CORRESPONDENCE

The Secretary,
The Institution of Civil Engineers.

CONTROL OF SEEPAGE THROUGH FOUNDATIONS AND ABUTMENTS OF DAMS

DEAR SIR,

In his Paper on "Control of seepage through foundations and abutments of dams" (*Géotechnique*, 11:3:161–182), Professor Arthur Casagrande has presented an analysis of the relative merits of grout curtains and foundation drains under concrete dams in his usual practical and precise manner. The Writer would like to add some data on observed uplift pressures under existing dams to the information included in the article. There are several corollaries to the problem, which also focus attention on Casagrande's recommendations regarding the need for re-evaluation of the current approach to control of seepage under concrete dams.

UPLIFT PRESSURES AT GIBSON AND OWYHEE DAMS

Average uplift pressures observed at Gibson and Owyhee dams by the U.S. Bureau of Reclamation are included in Fig. 12 of Casagrande's Paper. Observations under some of the blocks show extraordinary divergence from the average pressures shown in this figure, and it is interesting to examine them in light of the actual foundation treatment. This additional information was obtained from "Summary of uplift pressures at Bureau of Reclamation dams." Technical Memorandum 636, and other Bureau of Reclamation publications (Richardson, 1948; U.S. Bureau of Reclamation, 1949)*.

Gibson Dam in Montana, a 198-ft high concrete arch structure, was constructed during the period 1926 to 1929. Uplift pressure measurements at 2-week intervals were made from May 1930 to September 1942. The dam is founded on crystalline limestone in regular beds, which dip upstream 70° to 86°. A single row of grout holes at the upstream face and shallow drain holes parallel to the grout curtain, composed the principal foundation treatment. The drain holes connect with horizontal tile drains embedded in the dam, which presumably discharge at the downstream face of dam. However, records of the foundation treatment operations are not available (Richardson, 1948), and consequently the adequacy and performance of these provisions can be evaluated only by reviewing the uplift pressure measurements.

Uplift pressures observed under Gibson Dam at three locations, are shown in Fig. 1, which is reproduced from Fig. 26 of (Richardson, 1948). The holes for uplift observation pipes were drilled only 18 in. into the rock. In all sections of the dam shown in Fig. 1, Pipe D is the observation point closest to the foundation drain wells. Since uplift pressures appreciably greater than tailwater pressure occur downstream of the grout curtain, it is obvious that the grouting was not significantly effective in reducing uplift at any of the locations considered. The three sections of the dam shown in Fig. 1 illustrate three different cases of uplift phenomenon under a concrete dam:

Fig. 1(a): The maximum measured uplift pressure gradient varies almost linearly from reservoir level at the upstream face to tailwater level about 25 ft from the toe. The downstream portion of the foundation contact, for about 25 ft, seems to be either open and accessible

* See references on p. 348.

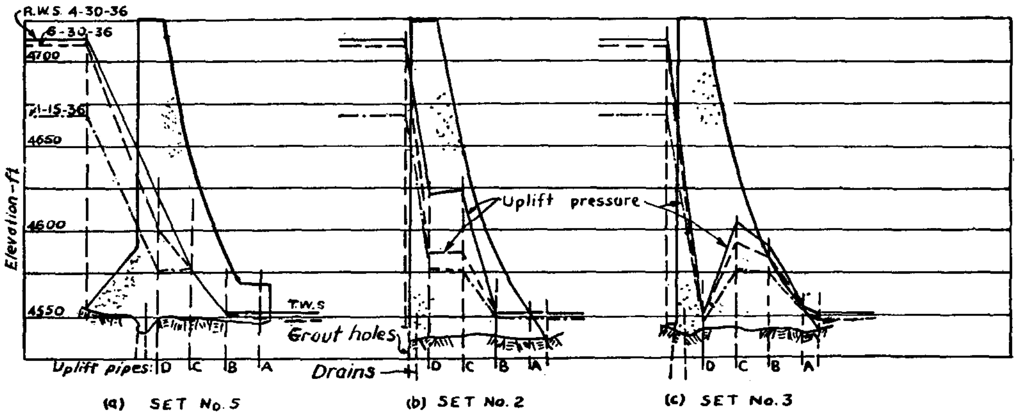

Fig. 1. Uplift pressures measured at foundation of Gibson dam in 1936. (Reproduced from Fig. 26 of Ref. 3.)

to the tailwater, or the top layer of the foundation is comparatively free-draining. The drain wells are either almost completely clogged, or are not sufficiently closely spaced and are also not deep enough.

