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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
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ALLUVIUM GROUTING PROVED EFFECTIVE ON ALPINE DAM

By Darius Bonazzi[1]

INTRODUCTION

The grouted foundation at the Notre-Dame de Commiers earthfill dam on the River Drac in southeast France is examined herein. The dam has a thin sloping core and is 40 m (131.23 ft) high for a total crest length of 300 m (984.26 ft). The fill volume is approximately 3,500,000 cu m (123,642,480 cu ft). The banks are composed of a mediocre-strength Lias shale and the river-bed is choked by a 50-m thick, highly pervious alluvium deposit with an approximate area of 7,000 sq m across the valley. A cross section of the dam is shown in Fig. 1.

It was decided at an early stage of the design that the foundations would be grouted by the same process used at the Serre-Poncon Dam on the River Durance, where the alluvium is similar.

The alluvial deposit is lenticular in structure, but no particular bed is extensive or sufficiently thick to be identified from one bank to the other. The material consists of sand and pebbles, with few blocks larger than shown on Fig. 2. The sand fraction is small and accounts for approximately 15%, except at great depth, where the beds are richer in fines.

The probable magnitude of the permeabilities was first roughly assessed by a large number of water-injection tests. The mean horizontal permeability was then deduced from ten pumping tests at different levels in beds from 10 m to 15 m (32.8 ft to 49.2 ft) thick, and gave the following results:

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[1] Engr., Coyne et Bellier, Bureau d'Ingénieurs-Conseils, Paris, France.

Depth, in meters	Permeability coefficient, in centimeters per second
0 to 4	10 to 1
4 to 10	$2 \text{ to } 4 \times 10^{-1}$
10 to 26	2×10^{-1}
below 26	3×10^{-2}

The proposed grouting method was still not sufficiently widespread for open bidding to be held and a negotiated contract was concluded with the contractor, who had acquired experience at Serre-Poncon.

In return, the client (Electricité de France), insisted that the consulting engineers and the contractor jointly propose a scheme for checking the degree of watertightness obtained with the process to guarantee the ultimate quality of the curtain. This guarantee had to be given before fill placement began. The assumption was that the foundation, at least in the river bed, could be completely grouted in one working season, thus dispensing with the gallery that is frequently provided to permit the resumption of grouting after reservoir filling. The checking scheme was to be written into the contract.

GROUT CURTAIN

A detailed description of the grout curtain is available.[2,3,4,5] The grouting procedure itself has been described by E. Ischy and R. Glossop.[6]

Briefly, the curtain consisted of five rows, a total of 33,000 ft, of boreholes staggered in depth (Fig. 1). Hole spacings along the rows were 3 m (10 ft) and the rows themselves were 3 m apart. The estimated curtain thickness is 15 m at the core contact and approximately 6 m (19.6 ft) in its deepest part. Three types of grout were used, namely, clay-cement, deflocculated clay, and silica gel, in that order.

The principles guiding the grouting operations were:

1. The quantities injected were limited in advance to a value computed from the voids in the material being grouted, which accounted for 30% to 40% of the total volume. A "radius" of effectiveness was aimed around each hole.
2. Grouting proceeded from the outside rows towards the center of the curtain, so that the grout was increasingly retained between two grouted "walls."
3. As the central section of the curtain was approached, the materials were also upgraded in quality. Starting with a clay-cement grout, manufactured

[2] Chanez, R., Le Genie Civil, Vol. 139, No. 1, 1962 (in French).
[3] Conte, J., and Chanez, R., Travaux, Special Number, 8th Internatl. Congress on Large Dams, 1964.
[4] "Comprehensive Report No. 4," French Committee on Large Dams, Proceedings, 8th Internatl. Congress on Large Dams, 1964 (in French).
[5] Tavernier, M., and Chanez, R., Travaux, April, 1965 (in French).
[6] Ischy, Ernest, and Glossop, Rudolph, "An Introduction to Alluvial Grouting," Minutes of Proceedings, Institution of Civ. Engrs., London, England, Paper No. 6598, 1962.

from local clay in the outer rows, the mix was improved by replacing part of the local clay with a finer clay in the intermediate rows and finally with a deflocculated and silica gel grout in the central row. The plant design enabled grout changes without interrupting operations.

