but

$$P_1 :: R_1 - 2p_1$$

$$P_2 :: R_2 + p_1 - p_2$$

$$P_3 :: R_3 + 2p_2$$

Therefore

$$P_1 :: R_1 - \frac{12E_1\, I_1}{5L} \quad (w_2 - w_1)$$

$$P_2 :: R_2 - \frac{6E_1\, I_2}{5L} (w_3 - w_2) + \frac{6E_1\, I_1}{5L} \quad (w_2 - w_1)$$

$$P_3 :: R_3 + \frac{12E_1\, I_2}{5L} \quad (w_3 - w_2)$$

These equations together with the previous equations for the deflections permit us to solve for the reactions in terms of the loads, and from these the deflections. We thus have all the information necessary for the design of such a foundation.

While this procedure is of necessity cumbersome when applied to ordinary structures, or even to the simple layout herein considered when the subgrade modulus is altered at one column point only (as might be assumed to study the influence of soft areas on the stresses within the structure), the designer may by cut and try methods arrive at a fairly satisfactory result. It should be kept in mind that the final deflections will lie part way between those resulting from considering the reactions equal to the applied loads, and those resulting from a set of reactions which will produce equal settlements.

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No. N-3 SOME FEATURES IN CONNECTION WITH THE FOUNDATION OF SVIR 3 HYDRO-ELECTRIC POWER DEVELOPMENT

Professor H. Graftio, Leningrad, in collaboration with Vattenbyggnadsbyran, VBB, Stockholm

Svir 3 is situated in the Svir river between the Lakes Onega and Ladoga, about 50 miles from the latter. The total fall of the river between the two lakes is about 90 ft., of which about 36 ft. are utilized in Svir 3 and the rest will be utilized in another power plant now in course of construction. A plan of Svir 3 is shown in Fig. 1. A concrete dam, 630 ft. long, is joined to the power house which is 425 ft. in length and situated near the left bank of the river. On both banks there are earth embankments with a total length of 5,000 ft. Between the power house and the left bank there is a lock.

In the power house four units, each 37,500 HP, are installed. To prevent drifting ice in the autumn and spring from accumulating in front of the intakes to the power station, a protective concrete structure has been built in front of the intakes.

Counting from the power station the gate openings in the dam are as follows:

(a) A sector gate 15.5 ft. high and 105 ft. wide.

(b) 3 gates, each 14.8 ft. high and 43 ft. wide. When opened, these gates are lowered on the downstream side of concrete stop logs.

(c) 4 gates, each 8.2 ft. high and 67 ft. wide. When opened, these gates are raised.

The gates mentioned under (b) and (c) are operated by a movable crane. The maximum discharge of the dam is about 80,000 cusecs.

The power station has been planned and carried out under the supervision of Professor Henry Graftio, assisted by his Russian engineering staff, Svirstroi, Leningrad, and by Vattenbyggnadsbyran (VBB), Consulting Engineers, Stockholm. For questions relating to certain foundation problems the assistance of Professor Karl von Terzaghi of the Technische Hochschule in Vienna was secured.

Ground conditions. In a very large region the ground consists, to a considerable depth, of Devonian deposits covered with quarternary formations of sand and clay the thickness of which does not exceed 10 feet in the river bed at Svir 3, but increases rapidly further downstream. This in conjunction with the greater bearing capacity of the Devonian deposits, and the desirability to found the concrete structures directly on these deposits, caused the location of the power plant at its present site.

