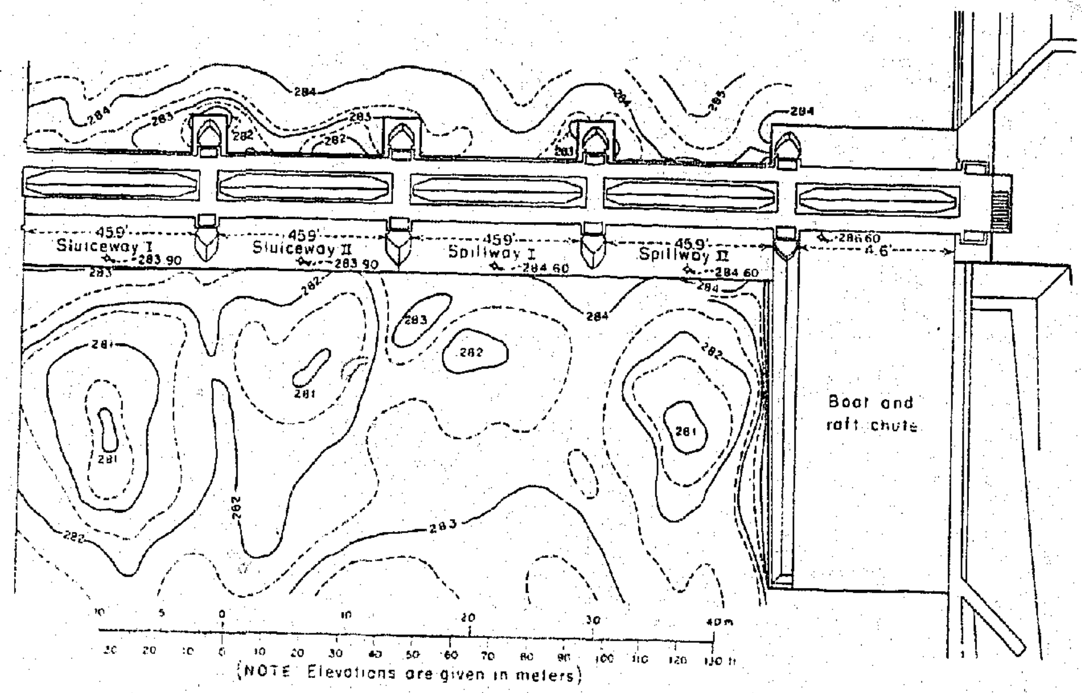



FIGURE 80 – SCOUR AT THE DAM ON THE MUR RIVER AT LEBRING, AUSTRIA, IN 1927.

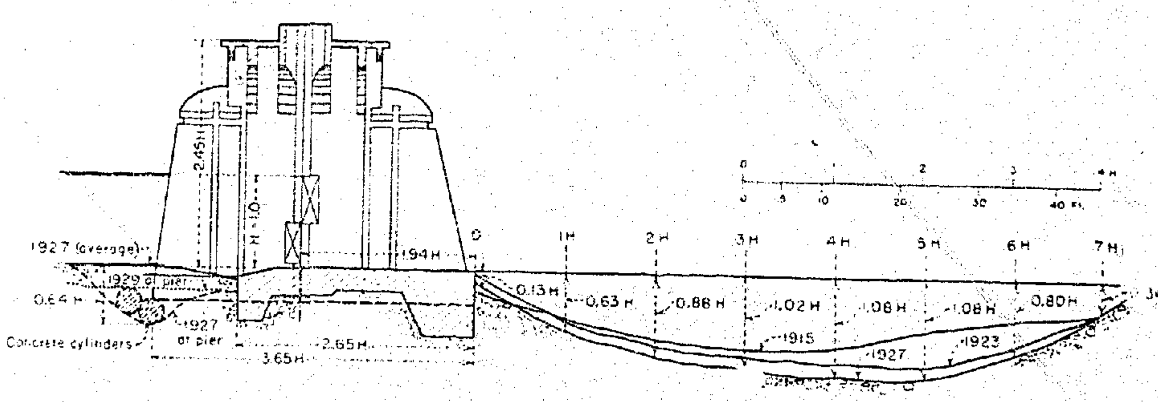



FIGURE 81 – SCOUR ABOVE AND BELOW THE DAM ON THE MUR RIVER
AT LEBRING, AUSTRIA. HEAD H = 10.7 FEET.

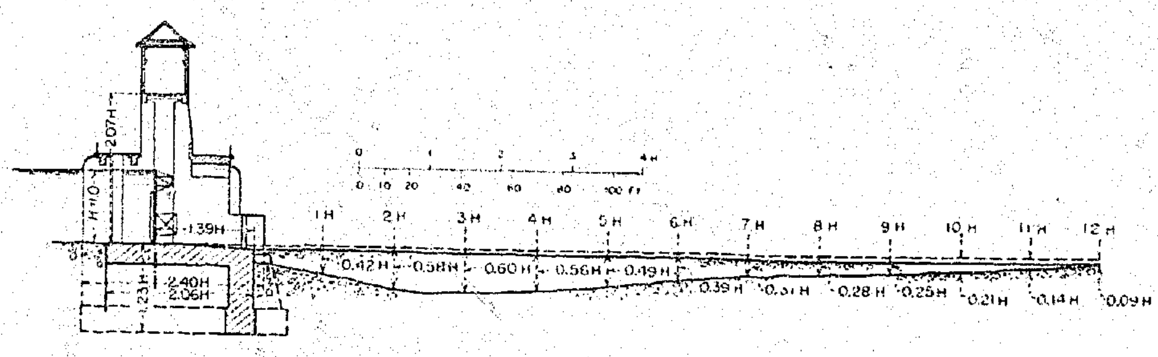



FIGURE 82 – SCOUR BELOW THE DAM ON THE INN RIVER AT JETTENBACH,
AUSTRIA, IN 1931. HEAD H = 27.9 FEET.

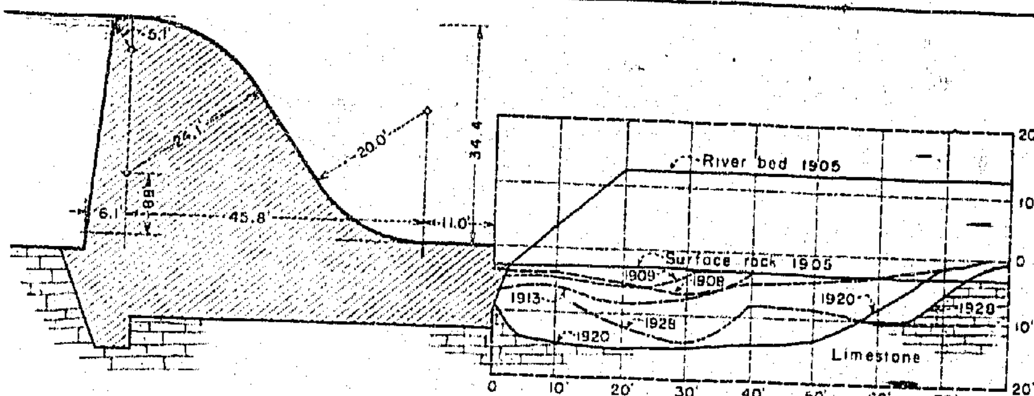



FIGURE 83 - SCOUR BELOW THE JULIAN GRIGGS DAM ON THE SCIOTO RIVER, OHIO. (MEASUREMENTS BY J. H. GREGORY).

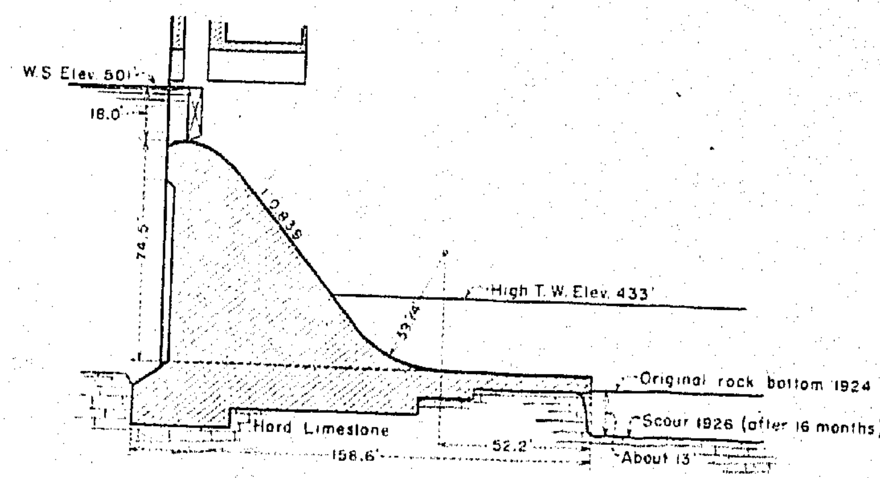



FIGURE 84 - CROSS SECTION OF THE WILSON DAM ON THE TENNESSEE RIVER SHOWING THE SCOUR IN THE LIMESTONE AFTER 16 MONTHS.

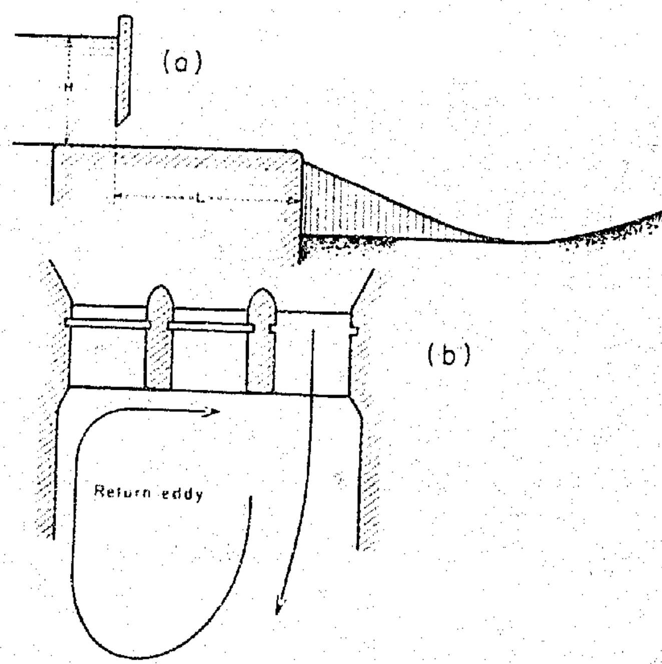



FIGURE 87 - RETURN EDDY PRODUCED BY UNSYMMETRICAL DISCHARGE.

The Wilson Dam[11] on the Tennessee River is likewise protected with a horizontal apron. During construction, the river was discharged over a completed portion of the dam and yet, before the completion of the entire structure, the remarkable erosion in the limestone shown in figure 84 occurred, extending in under the apron and necessitating extensive repairs.

## 4. SCOUR AT COMPLETE MODELS OF DAM

The experiments hitherto discussed were conducted in a channel with glass sides. The flow conditions were analogous to those at a dam where the discharge is uniformly distributed over the entire length of dam. Generally, the flow is not so distributed but may be discharged through a single gate whose width is only a fraction of the entire river width. In this case side rollers form at the sides of the discharging jet which transports bed material upstream. The scour acquires different forms, depending on the position of the gate relative to the entire length of the dam, and may deviate greatly from the scour formed in the glass channel. As a rule, the maximum depth of scour is less than that occurring for uniformly distributed flow, because the discharge from a single opening can spread out over the entire width of the stream in a relatively short distance.

The scour in all of the experiments conducted in the glass channel with different aprons formed a trough downstream from and in the immediate vicinity of the apron. On the other hand, in experiments on models of complete dams, the maximum depth of scour occurred at the end of the apron when a single gate was open. This change in the development of scour was caused by the side rollers on either side of the discharge from the gate. These rollers were maintained by energy withdrawn from the discharge jet and the high velocity at the periphery of the side roller along the shore and along the end of the apron scoured out a channel there. This scour threatened the shore line and exposed the end of the apron by removing that portion of the river bed cross-hatched in figure 87. River banks thus eroded may cave in, and, if so these side rollers will finally produce a fan-shaped widening of the river. An example of this is shown in figure 93 (according to J. K. Lehr[12]).

This undesirable side roller action can be diminished, as Theodor Rehbock[13] has remarked, provided the discharge can be distributed equally over the entire river width, and the great inequalities in the flow near the sluice openings can be avoided.

When the discharge emerges from a single gate, the velocity is distributed unequally over the river bed and, in addition to reshaping of the bed, a sorting of the bed material takes place. In general, only the largest grains remain at the place of maximum scour where the velocity is also a maximum. In figures 94 and 95 the sorting of bed material by the scour in two model experiments is represented by means of the contours of equal "effective" grain size, $K_{10}$.

## 5. SCOUR UPSTREAM FROM DAMS

It is generally known that the erosion at the nose of a bridge pier often extends to a considerable depth below the river bed. However, that a similar scour can occur on the upstream side of a movable dam is not widely recognized, although the scour may be extensive enough to endanger the dam.

Occasionally during experiments on models of various dams, it is possible to investigate the conditions favorable to the production of scour at the upstream side of dams. This scour occurs at the nose of the piers and also in front of the apron between the piers. The scour at the nose of the pier increases the farther the piers project upstream from the apron. If the discharge is equally distributed over the entire length of the dam, this scour is less than when water is released from one of the several gates. The flow conditions along the sides of the piers at the opening are such that an unsymmetrical scour occurs at the pier nose, and if the dam is located at a curve in a stream, the main channel in front of the dam is oblique to the direction of the river and, hence, the unsymmetrical scour is intensified. The form and dimensions of this scour are wholly different from that produced downstream from the dam.

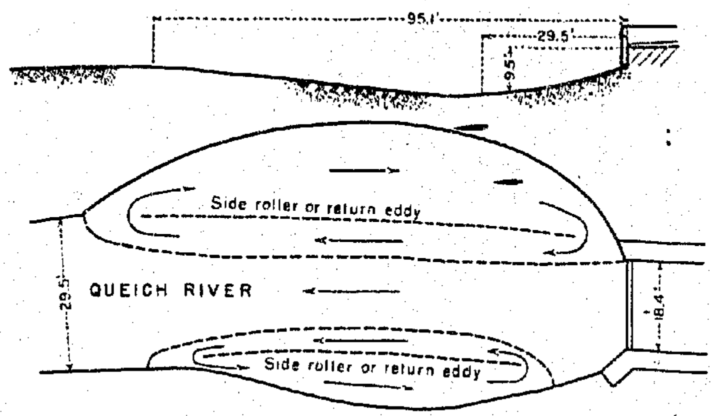



FIGURE 93 - SCOUR AT THE DAM ON THE QUEICH RIVER AT BANNE HOCHSTADT (ACCORDING TO J.K. LEHR).

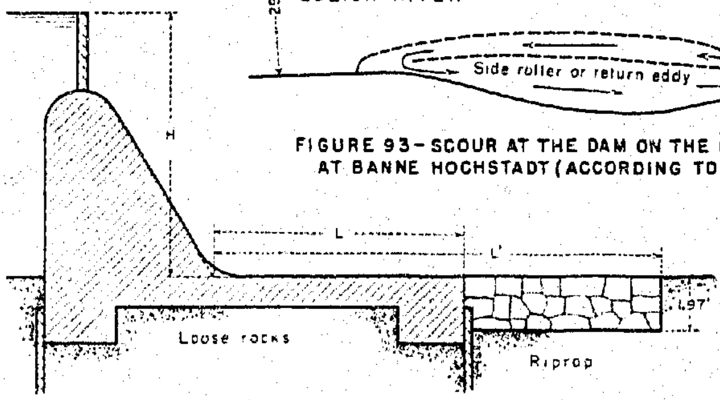



FIGURE 97 - PREVENTION OF SCOUR BY MEANS OF A LONG HORIZONTAL APRON (ACCORDING TO W. G. BLIGH).

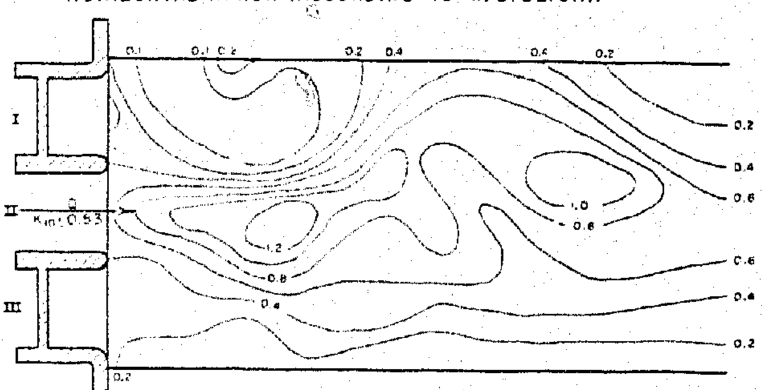



FIGURE 94 - SORTING OF BED MATERIAL DURING SCOUR. GATE II OPEN. THE "CONTOURS" ARE LINES OF EQUAL GRADATION OF MATERIAL. FIGURES INDICATE "EFFECTIVE" GRAIN SIZE, $K_{10}$, IN MILLIMETERS.

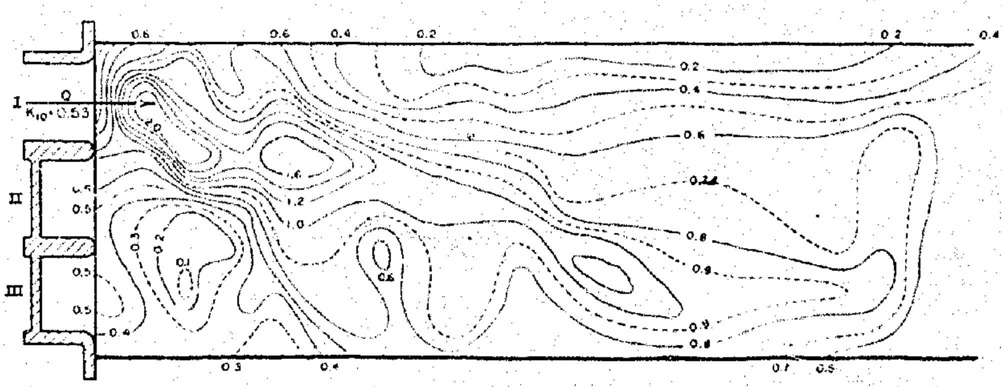



FIGURE 95 - SORTING OF BED MATERIAL DURING SCOUR. GATE I OPEN. THE "CONTOURS" ARE LINES OF EQUAL GRADATION OF MATERIAL. FIGURES INDICATE "EFFECTIVE" GRAIN SIZE, $K_{10}$, IN MILLIMETERS.

Although the scour at the upstream side never attains the depth incurred downstream from the dam, it must receive attention, and, in particular, if seepage under the dam is obstructed by a wall which, for special reasons, may have been placed at the downstream end of the apron rather than on the upstream side of the dam.

Scour on the upstream side of existing dams has been repeatedly observed. Such erosion is shown in figures 80 and 81 at the dam on the Mur River at Lebring. The scour at the pier in 1927 had already attained dangerous dimensions, extending in under the foundation (see figure 81). The deepest holes were filled with concrete cylinders, and two years later a deposition of silt was observed which eliminated the danger to the piers.

c. Sturctures for Preventing Scour

Various methods have been used or proposed for reducing the scour below dams. These are discussed in the following section. The effectiveness of these arrangements was investigated by means of model tests. So as to be comparable, the tests, unless otherwise noted, were conducted with bed material of the same average diameter, d, equal to 2.5 millimeters, the head, H, was maintained constant throughout at 0.92 feet, and the discharge, q, at 0.60 second-feet per foot. Unless otherwise noted, the length of the apron, L, was equal to 1.5H.

1. SCOUR WITH A VERY LONG APRON

An old but very costly method which has been used frequently for reducing the scour consists in simply extending the length of the apron. This serves to push the scour a distance downstream. Short aprons, later lengthened, were originally used at many dams. Whenever dangerous scour holes occurred they were artificially filled up and the apron lengthened.

The sole aim underlying the design of exceptionally long aprons was to cause a hydraulic jump to form on the apron, thus dissipating the energy of the stream there. The position of the hydraulic jump depends upon the roughness of the apron and cannot be predicted.

According to W. G. Bligh[14] many dams with long horizontal aprons without sills have been constructed in the United States, England, and its colonies. For computing the effective length, L, in feet, of an apron necessary for removing the scour a safe distance downstream, Bligh recommended the following formula:

$$L = 4c\sqrt{\frac{H}{13}} = 1.11\,c\sqrt{H} \qquad (66)$$

in which H (see figure 97) is the difference in feet between the elevations of the headwater and the horizontal apron; and c is the so-called Bligh coefficient which has the following values:

| | |
|---|---|
| Fine silt and sand as in the Nile River | 18 |
| Fine micaceous sand as in the Colorado and Himalayan rivers | 15 |
| Ordinary coarse sand | 12 |
| Gravel and sand | 9 |
| Boulders, gravel, and sand | 4 to 6 |

In order to diminish the depth of scour still further, according to Bligh, riprap not less than 1.83 feet thick should be added to the solid horizontal apron, and the total length computed from the following formula:

$$L' = 10c\sqrt{\frac{H}{10}} \times \sqrt{\frac{q}{75}} = 0.368\sqrt{Hq} \qquad (67)$$

in which q is the maximum unit discharge in second-feet per foot.

F. Kurzmann[15] designed a very long apron for the dam at Oberföhring on the middle Isar River. The pool shown in figure 98 was introduced in order to compel the hydraulic jump to form on the apron. The total length of the solid apron was 187 feet and the effective length 144 feet. This is somewhat longer than the length computed from Bligh's formula.

## 2. PREVENTION OF SCOUR BY MEANS OF A RISING APRON

The prevention of scour at dams by means of rising aprons has been attempted in numerous instances during the last two decades.

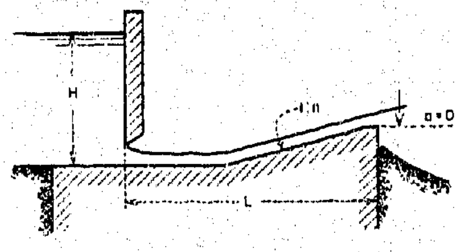



FIGURE 99 - MODEL OF A DAM WITH RISING APRON

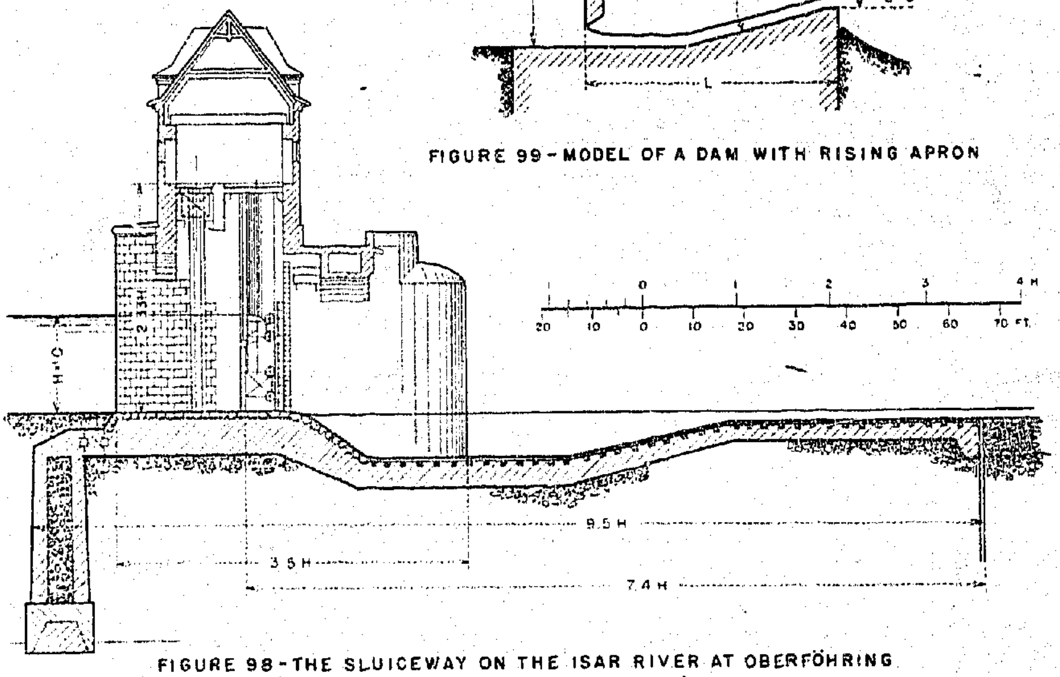



FIGURE 98 - THE SLUICEWAY ON THE ISAR RIVER AT OBERFÖHRING (ACCORDING TO S. KURZMANN).

