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Autor: Bjerrum, Laurits / Kjaernsli, Björn

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Design and Construction of Mänika Dam

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By **Laurits Bjerrum,** Dr. sc. techn., Director, and **Björn Kjaernsli,** Chief Engineer, Norwegian Geotechnical Institute, Oslo - Blindern, Norway

Introduction

The rapid development of hydroelectric power in Norway has, in recent years, resulted in the construction of numerous earth- and rock-fill dams. During the last five years about ten dams of this type, ranging in height from 30 to 90 metres, have been completed or are at the time under construction. Generally these dams are built with a core of moraine that is supported by rock-fill shells but in two instances the desired imperviousness was obtained by a frontal deck of asphaltic concrete.

Connected with the design and construction of a water retaining structure is the great responsibility of assuring that the completed structure will safely perform its intended function. This requires, therefore, that an appropriate study of the stability of the structure be made. Experience has shown that the engineering-geological problems involved in the construction of a dam are not primarily dependent on the volume and height of the dam, and equal attention to these problems is required for the design of minor auxiliary dams as well as for major dams. This fact is illustrated in the following description of the Mänika Dam in which the engineering-geological problems are emphasized in honour of *Professor Schnitter* on his 65th Anniversary.

The Mänika Dam is an auxiliary dam of the Skogfoss stage of the development of the Pasvik river which forms the border between Norway and the USSR. According to the agreement to develop the Pasvik river, which was signed by the USSR and Norway in 1958, the development of the Skogfoss stage would be a Norwegian affair and the hydroelectric development at Boris Gleb and Hestefoss would be an affair of the USSR, see Fig. 1 and 2.

The Skogfoss development consists of the 15 m high main dam across the Pasvik river and the 45000 kW generating station. The Mänika Dam forms a dyke across the eastern branch of the Pasvik river and is thus built in Russian territory.

Geology at the site

It is known that during the last ice age the dam site was covered by ice and depressed to a level below the existing sea. As the ice melted, sand and gravel were deposited in the melt water passing beneath the ice cover and morainic material was ablated. After the glacier receded, the area was flooded by the sea to approximately 100 m above present sea level and clay was deposited on the submerged land. This deposition of clay continued until the land was elevated above sea level. On the basis of this geological history, irregular subsurface conditions were anticipated at the site.

Investigations within a limited area at the dam site by means of shallow shafts excavated in 1958 confirmed the irregularity of the subsurface deposits. Generally sands and gravels were encountered in the test shafts, however, it was found that the ridges on the terraces consisted partly of morainic material and clay was encountered in depressions in the area east of the site where the ground was covered with peat. Seismic measurements indicated that the depth to bedrock was of the order of 15 to 20 m. Three borings at the site indicated that the depth to bedrock was at least 16 to 18 m. From these borings it was found that the subsoil along the assumed dam axis consisted of sand and gravel with cobbles and boulders. In one boring situated close to the river, the upper 3 m consisted of mud containing shells and in the upper portion of a boring 20 m from the river a layer of fine sand was encountered.