Fig. 1(b): The partial inadequacy of the drain wells in spacing and depth, coupled with relative ineffectiveness of the grout curtain are also demonstrated in this case. The drains, however, are more effective than for the block shown in Fig. 1(a). Downstream portion of the foundation contact exhibits the same characteristics as in Fig. 1(a).

Fig. 1(c): The uplift pressure diagrams for this block resemble the examples shown in Figs 2 and 9(f) of Casagrande's Paper. Partly because the wells are not deep enough, and also because the foundation is stratified, uplift pressure again builds up downstream of the drains. Since the maximum uplift pressure does not exceed a uniform hydraulic gradient from headwater to tailwater, the increase in uplift is not caused by a blocking discontinuity in the stratified foundation.

Owyhee Dam (U.S. Bureau of Reclamation, 1949) in Oregon, U.S.A., was built in 1928–32. It is a 417-ft-high, curved, gravity dam, founded on porphyritic or glassy rhyolite. Throughout the entire foundation and abutment areas the rock is described (Richardson, 1948) as full of "innumerable joints, fissures, and fractures forming a wide-spread system of channels through which water could travel."

Original foundation treatment consisted of a single-line grout curtain about 200 ft deep and inclined drain wells about 50 ft deep parallel to but downstream of the grout curtain. For design of the dam, "uplift pressure was assumed to vary uniformly from full reservoir static head at the upstream face to one-half of full static head at the line of drains, located $6\frac{1}{2}$ ft downstream from the axis, and from there to tailwater static head at the downstream face" (Richardson, 1948).

Uplift pressure measurements at Owyhee Dam were conducted from June 1934 to May 1948. The measurement system consisted of four lines of pipes, a total of nineteen pipes, each pipe located in a hole drilled 2·5 ft into the rock. The measurements were made from inspection galleries located about 30 ft above maximum tailwater level.

Early measurements showed uplift pressures over a large area of the base of the dam considerably in excess of the design value. Evidence of seepage through one of the abutments was visible in an area 200–500 ft downstream of the dam. Corrective measures were carried out from June to December 1936. These took the form of an extensive network of vertical and slanting grout holes drilled from galleries and shafts in the dam and from the ground surface.

Some of the holes were as deep as 500 ft, the average depth being about 300 ft. The grout take ranged from ten to ninety sacks per foot of hole length, for an average of about thirty sacks. Some additional drain wells, about 50 ft deep, were also drilled from the lowest gallery, after completion of the corrective grouting.

As a result of the additional grouting and drainage, visible seepage through the abutment was reduced, but not completely eliminated (Richardson, 1948, Fig. 113). To evaluate the effect of these measures on uplift, pressures observed in January 1936, that is before the additional treatment, and in January 1943, 6 years after completion of the treatment, are shown in Fig. 2, for two locations. These are adapted from Figs 115, 117, 119, and 121 (Richardson, 1948). Line 1, Fig. 2(a) is located adjacent to the abutment subjected to the extensive supplementary treatment, while Line 3 is approximately at the centre line of the dam nearly 200 ft away from the abutment under discussion. Supplemental drain holes were drilled along Line 1 normal to the axis of the dam, but none were drilled along Line 3.

Fig. 2(a): Since no observations were made upstream of the grout curtain, the measured uplift pressures are only indirect indicators of the inefficacy of the grout curtain. The shallow drain holes were partially effective, since the uplift pressures measured in 1936 are substantially less than the hydraulic gradient at, and downstream of the drains. Uplift downstream of the drains, however, is considerably higher than the tailwater head.

Uplift pressures measured in 1943 after completion of supplementary grouting in the abutment and additional drain wells along Line 1, show an appreciable reduction from those observed seven years earlier.