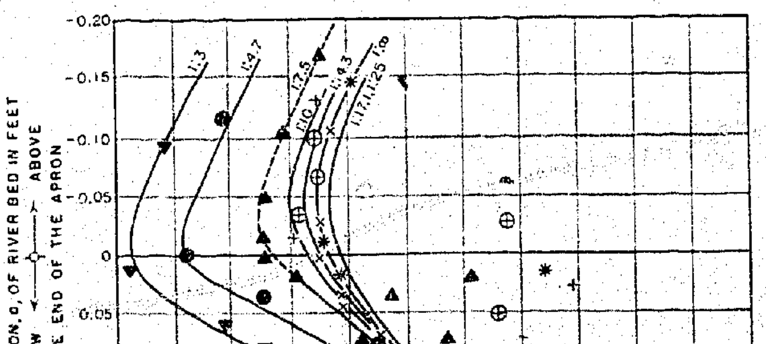



FIGURE 100 - MAXIMUM DEPTH OF SCOUR, T, AT A SLUICEWAY WITH AN APRON HAVING A RISING SLOPE. DIAMETER OF BED MATERIAL, d = 2.5 mm; HEAD, H = 0.92 FT; DISCHARGE, q = 0.60 SEC.FT/FT.

HYD-20



X-D-2021

Measurements of the erosion at such structures indicate no improvement over the use of horizontal aprons without sills. In order to verify this, models with rising aprons were next tested in the laboratory. These experiments were conducted in a glass channel on aprons with the following slopes: 1:00, 1:25, 1:17, 1:14.3, 1: 10, 1:7.5, 1:4.7, and 1:3. A cross section of the model is shown in figure 99. The apron began to slope at some distance from the gate, because this appeared more effective in preventing scour. Sand with an average diameter, d, equal to 2.5 millimeters was used for the bed material; the head, H, was held throughout these tests at 0.92 feet; and the discharge, q, at 0.60 second-feet per foot. The elevation, a, of the river bed was measured from the end of the apron.

Figure 100 shows that for the slopes tested the maximum depth of scour is not noticeably reduced below that for a horizontal apron. However, the diving jet appears less frequently and then at only low elevations of the river bed. Furthermore, a negative wave jet does not precede it. For many elevations of the river bed below the end of a rising apron, a true positive wave jet appears which produces a large scour but at a considerable distance downstream from the apron. The true positive wave jet appears initially at lower elevations of the river bed in the case of a rising apron than for a horizontal apron.

The dam at Pernegg on the Mur River serves as an example of the scour produced with a rising apron (figures 101 and 102).

### 3. PREVENTION OF SCOUR BY MEANS OF APRON SILLS

Prevention of scour can be effected by means of sills, either solid or dentated, placed at the end of the apron, or by double sills, one of which is placed at the end of the apron, the other between the end of the apron and the dam; furthermore, either solid or dentated sills may be used for double sills.

#### a. Prevention of Scour by Means of Solid Sills

The rectangular, trapezoidal, and triangular sills shown in figure 103 are examples of solid sills. Such sills deflect the

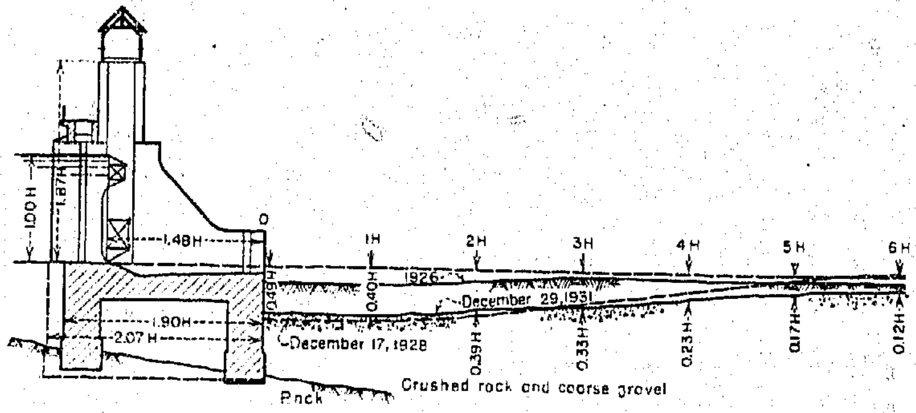



FIGURE 101 - SCOUR AT THE SLUICEWAY ON THE MUR RIVER AT PERNEGG, AUSTRIA. HEAD, H = 38.0 FT.

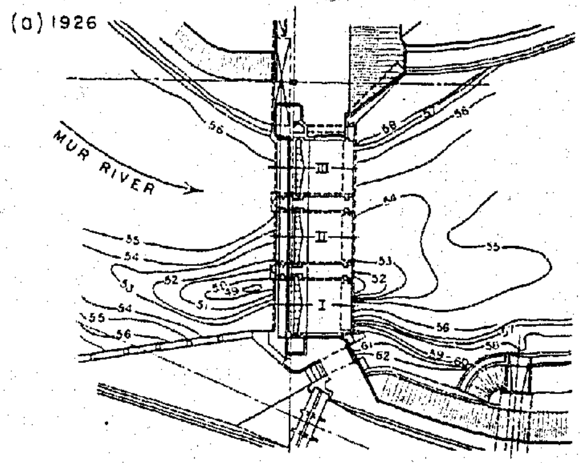
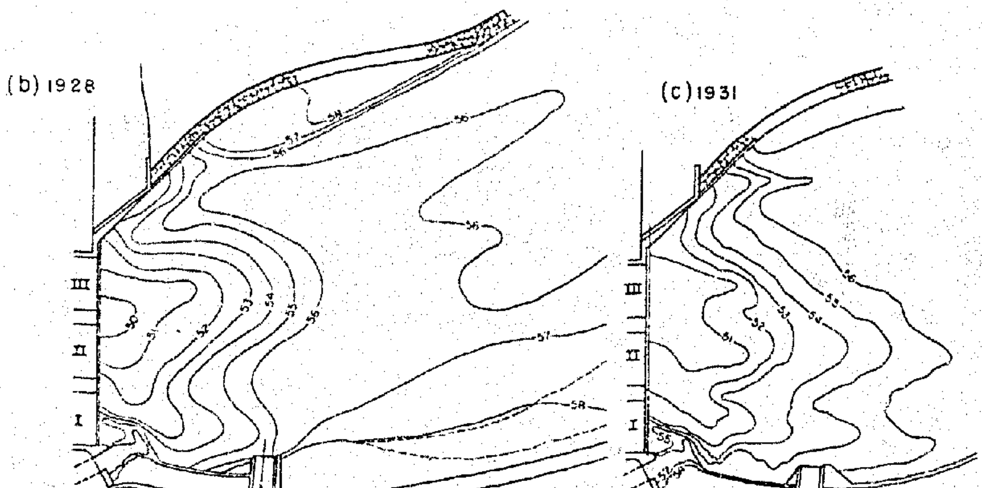



FIGURE 102 – DEVELOPMENT OF SCOUR AT THE SLUICEWAY ON THE MUR RIVER AT PERNEGG, AUSTRIA. (CONTOURS ARE AT ONE METER INTERVALS).



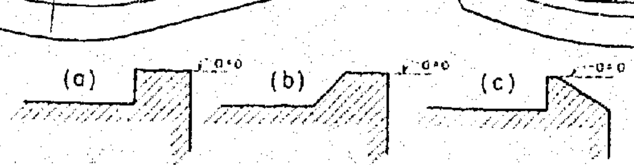




FIGURE 103 – SILLS: (a) RECTANGULAR; (b) TRAPEZOIDAL; (c) TRIANGULAR.

flow at the end of the apron upward and, more or less, stabilize the type of flow. The frequent shifting from one type of flow to another, experienced with a horizontal apron at many different tail-water elevations, can be reduced by the use of sills.

The effectiveness of each of these sills was investigated in the glass channel. The head, H, was held constant at 0.92 feet; the discharge, q, at 0.60 second-feet per foot; and the average diameter of the bed material, d, was 2.5 millimeters as before. The elevation of the river bed downstream from the dam was measured from the top of the sill.

The results of the tests on rectangular sills are given in figure 104 along with the comparable results for a horizontal apron without a sill. For elevations of the river bed higher than the top of the sill, the depth of scour was somewhat reduced below that obtained with a horizontal apron without a sill. For rectangular sills 32 and 40 millimeters (1.26 and 1.57 inches, respectively) high, the depth of scour remained smaller than for a horizontal apron without a sill, even when the elevation of the river bed was below the top of the sills, because small surface rollers were retained on the apron which persisted, and which were independent of the elevation of the bed. These surface rollers remain in a fixed position even if for low elevations of the river bed a vacuum forms under the jet as it leaves the apron. The experiments showed that transportation of detritus across the apron is not obstructed by rectangular sills.

The trapezoidal sill is more satisfactory than the rectangular sill because it is less subject to wear by material transported over the dam. The results of the experiments on trapezoidal sills, carried out under the same conditions as for the rectangular sills, are given in figure 106, and again along with the results for a horizontal apron without a sill. When the elevation of the river bed is above the top of the sill and also for small elevations below, the depth of scour is again less than that obtained for a horizontal apron without a sill, the amount of the reduction being about the same as for a rectangular sill. When the bed lies at a still lower elevation, a wave jet appears which shoots over the trapezoidal sill in a wide arc and the small surface roller vanishes. The jet then impinges against the bed, becomes a diving jet, and scours out a deep hole. The diving jet then continues. A negative wave jet does not develop either for a rectangular sill or a trapezoidal sill.

Figure 109 shows a dam[16] having a trapezoidal sill on the Neckar River and the scour after two years.

The experiments on triangular sills are not as complete as planned and yet the results shown in figure 110 indicate that they are more satisfactory than either rectangular sills or trapezoidal sills, because the flow is essentially more quiet. The experimental results also show that the height of a rectangular sill for all practical purposes is a relatively unimportant factor.

## b. Prevention of Scour by Dentated Sills

Theodor Rehbock,[17] introduced an important improvement for the prevention of scour in his well-known dentated sill which is also placed at the end of the apron. As figure 118 shows, a Rehbock dentated sill consists of teeth superimposed on a solid sloping sill. These sills ensure the formation of a surface roller on the solid portion of the apron, and, in addition to this they produce a high difference of velocity between the upper and lower portions of the jet as it leaves the apron, thus giving rise to an extensive dissipation of energy.

Figure 111 shows the velocity distribution when a Rehbock dentated sill is used. A ground roller always occurs underneath the jet downstream and adjacent to the dentated sill and transports a considerable amount of bed material upstream against the sill. The scour slopes gently away downstream from the dentated sill.

The results of the tests on Rehbock dentated sills of different heights are given in figure 112. The best results were produced by a sill 36 millimeters (1.42 inches) high. In figure 113 the depths of scour for a 36 millimeter dentated sill, a triangular sill of equal height, and a horizontal apron without a sill are shown together. This comparison clearly shows the effectiveness of the teeth and, above all, the excellent performance of a dentated sill.

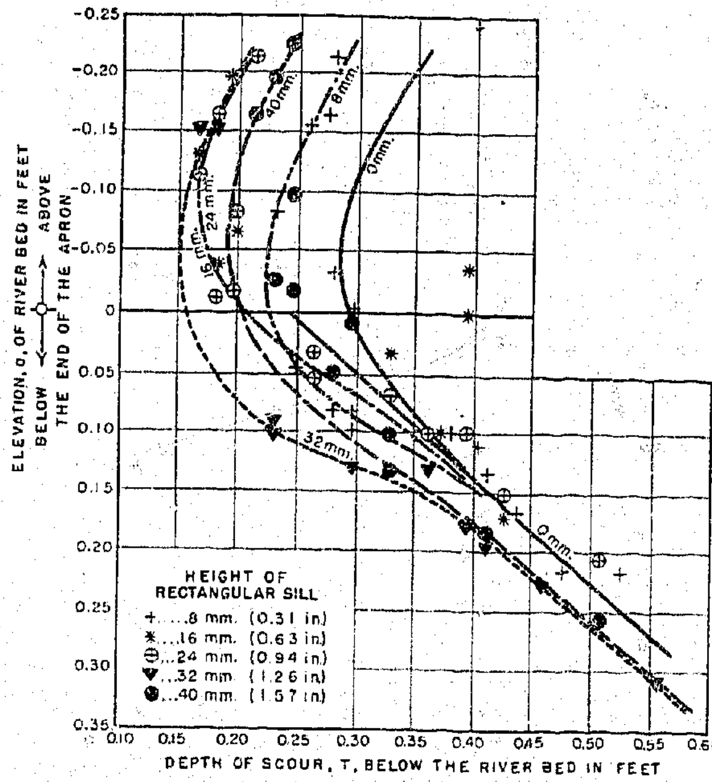



FIGURE 104 - MAXIMUM DEPTH OF SCOUR, T, FOR RECTANGULAR SILLS OF VARIOUS HEIGHTS. DIAMETER OF BED MATERIAL, d = 2.5 mm.; HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.

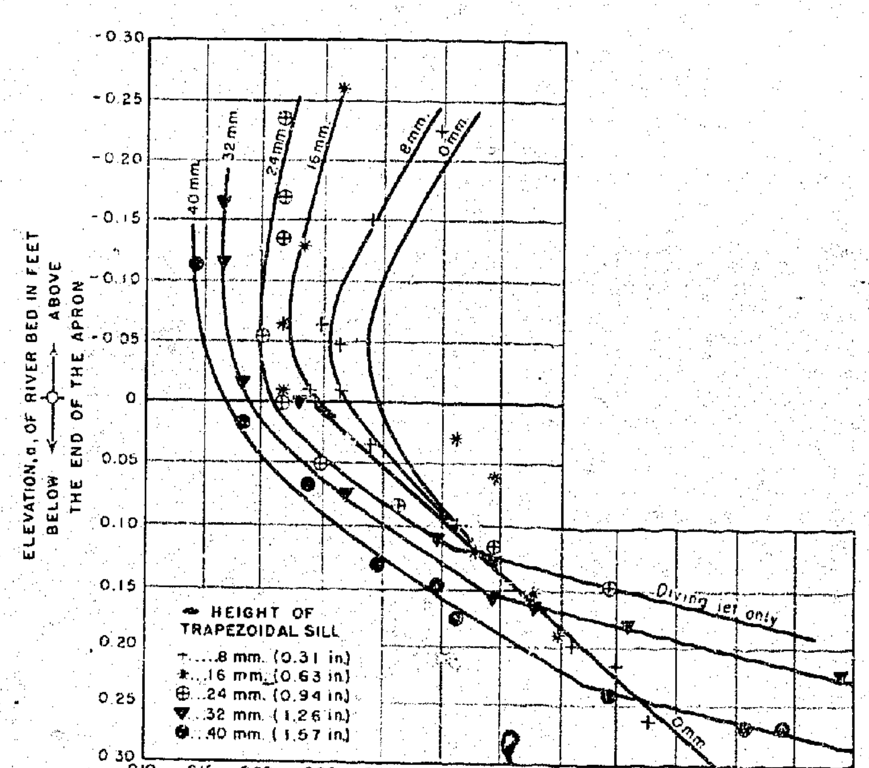



FIGURE 106 - MAXIMUM DEPTH OF SCOUR, T, FOR TRAPEZOIDAL SILLS OF VARIOUS HEIGHTS. DIAMETER OF BED MATERIAL, d = 2.5 mm.; HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.

HYD-20



X-D-2023

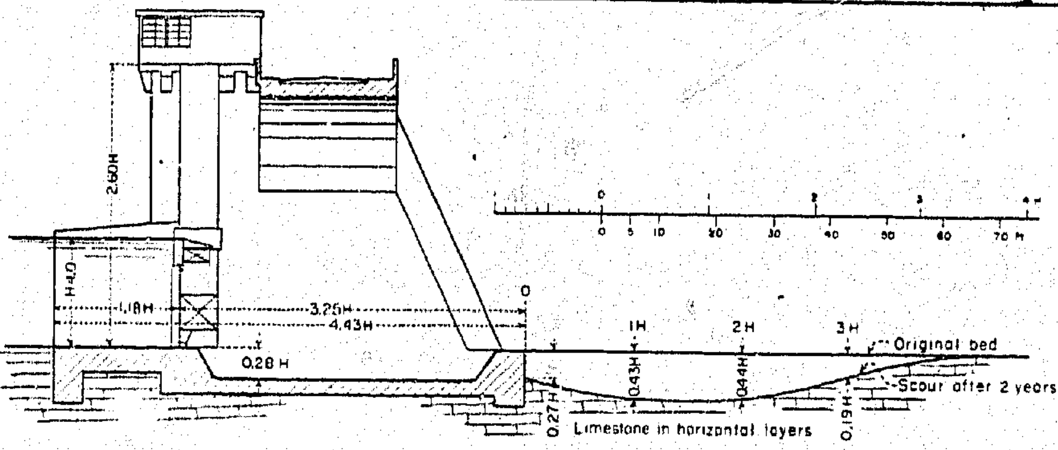



FIGURE 109 - SCOUR AT A DAM ON THE NECKAR RIVER (ACCORDING TO A. SCHÄFER).

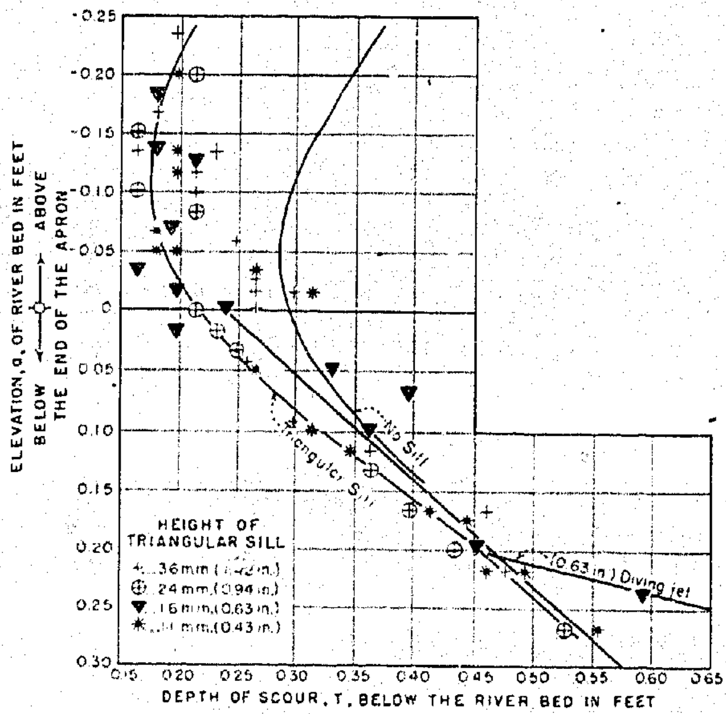



FIGURE 110 - MAXIMUM DEPTH OF SCOUR, T, FOR TRIANGULAR SILLS OF DIFFERENT HEIGHTS. DIAMETER OF BED MATERIAL, d = 2.5 mm.; HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.

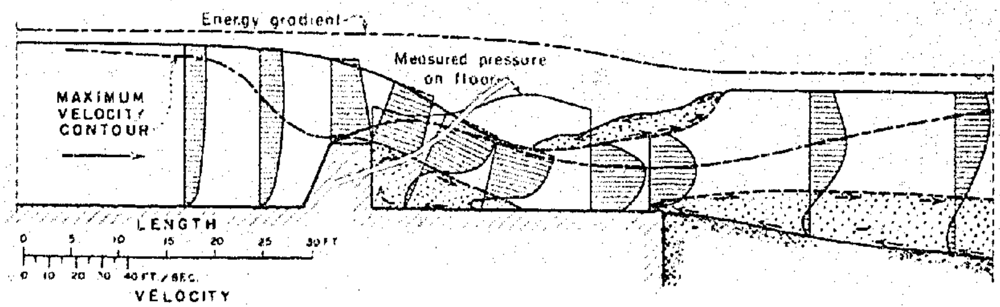



FIGURE 111 - EFFECT OF A REHBOCK DENTATED SILL ON THE VELOCITY DISTRIBUTION. SALIN WEIR IN LIZIN, BURMA (ACCORDING TO MODEL STUDIES BY TH. REHBOCK).

The surface roller vanishes if the bed lies deep below a Rehbock sill. Figure 115b shows the flow when the tail-water elevation is at about the same height as the teeth, and figure 115c the flow at still lower tail-water level. The depth of scour is not always smaller than with other types of sills, for at low tail-water levels the main jet is divided by the teeth into separate jets of different trajectories, between which there are large differences of velocity and much air is sucked in.

When a Rehbock dentated sill is used, for the elevations of the river relative to the apron usually found in practice, a negative wave jet does not appear at all, and a diving jet will not occur if the proper height for the teeth is chosen.

Finally, figure 125 shows the effect of lengthening the apron when using a Rehbock sill. This sill has found a wide use in the last ten years and has proved to be, in general, satisfactory. Figure 116 shows the Ryburg-Schwörstadt Dam on the Rhine River where a granite dentated sill is used.

According to the severity of the attack by the material transported over or through a dam, the Rehbock dentated sill is constructed from concrete, concrete with iron-protected corners, granite, cast iron, or cast steel, as shown in figure 118. The teeth are from 0.5 to 3.3 feet high. The width, b, at the foot of the solid section of the sill is as follows: for a light or average attack by detritus, b = 2.5h; for a severe attack, b = 2.75h; and for the most severe attack, b = 2.0h. The dimensions Z and t, of the top of the teeth are: for low teeth, 0.35h and 1.5h, respectively; and for high teeth, 0.10h and 1.1h, respectively. All corners on concrete and granite dentated sills are rounded to a radius of 0.9 to 1.6 inches.

The use of the Rehbock dentated sills for double sills and for stilling pools will be discussed later.

After the introduction of Rehbock's sill various engineers suggested other types of dentated sills, more or less similar to Rehbock's. Three of these are shown in figure 119: namely, those

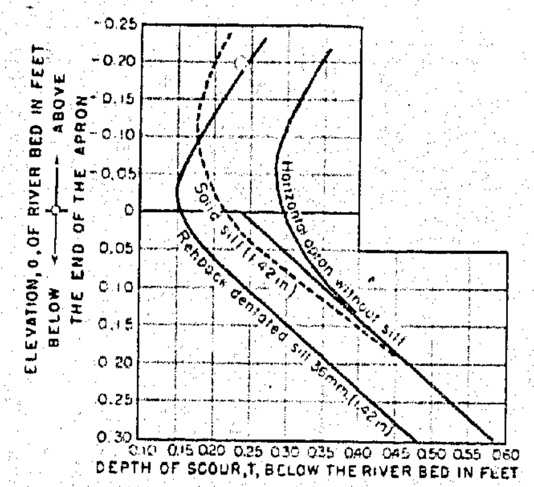



FIGURE 113—INFLUENCE OF THE TEETH OF THE REHBOCK DENTATED SILL ON THE DEPTH OF SCOUR. HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC. FT./FT.; DIAMETER OF BED MATERIAL, d = 2.5 mm.