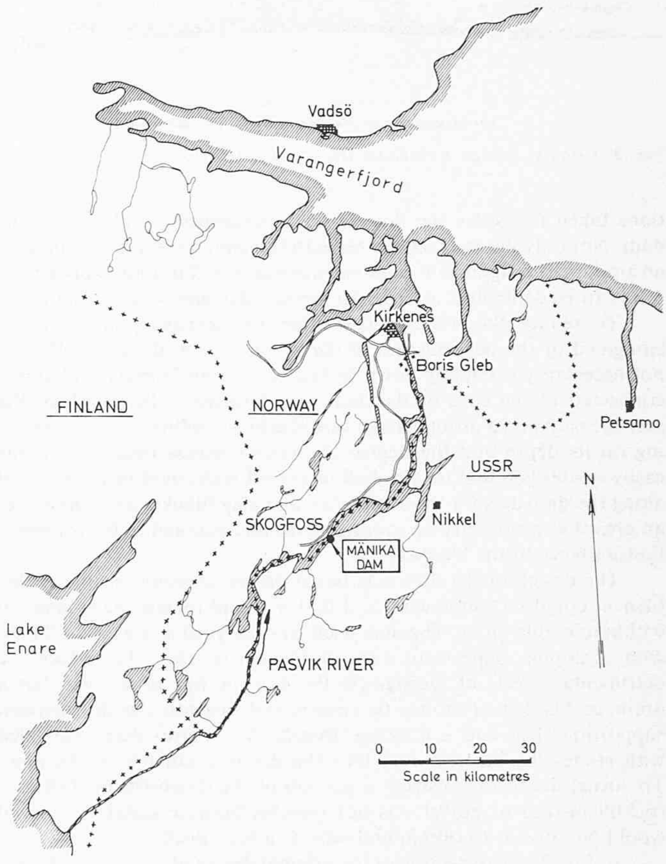

Fig. 1. Pasvik River, Plan, scale 1: 1 500 000

Design of the dam

On the basis of the site geology and the field investigations, it was evident that the dam had to be built on an irregular deposit of rather pervious material. Because of the great depth to bedrock it was also evident that a fill type dam should be constructed and special precau-

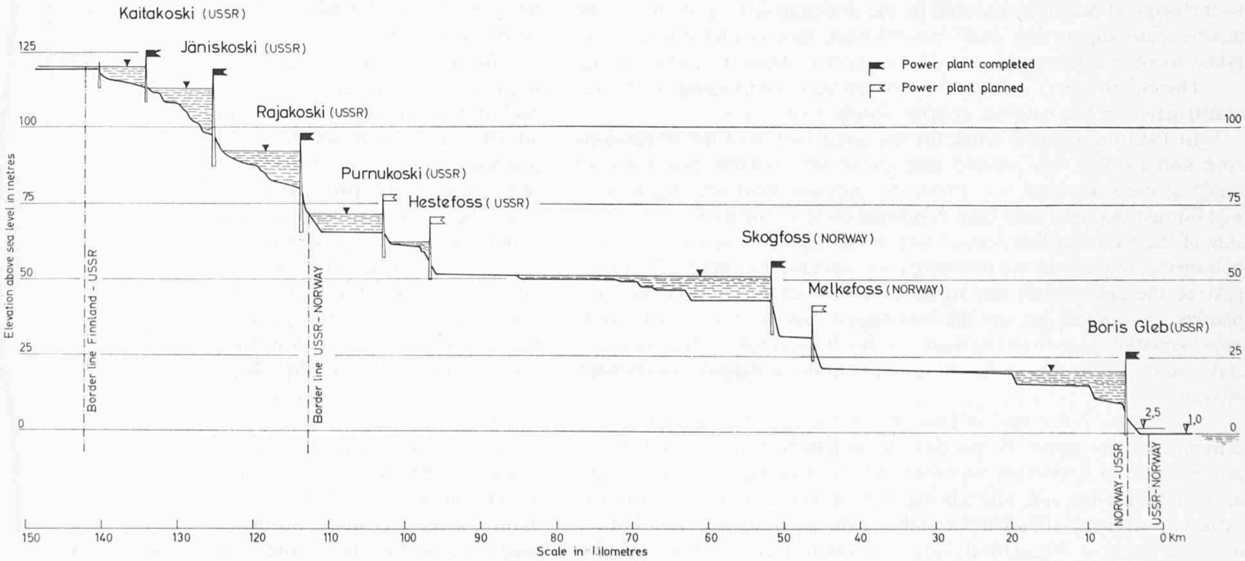

Fig. 2. Pasvik River, Profile, scale 1:1 000 000 / 1:2500. — Fig. 1 and 2 after B. A. Berdal and L. Kihl

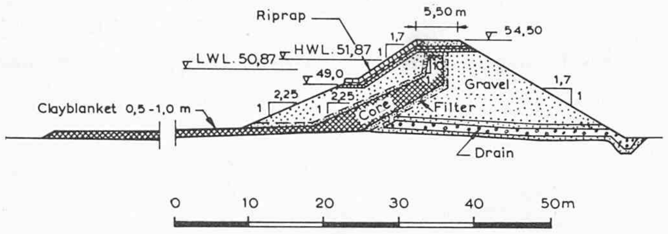

Fig. 3. Original Design of Mänika Dam, scale 1:1000

tions taken to reduce the flow of water underneath and around the dam. Not only did the seepage beneath the dam have to be reduced to an amount of insignificant economical value but of primary importance it had to be controlled in order to prevent damage to the structure.