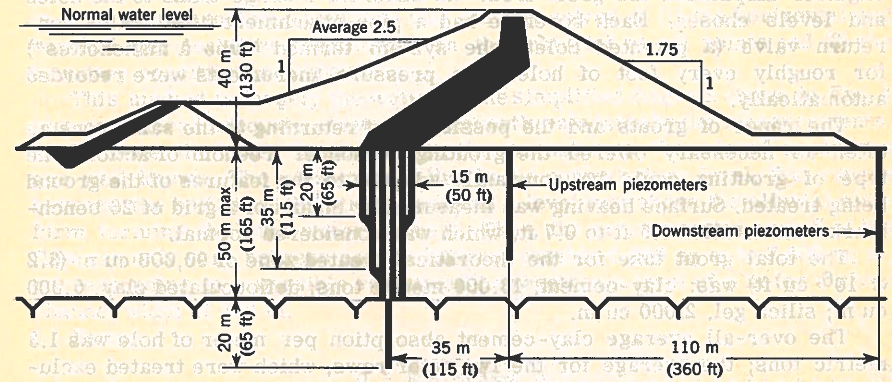

FIG. 1.—LOCATION OF MAIN PIEZOMETER ROWS IN TYPICAL DAM SECTION

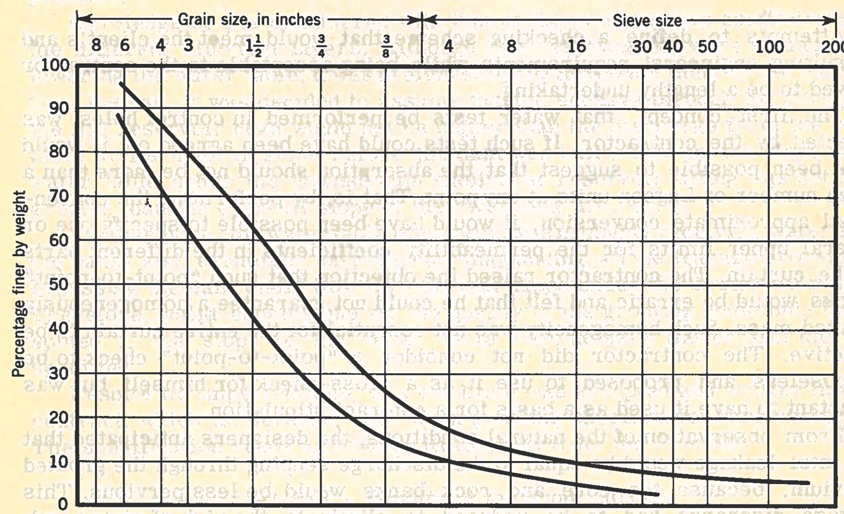

FIG. 2.—GRAIN SIZE DISTRIBUTION FOR ALLUVIUM

4. Frequent water-injection tests were performed to guide the choice of grout most suitable for completing the work in a given section.
5. Except for "open" pebble beds, the alluvium was considered incapable of being grouted with clay-cement grout. This grade was therefore used to

split the grouted zone using watertight partitions. In contrast, the silica gel was capable of sealing horizons of fine sand (between 0.3 mm and 0.1 mm).

The grout was manufactured in a single plant with pipe connections to the working platform. A system of eight pumps under the control of the grouting engineer dispatched the grout from the different storage tanks to the holes and levels chosen. Each borehole had a pipe attachment fitted with a non-return valve (a patented Solétanche system termed "tube à manchettes") for roughly every foot of hole. The pressure increments were recorded automatically.

The range of grouts and the possibility of returning to the same zone as often as necessary offered the grouting personnel freedom of action. The type of grouting could be constantly adapted to the features of the ground being treated. Surface heaving was measured by means of a grid of 26 bench-marks and totalled 0.5 ft to 0.7 ft, which was considered normal.

The total grout take for the theoretical treated zone of 90,000 cu m (3.2×10^6 cu ft) was: clay-cement, 13,000 metric tons; deflocculated clay, 6,000 cu m; silica gel, 2,000 cu m.

The over-all average clay-cement absorption per meter of hole was 1.3 metric tons; the average for the two outer rows, which were treated exclusively with this grout, was 1.85 tons.