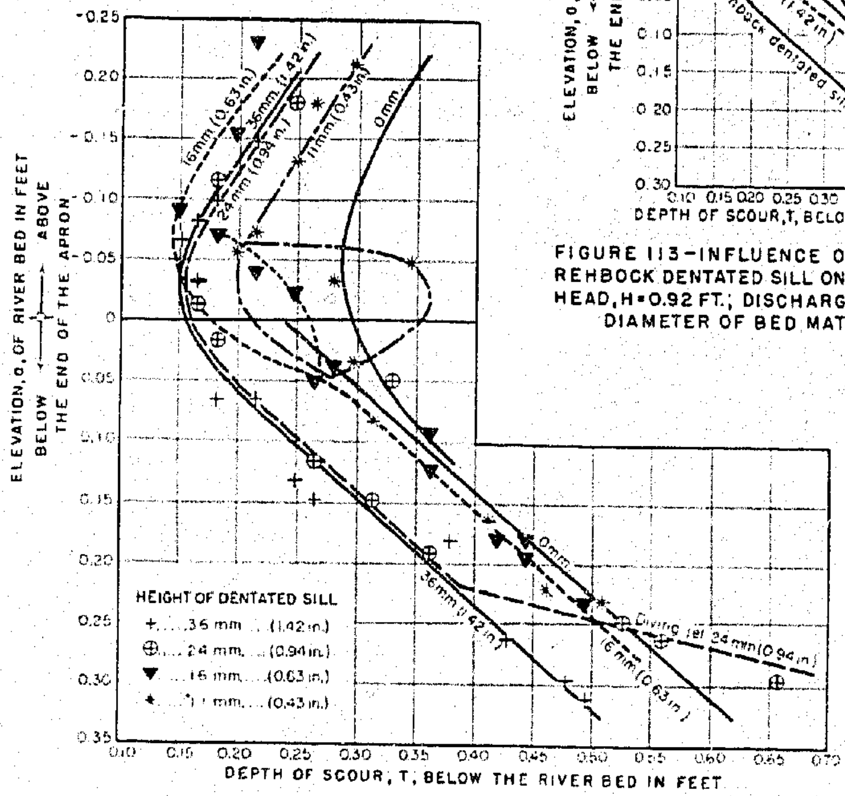



FIGURE 112 - MAXIMUM DEPTH OF SCOUR, T, FOR REHBOCK DENTATED SILLS OF DIFFERENT HEIGHTS. DIAMETER OF BED MATERIAL, d = 2.5 mm.; HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.

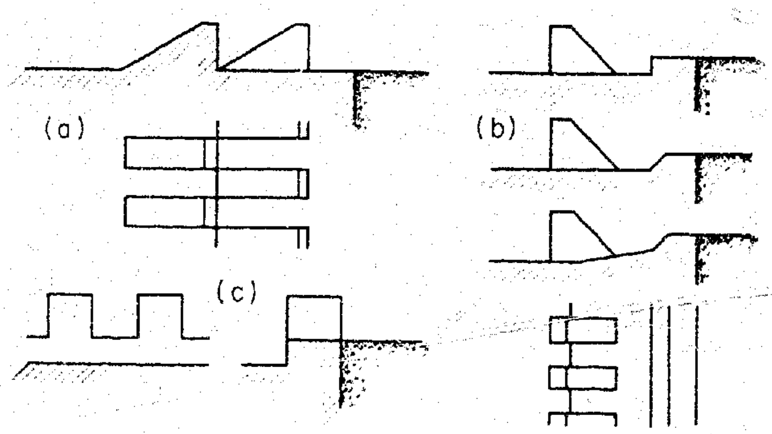



FIGURE 119 - OTHER TYPES OF DENTATED SILLS: (a) SMRCEK SILL; (b) THÜRNAU SILL; (c) RECTANGULAR DENTATED SILL.

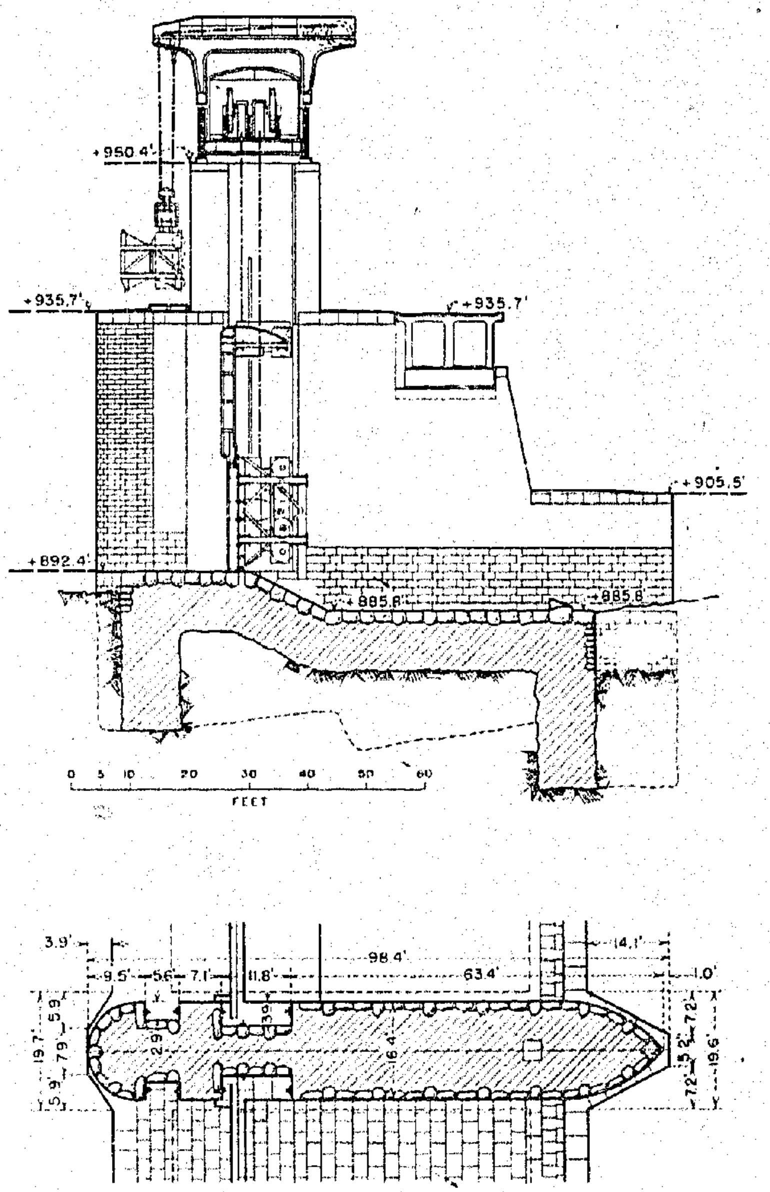



FIGURE 116 – CROSS SECTION OF THE RYBURG–SCHWÖRSTADT DAM ON THE RHINE RIVER (FROM N. KELEN "GEWICHTSSTAUMAUERN" J. SPRINGER, BERLIN, 1933).

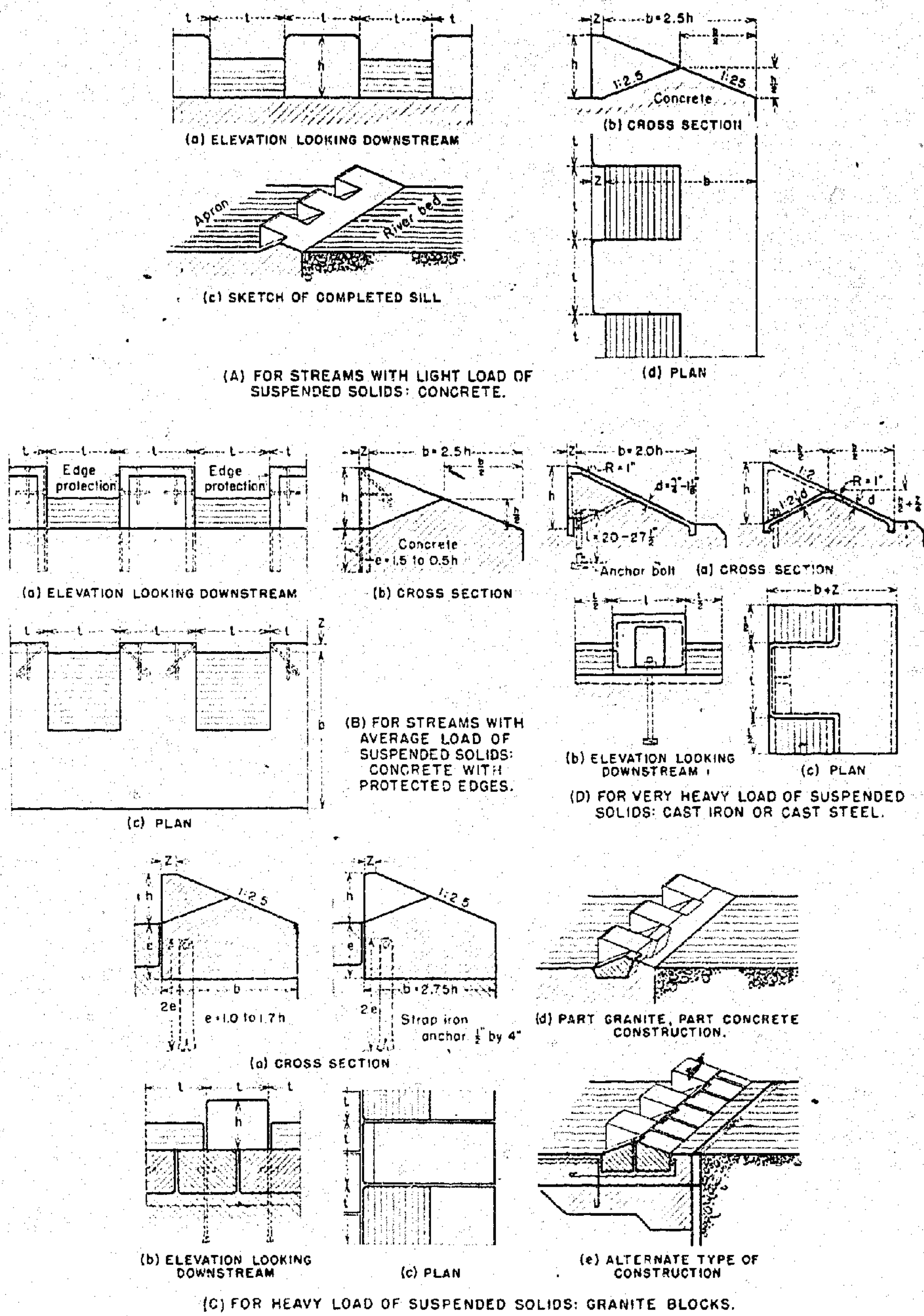



FIGURE 118 – CONSTRUCTION METHODS FOR THE REHBOCK SILL FOR DIFFERENT LOADS OF SUSPENDED SOLIDS.

HYD-20



X-D-2027

of C. Thurnau[18], A. Smrcek, and a rectangular dentated sill. Figure 120 shows the depth of scour for a rectangular dentated sill.

The unique dentated sill made by setting irregular stones in concrete is used on the Zwitta River near Hadlas. No rollers are formed, and the water, for the most part, is sprayed into the air, as when baffle piers are used.

## 2. Prevention of Scour by Double Sills

When a sill is placed at the end of the apron a positive wave jet is formed for a given elevation of the river bed. The maximum depth of scour occurs at a greater distance downstream from the dam and although the dam itself is not, as a rule, endangered, the river banks may be. The large depth of scour produced by the positive wave jet can be controlled by a double sill, provided the sill is so arranged that the first crest of the positive wave jet is compelled to lie entirely on the apron. The first sill must lie far enough downstream from the foot of the dam so that all of the flow over it is deflected to the apron, and the second sill at the end of the apron must lie far enough downstream from the first so that the wave jet does not overleap it. The first sill must lie at a minimum distance equal to the head, H, downstream from the dam and the second should be placed at least 1.5H farther downstream. Thus when a double sill is used, the minimum effective length, L, of the apron is equal to approximately 2.5H.

The double sill shown in figure 122 employs solid sills. It was first proposed by A. Ludin,[19] and has been used widely. Its performance is shown in figure 123, and, for the purpose of comparison, the depths of scour for a horizontal apron without sills of an effective length, L, equal to 1.5H, and for a single rectangular sill 32 millimeters (1.26 inches) high are also shown. The distance between the two sills in model (a) of figure 123 was too small and on that account the wave jet was deflected upward by the first sill and overleaped the end sill. Further experiments indicated that it is better to place the fore sill with its vertical face against the flow, because in the reverse position the

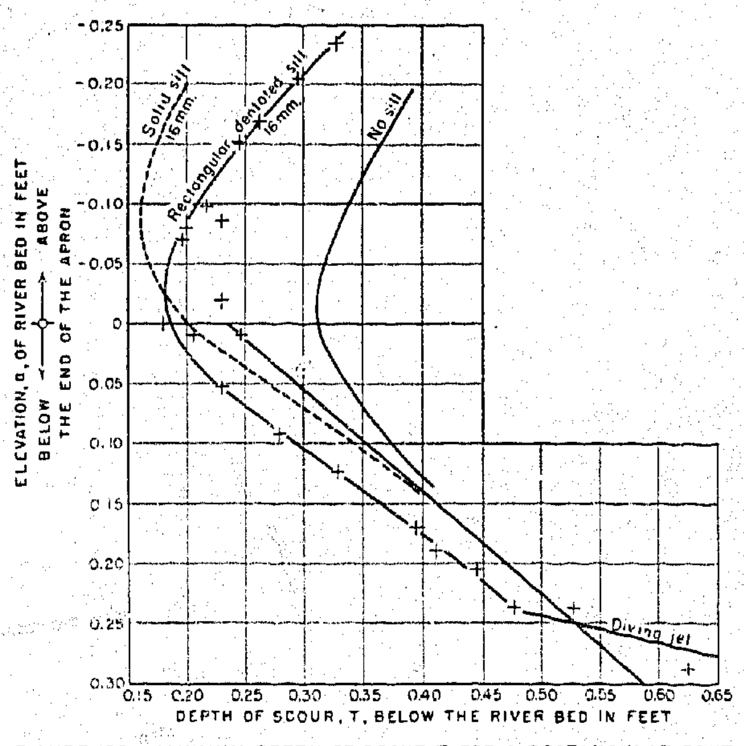
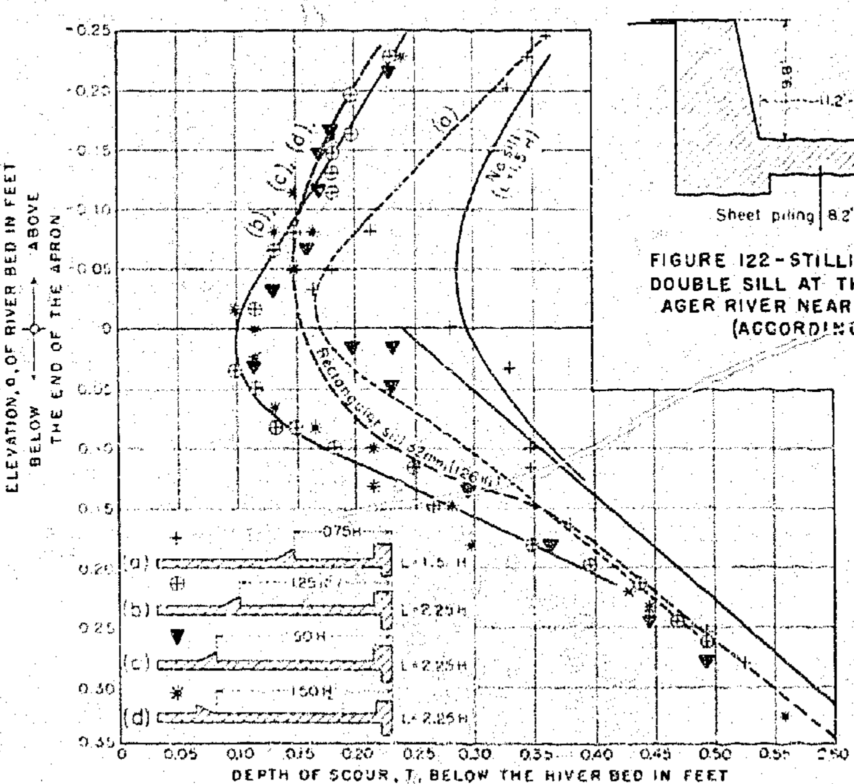



FIGURE 120 - MAXIMUM DEPTH OF SCOUR, T, FOR A RECTANGULAR DENTATED SILL WITH TEETH 16 mm. (0.93 in.) HIGH. DIAMETER OF BED MATERIAL, d = 2.5 mm.; HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.



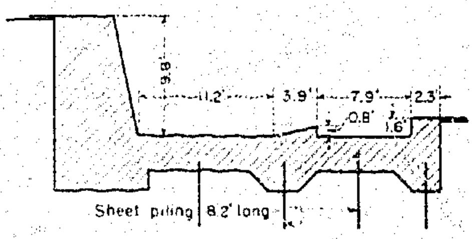




FIGURE 122 - STILLING POOL WITH LUDIN DOUBLE SILL AT THE DAM ON THE DÜRRE AGER RIVER NEAR ST. GEORGEN, AUSTRIA (ACCORDING TO K. KUICH).

FIGURE 123 - MAXIMUM DEPTH OF SCOUR, T, FOR A LUDIN DOUBLE SILL. RECTANGULAR END SILL, 32 mm. (1.26 in.) HIGH. TRIANGULAR FORE SILL, 16 mm. (0.63 in.) HIGH. L = EFFECTIVE APRON LENGTH. HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC.FT./FT.; DIAMETER OF BED MATERIAL, d = 2.5 mm.

formation of a wave jet is favored too much. An unsteady flow is formed by these double sills, which oscillates continually. The maximum depth of scour is advanced toward the apron by a double sill. If silt is carried over the dam, the space between the two sills is partly filled by it because of the ground roller formed there, but if the type of flow is altered, this material will be scoured out again.

Figure 125 shows a double sill made up of Rehbock sills and also the depths of scour for this double sill, a single Rehbock sill placed at the end of the apron, and a horizontal apron without a sill. Dentated sills effectively prevent the development of dangerous wave jets on the river bed, and it is therefore possible, with a Rehbock double-dentated sill, to place the two sills about 1H apart, thus permitting a somewhat shorter apron than with the Ludin double sill.

Finally, the proposal of A. Schafer[16], is mentioned. On the basis of model tests, he recommended the arrangement shown in figure 126, which consists of a row of baffle piers on a long horizontal apron, followed by a trapezoidal sill on a rising apron. He further declared that by lengthening the rising section of the apron beyond the trapezoidal sill, scour was effectively prevented because the jet, on leaving the apron, was deflected upward.

## 4. PREVENTION OF SCOUR BY MEANS OF RIPRAP

At small structures the prevention of scour is frequently attempted by means of riprap. All of which is placed downstream from the solid part of the apron. Stones used in the riprap must be large enough so that they cannot be moved downstream by the jet. Therefore, the use of riprap is confined to low-head projects.

A number of experiments were performed on two different thickness of riprap. The results are given in figure 127. The individual stones were granite of an average thickness of about five centimeters (1.97 inches). The diameter, d, of the bed material was 2.5 millimeters; the head, H, was maintained constant at

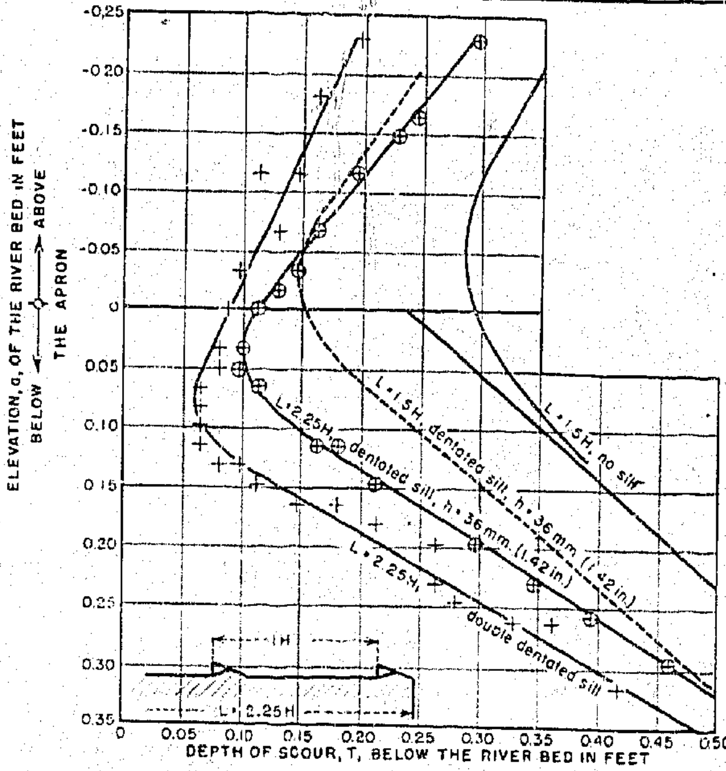



FIGURE 125 — MAXIMUM DEPTH OF SCOUR, T, FOR A DOUBLE REHBOCK DENTATED SILL, 36 mm. (1.42 IN.) HIGH. L = EFFECTIVE APRON LENGTH; HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC. FT./FT.; DIAMETER OF BED MATERIAL, d = 2.5 mm.

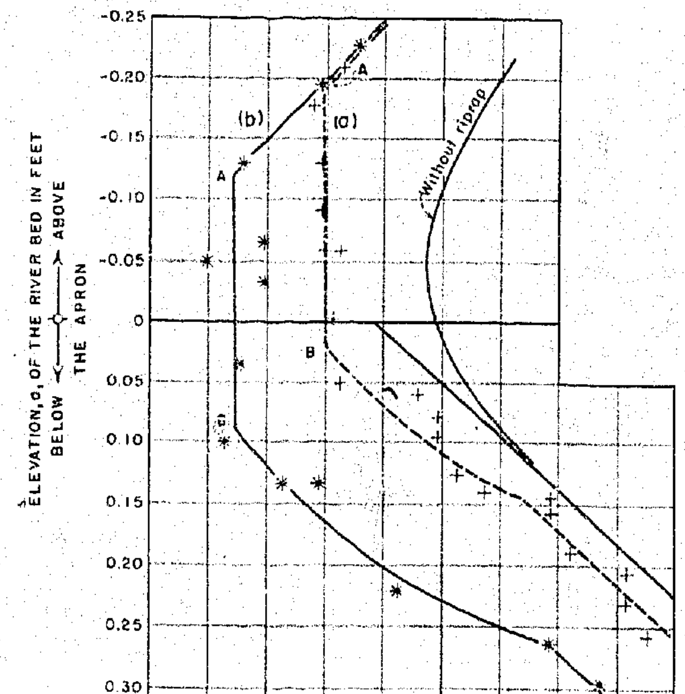



FIGURE 127 — MAXIMUM DEPTH OF SCOUR, T, WHEN RIPRAP IS USED AS A PREVENTATIVE AGAINST EROSION. DIAMETER OF BED MATERIAL, d = 2.5 mm.; DIAMETER OF STONES OF RIPRAP, 50 mm. (1.97 IN.); HEAD, H = 0.92 FT.; DISCHARGE, q = 0.60 SEC. FT./FT.; EFFECTIVE LENGTH OF SOLID HORIZONTAL APRON, L = 1.5H; (a) LENGTH OF RIPRAP, 1H; (b) LENGTH OF RIPRAP, 1.5 H; THICKNESS OF RIPRAP, 80 mm. (3.15 IN.). (A) THE SCOUR BEGINS TO FORM UNDER THE END OF THE RIPRAP; (B) AS A CONSEQUENCE OF THE WASHING OUT OF THE MATERIAL BETWEEN THE INDIVIDUAL STONES, THE RIPRAP BEGINS TO SINK.