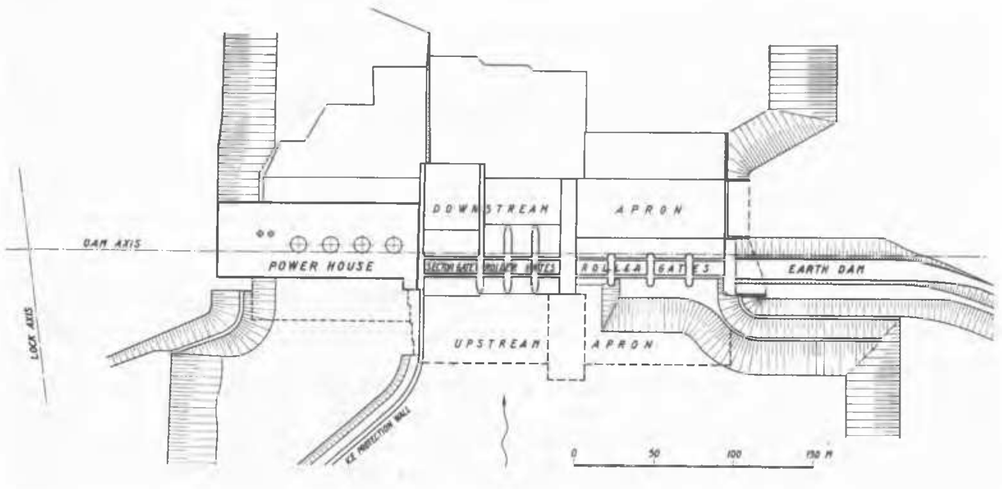

Fig. 1
General Plan

The Devonian ground consists of practically horizontal layers of alternating sand, clay and mixtures of both. The clayey layers are predominant, their aggregate thickness averaging 75% of the total depth of excavation. The various layers, which are from a few millimetres up to about one metre thick, are quite different in regard to consolidation and hardness. Generally the thick sand layers are hard, some of them transformed into sandstone. Also certain layers of clay containing much ferric oxide are consolidated to a great extent and nearly as hard as shale. Most of the layers are much less consolidated, some of them are compact and plastic, a few of them are soft. Some characteristic data on grading, ratio of voids, lower plastic limit, and flow limit, -- according to Atterberg --, are given below in Table 1 and Table 2.

TABLE I

Average grading of clayey layers

Size of particles in mm

	>1	1.0- -0.5	0.5- -0.25	0.25- -0.05	0.05- -0.01	0.01- -0.005	0.005- -0.001	<0.001
Compact, shaly clay	-	-	-	8.18%	27.35%	43.70%	4.91%	15.86%
Compact, plastic clay	-	-	-	-	-	-	-	27.6%
Soft clay	0.03%	0.07%	0.09%	6.00%	14.30%	38.40%	12.00%	29.11%

The colour of the different layers varies in all shades of red, brown, green and blue. With few exceptions the clayey layers are red or brown, whilst the sandy layers are bluish or greenish. The latter shades are caused by small quantities (less than 5%) of chlorite whilst the red and brown colours are due to small particles of ferric oxide being present in varying quantities. Chlorite exists in practically all layers. Due to the great variation in colour the cuts give, at a distance, the impression of continuous brownish deposits, mingled with greenish horizontal layers, all of which are of varying thickness. On closer examination, however, it will be found that most of the layers are discontinuous, that many of them disappear in the form of horizontal wedges, to reappear again at the same elevation. Although it has not been possible to ascertain exactly the same succession of layers in different cuts and pits, the relation between the different layers deposited at the same time is so close that it has not met with any particular difficulty to classify the layers in different groups.

Original sedimentary structure, such as drying cracks, ripple marks and miscellaneous irregularities, which characterize deposits made by running water, are common throughout the deposits. In places certain irregularities appear to have been produced by deformation, but are in most cases distinguishable from such secondary structure by their lack of planes of thrusting and faulting, features which will be described below in connection with the deformation of the deposits.

These features, and such extreme variation in colour and structure as must have been caused by vary-

TABLE II

Average ratio of voids, average lower plastic limit, and flow limit of the clayey layers, according to Atterberg

	Ratio of voids	Lower plastic limit	Flow limit
Compact plastic clay	15.5	15.5	25.0
Soft clay	21.0	19.0	33.0

ing chemical composition of the deposits and varying velocity of the water by which the deposits were brought, have made the geologists believe that the region in question is part of an old delta. In their opinion the extensive, comparatively thick and resistant layers of sand and sandstone were deposited during periods of exceptionally great floods, when the river rose over its banks, and spread coarse material of fairly equal grading over large parts of the delta.