TENTATIVE CHECKING SCHEMES

Attempts to define a checking scheme that would meet the client's and consulting engineers' requirements while being acceptable to the contractor proved to be a lengthy undertaking.

The first concept, that water tests be performed in control holes, was rejected by the contractor. If such tests could have been agreed on, it would have been possible to suggest that the absorption should not be more than a given number of Lugeon units at any point. That is, by performing the conventional approximate conversion, it would have been possible to specify one or several upper limits for the permeability coefficients in the different parts of the curtain. The contractor raised the objection that such "point-to-point" checks would be erratic and felt that he could not guarantee a homogeneously grouted mass. Such homogeneity was not essential for the entire curtain to be effective. The contractor did not consider a "point-to-point" check to be purposeless and proposed to use it as a cross-check for himself, but was reluctant to have it used as a basis for a contract stipulation.

From observation of the natural conditions, the designers anticipated that the total leakage would be equal to the discharge seeping through the grouted alluvium, because the core and rock banks would be less pervious. This leakage discharge had to be reduced to eliminate the risk of piping and, primarily, to prevent the phreatic surface in the downstream shell from rising sufficiently high to produce a flow parallel to the 1.75 in 1 slope, which would have been incapable of withstanding such a flow.

The contractor's rejection of the "point-to-point" check and the designers' preoccupation with the phreatic surface in the downstream shell combined to win acceptance for the concept of an over-all check based on water-table measurements of the entire site area both before and after grouting. These measurements were made with piezometers.

THE CHECKING SCHEME ADOPTED

The plotting of the flow net through the mass of alluvium in the rock channel is a three-dimensional problem that can be solved if (1) the horizontal and vertical permeability coefficients in the material at all points and (2) the boundary conditions are known.

This method is largely theoretical. The simplified diagram shown in Fig. 3 was considered adequate and was subsequently valadated by the measurements.

Because the river is never dry, the downstream "boundary" conditions are well known. The piezometric line is the normal river level, that ranges between the bottom of the stream bed and a level just above the alluvial platform forming the high-water channel. Thus, the cross-section area S of the flow can only vary between narrow limits of roughly 6,000 sq m to 7,000 sq m, corresponding to a possible variation in downstream level of 4 m for a river channel width of 250 m.

From

$$q = k(\text{average})\ S\ i \quad (1)$$

it is seen that the seepage discharge, q, is proportional to the slope of the water table, i. This was an initial approximation that proved satisfactory in practice.

Knowledge of q is considered with that of the permeability coefficients for the different alluvium layers. Although these coefficients were measured by lowering the water table, it was preferred not to introduce them in the calculation. Instead, it was decided to assume that the seepage through the curtain at the full reservoir head would not be higher than the discharge from the water table at its natural slope before the dam was built. That is, the slope of the water table under the downstream shell after reservoir filling was not to be greater than the over-all slope of the river-bed, i.e., 0.6%.

Furthermore, the water table in question was probably receiving water from the banks and, eventually, on reservoir impounding, all the leakage crossing or bypassing the dam would flow through this nappe under the downstream shell. Therefore, valid conclusions on the quality of the grouting could only be assumed if the slope of the water table were actually flatter than the limit specified.

Despite its imperfections, this condition was accepted and written into the contract, which is, perhaps, rare when grouting specifications are concerned. The specification reads as follows (translated from the French):

15-1-2-3 - Quality of the grout curtain

> . . . Its quality shall be such that the leakage discharges at the full reservoir head and for the permanent flow regime are equal at most to the discharge in the alluvium under the present mean flow conditions in the River Drac before construction commences. The slope of the water-table downstream of the curtain and under the dam shall not be greater than the present 0.6% slope of the water-table . . . (Notre-Dame