HYD-20



Y-D-2029

0.92 feet, and the discharge at q at 0.60 second-feet per foot. The experiments show that the depth of scour can be reduced by means of riprap composed of sufficiently large stones, but, as is to be expected, scour cannot completely be prevented. Other experiments have shown that the coarser the material of the river bed the more effective the riprap. Part of the flow is diverted into the spaces between the stones and thus against the river bed under the riprap. The amount of material scoured out from under the riprap increases with an increase in its size, and decreases the thicker the riprap.

If the river bed underneath the end of the riprap is scoured out sufficiently, the stones there begin to separate, and the end stones slip into the hollow and form a slope of perhaps less than 1:2. The closer the stones are fitted together the less will be the scouring underneath the riprap; and a riprap composed of several layers of stone is more effective than a thinner riprap. If the riprap is formed by throwing stones into the river bed, scour occurs at the upstream side of each stone similar to that at bridge piers. Often, the stones tip over into this cavity and completely disappear under the bed.

The depth of scour may be reduced by roughening the surface of the riprap. A true wave jet did not appear in any of the experiments. However, a diving jet did occur, and although it did not directly affect the shape of the scour, it sucessfully eroded the material from between the stones and disintegrated the riprap.

## 5. PREVENTION OF SCOUR BY MEANS OF FLOATING AND SHELF APRONS

R. Hofbauer and P. Puchner,[20] have observed that the extensive scour at the end of a log chute can be rapidly filled up by means of a so-called floating apron which prevents the logs from striking the bed. These floats are made of wood, the upstream section being solid and the downstream section in the form of an open grid with longitudinal openings. They are hinged to the end of the chute and hence slope upward, deflecting the logs from the river bed.

The author[21] conducted some experiments in a glass channel for the purpose of studying the action of these floating aprons. The experiments have further shown that the flow which is parallel to the float on the solid portion, in passing over the open grid portion releases silt, which passes through the slits in the grid.

Later, A. Läufer[22] conducted several experiments with floats. He placed particular emphasis on a slit between the concrete apron and the float. This space was present in the first model tested by the author. Apparently a vacuum exists at this slit, causing a flow upward through it and a reverse flow under the float, which carries the silt passing through the grid back to the dam. Läufer attributes the deposition of silt under the apron to this reverse flow. Figure 129 shows the float designed by Läufer and the silting below the dam on the Mürz River at Bruck.

During the preliminary work for the reconstruction of the dam on the Mur River at Bruck, where the first float for the prevention of scour had been installed, the author conducted new experiments to test the validity of the conclusions reached by A. Läufer. These experiments showed that the reverse flow observed by him underneath the floating apron took place only if a hydraulic jump occurred toward the downstream end of the float; in other words, if the tail-water level there was higher than at the slit (figure 130). Therefore, the reverse flow observed by Läufer was produced by this difference in head and not by a vacuum at the slit.

Further experiments have shown that with the proper proportioning of the float, its length, as far as the formation of scour is concerned, is relatively unimportant. If the dam directs the water parallel to the float, then it may consist entirely of a grid because the rapid flow does not pass through the float and scour the bed. Finally, it was shown that a grid with openings transverse to the direction of flow produced a satisfactory deposition of silt provided the jet released by the dam was parallel to the float. The existing scour is filled up only if sufficient silt is carried over the dam. Floats may also be used to reduce the formation of scour.

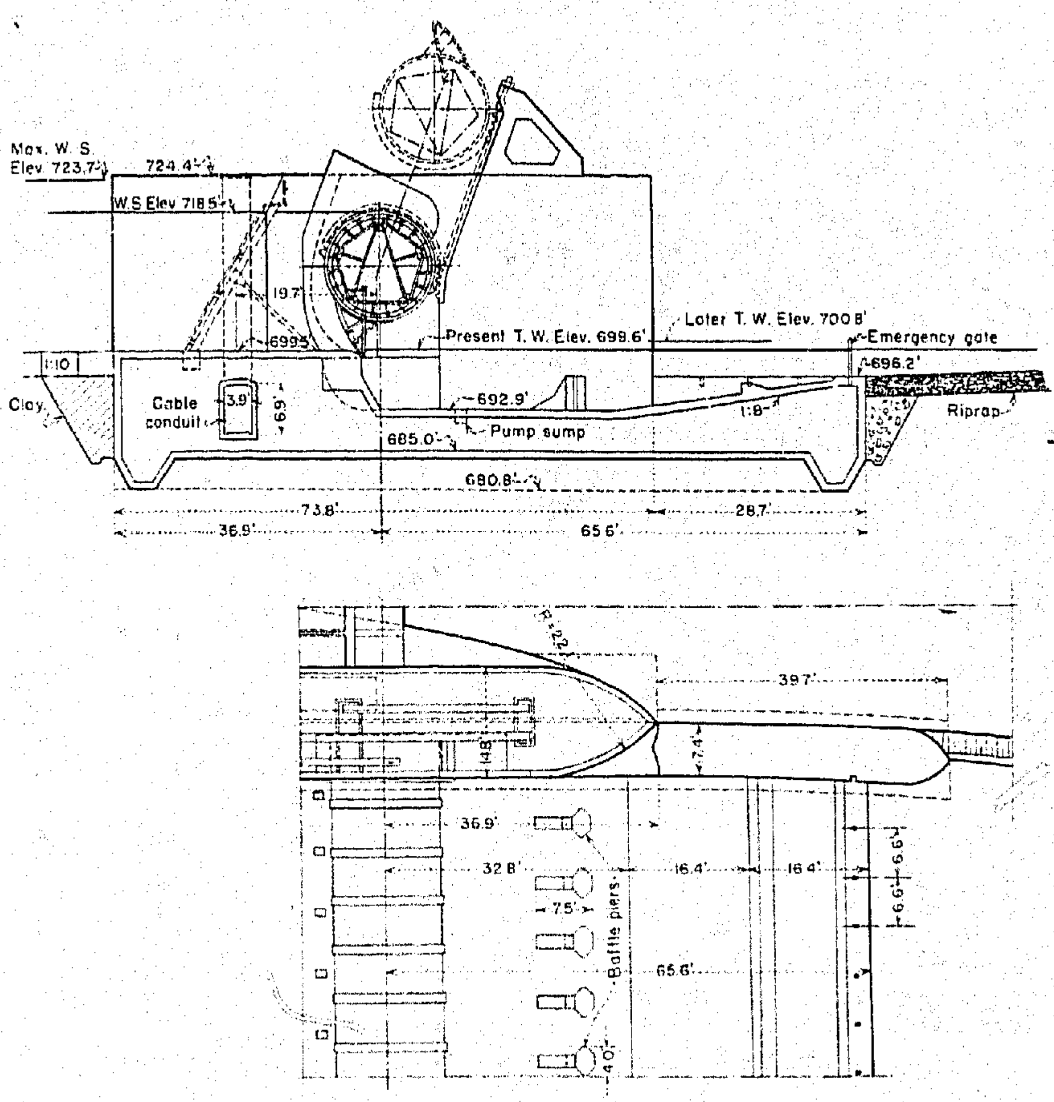



FIGURE 126 – ROLLER GATE DAM AT CANNSTADT WITH A SCHÄFER DOUBLE SILL (ACCORDING TO N. KELEN "GEWICHTSSTAUMAUERN", J. SPRINGER, BERLIN, 1933).

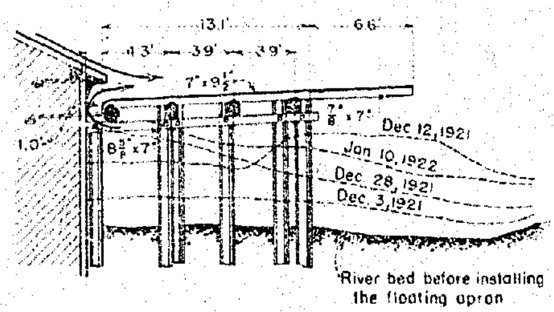



FIGURE 129 – OBSERVATIONS OF SILT DEPOSITION UNDERNEATH A FLOATING APRON ON THE MÜRZ RIVER IN AUSTRIA.

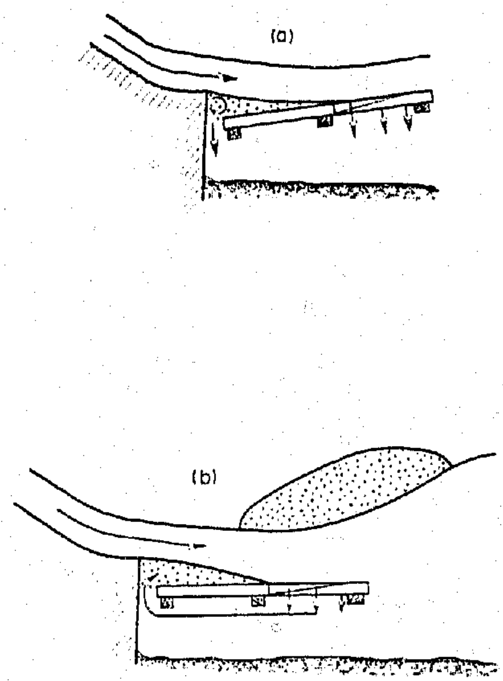



FIGURE 130 – FLOW CONDITIONS WITH A FLOATING APRON.

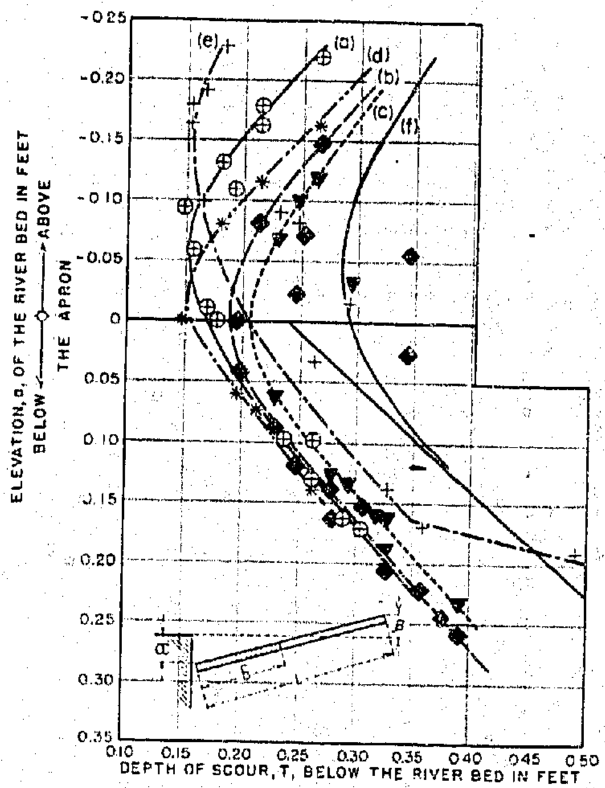



FIGURE 131 – MAXIMUM DEPTH OF SCOUR, T, FOR A FLOATING APRON OF LENGTH, L = 0.75 H. H = 0.92 FT.; DISCHARGE, q = 0.60 SEC. FT./FT.; DIAMETER OF BED MATERIAL, d = 2.5 mm.; α = 0.075 H. (a) β = -0.1 H, δ = 0.4 L (SOLID); (b) β = 0, δ = 0.4 L; (c) β = 0.075 H, δ = 0.4 L; (d) β = 0, δ = 0; (e) β = 0, δ = 0, RECTANGULAR SILL 16 mm. (0.63 IN.) HIGH ON END OF APRON; (f) HORIZONTAL APRON WITHOUT A FLOATING APRON ATTACHED.

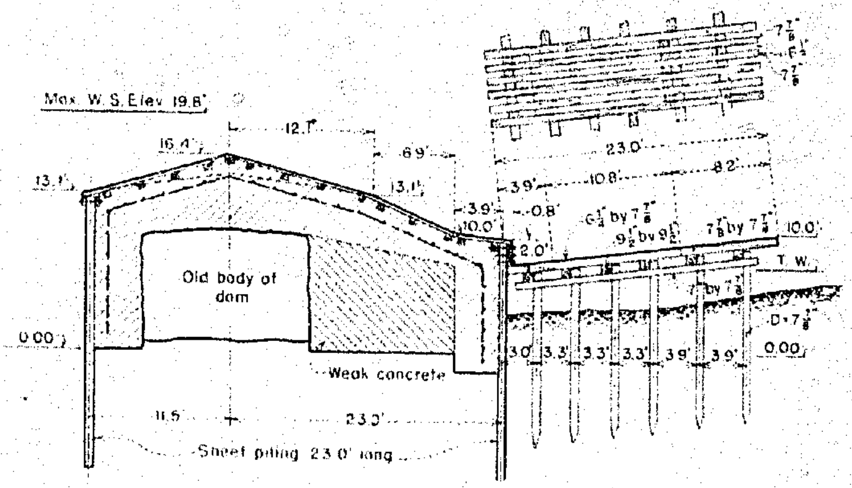



FIGURE 132 – CROSS SECTION OF THE DAM ON THE MUR RIVER AT BRUCK, AUSTRIA, WITH A SHELF APRON. DESIGN BASED ON MODEL TESTS.

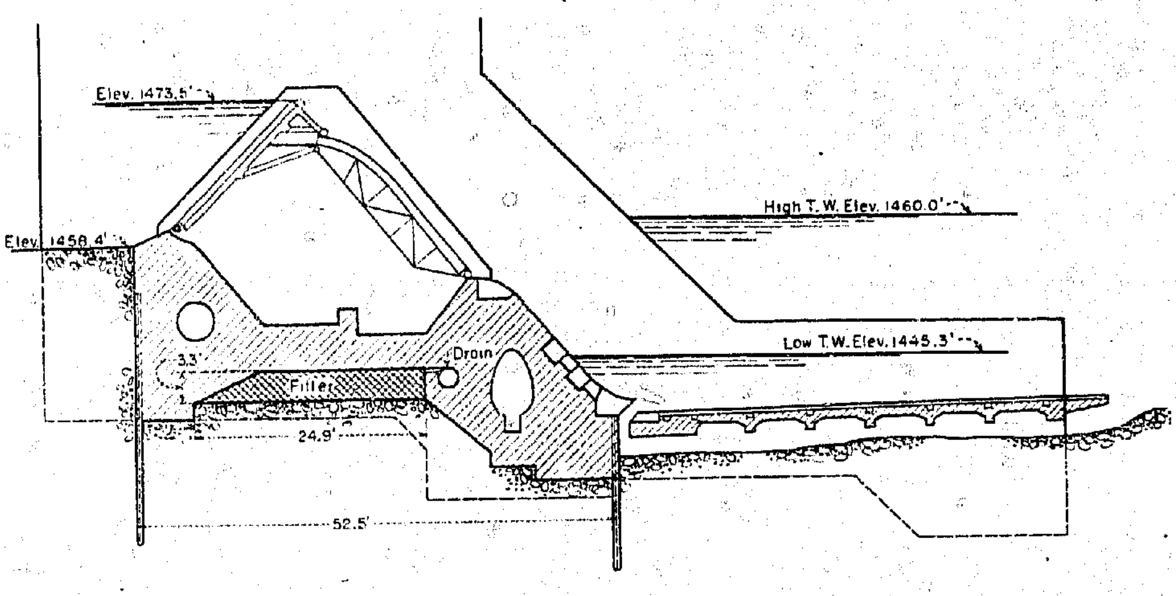



FIGURE 134-THE BEAR TRAP DAM AT HALLEIN, AUSTRIA, WITH A SHELF-APRON SUPPORTED BY REINFORCED CONCRETE (FROM A. SCHOKLITSCH, "DER GRUNDBAU", J. SPRINGER, BERLIN, 1932).

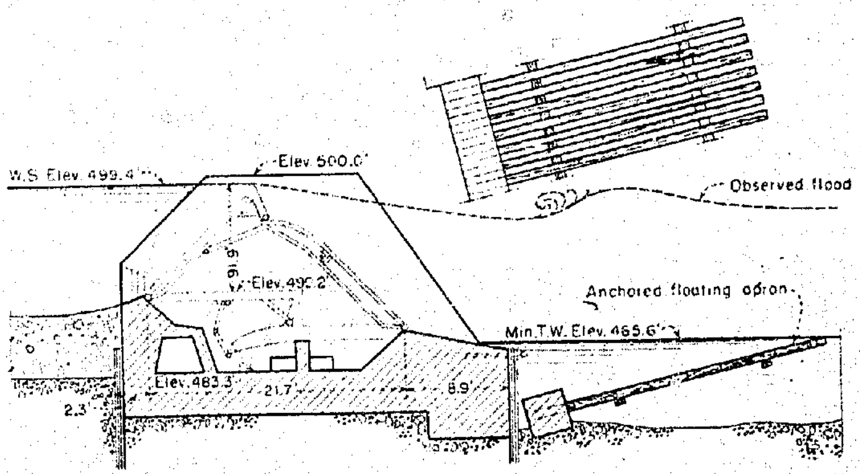



FIGURE. 135 - A BEAR-TRAP DAM WITH AN ANCHORED FLOATING APRON (ACCORDING TO HUBER AND LUTZ).

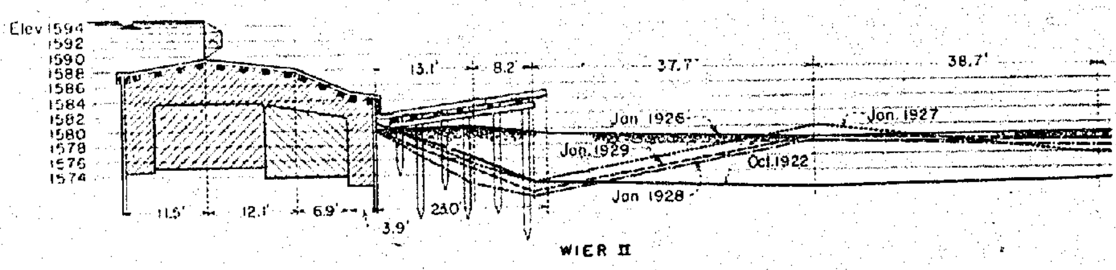



FIGURE 138 - SCOUR AT THE WEIR ON THE MUR RIVER AT BRUCK, AUSTRIA.

HYD-20



X-D-2032

The results of experiments on floating aprons and using clear water are given in figure 131 and show how much the formation of scour can be reduced by a float in comparison with a horizontal apron. However, if no silt is carried over the dam, scour cannot be completely prevented.

Floats for the prevention of scour have been built at a large number of projects and have proved satisfactory for reducing scour as well as for silting up the scoured bed. Examples of such structures are shown in figures 132 and 134.

These floats are always constructed from wood, and originally they were simply hinged to the dam. It had been shown that with water containing sand and silt, the free turning of the table was rapidly destroyed and accidents resulted. Later, shelves similar to the floats in design, and supported on piles (figure 132) or placed on rigid beams of iron or reinforced concrete, in turn supported by special walls (figure 134), were used.

If the nature of the river bed or an excessive scour does not permit the driving of piles, floats may be anchored at one end in order to cause the deposition of silt. Figure 135 shows an anchored float designed by Huber and Lutz in Zurich; one end was kept submerged by a concrete block, no other support being provided. The float thus sloped upward from the upstream end.

During the last few years considerable scour has occurred at the dam at Bruck in spite of a shelf. Several piles were left hanging from the shelf itself. Measurements of the scour are given in figure 138. An investigation disclosed that this erosion under the shelf and that occurring along the river bank was due to side rollers. After the old dam had failed, a considerable amount of the silt deposited in the reservoir was washed out and redeposited below the dam, raising the elevation of the river bed to an abnormal degree. The silting of the reservoir above the repaired dam and of a reservoir farther upstream had the effect that the silt-free flood water passing over the dam at Bruck lowered the bed considerably because the shelf had been rendered useless by the abnormally high river bed. This example illustrates the significant factor the elevation of the river bed plays in the prevention of scour by means of a shelf.

If the discharge, according to common practice, is released through but one of the gates, rollers will form at the sides of the discharging jet and the space occupied by them does not contribute to the direct flow. The return flow passes under the shelf and scours out the bed material there (figure 139). This can be prevented by building piers from the dam downstream to the end of the shelf. The discharge is then so distributed that the side rollers are considerably reduced in size. The form of the discharge jet can be modified by changing the slope of the shelf. If it lies flat, a hydraulic jump will occur on it, providing the river bed downstream from the dam lies high enough. A steeper slope, such as used at the dam at Bruck, can hinder the formation of the hydraulic jump and cause a true jet to form. As was to be expected, the old dam at Bruck was badly damaged by logs carried over the dam during occasional floods. These logs entered the surface roller located just below the dam where they were detained; they then turned 90 degrees about a vertical axis and rammed the main body of the dam, damaging it. However, logs pass downstream without halting and without damage to the dam, if a wave jet develops.

## 6. PREVENTION OF SCOUR BY MEANS OF WALLS PLACED DOWNSTREAM FROM THE DAM (CHECK SILLS)

The reduction of scour by means of a wall placed across the stream below the dam was first proposed by A. C. König,[23] as shown in figure 140, and P. Rosenberg has used such a wall at a dam at Zierberg south of Graz. A section of this dam and the scour developed at the end of the first year after the wall was put into operation, is shown in figure 141. During this year, however, no extraordinary flood occurred on the Mur River.

At first sight the prevention of scour by means of a wall, as proposed by König, seems to be a good scheme, since it is easy to construct and is easily installed at most dams already in service. Nevertheless, several experiments were undertaken for the purpose of investigating the efficiency of such a wall.

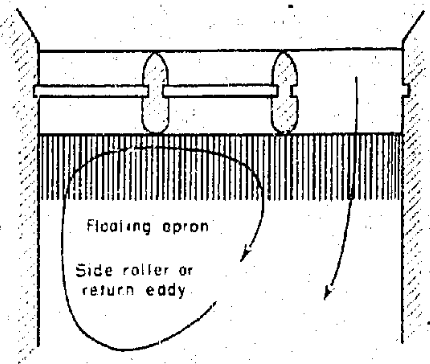



FIGURE 139 - SCOURING OF THE MATERIAL UNDER A FLOATING APRON BY A SIDE ROLLER.

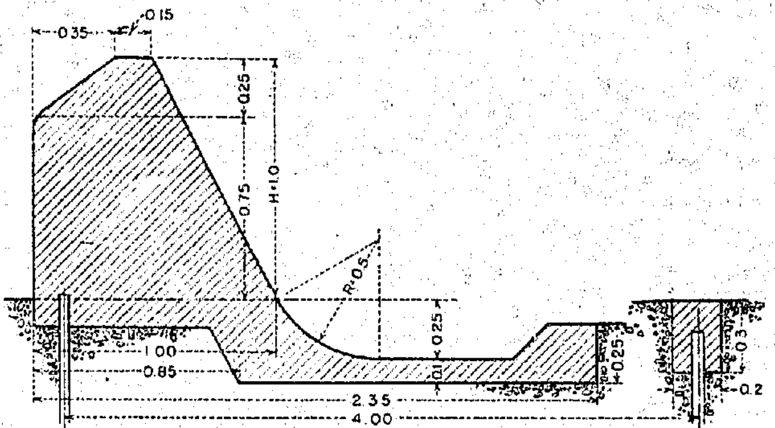



FIGURE 140 - DAM WITH CHECK SILL DESIGNED BY A.C. KÖNIG. (ALL DIMENSIONS ARE IN TERMS OF THE HEIGHT, H, TAKEN AS A UNITY).