Fig. 2(b): The general pattern of uplift pressure distribution at the foundation along Line 3 is similar to the pressures measured at Line 1 in 1936. The uplift pressures measured in 1943, after the completion of the supplemental grouting, do not show any decrease. On the

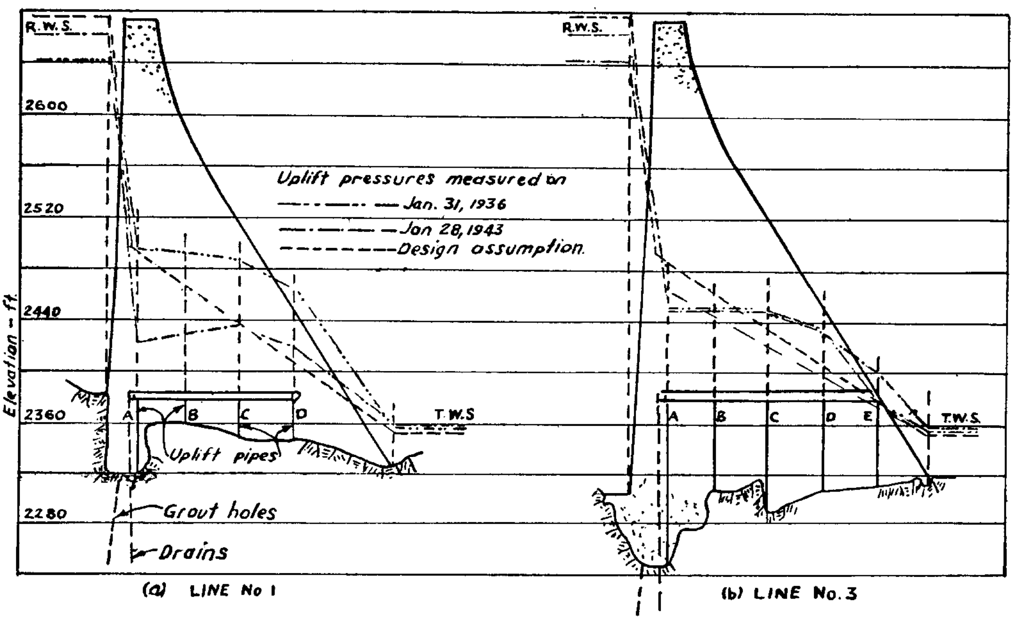

Fig. 2. Uplift pressures measured at foundation of Owyhee dam in 1936 and 1943. (Reproduced from Figs 115, 117, 119, and 121 of Ref. 5.)

contrary, there is a noticeable increase in the pressure under the downstream one-third of the base of the dam.

The following conclusions can be drawn from the measurements and foundation experience at Owyhee Dam:

1. There is no evidence of the beneficial effect of deep foundation grouting on uplift pressures under the dam.
2. The extensive, and expensive, additional grouting in the left abutment may have reduced the seepage flow through this area, but it did not completely stop it. Factual quantitative data are not available.
3. The drain wells along the axis of the dam, although not sufficiently deep, reduced uplift pressure under the dam appreciably.
4. Reduction in uplift pressure measured at Line 1 subsequent to completion of the supplemental foundation and abutment treatment should be attributed to the additional drain holes drilled along this line.
5. A network of drain wells drilled from tunnels driven into the left abutment would have provided a more effective control of the seepage flow through, and uplift pressure in the abutment than the additional grouting.

The uplift pressure measurements at Gibson and Owyhee dams also suffer the same shortcomings as the other examples cited by Professor Casagrande. The more important deficiencies of these programmes are the complete absence of seepage flow measurements, and the omission of uplift pressure observation pipes upstream of the grout curtain. The observation pipes, which are provided, generally penetrate only a few feet below the foundation and the measured pressures, therefore, indicate the pressures at the foundation contact only. Piezometers, which are so essential to the study of seepage through the foundation and the abutment of dams, have rarely been provided in the past.

TIME FACTOR AND THE SEEPAGE PHENOMENON

Two questions arise when influence of time on various aspects of the seepage and uplift phenomena is examined:

(*a*) How long would it take maximum uplift pressures to develop?