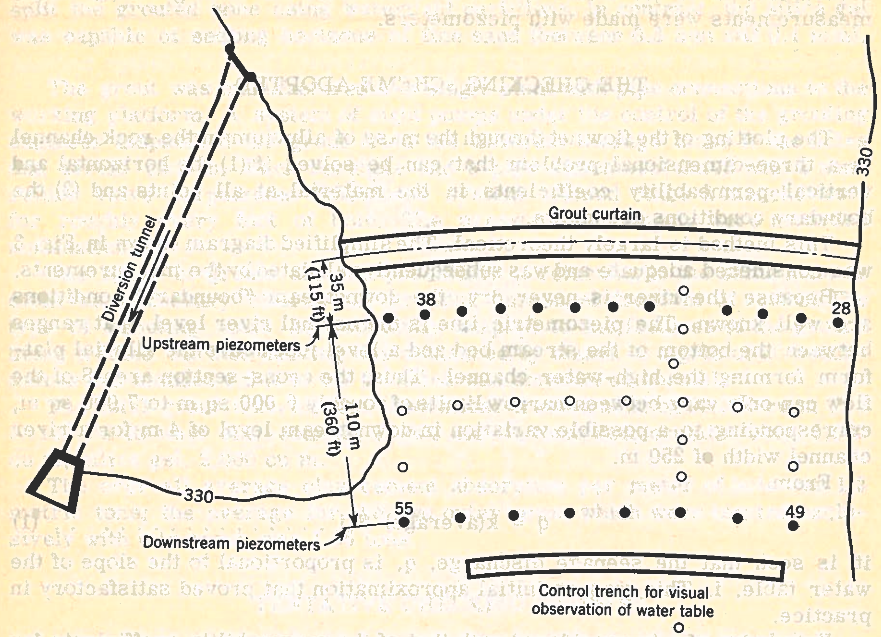

FIG. 3.—PLAN VIEW OF PIEZOMETER NETWORK

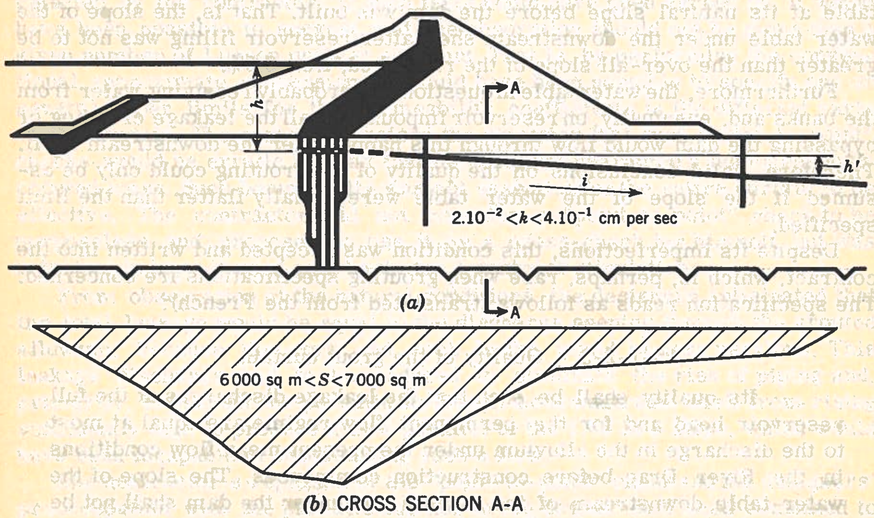

FIG. 4.—FLOW DIAGRAM DOWNSTREAM OF GROUT CURTAIN

de Commiers Dam—Negotiated Civil Engineering Contract. Special construction requirements—1960).

The aim thus defined was secured, as will be seen.

Thus, reference was only made to the state existing before construction and $q_2/q_1 = (K\ i_2\ S_2)/(K\ i_1\ S_1)$ or virtually $q_2/q_1 = i_2/i_1$, in which the subscripts