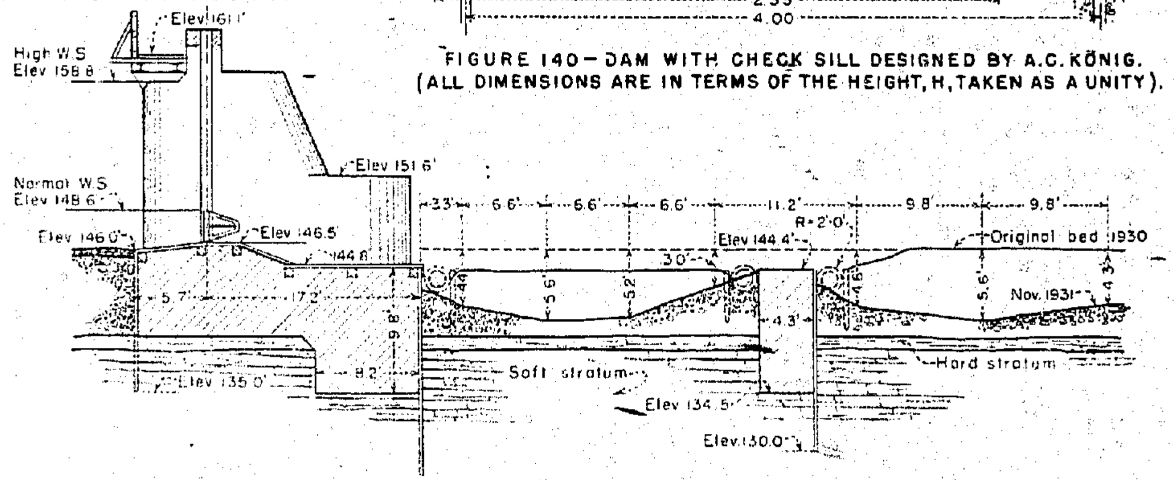



FIGURE 141 - SCOUR AT THE SLUICEWAY ON THE MUR RIVER AT ZIERBERG, AUSTRIA (DESIGN AND MEASUREMENTS BY REDLICH AND BERGER).

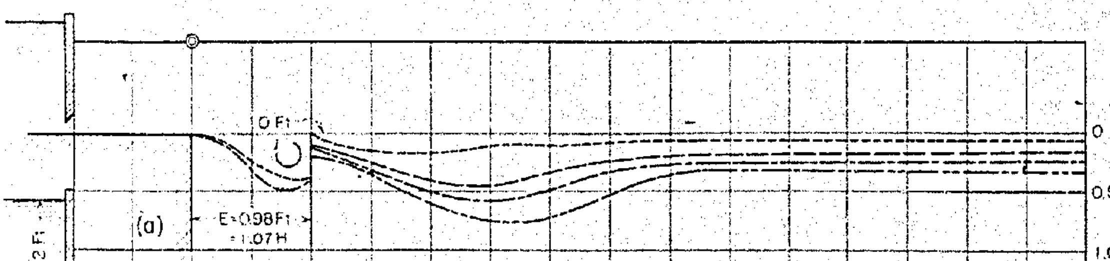



FIGURE 142 - SCOUR AT A "KÖNIG" CHECK SILL. HEAD, H=0.92 FT.; DISCHARGE, q=0.60 SEC.FT./FT.; DIAMETER OF BED MATERIAL, d=6.2 mm. (ALL DIMENSIONS ARE IN FEET).

HYD-20



X-D-2033

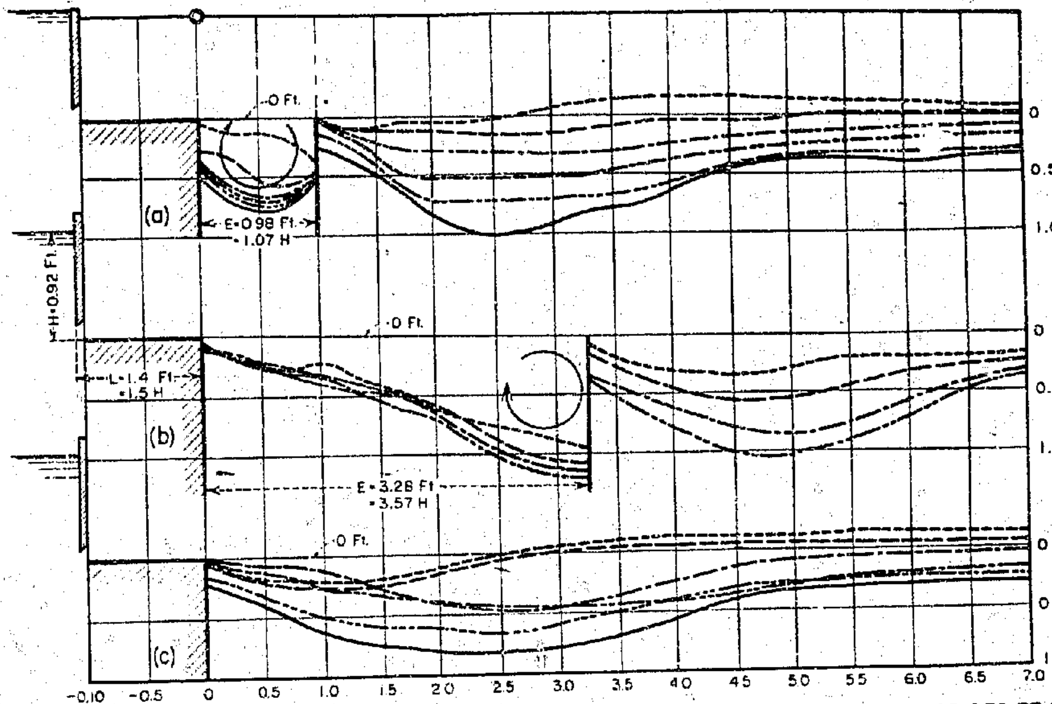



FIGURE 143 – SCOUR AT A "KÖNIG" CHECK SILL. HEAD, H= 0.92 FT.; DISCHARGE, q= 0.60 SEC. FT./FT.; DIAMETER OF BED MATERIAL, d= 2.5 mm. (ALL DIMENSIONS ARE IN FEET).

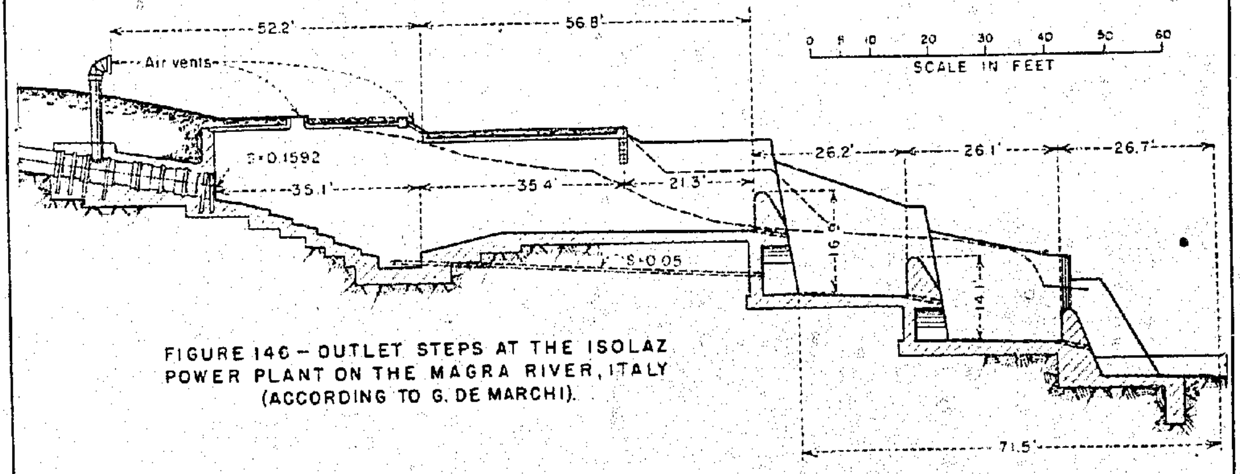



FIGURE 146 – OUTLET STEPS AT THE ISOLAZ POWER PLANT ON THE MAGRA RIVER, ITALY (ACCORDING TO G. DE MARCHI).

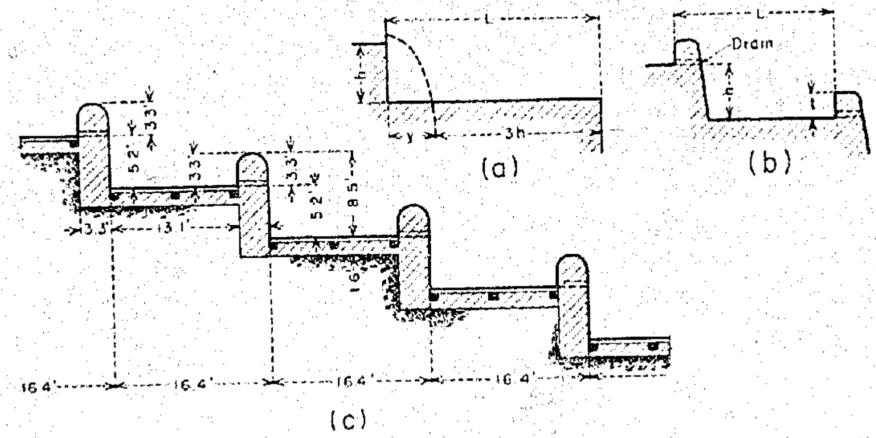



FIGURE 147 – OUTLET STEPS: (a) WITHOUT A STILLING POOL, (b) WITH A STILLING POOL AND (c) STEPS WITH STILLING POOLS IN CASCADE.

These tests were carried out in the glass channel with a head, H, equal to 0.92 feet, and a discharge, q, equal to 0.60 second-feet per foot. The model of the dam was fitted with a horizontal apron without a sill. The effective length of the apron was 1.5 times the head. Two different sizes of bed materials were used; they were uni-granular sands with an average diameter of 2.5 and 6.2 millimeters, respectively. The wall was first placed at a distance, E, from the end of the apron, equal to 1.07H, or 0.98 feet, and secondly, at 3.57H or 3.28 feet. The top of the wall was, first, even with the apron, then, 0.0715H or 0.79 inches above the apron, and, finally, an equal distance below. The elevation of the river bed downstream from the dam was not altered in any of the experiments. Finally, the crest of the wall was beveled in one series of tests, and, as a basis for comparison, each series of tests performed with a wall was repeated without a wall for both sizes of bed material. In all, seventy experiments were made on on this model.

The results of the experiments with the crest of the wall at the same height as the horizontal sill are shown in figures 142 and 143. It is seen that the wall causes a ground roller to form just upstream from itself. This roller transports considerable material upstream, transferring it to the discharge jet which carries the silt over the wall. The activity of these rollers increases as the wall is moved toward the apron. The finer the bed material, the deeper the scour will be between the apron and the wall. Beveling the crest of the wall does not reduce the ground roller to any great extent. With fine bed material and the near position of the wall, the maximum depth of scour is not much less than that formed without a wall, and if the wall is very close to the dam, the scour may even threaten the dam itself. The larger the bed material, the less is the deveolpment of scour, and the greater is the difference between the depths of scour occurring with and without a wall. However, the larger the bed material, the greater will be the abrasive effect on the wall itself. A deep cut-off wall must be built at the end of the apron on account of the scour occurring between the apron and the wall, and, for the same reason, the wall itself must have a deep foundation. It is possible to lower the cut-off wall at a fraction of the cost of lowering both the cut-off wall and the König wall, and furthermore, the safety of the dam is raised by the former. In addition to this, the König wall produces a length of scour which is almost double that occurring without a wall; it therefore increases the cost of protecting the river banks downstream from the dam.

## III. ENERGY DISSIPATORS

Energy dissipators are used for transforming the kinetic energy of the water into heat if the water passing through the dam is free of silt. They should be designed to dissipate the energy within themselves.

### a. Outlet and Spillway Steps (Cascades)

Formerly, if the overfall height at a spillway or outlet was large, it was almost invariably divided up into a number of smaller overfalls forming a stepped outlet or cascade similar to a stepped dam. Now, they are only built in exceptional places where rock is accessible at the location. Such sites permit their construction with but a small demand for other building materials. Figure 146 shows a cross section of the cascade at the Isolaz power plant on the Magra River.

As figure 147 shows, the cascade may be made up of simple steps or, better, of a series of tumble bays which are provided with drains to prevent the basins from filling with ice in winter. According to Thomas Rumelin[24], these basins ought to be at least 6.5 feet deep; but there are numerous cascades with a much smaller basin depth, t. The length of the basin of the tumble bays should be three times the step height, and for simple steps, four times the height. The range, y, of the trajectory of the overfalling nappe cannot be computed because of the disturbed condition of the water in the basin over the step.

Instead of basins, the dam at Muhleberg is equipped with baffle piers arranged on the two steps in order to improve the performance of each step (figure 148). This construction has not proved satisfactory because the water is sprayed by the baffle piers on the upper step and because the piers themselves are subject to considerable erosion.

b. Counterflow Energy Dissipators

A counterflow energy dissipator consists of a basin in which the water is directed from openings placed opposite to each other. These discharge jets, flowing in opposite directions, produce a very turbulent flow.

The Rossnow power plant in Pomerania (figure 150) is an example. In order to increase the turbulent action, water may also be discharged over the two steps.

Another example[25], designed on the basis of model experiments, is found at the end of a tunnel at the Toging power plant on the Inn River. As is seen in figure 152, the two side jets entering the tub are directed at each other, and the middle jet rebounds from the concrete pier.

A third example of larger dimensions, designed by H. Corrazza[26], has been constructed at the Kardaun power plant on the Eisak River (figure 154). The two sets of nozzles are directed toward each other. The number of nozzels in the lower set is twice the number in the upper set.

Finally, figure 155 shows a design by E. Meyer-Peter[27] for the Limmat power plant near Wettingen. Here, the two jets are not directed axially against each other.

c. Energy Dissipation in a Hydraulic Jump on a Horizontal Floor

If water is discharged under a large enough head into a channel having a horizontal floor, so that its velocity, $V_1$, is greater than $\sqrt{gd_1}$, $d_1$ being the depth of flow, it moves as shooting flow. At a certain place in the channel, a hydraulic jump in which the shooting flow changes to streaming flow occurs (figure 156). A surface roller develops on top of the hydraulic jump and dissipates an amount of kinetic energy, $H_v$, into heat.

The depth, $d_2$, of the streaming flow can be computed by equating, for a unit width of channel, the resultant hydrostatic force acting on the slug of water bounded by the jump to the rate of change of momentum in the jump, assuming a horizontal floor, uniform distribution of velocity in the cross sections just below and just above the jump, and neglecting friction at the wetted perimeter.

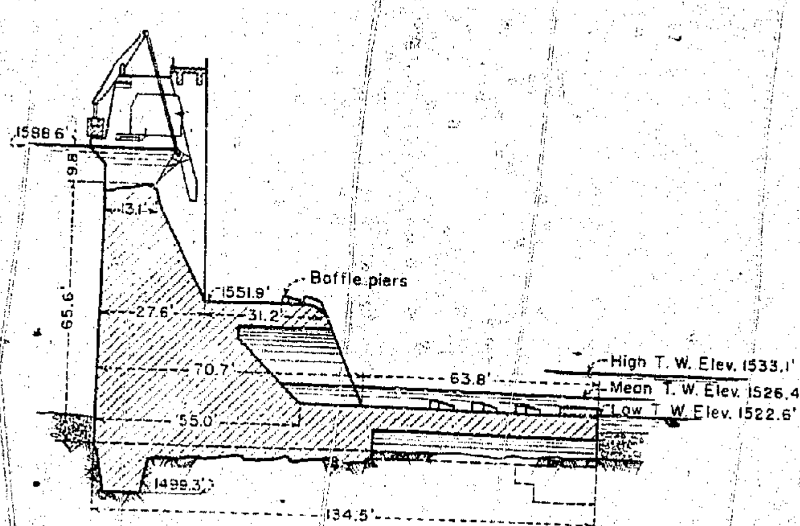



FIGURE 148 — CROSS SECTION OF THE MUHLEBERG DAM.

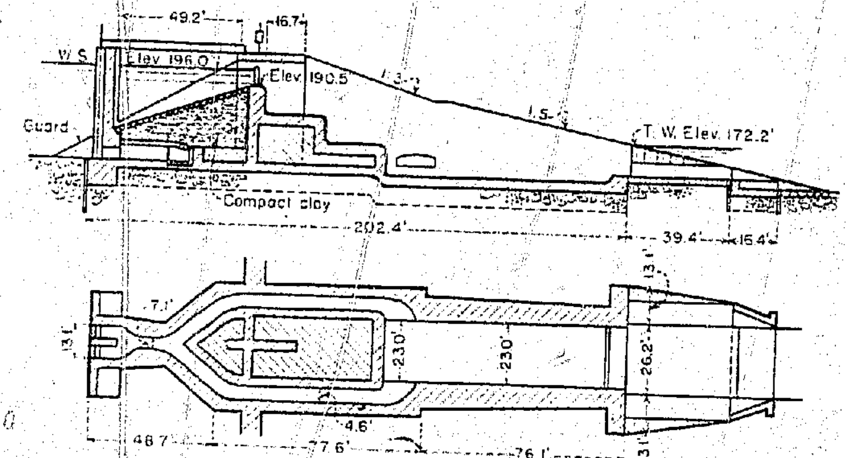



FIGURE 150 — THE ENERGY DISSIPATOR AT THE ROSSNOW POWER PLANT.

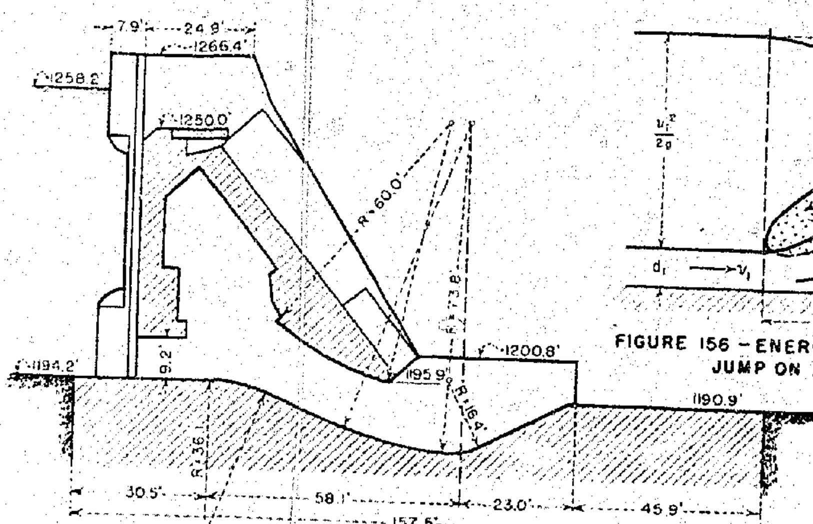



FIGURE 155 — CROSS SECTION OF THE DAM AT WETTINGEN ON THE LIMMAT RIVER (ACCORDING TO E. MEYER-PETER).

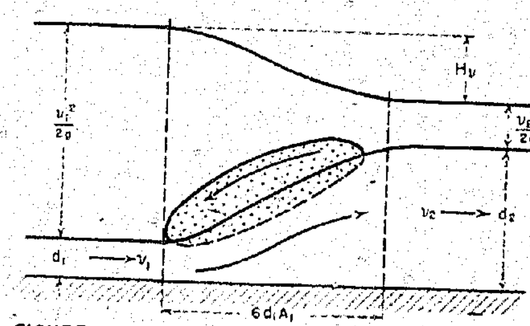



FIGURE 156 — ENERGY DISSIPATION IN A HYDRAULIC JUMP ON A HORIZONTAL FLOOR.





X-D-2035

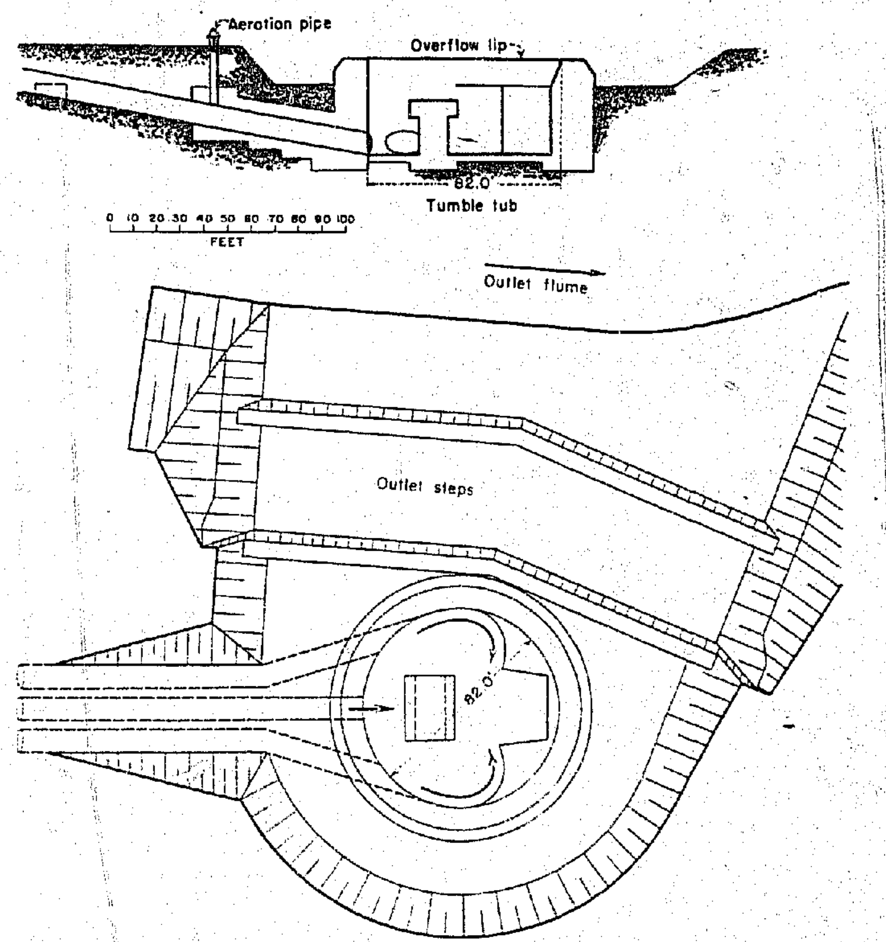



FIGURE 152 – ENERGY DISSIPATOR OF THE POWER PLANT AT TOGING ON THE INN RIVER.