Geological disturbances of the Devonian ground. At first sight the system of layers disclosed in cuts and pits exhibits so few irregularities that not only the layman would be tempted to consider the Devonian soil as practically undisturbed. However, this impression is erroneous. A thorough investigation disclosed the fact that in many places radical disturbances have completely altered the original structure of the layers. These disturbances may be divided into the following three groups:

(a) Faulting of the layers at an angle of less than 45° to the vertical, caused by pressure from above.

(b) Practically horizontal thin sliding zones between adjacent relatively hard layers.

(c) Broken and thoroughly kneaded layers of a thickness of up to 1 foot.

In the layers relating to group c there is a clearly defined thrusting and faulting, always in the main

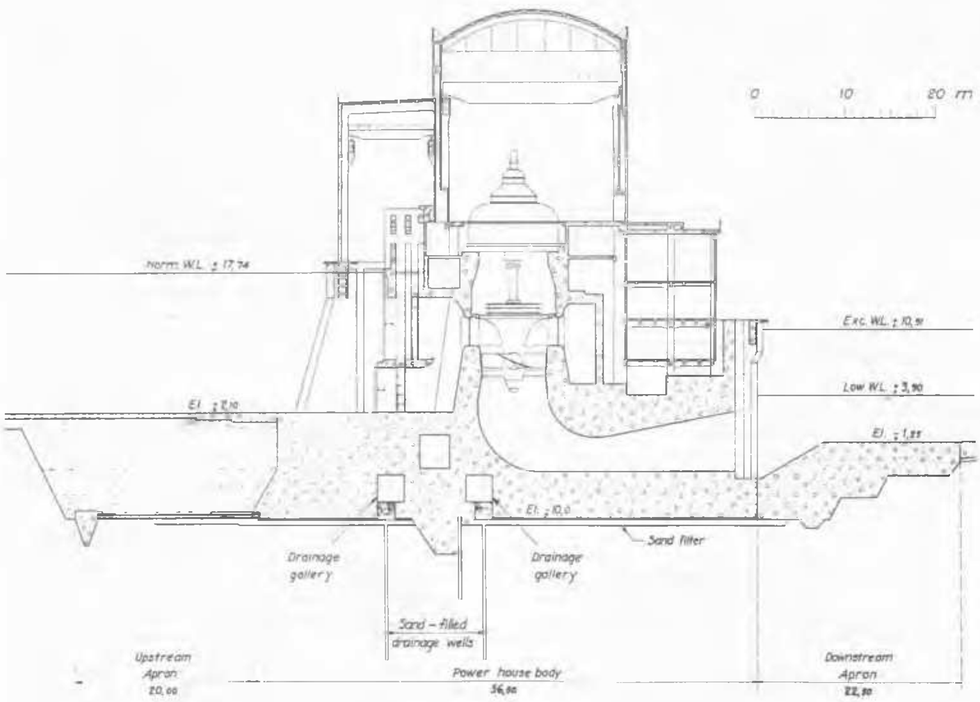

Fig. 2
Cross section of power house

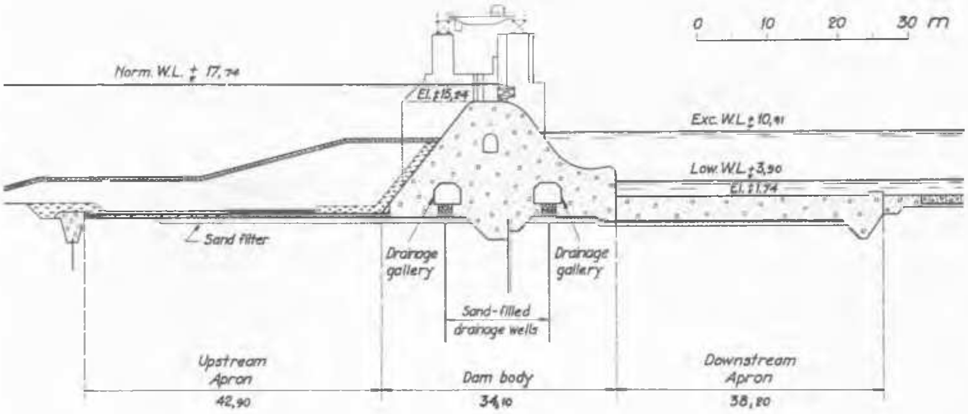

Fig. 3
Cross section of right hand part of dam

direction north-west to south-east. Occasionally small parts that formerly belonged to adjacent hard layers are kneaded into the broken up layers. This in conjunction with the faulting mentioned under (a) testifies that the Devonian ground has been subjected to simultaneous vertical and horizontal forces of a considerable magnitude. That the disturbances were caused by the ice-cap is very likely, because during the Pleistocene period the ice moved in practically the same direction as can be traced in the broken up layers. It may be assumed that certain Devonian layers brought along with the ice-cover started to slide on substrata the shearing strength of which was particularly small, and that parts of the harder layers bounding on these strata were broken off and kneaded into the layers where the rupture occurred. As the movement was very slow, these latter layers gradually became more consolidated than other layers of a small shearing strength, which in their turn were subjected to demolition. Finally a system of more or less parallel sliding zones extending continuously over large areas was created.

At present the Devonian ground comprises a series of fairly hard and practically undisturbed, nearly horizontal layers of comparatively great thickness, intermingled with thin broken and kneaded zones. The thickness of the unbroken layers varies between 2.5 ft. and 15 ft.

It follows that the moving ice-cap has subjected the Devonian ground to a gigantic test of its shearing strength, and indicated the layers that offered the least resistance to shearing.