To reduce the seepage loss either the seepage path could be elongated or the perviousness of the ground reduced. A cut-off wall, not necessarily extending down to bedrock, or an impervious blanket connected to the core of the dam would increase the length of the seepage path and a grout curtain would give a combined effect depending on its depth and the degree of imperviousness obtained. It was easily established that only a shallow cut-off wall could be constructed along the dam axis for the same price of a clay blanket extending over an area that would give an average hydraulic gradient of 0,1 for water flowing beneath the blanket.

The design of the dam was based on the assumption that a clay blanket could be constructed and that it would reduce the seepage to within tolerable limits. The dam itself was designed as a gravel-fill dam with a sloping impervious core of morain or clay. To reduce the detrimental effects of seepage on the dam, it was prescribed that a drainage blanket of stones be constructed beneath the downstream supporting shell and a drainage trench, two metres deep and filled with stones, be incorporated into the downstream toe of the dam. The usual desire to construct a portion of the downstream shell as a rockfill instead of gravel was not possible because suitable material would be difficult to obtain and only at a high price.

In Fig. 3 a cross section of the original design of the dam is shown. During construction, however, it became necessary to change the design.

Construction of the dam

Construction started in 1962 with the stripping of the foundation area for the dam and the borrow pit on the left shore upstream of the dam; in the same season a cofferdam was erected. As the riverbed downstream of the cofferdam dried out, several water springs developed. Most of these springs were situated just downstream of the cofferdam but one spring was discovered as far as 200 m downstream of the cofferdam approximately at the axis of the Mänikadam. At first these springs did transport fines and as a result gently sloped cones of fine material were built up around each spring. A state of equilibrium with no transportation of material seemed to be reached after a certain time elapse or when a certain height of the cone was established. The leakage out of the springs was estimated to be of the order of 5–10 l/s. The spring situated close to the dam axis would according to the original design be covered by the drainage layer underneath the downstream supporting shell, nevertheless, special precautions were taken to reduce the possibility of transport of fines from this spring.

During the first season the downstream supporting shell was constructed up to a level of approximately 49,0 m a. s.

In 1963 preparatory work for the construction of the impervious core and blanket was started and convenient borrow pits for core material were searched for. From the previous field investigations it was known that clay had been deposited close to the dam on the right side of the river but this deposit was unfortunately covered with up to two metres of peat. In the riverbed, clay was also encountered within a part of the area which was to be covered with the impervious clay blanket. A borrow pit on the Norwegian side at the Pasvik river, approximately 5 km from the dam, was finally selected. At this location, clay could be found in sufficient quantity under a shallow overburden of peat.

Soon after placement of this clay in the dam had started it was evident that this material was difficult to handle and compact. What was even more important was that the clay lost most of its strength upon remoulding and after being put in place it did not possess sufficient strength to secure a stable dam as designed. Laboratory tests showed that the natural water content of the clay was 50%, the liquid and plastic limits were 50% and 24% respectively. According to unconfined compression tests and cone tests the shear strength of undisturbed samples was 4 t/m² and only 0,34 t/m² for remoulded samples, that is, the clay had a sensitivity of twelve. In the remoulded state the shear strength varied, depending on the moisture content, between 0,6 t/m² and 0,1 t/m² at moisture contents of 43% and 60% respectively.

According to the original design, if the dam was to have a sufficient factor of safety against sliding the shear strength of the core material had to be approximately 4,0 t/m². Since the clay was placed with a shear strength less than ten percent of this required value and because oedometer tests on the clay had shown that the coefficient of consolidation for this material was only $5 \cdot 10^{-7}$ cm²/s, it was evident that the core material could not gain sufficient strength due to consolidation during the period of construction.

In principle two alternative solutions had to be investigated: could the strength of the clay be increased by any practical means, or could the design be altered so that a safe structure would be obtained.

To study the possibility of strengthening the clay, tests were carried out in which the clay was mixed with cement and lime. It could be shown that such a mixture gained strength with time depending on the amount of cement or lime mixed with the clay. After an elapse of about one hour, a mixture which contained more than approximately two percent cement or lime would develop sufficient strength to assure the stability of the dam. However, it is one thing to mix cement with clay in the laboratory and another thing to get the mixing done in the field and it was understandable that the contractor was sceptical of this solution and preferred a less inconvenient solution.