(*b*) Would uplift pressures fluctuate instantaneously throughout the area under pressure, with changes in reservoir and/or tailwater levels?

The time required for saturation of the foundation as a whole, or of particular strata, or of the dam-foundation contact, and consequent development of uplift pressure would depend upon the nature of the materials and the dimensions of the structure involved. The Writer has not come across any reliable observed data which could assist in the comparison of rate of development of uplift pressures in different types of foundation rock. Some foundations and abutments may be almost completely impermeable, and so formed that sufficient saturation and uplift build-up would not take place during the lifetime of the dam. Such formations, however, are rare. Most rock formations on which concrete dams are founded are only relatively impermeable and are susceptible to seepage penetration and saturation in a matter of months or a few years. A dangerous situation can develop at the dam where critical uplift pressures may occur 10 or 15 years after completion of the dam. Inspection at a project may slacken off after a number of years of apparently satisfactory performance and measurement of uplift pressures may be discontinued. A slow and gradual increase in uplift pressures may go unnoticed. In such cases, as indeed at all concrete dams, it is imperative to conduct the uplift measurements on a long-term basis, preferably as long as the dam is in existence and in operation.

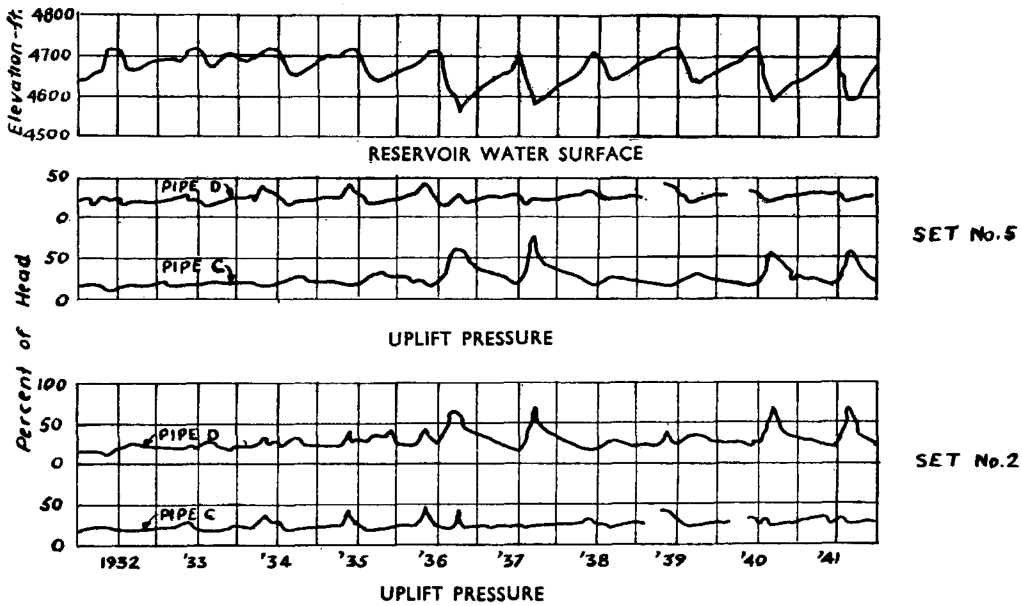

Fig. 3. Gibson dam. Fluctuation of reservoir water surface and measured uplift pressures. (Reproduced from Figs 30 and 31 of Ref. 5.)

Uplift pressure observations at Gibson Dam exhibit an interesting characteristic. In Fig. 3 the reservoir water surface fluctuations and uplift pressures at different observation pipes are shown. The locations of observation pipes and lines (sets) are shown in Fig. 1.

At observation pipes D and C, the maximum uplift pressures do not occur at the time the reservoir reaches its maximum level. On the contrary, the peaks in uplift pressures show a tendency to lag two to three months behind the peaks in the reservoir surface. The minimum uplift pressures, on the other hand, trail the minimum reservoir levels by about six to eight months. These characteristics are not consistent and did not occur all the time or at all observation points. However, the time-lag correlation between reservoir surface fluctuation and the build-up as well as dissipation of uplift pressures is distinctly noticeable at some locations at Gibson Dam.