<u>The resistance of the ground with reference to vertical load and sliding.</u> A close examination confirmed the first impression that the capacity of the layers to sustain vertical loads was great enough to make it unnecessary to extend the base surface of the structures over greater areas than required for other reasons. It is true that the smallest bearing capacity ascertained by loading an area of 1.5 sq. ft. amounted to 7.11 lbs./sq. inch only, whilst the greatest vertical load on the ground under the power-house amounts to 64 lbs./sq. inch. However, considering that each individual weak layer is as a rule horizontal, very thin, and embedded in hard thick layers, from between which it cannot be pressed out, it was not deemed necessary to base the computation on the weak layers to which the aforesaid low test-figure applies.

On the other hand the existence of the weak layers was very disquieting in regard to the ability of the ground to resist horizontal forces. Tests comprising the sliding zones just below the base of the structures were carried out, partly in cuts, partly in laboratories, in every case on specimens having a surface of 200 x 300 mm and a height of 150 mm.

Due to the long time required for the consolidation of the specimens while subjected to vertical load, and also to the lack of a typical limit at which sliding occurred when the horizontal stress was slowly increased, no reliable result could be obtained unless each test was extended over at least 10 days. As a matter of fact elastic deformation and sliding intermingled.

The tests displayed that the resistance increases very nearly in proportion to the vertical load, and that the coefficient of friction in the weak layers slightly exceeds 0.2, which figure has served as a basis for the computation of the stability of the structures.

<u>The properties of the ground in respect of swelling and contraction.</u> Great difficulties were due to the clay being apt to absorb water and swell when relieved of load, respectively to discharge water and contract when reloaded. Already in the first stages of design it was evident that considerable settlements of the structures must be expected, and therefore extensive series of tests were made in order to determine the elastic properties of the ground. By these tests the methods developed by Professor Terzaghi were applied, first using the ring apparatus, and later on the Oedometer. A comprehensive description of this part of the work is published in another report submitted to the Conference. (See Paper No. D-3)

The results of the tests may be summarized as follows.