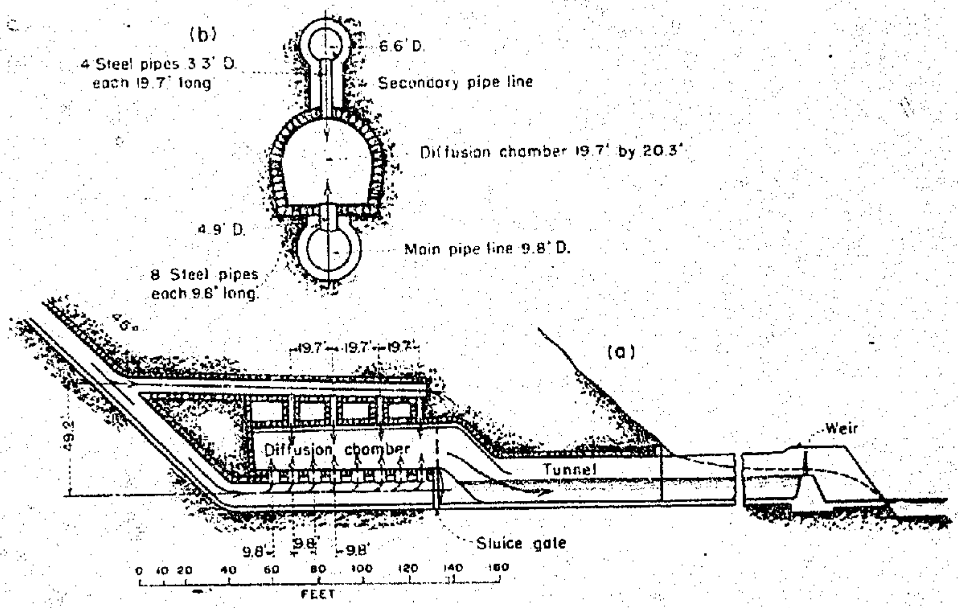



FIGURE 154 – DIFFUSION CHAMBER ENERGY DISSIPATOR AT THE KARDAUM POWER PLANT ON THE EISACK RIVER. (a) LONGITUDINAL SECTION, (b) CROSS SECTION (ACCORDING TO M. H. CORRAZZA).

Thus:

$$\frac{\gamma d_2^2}{2} - \frac{\gamma d_1^2}{2} = \frac{\gamma q v_1}{g} - \frac{\gamma q v_2}{g} \quad \text{. . . . . . . . . . . . . . (68)}$$

Substituting, $q = v_1 d_1 = v_2 d_2$ . . . . . . . . . . . . . . . . . (69)

we have, after simplifying:

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}} \quad \text{. . . . . . . . . . . . . (70)}$$

Introducing the dimensionless number,

$$A = \frac{v_1}{\sqrt{gd_1}} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . . . (71)}$$

we have:

$$d_2 = \frac{d_1}{2}\left(\sqrt{8A_1 + 1} - 1\right) \quad \text{. . . . . . . . . . . . . . . . . . . . (72)}$$

M. Merriman[28] has deduced the following empirical formula:

$$d_2 = \sqrt{\frac{2v_1^2 d_1}{g}} = 1.41 A_1 d_1 \quad \text{. . . . . . . . . . . . . . (73)}$$

Later, K. Safranez[29] obtained for this form of equations:

$$d_2 = 0.435 \frac{q}{d_1} = 1.36 A_1 d_1 \quad \text{. . . . . . . . . . . . . . . . (74)}$$

The amount of energy dissipated by the hydraulic jump is equal to the drop in the energy gradient and, according to P. Nemenyi[5], can be computed from:

$$H_v = \left( \frac{v_1^2}{2g} + d_1 \right) - \left( \frac{v_2^2}{2g} + d_2 \right) \quad \text{. . . . . . . . . . . (75)}$$

Substituting equation (72) in this equation and simplifying, we have:

$$H_v = \frac{d_1}{16} \frac{\left(\sqrt{8 A_1^2 + 1} - 3\right)^3}{\sqrt{8 A_1^2 + 1} - 1} \quad \text{. . . . . . . . . . . . . . (76)}$$

The energy loss per unit of total energy head, $H_1$, equal to

$$\frac{v_1^2}{2g} + d_1$$

is:

$$H_v \text{ (per unit of } H_1) = \frac{12.5}{2 + A_1^2} \frac{\left(\sqrt{8 A_1^2 + 1} - 3\right)^3}{\sqrt{8 A_1^2 + 1} - 1} \quad \text{. . (77)}$$

It should be pointed out that the approximate formulas of Merriman or Safranez will not give a zero energy loss when the flow is at the critical depth ($A = 1$) as is required from theoretical considerations.

K. Safranez found that the length of the jump could be expressed by the following empirical formula:

$$L = 6\, d_1\, A_1 \quad \text{. . . . . . . . . . . . . . . . . . . . . . . . . (78)}$$

d. Energy Dissipation by Means of Baffle Piers

Under certain conditions, shooting flow on a horizontal, smooth floor moves forward for a considerable distance before a hydraulic jump takes place. In order to reduce the distance within which energy is dissipated, some means must be adopted to force the development of rollers.

This can be accomplished by placing baffle piers on the apron, such as the baffle piers of the Rempen[30] Dam of the Waggital plant (figure 157). These caused a jump to form near the toe of the dam.

At many projects, baffle piers have not proved satisfactory because the discharge often shoots upward at the piers, or even sprays into the air.

## e. Energy Dissipation in Stilling Pools

A hydraulic jump with its attendant surface roller can also be compelled to form on a horizontal apron by means of a sill. Simple stilling pools have been used for a long time. They have proved, for the most part, satisfactory. Many times, however, they have been disappointing because, until recently, no rules for designing them were known. The author[31] attacked this problem in the laboratory, using simple stilling pools and solid end sills.

The tests showed that the discharge jet moves forward in the stilling pool (figure 158a) between a large surface roller and a small ground roller, and as it proceeds, it gradually slackens and becomes thicker. At a certain unit discharge, q, the jet carries the surface roller out of the stilling pool, shoots over the end sill with almost undiminished velocity (figure 158b), and strikes the river bed farther downstream, producing a deep scour. A comparison of the types of flow directly before and after the disappearance of the surface roller shows clearly the important role played by the surface roller in the dissipation of energy.

Preliminary tests showed that in most cases, for a length of stilling pool equal to two-thirds of the height of the dam, P, the dissipation of energy was satisfactory providing the height, S, of the end sill was chosen correctly. Let q be the maximum unit discharge in second-feet per foot; then if the length of stilling pool, L, is calculated from the formula,

$$L = 2/3\,P \qquad (79)$$

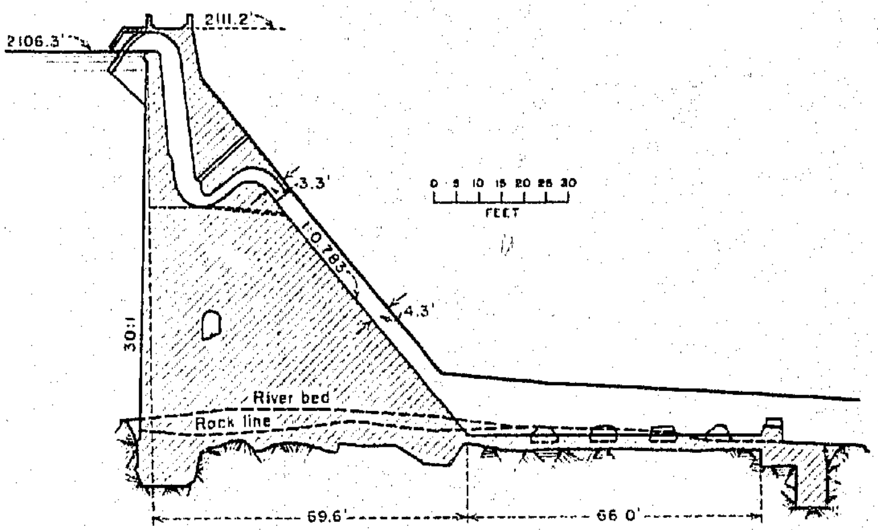



FIGURE 157 - THE REMPEN DAM WITH BAFFLE PIERS FOR ENERGY DISSIPATION.

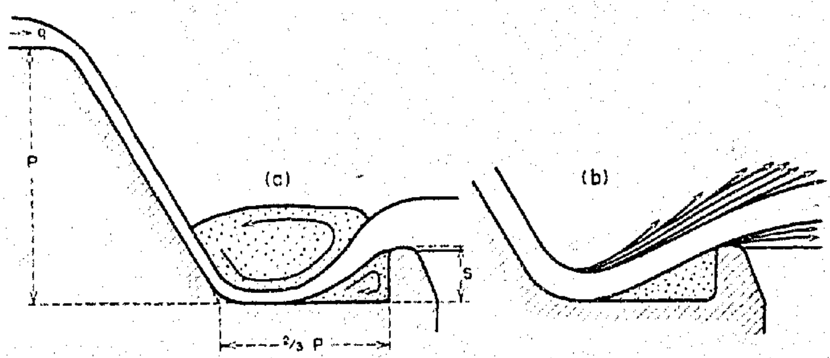



FIGURE 158 — A SIMPLE STILLING POOL. (a) GOOD ACTION, (b) BREAK-DOWN.

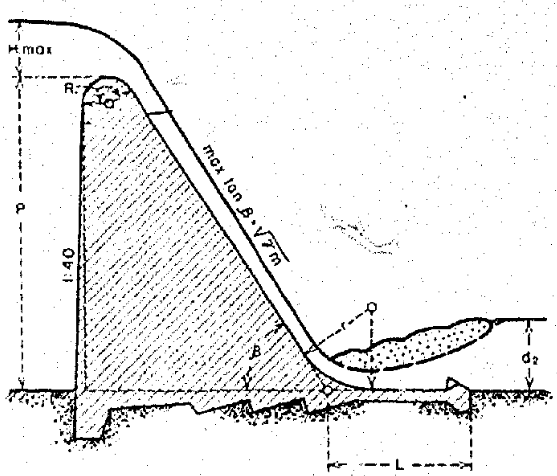



FIGURE 159 - ARRANGEMENT OF A REHBOCK DENTATED SILL FOR AN OVERFLOW SPILLWAY. $\gamma_m$ = SPECIFIC GRAVITY OF THE MATERIAL IN THE STRUCTURE.

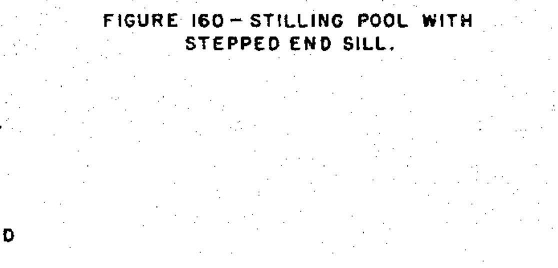



FIGURE 160 - STILLING POOL WITH STEPPED END SILL.

HYD-20



X-D-2037

The minimum height of end sill is given by:

$$S = 0.6 \; q^{1/2} \left(\frac{P}{g}\right)^{1/4} \ldots\ldots$$

Instead of a solid sill, a Rehbock dentated [illegible] placed at the end of a simple stilling pool. Using the [illegible] figure 159, Rehbock recommends that the height of the [illegible] puted from:

$$h = 0.08 \; (H_{max.})^{2/3} \; p^{1/3} \ldots\ldots$$

and the length of the pool from:

$$L = 2 \; H_{max.} + 1/8 \; p \ldots\ldots$$

The minimum radius, R, of the crest of the spill[illegible] should be:

$$R = 0.5 \; H_{max.} \ldots\ldots$$

and the minimum radius, r, of the bucket:

$$r = 1.5 \; H_{max.} \ldots\ldots$$

These formulas are valid provided that:

$$H_{max.} < \frac{P}{2} \text{ and } d_2 > 1.2 \; H_{max.}$$

Finally, J. Smetana[32], on the basis of his own and the earlier tests of E. Beyerhaus, recommends the stepped sill shown in figure 160. The height of a step, a, is one-half its width, b. The length of the stilling pool is computed from the formula,

$$L = 2.72 \; d_1 \left(\sqrt{1 + 8A_1} - 3\right) \ldots\ldots (85)$$

and the depth of the floor of the stilling pool below the tail-water elevation from:

$$y = 0.6 \; d_1 \left(\sqrt{1 + 8A_1} - 1\right) \ldots\ldots (86)$$

in which

$$A_1 = \frac{v_1}{\sqrt{gd_1}} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . . . (87)}$$

$v_1$ is the velocity, and $d_1$ the depth of the shooting flow.

In America, many other types of stilling pools have been built, and although of diverse shapes, many of them possessed no special value for the dissipation of energy.

The observations on the flow in the model of the simple stilling pool in figure 158 showed that the jet first clung to the smooth floor, then, at a short distance from the end sill, it rose and passed over the sill. A small ground roller formed in front of the sill. As a means of forcing the development of both a surface roller and a efficient ground roller, the author[31] placed a step (figure 161) at the end of the bucket of the dam. The jet now flowed between rollers and over the end sill and was gradually retarded along its path.

Figure 161a shows the surface and ground rollers at low discharges. As the discharge increases, the jet assumes a wave form, providing the height of the end sill is small. The surface roller now lies in the second wave trough. If the height, S, of the end sill is greater than one-tenth the dam height, P, the type of flow changes to that shown in figure 161c. For small sills this is attained at still higher discharges. If the discharge is increased beyond the amount necessary for this condition, the surface roller is suddenly scooped out of the stilling pool (figure 161e), and the jets shoot over the end sill, strikes the river bed farther downstream, and produces a deep scour. At this discharge, the stilling pool fails as an energy dissipator. With this design, the gradual expansion of the jet proceeds without a hydraulic jump forming. The flow at the end sill is mixed with air bubbles.

The following simple rules for the design of this stilling pool are based on a large number of model tests, and are valid for any length of pool from one-half to one times the height, P, of the dam.

Thus, using the symbols in figure 161f, a must be chosen between 0.5 and 1; the height, S, of the end sill measured from the elevation of the step at the end of the bucket depends on the maximum unit discharge, q, and is computed from:

$$S = \beta\, q^{1/2} \left(\frac{P}{g}\right)^{1/4} \qquad (88)$$

but must be at least equal to 0.1P. The coefficient $\beta$ depends on the elevation, $\varepsilon P$, of the step above the floor of the stilling pool, and can be determined from the curve in figure 162.

---

Translator's note: - Either $\varepsilon$ or $\beta$ may be made arbitrary subject only to the prescribed limit of S, namely:

$$S > 0.1P$$

Hence, the dimensions of a Schoklitsch stilling pool may be varied between wide limits. One set of limiting dimensions is given by $\varepsilon = 0$, which corresponds to a sill on a horizontal apron (see figure 158). Then $\beta = 0.6$, and equation 88 is identical with equation 80.

---

The minimum radius of the bucket for satisfactorily deflecting the jet in front of the step is given by:

$$r = \rho\, P = 0.15P \qquad (89)$$

The height, S, of the end sill governs the formation of the rollers in the stilling pool, so necessary for the dissipation of energy, even if the jet shoots over the end sill; therefore, the elevation of the tail-water is unimportant. However, if no scour is to occur, the jet should not shoot over the end sill. The top of this sill could be placed at the elevation of the river bed downstream from it or, better still, due to the later unavoidable deepening of the river bed, it may be placed at a still lower elevation.

Figure 163 shows a dam with this type of stilling pool. It has thus far proved satisfactory. Another example is given in figure 165.

The difficulties in designing an energy dissipator for the Fernegg power plant (figure 165) necessitated special model tests. Water was discharged into the stilling pool not only from the sluice gates during sluicing operations but also over the spillway. In order that the overfalling nappe produce no shock to the structure, it must be so directed that, independent of the amount of the discharge, it strikes the downstream face of the dam tangentially or as nearly tangentially as possible. For this purpose, the transition profile between the sluice gates and the stilling pool was designed to correspond to the under profile, determined from model tests, of the jet discharge over the spillway. Finally, the model was designed to prevent sand and gravel, occasionally discharged during sluicing operations, from obstructing the stilling pool and the draft-tube outlets at the powerhouse.

### f. Nozzle Energy Dissipators

In nozzle energy dissipators, water passes from a nozzle into a space whose cross section is several times the cross section of the nozzle. The total energy loss can be computed more or less accurately, depending on the design, from the formula of J. C. Borda[33]. Borda proposed that water emerging from the smaller pipe strikes against the retarded flow in the larger pipe with inelastic impact. The amount of mechanical energy converted into heat, using the symbols in figure 166, is computed from the following formula:

$$H_v = \frac{(v_1 - v_2)^2}{2g} = \frac{v_2^2}{2g}\left(\frac{A_2}{A_1} - 1\right)^2 \qquad (90)$$

K. Banninger[34] in his experiments, in which he used ratios of cross-sectional areas, $A_2:A_1$, from 1.1 to 10, found this relation to be valid.

Passing from the case of a small pipe to a nozzle of equal cross-sectional area, $A_1$, we find that the energy loss is somewhat greater than that computed from formula 90 because the jet contracts on leaving the nozzle.

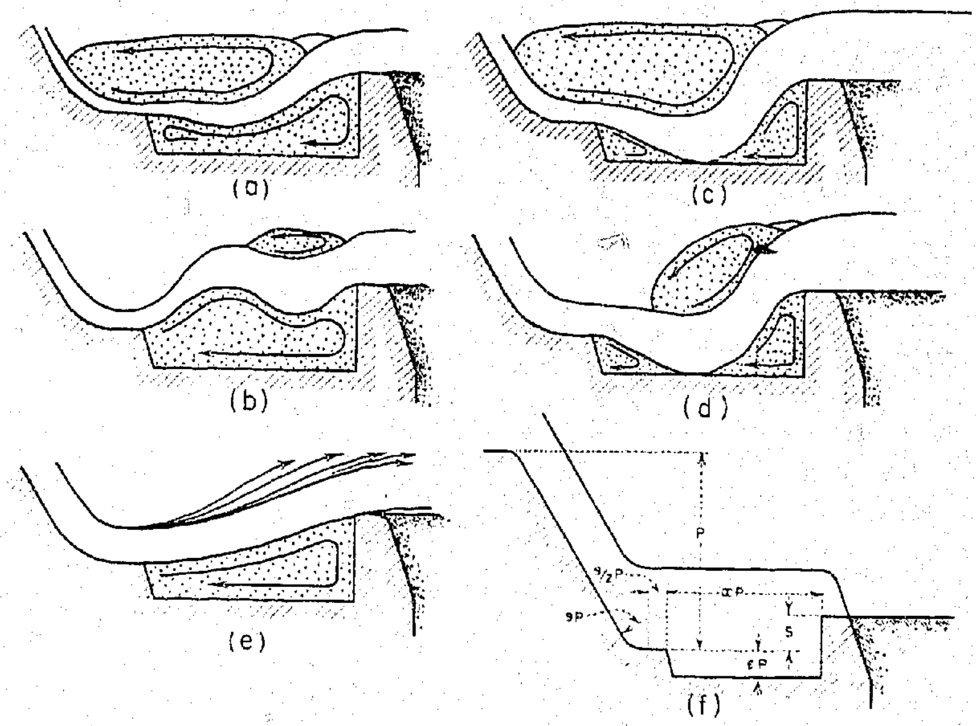



FIGURE 161 - THE SCHOKLITSCH STILLING POOL. (a) TO (e) TYPES OF FLOW FOR VARIOUS DISCHARGES, AND (f) SYMBOLS FOR DESIGN.

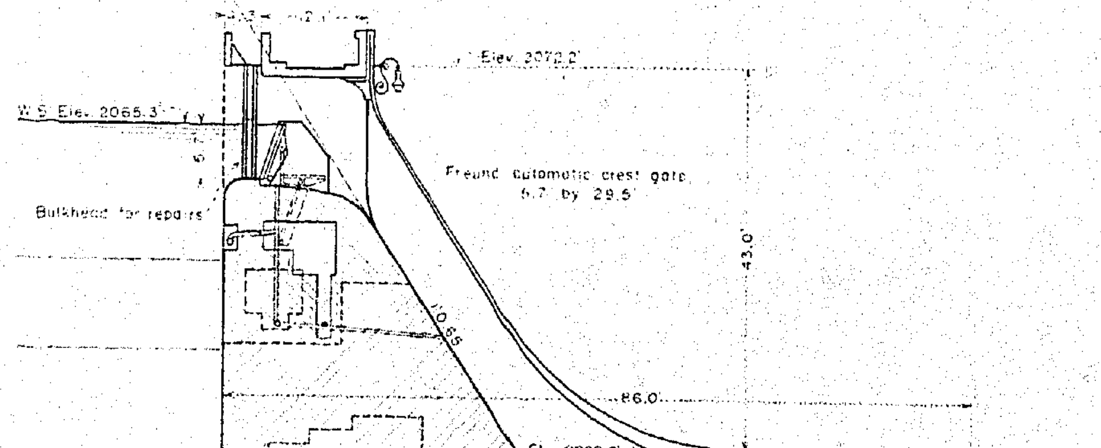



FIGURE 163 - CROSS SECTION OF THE LANGMANN DAM WITH A SCHOKLITSCH STILLING POOL.

HYD-20



X-D-2038

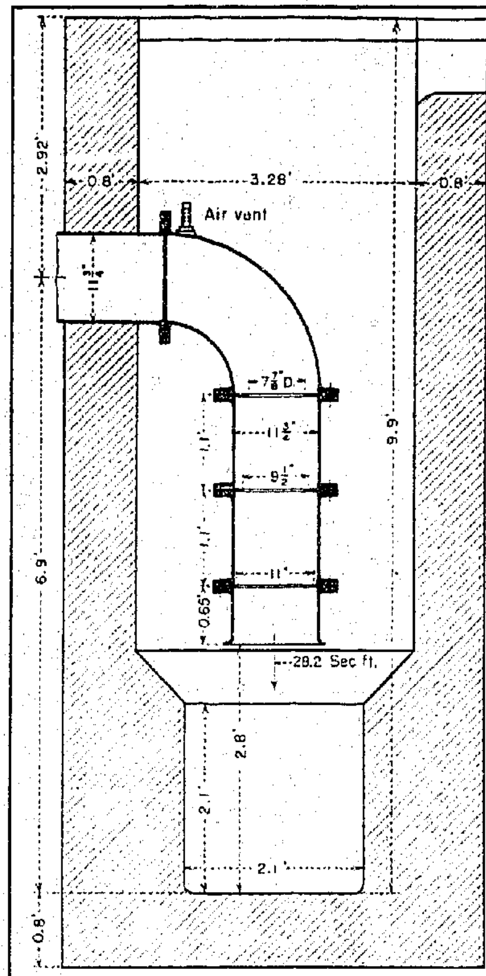



FIGURE 167 - POEBING ENERGY DISSIPATOR AT THE ROHRENDORF POWER PLANT NEAR BERNECK, BAVARIA. HEAD = 387.1 FT.; DISCHARGE = 28.2 SEC.FT., SQUARE DESIGN.

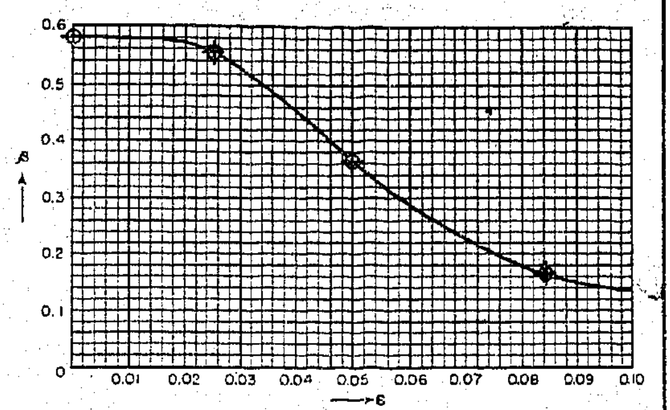



FIGURE 162 - RELATION BETWEEN ε AND β

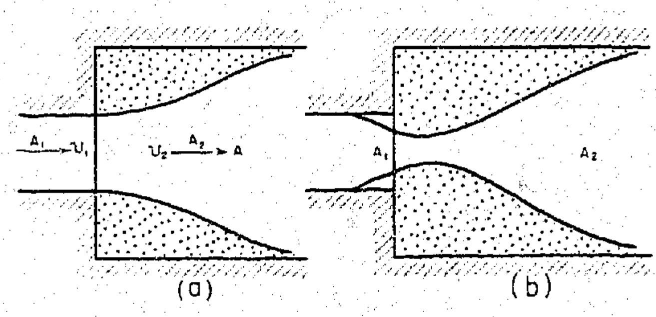



FIGURE 166 - ENERGY DISSIPATION AT A SUDDEN ENLARGEMENT.