Since it was not practical to improve the shear strength of the clay it became necessary to find an alternate design which would provide a stable structure in spite of this undesirable property of the clay. The first design change considered consisted of a flattening of the upstream slope of the supporting shell possibly together with a surcharge placed on the upstream blanket. It was evident, however, that a flattening of the slope or a counterweight would not be suitable and would only increase the mass which would take place in a slide. The only benefit of such a solution would be the consolidation of the clay under an increased load but the clay would only gain very little strength due to

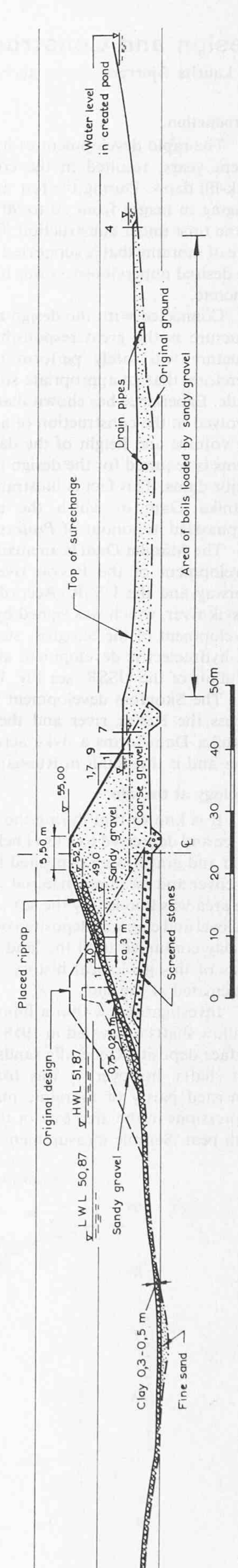

Fig. 4. Mänika Dam as constructed, scale 1:1000

consolidation during construction because the dam would be completed within a few weeks. If strength sufficient to insure the stability of the dam had to be obtained in this manner then the construction work had to be performed at a very slow rate which again was more a theoretical than a practical solution.

After realizing the ineffectiveness of a flatter upstream slope and of a counterweight, consideration was given to the possibility of eliminating the internal clay core and instead placing the clay as an impervious facing on the upstream slope without any surcharge as an extension of the impervious blanket. It was felt that the clay would be stable on a slope of 1 vertical to 3 horizontal and the design which was agreed upon is shown in Fig. 4. At this stage, however, the loaded area downstream of the dam in figure 4 was not included in the design. To take advantage of the gravel fill that had already been constructed the dam was moved downstream so that the fill could be incorporated into the new structure simply by flattening the upstream slope of the fill to 1 on 3. To protect the clay facing from the deteriorating effect of ice and waves the clay from the top of the dam down to 1,0 m below the lowest regulated water level was protected by a riprap of stones with a minimum weight of 50 kg placed on a layer of sand and gravel; below this level, the facing and clay blanket were left unprotected.

According to this revised plan, the construction work could continue without interruption and the gravel and clay were placed during the remaining weeks of the 1963 season. The manner in which the clay could be handled was limited because of its softness. After being dumped from lorries, the clay was levelled by a bulldozer; however, the bulldozer could not travel on the soft clay and it was necessary for it to keep its belts on solid ground and level out the clay with its blade as it travelled backwards. Afterwards the clay was further levelled or compacted to form a continuous and impervious blanket by systematic passes of a very light tractor that travelled on belts. During the autumn additional treatment of the clay blanket could be carried out by means of the bulldozer as the upper 15–20 cm was frozen.

When construction commenced again in the summer 1964 the clay blanket and facing were inspected and it was found that the clay had sufficient strength to permit additional compaction by systematic passes of the bulldozer. During the summer the riprap was placed and the construction was completed with the placement of top soil on the downstream slope on which grass was seeded.

Two overflow weirs were constructed downstream of the dam for seepage measurements and five standpipes, numbers 1 to 5 in Fig. 5, were installed for measuring the ground water level downstream of the dam.

Before closing the main branch of the river at the Skogfoss Dam and raising the water level in the reservoir, the seepage and standpipe levels were measured for a water level upstream of the Mänika Dam which very nearly corresponded to the normal river level that existed prior to construction of the dam. For example, at the end of May 1964 the water level in the Pasvik river upstream of the cofferdam was 44,12 m above sea level and the water level in the pond between the cofferdam and the Mänika Dam was 43,55 m above sea level. Under these conditions, the seepage at the two overflow weirs was found to be 4,2 l/s and 10,3 l/s respectively, see Fig. 6a. The standpipe levels at the five measuring points for the same situation is shown in Fig. 6b.