UPLIFT IN A CONCRETE DAM

The phenomena of seepage through, and uplift in the body of a concrete dam are analogous to the seepage problems in the foundation of the dam. Mass concrete used in dams is a fairly homogeneous material with a very low permeability. Discontinuities which may develop into zones of comparatively high permeability can occur at the horizontal construction joints. Saturation of the dam and seepage through the construction joints would be visible at the downstream surface. Excessive wetting of the downstream surface can accelerate the deterioration of concrete where the dam is exposed to extreme changes in atmospheric temperature and humidity, and to freezing and thawing.

To avoid the detrimental consequences of seepage through a concrete dam, and also to reduce the effective uplift pressures in the dam, it is almost universal practice to form vertical drains in the dam near its upstream face. U.S. Bureau of Reclamation's Glen Canyon Dam (April 1957), now under construction, may be taken as a typical example. Body drains, 5-in. diameter, and formed at 10-ft centres would extend from top to bottom along the entire upstream face of the dam. The drainage complex is located approximately 35 ft from the upstream surface. The drains terminate in the foundation drainage and grouting galleries. Drains are also formed in transverse or radial contraction joints.

Observation pipes to measure uplift pressures at the construction joints have been installed in several Bureau of Reclamation dams. As a typical testimony of the efficacy of body drains, the following observations on Hoover Dam are quoted from p. 17 of Richardson (1948).

> "All uplift pipes at the horizontal joints in the interior of the dam are dry and have been so except for one to two occasions, and show no pressure to date. . . . Since these pipes have remained dry for most of the time and show no pressure over the period of 12 years since the system was first placed in operation, it is probable that they will remain so indefinitely."

This indicates that the uplift pressures in the dam are much lower than those used in design, which assumed that "uplift pressures vary from full reservoir pressure at the upstream face to tailwater or zero at the downstream face . . . and act over two-thirds the area of the base." Present design criteria (Kirn, 1953) of the Bureau of Reclamation for uplift pressures in the dam are identical to those for its foundation. Data are not available on amount of seepage through the drainage system in the body of the dam.

The excellent performance of formed drains within concrete dams is a demonstration of the seepage and uplift pressure control that can also be affected in abutments and foundations of dams if adequate drainage systems are provided.

UPLIFT PROBLEMS AT DAMS LOCATED ABOVE STEEP DROPS

Dams, and especially spillway control structures and power intake structures, are often located above natural or excavated steep drops. Two such examples are shown in Fig. 4.

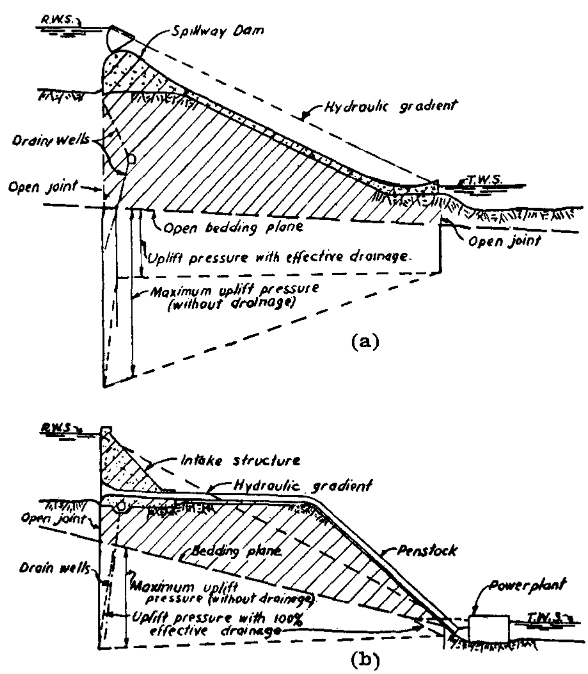

Fig. 4(a). A spillway located on a steep slope;
(b). An intake and penstocks located on a steep slope