When, due to varying loads, clay or clayey layers absorb or discharge water, a change in the volume of voids is effected. The corresponding upheaval or settlement of the ground is nearly fully elastic and characterized by a great hysteresis. The aptitude of the soil to contract and swell varies and is depending upon the consolidation of the layers and upon their sand content. A diagram typical for most of the clayey layers is shown in Fig. 4. To give an idea of the compressibility of the ground it may be mentioned that settlements measured so far have reached: Under the power house 10 in., under the dam 9 in., under the lock 11 in.

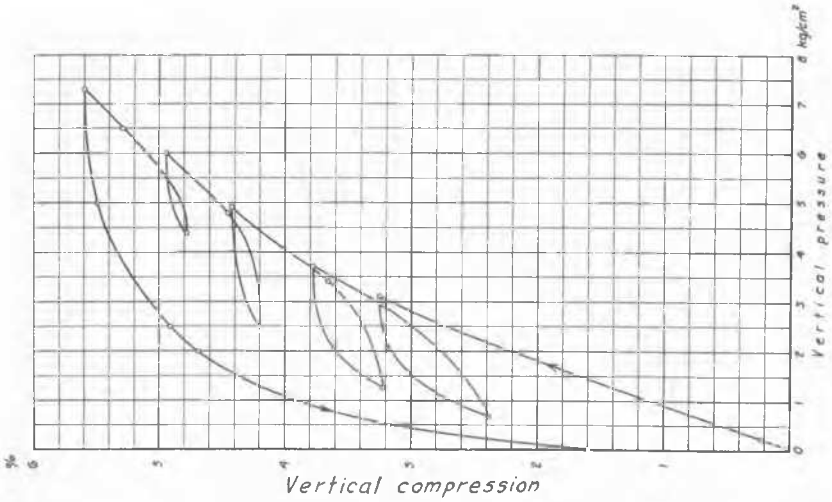

Fig. 4. Typical compression curve obtained with Devonian clay in Ödometer apparatus

<u>Subterranean water.</u> The groundwater at Svir 3 derives its sources from vast table-lands in the region between the lakes Onega and Ladoga. Due to the existence of alternately permeable and almost impermeable horizontal layers,

the latter of which separate, more or less completely the permeable layers from each other, the artesian pressure at the site of the power house and dam varies with varying depth, as a rule increasing with the depth. The main flow of the subterranean water is directed towards the river bed, and as the surface of the Devonian ground rapidly retreats downstream of the chosen site, it is but natural that the groundwater in the upper layers at this site emerges into the river at a comparatively small distance. These main features have been ascertained through extensive borings and measurements.

As a rule the flow of subterranean water is confined within the innumerable hair cracks in the sandy layers. But as there is also a vertical flow, it must be concluded that even clayey layers let water through. After various observations, experiments and computations it was assumed that the permeability is about 400 times greater in the horizontal than in the vertical direction. For the ground, taken as a whole, the permeability in the horizontal direction corresponds with Darcy's coefficient $4 \cdot 10^{-6}$ m per sec.

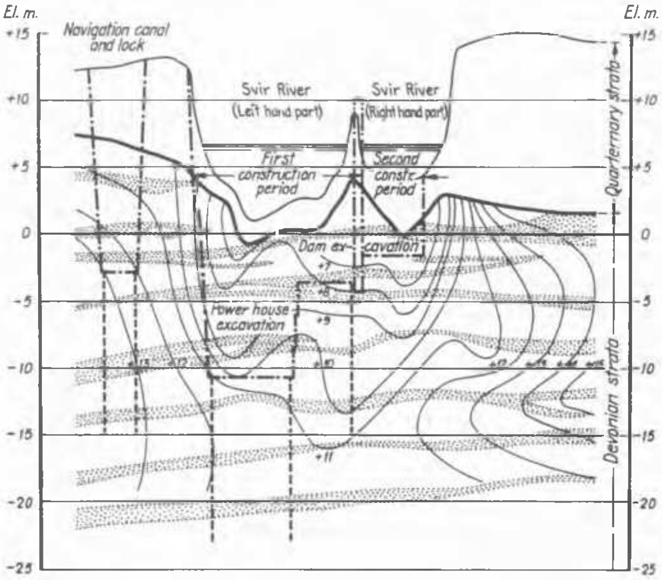

Fig. 5
Groundwater regimen at dam site in natural state according to measurements