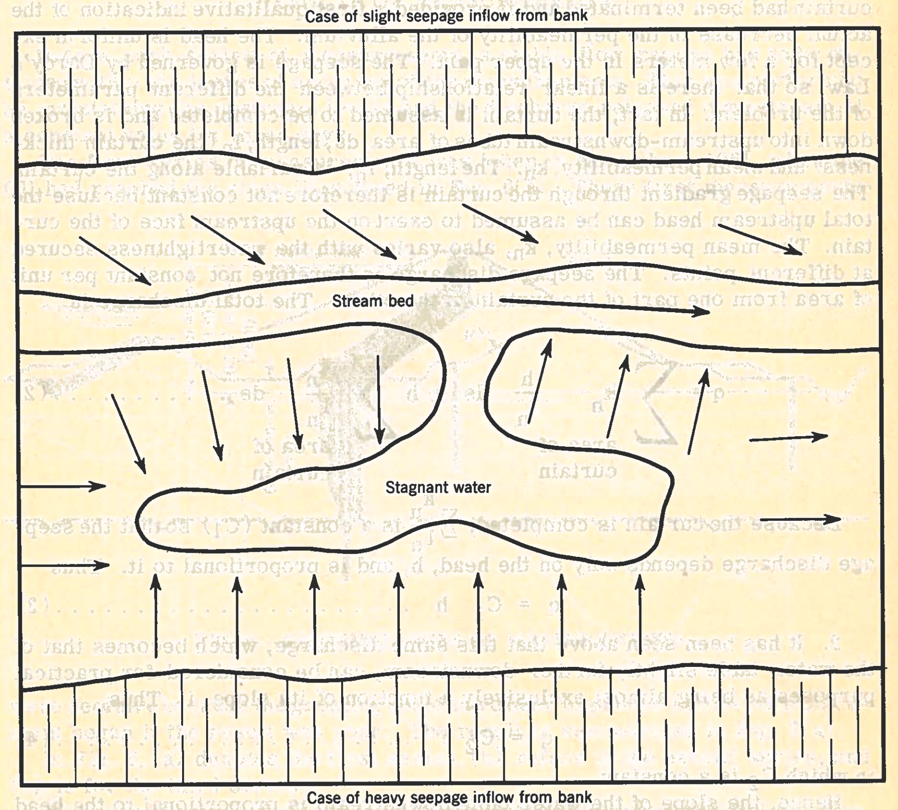

FIG. 5.—POSSIBLE COMPLEX FLOW PATTERNS IN A WATER TABLE DIRECTLY INFLUENCED BY A WATER COURSE

1 relate to the initial state before construction and the subscripts 2 to the state after construction.

This reference to the natural discharge from the water table seemed satisfactory at first. By definition, the stability of the alluvium was not affected by such a discharge. However, although this choice seemed logical, it was found to be arbitrary.

In Fig. 4, where the arrows indicate the probable direction of the seepage currents in a given case, the natural discharge from the water table does not have as direct a definition as was expected.

The measurements actually used for checking this condition are direct.

1. After the grouting has been completed, the water table in the alluvium is depleted on the downstream side, so that a head, h, appears on the curtain. This depletion effect was observed at a height of 5 m immediately after the curtain had been terminated and it provided a first qualitative indication of the actual decrease in the permeability of the alluvium. The head is uniform except for a few meters in the upper part. The seepage is governed by Darcy's Law, so that there is a linear relationship between the different parameters of the problem. In fact, the curtain is assumed to be completed and is broken down into upstream-downstream tubes of area, ds, length, l_n (the curtain thickness) and mean permeability, k_n. The length, l_n, is variable along the curtain. The seepage gradient through the curtain is therefore not constant because the total upstream head can be assumed to exert on the upstream face of the curtain. The mean permeability, k_n, also varies with the watertightness secured at different points. The seepage discharge is therefore not constant per unit of area from one part of the curtain to the other. The total discharge is

$$q = \sum_{\substack{\text{area of} \\ \text{curtain}}} k_n \frac{h}{l_n} ds = h \sum_{\substack{\text{area of} \\ \text{curtain}}} \frac{k_n}{l_n} ds \quad \ldots\ldots\ldots\ldots (2)$$

Because the curtain is completed, $\sum \frac{k_n}{l_n}$ is a constant (C_1) so that the seepage discharge depends only on the head, h, and is proportional to it. Thus

$$q = C_1 \; h \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

2. It has been seen above that this same discharge, which becomes that of the water table slightly further downstream, can be considered for practical purposes as being almost exclusively a function of its slope, i. Thus,

$$q = C_2 \; i \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which C_2 is a constant.