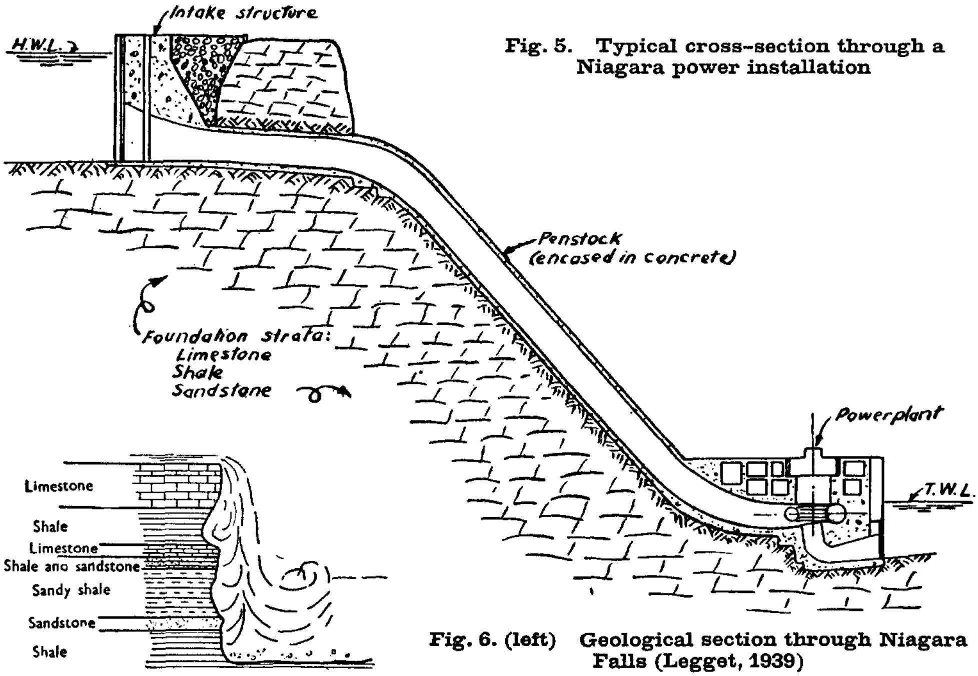

Fig. 5. Typical cross-section through a Niagara power installation

Fig. 6. (left) Geological section through Niagara Falls (Legget, 1939)

The problems of seepage control and uplift under the dam and through the rock strata below and the slope downstream of the dam, present a special corollary to the study of hydraulic seepage under concrete dams. If the foundation rock is jointed, as is often the case, the entire foundation block (shaded areas in Fig. 4), would eventually be subjected to full lateral hydrostatic pressure. If no foundation drainage is provided, uplift pressure varying linearly from full reservoir pressure at the upstream open joint to zero or tailwater pressure at the likely downstream exit point, could ultimately develop under this block. The development of uplift pressure would be facilitated where the rock formations are laminated and the dip of the strata is in the downstream direction. If the bedding planes are interrupted by a discontinuity, a condition similar to that shown in Fig. 9(h) of Casagrande's paper can develop.

In such cases it is essential to examine the stability of the dam and the foundation block as one integral unit. A three-dimensional study which would take into consideration all the joint systems in the rock would be necessary. To improve and insure the stability of the structure and the foundation it rests on (as shown in Fig. 4), an extensive drainage system should be provided, which would extend either to the level of the base of the rock slope or sufficiently into any known open and permeable bedding planes or strata where uplift build-up could occur. The drainage system should be able to effectively interrupt all channels of seepage flow toward the structure and its foundation.

Several of the power plants located in New York and Ontario, below the Niagara Falls, represent situations parallel to the case discussed above. Fig. 5 shows a typical cross-section through some of the power installations in this area. The intake structure is situated at the terminus of a forebay or supply channel and the powerhouse is located at the bottom of the cliff. The penstocks are located on the surface of the slope and encased in concrete. The surface of the cliff, between the penstocks, is often paved with concrete.

The cliffs on both sides of the Niagara River have geological characteristics similar to those at the Niagara Falls. A typical geological section (Leggett, 1939) through the Falls is shown in Fig. 6. The most predominant formations are limestones and shales with occasional strata of sandstones and dolomite. All strata are almost horizontally bedded and laminated. The limestones are harder than the shales. Joint systems, normal to the bedding planes, occur in most of the strata.