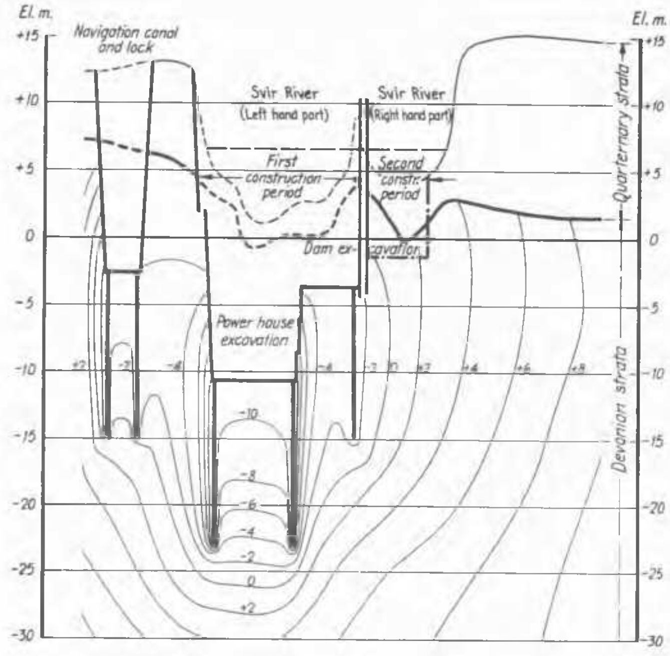

Fig. 6
Groundwater regimen at dam site after excavation according to computations

In Fig. 5, representing a cross section of the ground at the dam, the artesian pressure is shown by equipotential lines, the vertical scale being 20 times greater than the horizontal.

Measures taken during the construction period in order to safeguard the works against subterranean water pressure. The great subterranean water pressure involved risks of uplift and loosening of the ground at the bottom of the pits, not only when water was pumped out from within the coffer dams, but also in the course of excavation. In the first instance the prohibitive measures to be taken would have to reduce this pressure. But it was deemed advisable to make them serve also to reduce the settlements in the ground and increase the rate of settlement of the structures during the construction period and thus reduce the settlements after the completion of the work.

Several proposals were made. One comprised the sinking of deep shafts, at great distances from each other, from which the subterranean water would be pumped out. For several reasons this scheme was abandoned: It seemed a rather difficult task, how to restore the ground, when the shafts were no longer needed. In the case of a breakdown of the pumps or the electric supply, difficulties were to be expected. The shafts would hamper the use of Slack Line Tower Excavators for further excavation work etc. The following solution was finally decided upon, which proved satisfactory in every respect.

Round the pit, at comparatively small distances from each other, vertical wells of 12 in diameter were sunk through the various layers down to a depth of about 40 ft from the base of the structures, and into these wells perforated pipes surrounded with filter cloth and sand were immediately inserted in order to prevent the walls from falling in, and the fine material in the ground from being washed out. Throughout the excavation period these "sand wells" constituted a practically free communication between the pit and all pierced water-bearing layers, and reduced automatically the subterranean water pressure in the same proportion as the excavation proceeded.

A detailed account of the calculations relating to this matter is rendered by A. F. Samsioe in Zeitschrift für Angewandte Mathematik und Mechanik, April, 1931, Band 11, Heft 2: "Einfluss von Rohrbrunnen auf die Bewegung des Grundwassers", and in Fig. 6 is shown the calculated pressure in the ground at the end of the excavation of the pit.

In these calculations as well as in later graphical determination of the pressure and amount of subterranean water the following simplification was introduced, which may be of interest.