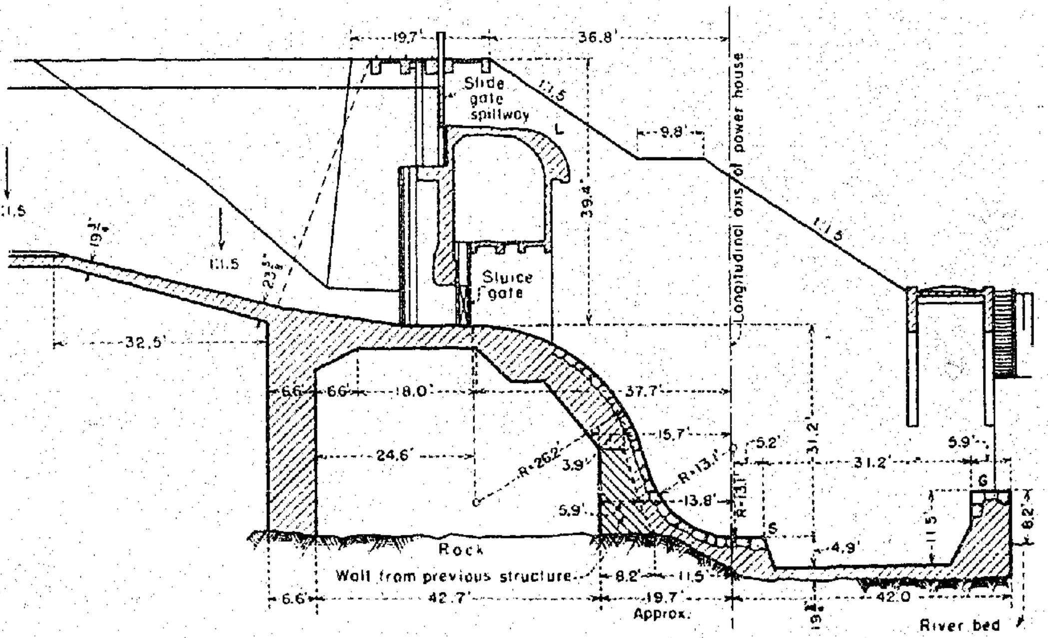



FIGURE 165 - CROSS SECTION OF THE SPILLWAY AND SLUICEWAY OF THE POWER PLANT ON THE MUR RIVER AT PERNEGG, AUSTRIA, SHOWING A SCHOKLITSCH STILLING POOL.

HYD-20



X-D-2039

O. Poebing[35] designed an energy dissipator in which he placed a number of pipe orifices whose diameters increased toward the end of the pipe, for the Rohrendorf power plant (figure 167). If these orifices are far enough apart, the energy loss at each can be computed from equation 90. If they lie close to each other, as in Poebing's energy dissipator, the jet does not expand to the full cross section of the pipe between the orifices, and the loss of energy resulting from each "impact" is smaller than that computed by equation 90. However, torus-shaped rollers form around the jet and increase the amount of energy dissipated. The kinetic energy remaining in the jet is further dissipated at the exit of the pipe where the water discharges into a concrete tank.

An effective method of dissipating the kinetic energy of the water issuing from a pipe, according to experiments by the author, is afforded by the energy dissipator shown in figure 168. Furthermore, the basin can be used as a water rheostat for loading the generator during acceptance and check tests. It can be constructed in the form of a well.

Another type of energy dissipator employed in connection with the waste water from the relief valve of the pressure regulator of a turbine is shown in figure 169. It is used at an American 2460-foot head, 40,000-horsepower Pelton turbine installation[36].

Figure 170 shows a nozzle energy dissipator designed by J. Heyn. No installation of this type is known.

Finally, figure 171 shows the energy dissipator designed by F. Kreuter[37], the so-called Kreuter-brake. In addition to the impact loss, energy is dissipated by the torus-shaped ground and surface rollers.

### g. Escher, Wyss and Company's Spiral Eddy Energy Dissipator

Escher, Wyss and Company[38], turbine manufacturers, employ the principle of energy dissipation involved in the large difference in velocity between two adjacent layers of water, as is shown in figure 172. The water is directed from two sets of guide vanes, L-1 and L-2, so that the two jets rotate in oposite directions as they pass down the draft-tube-like pipe. In order to prevent corrosion by cavitation, the flow is aerated by means of a single inlet at the center of the pipe.

### h. Energy Dissipation in Pipes and Flumes

Water flowing through a pipe or flume at high velocities experiences a considerable energy loss, due to the friction along the wetted perimeter. If this rapid flow is mixed with air, as, for instance, in an aerated pipe or sluiceway, it is retarded as a consequence of the large amount of air absorbed.

#### 1. Energy Dissipation in an Unaerated Pipe at High Velocity

Several experiments conducted by the author prove conclusively that the loss of energy in smooth brass pipes can be extraordinarily high. Velocities as high as 82 feet per second and Reynolds' numbers as high as 500,000 were reached. The results of these experiments fit best the formula for the friction loss given by H. Blasius[39]; namely,

$$H_f = \frac{\quad}{2g} \frac{V^2 L}{D R^{1/4}} \quad \text{. . . . . . . . . . . . . . . . . . . (91)}$$

in which V is the velocity, L the length of the pipe, D its diameter, and R Reynolds' number, or:

$$R = \frac{VD}{\quad} \quad \text{. . . . . . . . . . . . . . . . . . . . . . (92)}$$

a is a coefficient. Blasius gives it a value of 0.3164, while the author's tests give an average value of 0.3160 for Reynold's numbers from 2,200 to 500,000 and thus verify the Blasius coefficient.

 is the kinematic viscosity or:

$$= \frac{\eta g}{\gamma} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . (93)}$$

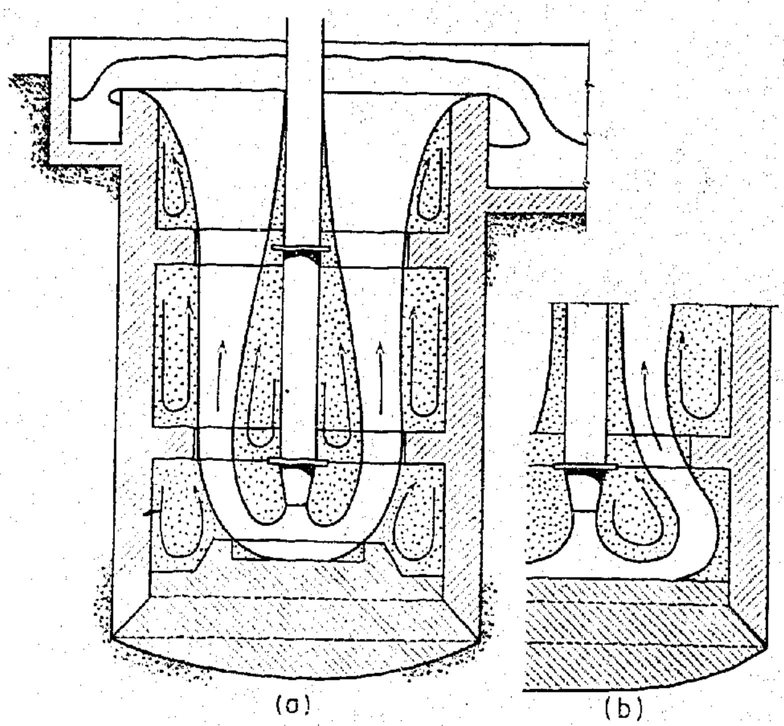



FIGURE 168.- NOZZLE ENERGY DISSIPATOR.

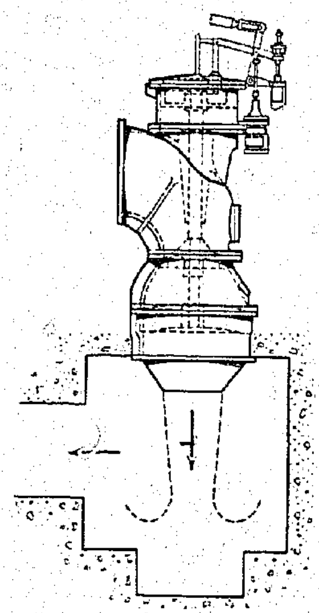


FIGURE 169-PRESSURE REGULATOR EMPLOYING A NOZZLE ENERGY DISSIPATOR.

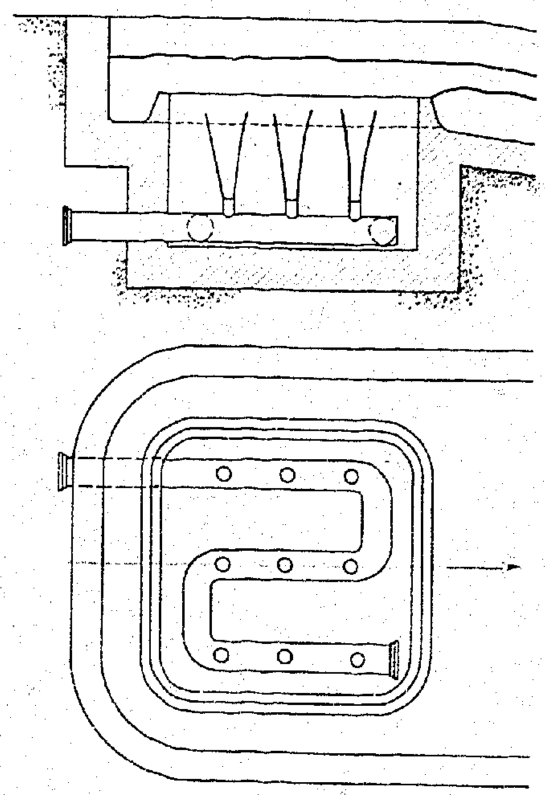


FIGURE 170 - NOZZLE ENERGY DISSIPATOR DESIGNED BY J. HEYN.

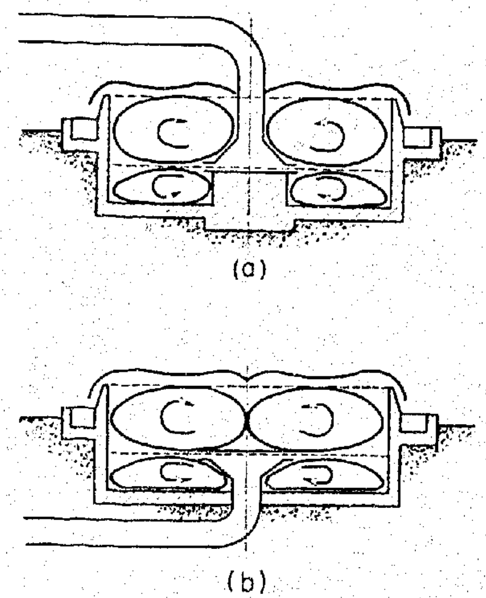



FIGURE 171 - KREUTER ENERGY DISSIPATOR.

in which $\gamma$ is the density of water in pounds per cubic foot, g the acceleration of gravity in feet per second, and $\eta$ the viscosity of the water in pounds/seconds per square foot, which can be computed from the following formula due to J. L. Poiseuille[33]:

$$\eta = \frac{0.00003716}{0.4712 + 0.01435\ T + 0.0000682\ T^2} \quad \ldots\ldots\ldots\ldots \quad (94)$$

where T is the temperature in degrees Fahrenheit.

If water at 14° C. (57.2° F.) flows through a pipe one-tenth meter (3.94 inches) in diameter and at a velocity of 25 meters per second (82 feet per second), the friction loss will be 2.65 meters of water per meter length of pipe (2.65 feet per foot). At high velocities of flow, such a pipe is suitable for an energy dissipator. However, if the diameter of the pipe is increased to one meter (3.28 feet), other conditions remaining the same, the friction loss is only 0.15 meter per meter (0.15 foot per foot). Therefore, smooth pipes are suited to the purpose of energy dissipators only if they are small and the velocity high.

The energy remaining at the end of the pipe can be dissipated in a device of the proper type; for example, a nozzle or counterflow energy dissipator.

## 2. Energy Dissipation in Aerated Pipes and Open-Channel Chutes

Water in an aerated pipe flowing partly full or in a steep chute absorbs a large amount of air at a certain velocity. This mixture of air and water presents a foamy appearance and travels much more slowly than pure water; in other words, in accordance with different laws. R. Ehrenberger[40], in tests on models, found that the critical velocity at which air is absorbed lies between 11.5 and 14.8 feet per second. The mechanics of the penetration of air into rapidly flowing water is not yet fully understood. The author succeeded in obtaining in the laboratory several photographs which show the extent to which air has penetrated into the upper surface layers of the flow. However, it is known that these upper layers of flow will absorb air only if a vacuum exists in them.

The maximum velocity heretofore observed at which a water-air mixture flows in a steep chute is 77.1 feet per second, and that portion of the water-air mixture occupied by the water (see table 15) amounts to from 20 to 55 percent of the total volume.

R. Ehrenberger[40] has investigated the motion of a water-air mixture in the laboratory and has derived the following formula, based on his measurements, which is in good agreement with observations on aerated penstocks and chutes at water-power plants:

$$V = 97 \; R^{0.52} \; (\sin \alpha)^{0.4} \quad \text{. . . . . . . . . . . . . . (95)}$$

in which V is the average velocity in feet per second of the mixture in a smooth chute; R the hydraulic radius, in feet, for the entire flow (cross-sectional area, $A_1$, divided by the wetted perimeter, P); and $\alpha$ the angle between the chute and the horizontal. The proportion of the whole volume, $\mu$, in percent, occupied by the water expressing R in feet is as follows:

$$\text{if } \sin \alpha < 0.476, \text{ then } \mu = \frac{Q}{AV} = 0.42 \; R^{-0.05} \; (\sin \alpha)^{-0.26} \quad \text{. . (96)}$$

$$\text{if } \sin \alpha > 0.476, \text{ then } \mu = \frac{Q}{AV} = 0.30 \; R^{-0.05} \; (\sin \alpha)^{-0.74} \quad \text{. . (97)}$$

The discharge, Q, in second-feet can be calculated from the following empirical equations:

$$\text{if } \sin \alpha < 0.476, \text{ then } Q = 41.3 \; \frac{A^{1.47}}{P^{0.47}} \; (\sin \alpha)^{0.14} \quad \text{. . . . . . (98)}$$

$$\text{if } \sin \alpha > 0.476, \text{ then } Q = 28.9 \; \frac{A^{1.47}}{P^{0.47}} \; (\sin \alpha)^{0.34} \quad \text{. . . . . . . (99)}$$

Chutes for bypassing water have been built with smooth concrete surfaces or have been lined with wood.

TABLE 15. OBSERVATIONS ON THE FLOW IN CHUTES

| Location of Chute | Length of Chute, L | Head, H | Slope | Width of Chute | Depth of Chute | Discharge of Water Only | Cross-Sectional Area, Air + Water | Discharge of Air + Water | Column 6 / Column 8 | Actual Observed Velocity | Reference |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Feet | Feet | | Feet | | Ft.$^3$/sec. | Feet$^2$ | Ft.$^3$/sec. | Percent | Ft.$^3$/sec. | |
| Glenwood Power Plant | 236 | 197 | 25° 20' | 12.5 | 3.9 | 494 | - | - | - | 68.9 | Engineering Record 1907 |
| Kitting | - | 52 | - | 59 | - | 44 | - | - | - | 39.4 | Wasserkraft-Jahrbuch 1924, p. 392 |
| Dago | - | - | | 3.3 | - | 49 | 2.2 | - | 64 | 33.1 | Ehrenberger, R: Wasserwirtschaft 1930, p. 573 |
| " | - | - | 13° 50' | 3.3 | - | 26 | 1.4 | - | 66 | 28.2 | |
| Benkok | - | - | 13° 50' | 3.3 | - | 212 | 7.2 | - | 38 | 77.1 | |
| " | - | - | - | 3.3 | - | 102 | 4.0 | - | 38 | 67.2 | " |
| Mallnitz | - | - | - | 6.8 | - | 166 | 8.4 | - | 54 | 68.6 | |
| | - | - | - | 6.8 | - | 134 | 7.9 | - | 54 | 63.0 | " |
| Ruetz Power Plant | 308 | 427 | 38° | Trapezoidal shape Top width - 15.1 Bottom - 8.2 | 5.2 | 36 | 2.6 | 175 | 20 | 68.6 | Innerebner, K.: Wasserkraft und Wasserwirtschaft 1924, p. 410 |
| " | 308 | 427 | 38° | | 5.2 | 52 | 3.0 | 205 | 25 | 65.9 | |
| " | 308 | 427 | 38° | | 5.2 | 56 | 3.9 | 209 | 27 | 53.5 | " |
| " | 308 | 427 | 38° | | 5.2 | 98 | 3.8 | 249 | 39 | 70.2 | " |
| " | 308 | 427 | 38° | | 5.2 | 132 | 7.4 | 519 | 25 | 70.2 | " |
| " | 308 | 427 | 38° | | 5.2 | 159 | 8.3 | 583 | 27 | 70.2 | " |
| " | 308 | 427 | 38° | | 5.2 | 191 | - | - | 64 | 70.2 | Wasserkraft-Jahrbuch 1924, p. 383 |

TABLE 16. OBSERVATIONS ON THE FLOW IN PIPE CHUTES

| Location and Material of Chute | Length of Pipe Chute, L Feet | Head, H Feet | Diameter, D Feet | Maximum Discharge, Q Sec.-Feet | Average Velocity V Feet/sec. | Reference |
|---|---|---|---|---|---|---|
| Lontsch Power Plant Welded Steel | 1089 | - | 2.3 | 282 | 65.6 | - |
| La Derniere Iron | 2533 | 778 | 2.8 to 4.6 | 3.53 | 60.0 | - |
| Alz Power Plant Iron | 492 | 174 | 9.8 | 1060 to 1410 | - | Wasserkraft-Jahrbuch 1925,-p. 390 |
| Toging Three Iron Pipes | 466 | - | 7.2 to 9.8 | 2650 | 47.6 | Wasserkraft-Jahrbuch 1925, p. 395 |

Pipe chutes may be constructed of either iron or wood. An example of an iron-pipe chute is shown in figure 178. Air can be introduced into the pipe through aeration pipes. The high velocity flow can be prevented from entering an aeration pipe by means of a metal baffle plate such as that shown in the upper right-hand corner of figure 178.

When water issues from the end of a pipe or open channel chute at a high velocity, it will produce a deep scour unless some further means of protection is provided. Figure 179 shows measurements of the scour produced at the end of a pipe chute[41]. In order to prevent this scour, the water emerging from the chute should be conducted through a nozzle or counterflow energy dissipation or any other device for dissipating energy, such as these shown in figures 146 and 180.

### i. Miscellaneous Energy Dissipators

The energy dissipators already described are designed to prevent dangerous scour. There are, however, other energy dissipators designed to dissipate the kinetic energy of the water more or less completely with no reference to scour.

#### 1. Energy Dissipators for Filling and Emptying Lock Chambers

Culverts with laterals were at one time almost exclusively used for filling locks. If the lock floor was paved, the sudden increase in cross section from the culvert to the lock chamber served to dissipate the kinetic energy of the inflow. In particular instances, if the foundation of the lock was sufficiently impermeable, it was attempted to spare the cost of concreting the chamber floor; in such cases considerable scouring of the bottom occurred near the exit of each inlet lateral. This necessitated concreting of the floor.

In recent times, in order to spare the cost of culverts and laterals, the tendency has been to eliminate them altogether and to use slide gates for both filling and emptying the lock chamber. When slide gates are used, some means must be adopted to prevent dangerous flow conditions both in the lock chamber itself and also at the outlet.

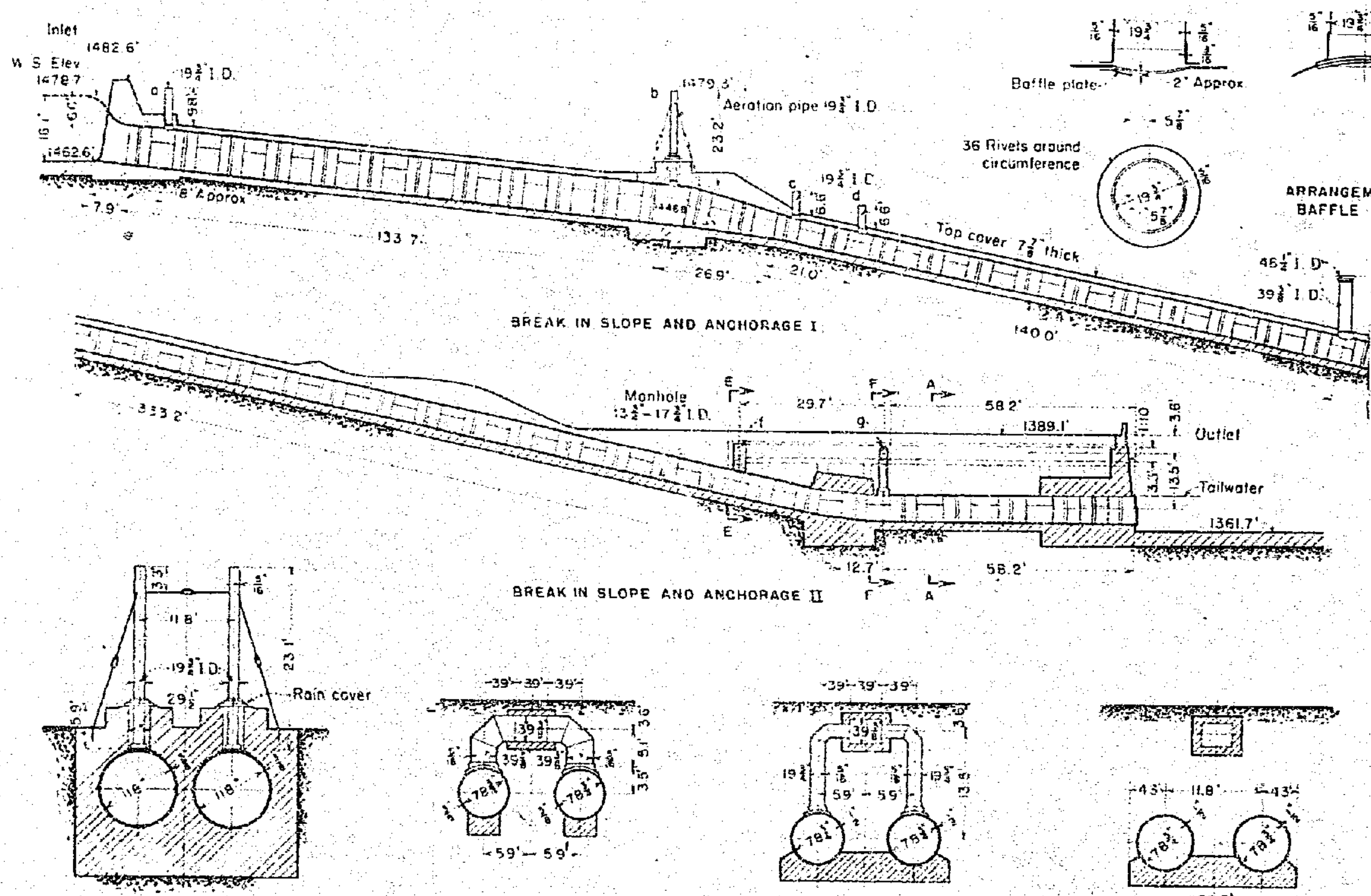



FIGURE 178 — THE PIPE CHUTE AT MARGARETHENBERG (ACCORDING TO K. DANTSCHER, BAUINGENIEUR, 1921).