The possibility of seepage into the foundation rock from the forebay, and the likelihood of consequent development of uplift pressure in the various strata need to be considered in checking the stability of such installations. A drainage tunnel under the intake structure halfway between the headwater and tailwater levels, with vertical drain holes drilled upward and downward from the tunnel, should provide satisfactory insurance against excessive uplift pressures which would endanger the stability of the cliff above the power plant. The Writer is not aware if any programme of measuring piezometric pressures in the foundation rock are in force at the existing power plants along the Niagara River. It is quite possible that uplift pressures have generally not been a problem at these power plants because the limestone and sandstone strata may act as adequate drains. The Writer however, suspects that critical uplift pressures, coupled with erosion of exposed shales, are a major cause of the several notable cliff slides that have occurred in this region in the past.

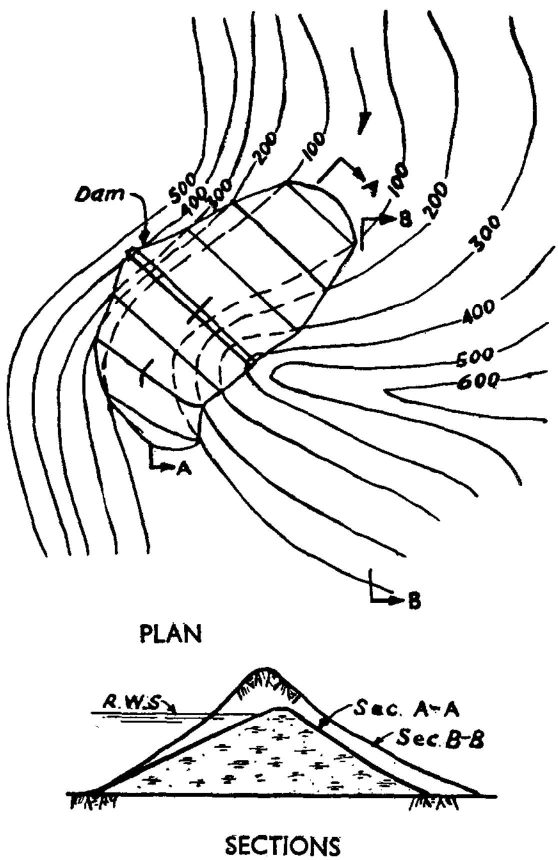

Fig. 7. Plan of a dam located at a sharp bend in the river

SEEPAGE THROUGH ABUTMENTS OF DAMS

In his Paper Professor Casagrande has pointed out the significance of seepage and its control through abutments of very thin arch dams. Hydraulic seepage through the natural abutments of a dam located in a sharp bend in the river, or at a location where one or both abutments may be comparatively narrow and steep spurs, also deserves special attention. Fig. 7 shows the general plan of a rockfill dam located in a Z-bend in the river. The left abutment, section B-B, is a narrow ridge which offers excellent sites for locating the diversion tunnel, spillway and the power installation. The cross-section of the natural abutment is only slightly more massive than that of the rockfill dam (section A-A). Actually, the type of dam is not so important as far as seepage through the abutment sufficiently away from the dam, say at section B-B, is concerned.

For checking the stability, seepage flow, and uplift forces, the abutment promontory should be investigated as carefully as a dam. This natural dam may or may not have in the past sustained reservoir pressures as high as those to which it would be subjected when the dam is completed. While the slopes of the natural dam may be flatter and outwardly more stable than that of the man-made embankment, the quality of natural construction may be inferior and the dip and orientation of the composite strata may adversely affect its stability. Such abutments may need drainage facilities to control seepage flow and uplift pressures even more than the foundations under the dam.

Yours faithfully,

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27 June, 1962.

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The Secretary,
The Institution of Civil Engineers.

STRESS/STRAIN RELATIONS FOR PARTLY SATURATED SOIL

DEAR SIR,

In considering the mechanics of a small cylindrical element of partly saturated soil, the axial total stress σ_1, the lateral total stress σ_3, the pore-water pressure u_w, and the pore air pressure u_a must all be specified. As Jennings and Burland have mentioned in their recent