Hence, the slope of the water table downstream is proportional to the head on the curtain, namely

$$i = \alpha \; h \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which α is a constant. The permissible limit for α is derived from the contract: At the normal reservoir level, the head on the curtain is 40 m (131.23 ft). It is stipulated that the slope of the water table shall not exceed 0.6%. Thus

$$i = \alpha \; h = 40 \; \alpha \leqslant 0.006$$

The result, in limit value, is $\alpha = 1.5 \times 10^{-4}$, with h in meters. However, it is simpler to refer directly to the mean level differential, h', between the two rows of piezometers parallel to the curtain, i.e., the "upstream row" and the "downstream row" on Figs. 1 and 2, which are 110 m (360.9 ft) apart. With i = h'/110, with h' in meters,

$$\alpha = \frac{i}{h} = \left(\frac{h'}{h}\right)\left(\frac{1}{110}\right)$$

The permissible limit for h'/h = 110 times the permissible limit for α is 0.017.

RESULTS OF MEASUREMENTS

Before each series of measurements, a stable flow regime had to be obtained within the degree of accuracy of the measurements. This was performed by maintaining the upstream levels and the discharge released downstream at a constant value for three days.

The first series of measurements was taken in November, 1961, when the fill had reached the stage illustrated in Fig. 6(a). These first measurements

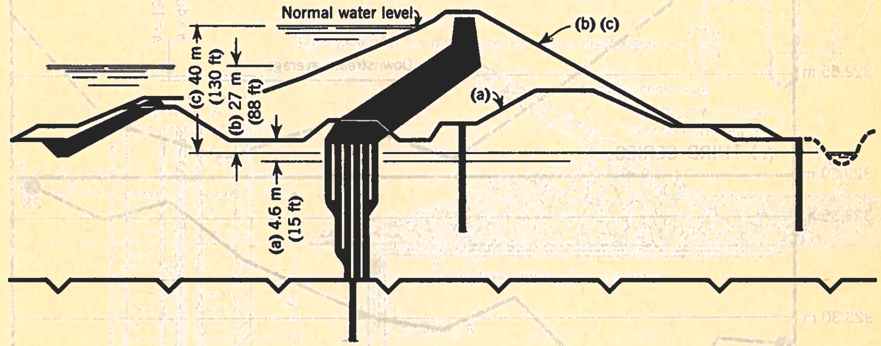

FIG. 6.—CIRCUMSTANCES OF MEASUREMENTS

were decisive because the grouting could have been improved before core placement began if the result was poor. The result is summarized in Fig. 7(a).

In Fig. 6, (a) denotes the first series, (b) refers to the second series, and (c) is for the third series.

The second series of measurements was taken after completion of the dam, during partial reservoir filling in November, 1963. The head, h, on the curtain was 27 m [Fig. 6(b)]. The measured level differential was approximately 10 cm [Fig. 7(b)].

The third series was taken after complete reservoir impounding in July, 1964, and confirmed the previous results.

Fig. 7(c) shows the difference, h', between the two known profiles of the water table for the normal 40-m head on the curtain. The water table always assumes the curved shape denoting the current's convergence towards the center line of the valley.

An air-injection system (Fig. 8) was used to measure the water table level in the piezometers placed at the start of the measurements and subsequently buried in the fill. This system was devised by the client's staff.