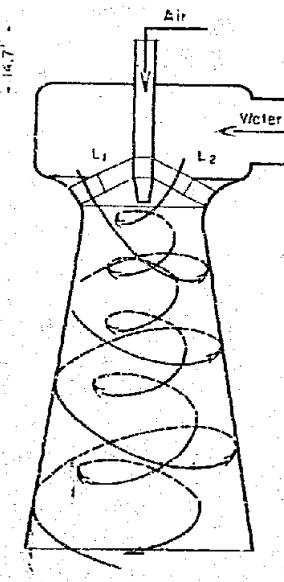



FIGURE 172 — ESCHER, WYSS AND COMPANY'S SPIRAL EDDY ENERGY DISSIPATOR.

HYD-20

X-D-2041

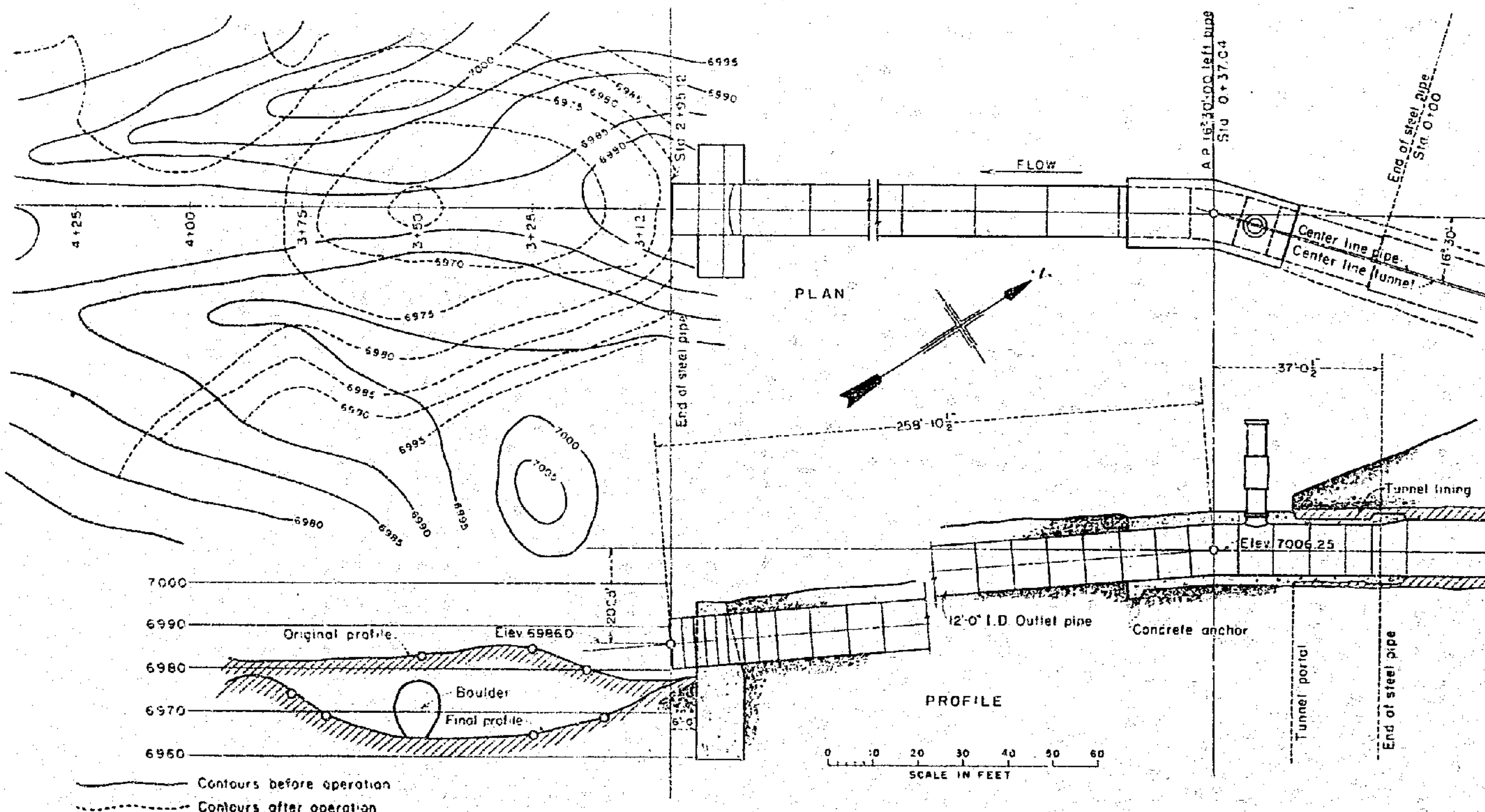



FIGURE 179 - PLAN AND PROFILE OF FLORENCE LAKE TUNNEL SHOWING SCOUR
(SEE TRANS. A.S.C.E., Vol. 93, 1929, P 515).

HYD-20.

X-D-2042

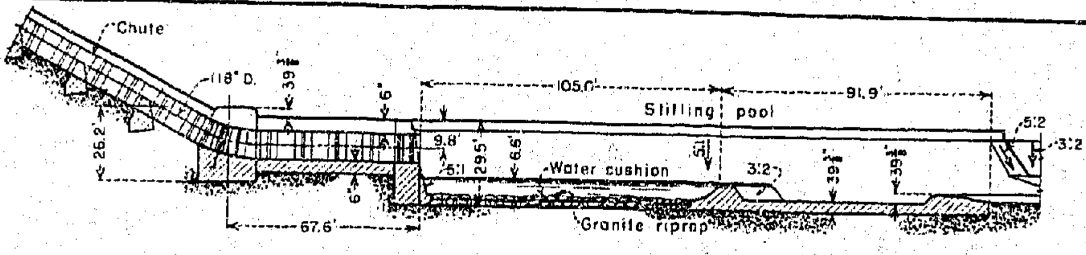



FIGURE 180 - PIPE CHUTE WITH ENERGY DISSIPATOR AT THE BURGHAUSEN POWER PLANT (ACCORDING TO F. KENNERKNECHT).

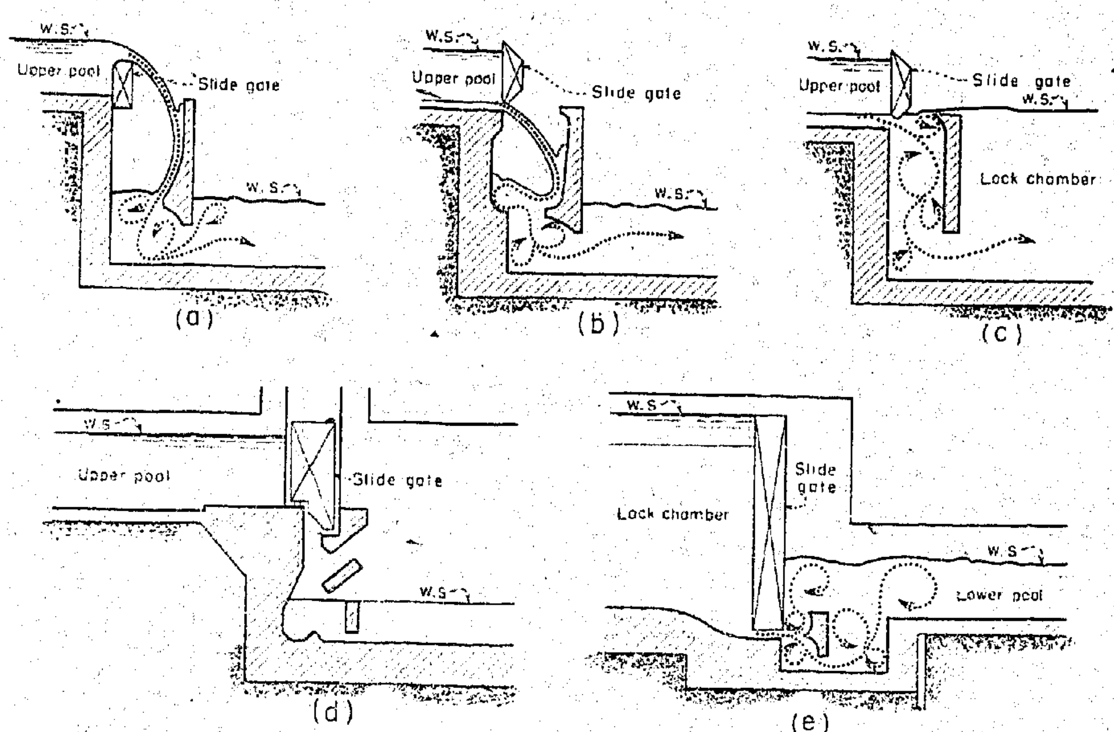



FIGURE 182 - ENERGY DISSIPATORS FOR FILLING AND EMPTYING LOCK CHAMBERS (a,b,c, AND e, ACCORDING TO BURKHARD, AND d, ACCORDING TO VAN RINSUM).

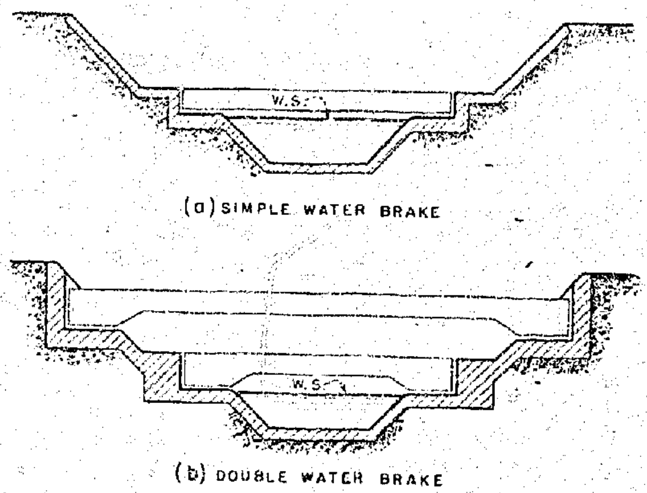



FIGURE 185 - WATER BRAKES DESIGNED BY A. RAMSHORN.

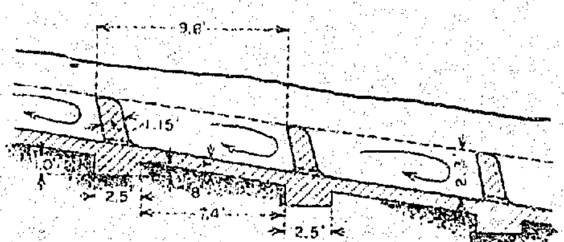



FIGURE 183 - LONGITUDINAL SECTION OF A CHUTE WITH BAZIKA SILLS.

HYD-20



X-D-2043

Figures 182a to c, and e, show examples of energy dissipators designed by E. Burkhardt[42] for this purpose, and figure 182d shows another example designed by Van Rinsum[43]. Both designs are based on model tests. Their main feature consists of the use of sills and baffles.

## 2. Energy Dissipators in Flumes

The slope of a flume for a given head should be made as large as possible in order to reduce the length of the flume and hence the cost of construction. E. Bazika[44] has placed herringbone ribs across the floor of the flume (figure 183). These cause ground rollers to form along the entire length of the flume. These ribs are constructed of granite or concrete and have proved entirely satisfactory on certain reaches of the Elbe in Bohemia. Even with very high heads, the use of these Bazika ribs has permitted a channel slope of 1:10. In order to prevent floating objects from adhering to the ribs, wood skids about 30 inches wide were laid on top of the ribs longitudinally along the flume throughout its entire length.

## 3. Energy Dissipation in Concrete Flumes by Means of the So-Called "Ramshorn Water-Brake"

The low-water flow in a concrete flume should have a tractive force of at least 0.05 pound per square foot in order to prevent silting of the flume, and during periods of high water, according to the experience at the power plant at Essen, the tractive force should not increase above 0.5 pound per square foot in order to prevent damage to the concrete lining of the flume. In order to retard the flow during flood times without retarding it during periods of low water, A. Ramshorn[45], on the basis of model tests, proposed the use of the "water-brake" whose construction is shown in figures 185. These water-brakes reduce the velocity of flood flow by 40 to 75 percent of that of an unobstructed flow. To prevent large waves, they should be placed every 65 feet for a slope of 0.005, every 130 feet for a slope of 0.003, and every 165 feet for a slope of 0.002. They are constructed from reinforced concrete or, in smaller channels, from wood. They have proved entirely satisfactory at the Emscher Company's plant at Essen.

Furthermore, the flow at drops in unlined canals can be controlled by water-brakes. Ramshorn even placed a water-brake upstream from a drop in order to reduce the head; otherwise scour might develop. Immediately below the drop a double brake was placed, and several simple brakes were placed farther downstream. With such an arrangement, damage to the concrete lining, often occurring at unprotected drops, can be prevented.

### 4. The Energy Dissipator at the Emscher Company's Plant at Graz

Special types of energy dissipators have been constructed by the Emscher Company[45] at several drops in an inlet flume in order to protect the lining. The action of these dissipators is similar to that of the Ramshorn water-brake; they do not retard low-water flow, but during flood flow they cause a copious development of rollers.

- - -

REFERENCES


1. Prandtl, L. Die Entstehung von Wirbeln in einer Flüssigkeit mit Kleiner Reibung (The Origin of Vortices in a Fluid with Small Internal Friction); Zeits. für Flugtechnik und Motorluftschiffahrt, vol. 18, 1927, no. 21.

2. Rehbock, T. Betrachtung über Abfluss, Stau und Walzenbildung bei fliessenden Gewässern (Considerations Concerning Discharge, Backwater and Roller Formation); Julius Springer, Berlin, 1917.

3. Safranez, K. Energieverzehung der Deckwalze (Energy Dissipation in a Hydraulic Jump); Bauingenieur, vol. 11, 1930, p. 352.

4. Kozeny, J. Wassersprung und Energieumwandlung (The Hydraulic Jump and Energy Transformation). Wasserkraft und Wasserwirtschaft, vol. 27, 1932, p. 9.

5. Nemenyi, P. Wasserbauliche Strömungslehre (Principles of Flow for Hydraulic Structures); J. A. Barth, Leipzig, 1933.

6. Schoklitsch,A Kolkbildung unter Überfallstrahlen (Scouring Caused by Overfalling Nappes); Die Wasserwirtschaft, vol. 25, 1932, p. 341.

7. Roth, H. Kolkerfahrungen und ihre Berüchsichtigung bei der Ausbildung beweglicher Wehre (Scour Phenomena and Their Consideration in the Construction of Movable Dams); Schweizerische Bauzeitung, vol. 70, 1927, p. 18.

8. Kuich, K. Über die Regulierung der nicht ärarischem Gewässer Oberösterreichs in der Nachkriegszeit (The Regulation of Non-Public Streams in Upper Wasserwirtschaft, vol. 23, 1930, p. 513.

9. Meyer-Peter,E. Die technische Entwicklung der hydroelektrischen Anlagon in der Schweiz, usw. (Technical Developements at the Hydroelectric Plants in Switzerland, etc.); Schweizerische Bauzeitung, vol. 89, 1927, p. 107.

10. Gregory,J.H. , Hoover, G. B. and Cornell, C. B. The O'Shaughnessy Dam and Reservoir; Trans. A. S. C. E., vol. 93, 1929, p. 1428.

11. Oram, J. P. Concentrated Flow Erodes Rock below Wilson Dam; Engineering News-Record, vol. 98, 1927, p. 190.

12. Lehr, J. K. Natürliche Sturzbecken (Natural Stilling Pools); Wasserkraft und Wasserwirtschaft, vol. 19, 1924, p. 101.

13. Rehbock, T. Die Ausbildung der Sturzbetten bei Überfallwehren und Talsperren (The Design of Stilling Pools for Overfall Weirs and Dams); Communications of the First Congress on Large Dams, Stockholm, 1933.

14. Thomas, B.F. and Watt, D. A. Improvement of Rivers, John Wiley and Sons, Ed. 2, 1913, p. 375.

15. Anonymous Die Dürchfuhrung der Bauarbeiten beim ersten Ausbau der Wasserkraftanlagen der Mittleren Isar A. G. München (Constructional Procedures for the First Hydroelectric Project of the Middle Isar Company, Munich); Published by the Mittlere Isar A. G., no.3.

16. Schäfer, A Die Energievernichtung an Wehranlagen (Energy Dissipation at Dam Sites); Die Bautechnik, vol. 7, 1929, p. 263.

17. Rehbock, T. Die Bekämpfung der Sohlenauskolkung bei Wehren durch Zahnschwellen (Combating River-Bed Scour at Dams by Means of Dentated Sills); Festschrift der Technische Hochschule Karlsruhe, 1925. See also: Zeits. des V.D.I., vol. 69, 1925, p. 1382 and also: Schweizerische Bauzeitung, vol. 87, 1926, pp. 27 and 85.

Also: Die Verhütung schädlicher Kolke bei Sturzbetten (Prevention of Destructive Scour at Stilling Pools); Bauingenieur, vol. 9, 1928, nos. 4 and 5.

18. Thürnau, K. Anlage zur Wasserkraftvernichtung und zur Verhütung von Auskolkungen unterhalb der Toskammern von Wehren oder ähnlichen Anlage in geschiebeführenden Wasserläufen (Devices for Power Dissipation and Prevention of Scouring below Tumble Bays or Similar Structures on Silt-Laden Streams); Die Bautechnik, vol. 8, 1930, p. 498. (Patentbericht)

19. Ludin, A. Kolkverhütung an Wehren (Scour Prevention at Dams); Zeits, des V.D.I., vol. 71, 1927, p. 161.

20. Hofbauer, R. Ein Mittel zur Bekämpfung der Wirbelbildung und Kolkbildung unterhalb der Stauwerke (A Means of Combating the Formation of Whirlpools and Scour Below Dams); Zeits, des Österreich. Ing. und Arch. Vereines, vol. 67, 1915, p. 109.

21. Schoklitsch, A. Kolkbildung und Kolkabwehr (Scouring and Prevention of Scour); Wasserkraft und Wasserwirtschaft, vol. 23, 1928, p. 217.

22. Läufer, A. Kolksichere Sturzböden (Scour-Proof Stilling Pools); Wasserkraft und Wasserwirtschaft, vol. 18, 1923, p. 23.

23. Smith, G.E. Trans. A.S.C.E. vol. 73, 1911.
also:
Wegmann, E. The Design and Construction of Dams; John Wiley and Sons, Ed. 7, 1922, p. 413.

24. Rümelin, T. Wasserkraftanlagen I (Water Power Plants, I); Sammlung Göschen, no. 665, p. 87.

25. Kennerknecht, F. Die Energievernichtungsanlage des Innwerkes (Device for Energy Dissipation of the Inn Power Plant); Wasserkraft und Wasserwirtschaft, vol. 21, 1926, p. 148.

26. Corrazza, M.H. Das Grosskraftwerk Kardaun am Eisack (The Large Kardaun Power Plant on the Eisack River); Wasserkraft und Wasserwirtschaft, vol. 24, 1929. p. 39.

27. Meyer-Peter, E. Die hydraulischen Modellversuche für das Limmatkraftwerk Wettingen der Stadt Zürich (Hydraulic Model Experiments for the Wettingen Power Plant of the City of Zurich); Schweizerische Bauzeitung, vol. 89, 1927, p. 275.

28. Kennison, K. The Hydraulic Jump in Open Channel Flow at High Velocity, Trans. A.S.C.E., vol. 80, 1916, p. 338.

29. Safranez, K. Wechselsprung und Energievernichtung des Wassers (The Hydraulic Jump and Energy Dissipation in Water); Bauingenieur, vol. 8, 1927, p. 898.

Also: Untersuchungen über den Wechselsprung (Investigation of the Hydraulic Jump); Bauingenieur, vol. 10, 1929, nos. 37 and 38.

30. Safranez, K. Das Kraftwerk Wäggital (The Wäggital Power Plant); Schweizerische Bauzeitung, vol. 98, 1931, p.307.

31. Schoklitsch, A Energievernichter (Energy Dissipators); Wasserkraft und Wasserwirtschaft, vol. 21, 1926, p. 108.

32. Smetana, J. Podhrazi Vodni Nadrze na Rece Blanici u Husince (The Stilling Pool of the Dam on Tributary of the Vltava River near Husinec); Bul. of the T. G. Masaryk Institute of Hydraulic Research at Prague, no. 10, 1934.

33. Forchheimer, P. Hydraulik (Hydraulics); Teubner, Leipzig, and Berlin, Ed. 3, 1930, p. 319.

34. Banninger, K. Studien und Versuche über Umsetzung von Geschwindigkeit in Drück, usw. (Studies and experiments on the Converting of Velocity Head into Pressure Head, etc); Zeits. für das gesamte Turbinenwesen. vol. 3, 1906, p. 12.

35. Poebing, O. Ein neuer Energievernichter (A New Energy Dissipator); Wasserkraft und Wasserwirtschaft vol. 19, 1924, p. 373.

36. Poebing, O. Ein amerikanisches 40,000 PS - Peltonturbinenaggregat für 725 m. Gefälle (An American 40,000 Metric Horsepower Pelton Turbine Installation for a 725 Meter Head); Wasserkraft und Wasserwirtschaft, vol. 23, 1928, p. 257.

37. Kreuter, F. Hydraulische Bremse (Hydraulic Brakes); Wasserkraft und Wasserwirtschaft, vol. 15, 1920, p.65.

38. Anonymous Einrichtung zur Unschädlichmachung Kinetischer Flüssigkeitsenergie usw. Patentschau, Escher, Wyss et Comp. (Arrangement for Dissipating the Kinetic Energy of Water, etc. Patent Review, Escher, Wyss and Co.); Die Bautechnik, vol. 11, 1933, p. 410.

39. Blasius, H. Das Ähnlichkeitsgesetz bei Reibungsvorgängen (The Law of Similitude for Frictional Processes); Zeits. des V.D.I., vol 56, 1912, p. 639.

40. Ehrenberger, R. Eine neue Geschwinfigkeitsformel für kunstliche Gerinne mit starken Neigungen (Schussternen) usw. (A New Velocity Formula for Artificial Channels Having Steep Slopes (Chutes), etc.); Die Wasserwirtschaft, vol. 23, 1930, p. 573.

41. Steele, I. C. and Monroe, R. A. Baffle Pier Experiments on Models of Pit River Dam; Trans. A.S.C.E., vol. 93, 1929, p. 451.

42. Burkhardt, E. Beobachtungen und Erfahrungen an der umlauflösen Doppelschleuse Ladenburg des Neckarkanals (Observations and Experience on the Double Discharge Sluice at Ladenburg on the Neckar Canal); Die Bautechnik, vol. 6, 1928, p. 447.

43. Rinsum, A. van, Die Twenthe-kanäle (The Twenthe Canals); Die Bautechnik, vol. 11, 1933, p. 588.

44. Bazika, E. Einrichtung zur Verringerung der Durchflussgeschwindigkeit in kunstlichen Gerinnen (Device for Decreasing the Discharge Velocity in Artificial Channels); Austrian Patent No. 56469, Class 84.

45. Ramshorn, A. Die Energievernichtung bei Abstürzen und Schussstrecken in offenen Abwasserkanälen (Energy Dissipation in Stilling Pools and Open Channel Chutes); Die Bautechnik, vol. 10, 1932, p. 139.

AUTHOR INDEX

## AUTHOR INDEX (Continued)

SUBJECT INDEX (Continued)