As the permeability of the ground is about 400 times greater in the horizontal than in the vertical direction, a cross section of the ground may be drawn to a vertical scale 20 times greater than the horizontal scale, thereby obtaining the picture of a ground where the permeability is the same in all directions. Consequently the flow of water can be treated graphically as if it took place in a homogeneous material. It should be noted, however, that when the quantity of water flowing to the wells is estimated, the coefficient of permeability has to be dealt with as the geometrical mean of the coefficients of flow in the vertical and in the horizontal direction, respectively.

Hence
$$k = \sqrt{k_v \cdot k_h}$$

where k_v represents the permeability in the vertical and k_h that in the horizontal direction.

Method of foundation. When a vertical cut is made in Devonian ground the material exposed becomes relieved of certain stresses. The clay expands inevitably in the horizontal direction and causes the hair cracks in adjacent sandy layers to open considerably. These latter layers react in their turn upon the clayey layers, and a general disintegration takes place near to the exposed surface. The process is accelerated by the pressure of the subterranean water. After a very short time hard lumps of clay become pulpy, later on soft enough to be wholly disintegrated by the emerging subterranean water, and particles of clay start to slide down with the water to the foot of the cut. Heavy rains cause further acceleration, particularly if preceeded by long dry periods. This is explained by the following phenomenon.

When dry specimens of a hard sandy layer were brought into water, capillary forces compelled the water to enter into its pores, subjecting thereby the air to an ever increasing pressure which soon became great enough to break the material, whereby the air escaped, emitting a hissing sound. Finally the test specimens became completely devoid of cohesion and turned into heaps of sand.

In the case of a deep foundation with concrete structures transferring the horizontal pressure to vertical or steep walls of Devonian ground, it would have become necessary to safeguard these walls of very great extent against deterioration of the aforesaid nature.

Even if it were possible to forestall dangerous deterioration of the ground near to the surface of the side walls of the cut, the risk remained that due to the release of lateral pressure the side walls, if very high, would slide towards the cut. During the excavation for the lock much trouble was caused by a great body of Devonian ground starting to move on a nearly horizontal sliding plane. It is apparent that in the downstream side walls of deep cuts which after the impounding of the water must serve as buttresses against horizontal pressure slides must be avoided, lest for the creation of a sufficient passive earth pressure great horizontal movements be needed. In the upstream side walls slides would cause loosening of the ground and increase permeability.

As shown in Fig. 2 and 3 the foundation problem was solved by giving the power house and the dam a base as flat as possible. To ensure thereby the necessary resistance in regard to sliding these structures were anchored to thin aprons loaded with puddled clay, Devonian material, layers of sand and water. Apart from the required addition to the sliding resistance, this design provided a desirable tightening of the ground upstream of the concrete structures.

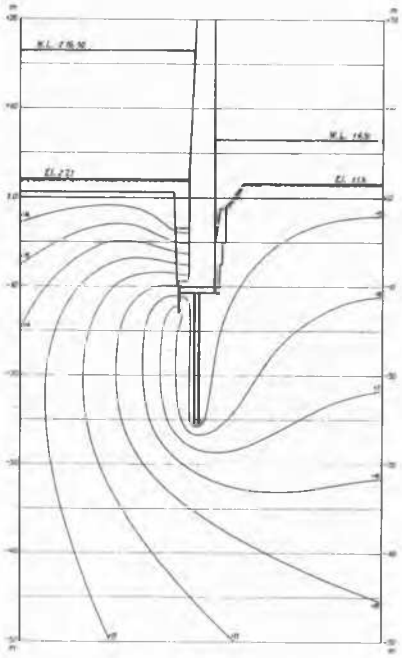

Fig. 7
Computed groundwater pressure in cross section of power house

In order to utilize as far as possible the load of the various elements of construction a sand filter was arranged under almost the whole base of the concrete structures, including part of the aprons. Via two galleries this filter is drained to the river downstream of the plant. The filter serves also the purpose of preventing the finer particles in the ground from being carried away by the subterranean water, which after the raising of the water level upstream of the dam flows at a greater velocity than before.