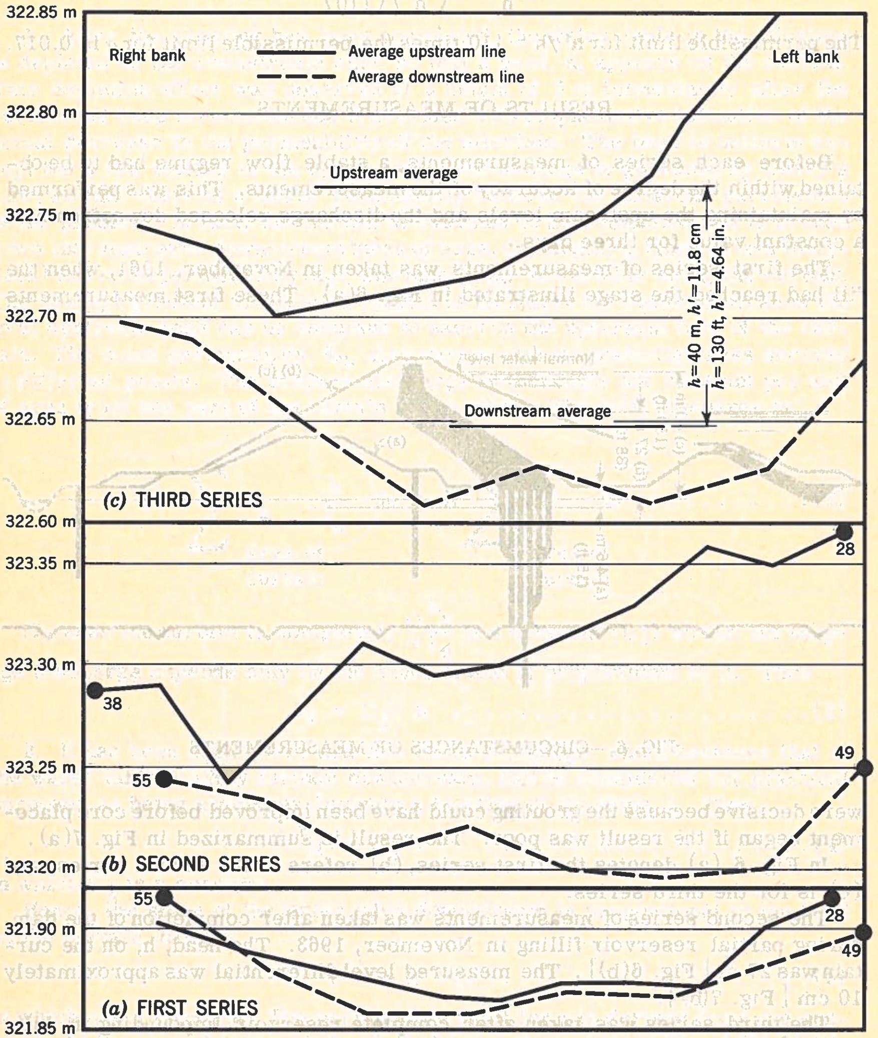

FIG. 7.—WATER TABLE PROFILES PARALLEL TO GROUT CURTAIN AS RECORDED ALONG UPSTREAM AND DOWNSTREAM PIEZOMETERS

The air discharge in the circuit is sufficiently low (approximately 1 l per hr) for the head losses to be negligible (less than 1 cm of water). The air pressure at the inlet to the circuit indicates the level differential between the level Z_o of the hole linking the two plunger tubes and the free surface, Z, of the water in the piezometer, i. e., in the surrounding alluvium if the pressure is hydrostatic, which it is in this case. Level Z_o was chosen because it was generally under the assumed water table level. Level Z_o was carefully noted when the piezometers were placed. The air was dried before being injected to avoid condensation. The water table level was then obtained by reading the air pressure and adding a constant.