According to schedule, impounding of water in the reservoir would not begin before August of the same year. In the meantime it was considered of interest to raise the water level in the pond downstream of the cofferdam and observe the performance of Mänika Dam. In the middle of July the water level was raised by means of pumps to 45,5 m above sea level which was 1,5 m higher than the water level in the Pasvik river. At the two weirs the leakage increased from 4,2 to 7,0 l/s and from 10,3 to 15,7 l/s respectively, and, as seen in Fig. 6b, a general rise in standpipe level was as well observed. The results of this «test filling» did not present any basis for being concerned about the performance of the dam but, of course, it was recognized that the water level would be raised an additional 6,5 m and that trouble could develop before the highest regulated water level was reached.

Preparations for trouble that could possibly develop after the main branch of the river was closed were discussed at an early stage. It was recognized that the construction road across the Pasvik river downstream of the Skogfoss Dam could be washed out if the gates in the Skogfoss Dam had to be opened. Therefore it was considered advantageous to place in advance a stockpile of rock at the Mänika Dam site. In addition, vehicles for loading and transport were to be taken over to the Russian side of the river if any situation developed that indicated

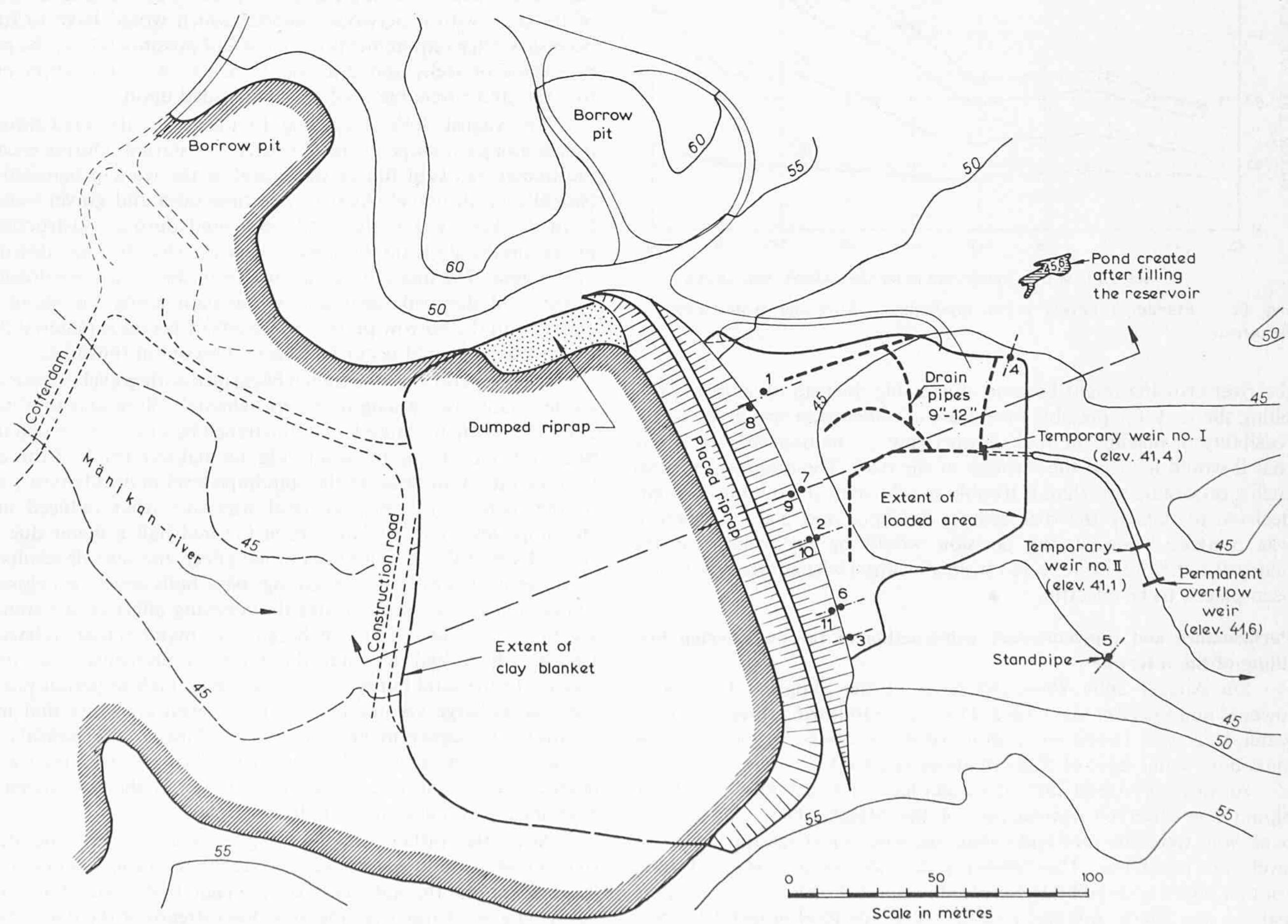

Fig. 5. Mänika Dam in Plan, scale 1:2500

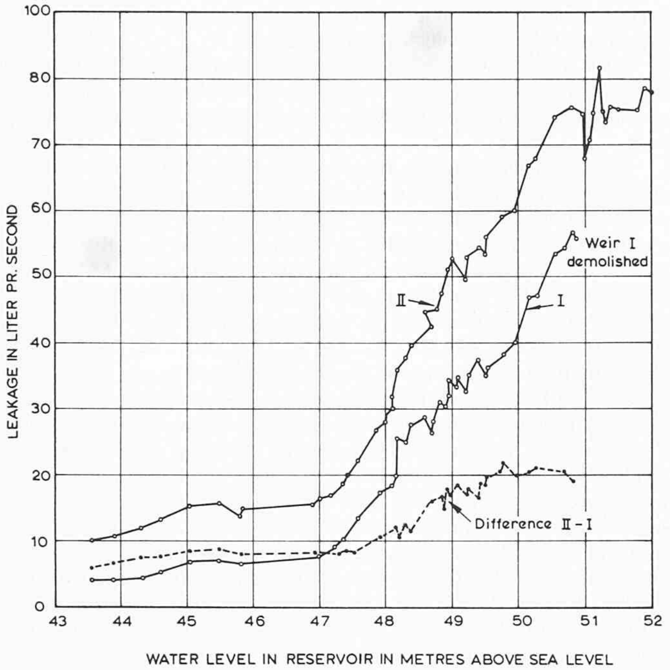

Fig. 6a. Leakage measured at temporary Weirs I and II versus Water Level in Reservoir

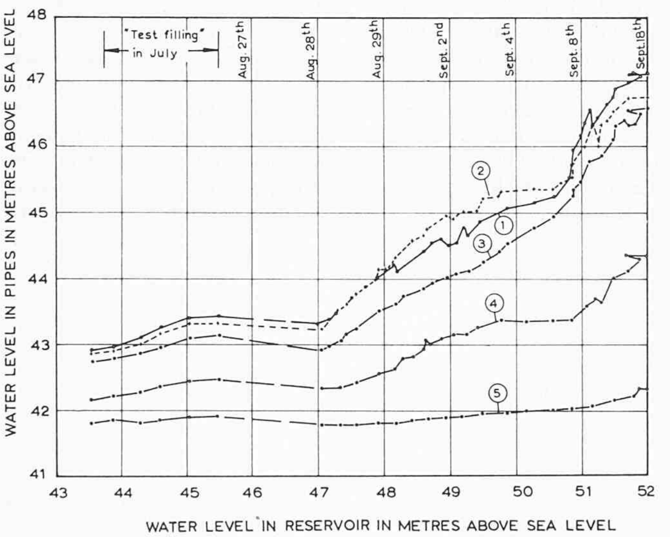

Fig. 6b. Standpipe Levels in Installations 1—5 versus Water Level in Reservoir

the river crossing might become impassable. Instead of simply stockpiling the rock for possible future use consideration was given to the feasibility of placing the rock downstream of the dam in such a way that it would increase the stability of the dam. The argument against such a procedure was that, if trouble should arise, it would be a simple decision to remove the rock from a stockpile and place it where it was needed; however, the decision would be problematic if the material could only be removed from a location where it had purposely been placed to be effectful.