A further increase of the sliding resistance has been attained by means of two rows of sand wells discharging into the drainage galleries. As will be seen in Fig. 7, the effect of these wells is considerable, particularly due to the greater permeability of the ground in the horizontal direction.

Means to adapt the design and construction of the plant to the elastic properties of the ground. It is obvious that the consecutive erection of a power plant on an elastic ground in which the settlement in some places attains 11 in. may cause anxiety in regard to the possible occurrence of cracks in the concrete structures, the opening of joints, and so forth. Therefore it was necessary to attempt to find out beforehand how the settlement under the various parts of the concrete structure would proceed during the different stages of the construction work, and later on, in the course of impounding the water upstream of the plant. The calculations relating hereto were chiefly made in the following way.

In a number of places along certain cross sections the change of the vertical pressure in the ground was computed on the assumption that the ground under the concrete structures consisted of an elastic homogeneous material of unlimited extent. The settlement of the surface of the ground was calculated on the basis of the above-mentioned Oedometer tests, and according to a method described by A. Frey Samsioe Dr. Eng. in a special report to the congress. (Paper No. D-3)

There was never a doubt but that the result of these calculations must be uncertain: In the first instance, because the ground is not homogeneous, in the second because the subterranean water pressure during different stages of construction work influences the ground in a manner that cannot be adequately dealt with in mathematic formulas. In order to ensure every obtainable means of adjusting the method of calculation to existing conditions it was deemed necessary to observe very closely the movements of the ground during the early stages of the work. At different depths in the ground gauges were arranged and levelled at close intervals, as the excavation and concreting proceeded, and later on auxiliary gauges were applied to various concrete structures.

Through these continuous investigations it was established that the actual settlements did not reach the limits originally computed. In the sense of relative values, however, the agreement was very close. That the computed settlements proved too great may be due partly to the unequal consolidation of the different Devonian layers, partly to the fact that the ground was loaded with concrete immediately after the excavation had been completed, and thereby dispossessed of the possibility to swell to its ultimate volume. However, it was very useful to be able to establish, at an early stage, that everywhere 35 per cent of the computed settlements corresponded very nearly to the actual settlements.

For instance it was calculated that the power house after the installation of the first turbine would be subject to uneven settlement, resulting in an inclination of about 0.75 °/oo, the top portion of the power house moving upstream. The last observation was made when the water had reached a level 4 ft. below the ultimate, at which time the inclination amounted to 0.65 °/oo.

Thus means were provided for planning and organizing the work in a manner promising the least possible deformation of the structures. In the power house and the dam, the total length of which is about 1,055 ft., there is only one expansion and contraction joint.

Generally the concreting was carried out in monoliths, the lower parts in smaller units separated from each other by open joints which, when suitable in regard to settlements, were concreted.

For certain reasons it became necessary to construct the power house and the concrete dam in advance of the aprons to which they were to be anchored. Therefore, and because the aprons would be subjected to an increasing load until the water above them reached its ultimate level, very uneven settlements were to be expected in the transition zones between the aprons and the other structures. Obviously it was of paramount importance to ensure here a most elastic watertight connection.

This was effected by making the chief constituent of the connection of asphalt concrete on top of which a layer of pure asphalt was laid. As the combined load of earth fill and water subjects the asphalt to a considerably higher pressure than the water pressure corresponding to the difference between the upstream and downstream water levels this very flexible connection may be deemed absolutely watertight as long as the asphalt remains, which should be for ever, the asphalt being well protected against exterior agents of injury.

The connection between the aprons and the concrete cut off walls upstream of the aprons are designed on the same principle; also the expansion and contraction joints between different parts of the aprons. It may be noted that in every individual case great care has been taken to prevent the asphalt from flowing sideways if, for one reason or another, the vertical load on the asphalt sheet should change from place to place.

The aprons and the lower part of the upstream surfaces of the power house and the dam were covered by asphalt mats, and on top of these layers of puddled sandy clay were arranged. According to observations made there is every reason to believe that the watertightness in these parts is perfect.