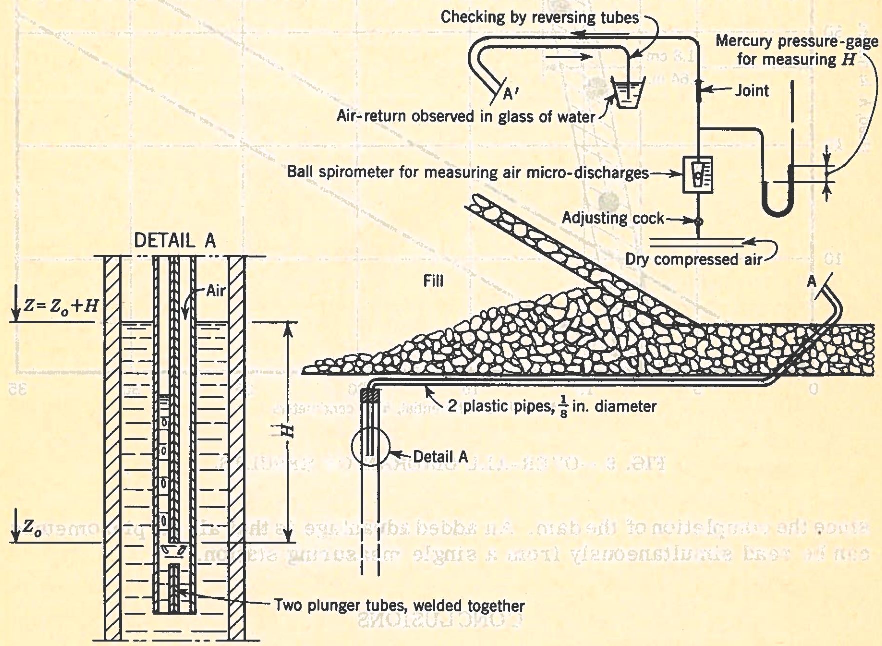

FIG. 8.—AIR-INJECTION MEASUREMENT SYSTEM

Unfortunately, level Z_o is influenced by foundation settlement because the piezometers of the upstream row have been driven downwards by the settlement of the natural alluvial layers under the weight of the fill. The surface settlement of the foundation is approximately 15 cm (6 in.). The piezometer tubes, which are linked by lateral friction, both to the surface layers and the deeper layers of the alluvium, are compressed lengthwise by the settlement of these layers. They are displaced downwards to an extent that is less than the settlement of the surface layers but cannot be known exactly. If allowance is not made for this displacement, the level of the water table in the upstream profile, and hence its slope, will be overestimated.

Fortunately, the results themselves point to the value of the correction that must be made. Hence, it has not been necessary to install new piezometers

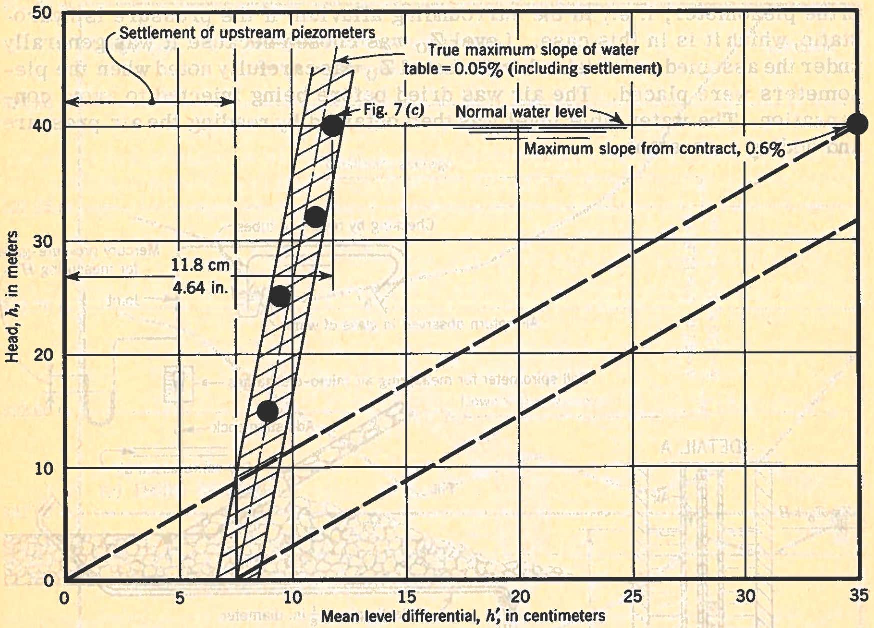

FIG. 9.—OVER-ALL DIAGRAM OF RESULTS

since the completion of the dam. An added advantage is that all the piezometers can be read simultaneously from a single measuring station.

CONCLUSIONS

The regular growth of h' as a function of h suggests that the question of the leakage through the entire dam can be reduced to that of the leakage through the grouted alluvium, as had been supposed from the beginning.

The relationship between h' and h seems to be generally linear, but with a fixed term that is probably caused by the settlement of the upstream piezometers. It seems from the graphs that approximately 7.5 cm (3 in.) must be deducted from the various measurements for h', whereby the slope of the water table for the upstream level at the normal reservoir elevation would be only 0.05% instead of the 0.6% stipulated in the contract and the permeability would be

approximately ten times lower to assure results, rather than using the original limits. A better result could not be anticipated.

ACKNOWLEDGMENTS

The Notre-Dame de Commiers Dam is under the control of Electricité de France, Région de' Equipement Hydraulique, Alpes I - Lyons. The grouting contractor was the Etablissements Soletanche and the consulting engineers were Coyne et Bellier, Bureau de' Ingénieurs-Conseils, Paris, France.

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DISCUSSION

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