Performance and supplementary construction of the dam during first filling of the reservoir

On August 26th, 1964, the gates at the Skogfoss Dam were lowered and the river was closed. During the following three weeks the water level was raised from short of 45 m above sea level to the maximum water level of 51,87 m above sea level.

As early as August 28th, at a water level of 47,0 m above sea level, piping was observed downstream of the Mänika Dam. On the left bank boils were observed and a slight increase in leakage and standpipe level was registered. The following day the water level had risen another metre and the boils increased in number as well as in size; the leakage was almost doubled and the standpipe level in installations 1 and 2 had risen as much as the reservoir level, Fig. 6b. Downstream of the dam a pool with a water level at approximately 41,5 m was created; its water level was controlled by the overflow weir I. Most of the boils were concentrated close to this pool along the left bank; the largest boil was approximately 10 metres downstream of the dam. The water flowing out of the boils carried a relatively large amount of fine sand.

It was beneficial to dig a hole in this area to relieve the water pressure and an excavation machine was set into operation. A trench which started near the pool was opened in the area containing the largest boils. The trench was excavated through a layer of coarse material down to approximately one metre in fine material, a narrow-graded fine sand 90% of which had a grain size between 0,06 and 0,2 mm. At a certain stage during the excavation of the trench a regular hydraulic failure could be observed in the bottom, the amount of water boiling out increased remarkedly and the amount of sand transported with it gave reason for being concerned. The leakage measured at the weirs increased significantly and the water level in standpipe 1 nearest to the trench dropped. It was found necessary to backfill the trench as rapidly as possible to reduce the transport of fines; therefore, it was filled with sandy gravel the same evening.

During the course of excavating the trench the reason for the development of the boils became apparent. The seepage from the reservoir built up an excess pressure downstream of the dam because of the presence of the relatively impervious layer of fine sand resting on coarser material probably communicating with the reservoir. It was further observed that this pressure had to be relieved by excavating through the layer of sand in a water-filled trench if a dangerous transport of material was to be avoided. The principal question at this stage was whether the raising of the water level in the reservoir should go on or not. It was decided that the storing should continue, but mainly during the daytime when the behaviour of the dam could be observed. It was obvious, however, that at this stage the dam as constructed could not be regarded as a structure capable of safely impounding water that had to be raised an additional 4,0 m, that is, for an increase in head between the storage level and tail water in the pool from 6,5 to 10,5 m or an increase of 60%.

Several measures for securing the stability of the dam were discussed. The tail water level could be raised and thereby reduce the head difference across the dam, one could load the area downstream of the dam with a pervious material which would have to fulfil the necessary filter requirements or one could possibly relieve the pressure by means of wells and drainage trenches. A combination of relief trenches and surcharge loading was decided upon.

On August 30th, according to this plan, the excavation of a trench along the slope of the left bank was started. During excavation the trench was kept full of water and as the work progressed it was backfilled with gravel. At the same time sand and gravel were taken from the borrow pit, that had been used during construction, and placed on the slope on the same side of the river bed just downstream of the dam. The fine sand excavated from the trench was loaded into lorries and dumped upstream of the dam along the shore of the reservoir in the borrow pit because layers of pervious material through which leakage could occur had been observed in this area.

After several days the trench backfilled with gravel extended along the left bank over a length of approximately 30 m and to a depth of 2 to 3 m. Along its entire length the trench had been excavated into the fine sand layer. It can be seen in Fig. 6b that as a result of this excavation the rate of increase of the standpipe level in installation 1 relative to the increase in reservoir level was somewhat reduced and the standpipe level had probably been lowered half a meter due to this work. Even if this was an intentional effect, the overall results of the work were somewhat disappointing. New boils developed close to the trench and it was obvious that the relieving effect of the trench was less than anticipated primarily because the material used to backfill the trench, which had to fulfil the filter requirements, was pervious relative to the sand but was not pervious enough to permit passage of the relative large volume of seepage. It even appeared that in some locations an excess pore pressure was developed in the backfill material in the trench. As a means of improving drainage from the trench the decision was made to excavate a narrow and shallow trench in the backfilled gravel and install a drainage pipe.

After the rather disappointing experience with the drainage trench and as more boils developed as the reservoir level increased, it was evident that the stability of the dam had to be insured first of all by applying a surcharge over the area downstream of the dam. Paced by the development of new boils the loading of this area with gravel

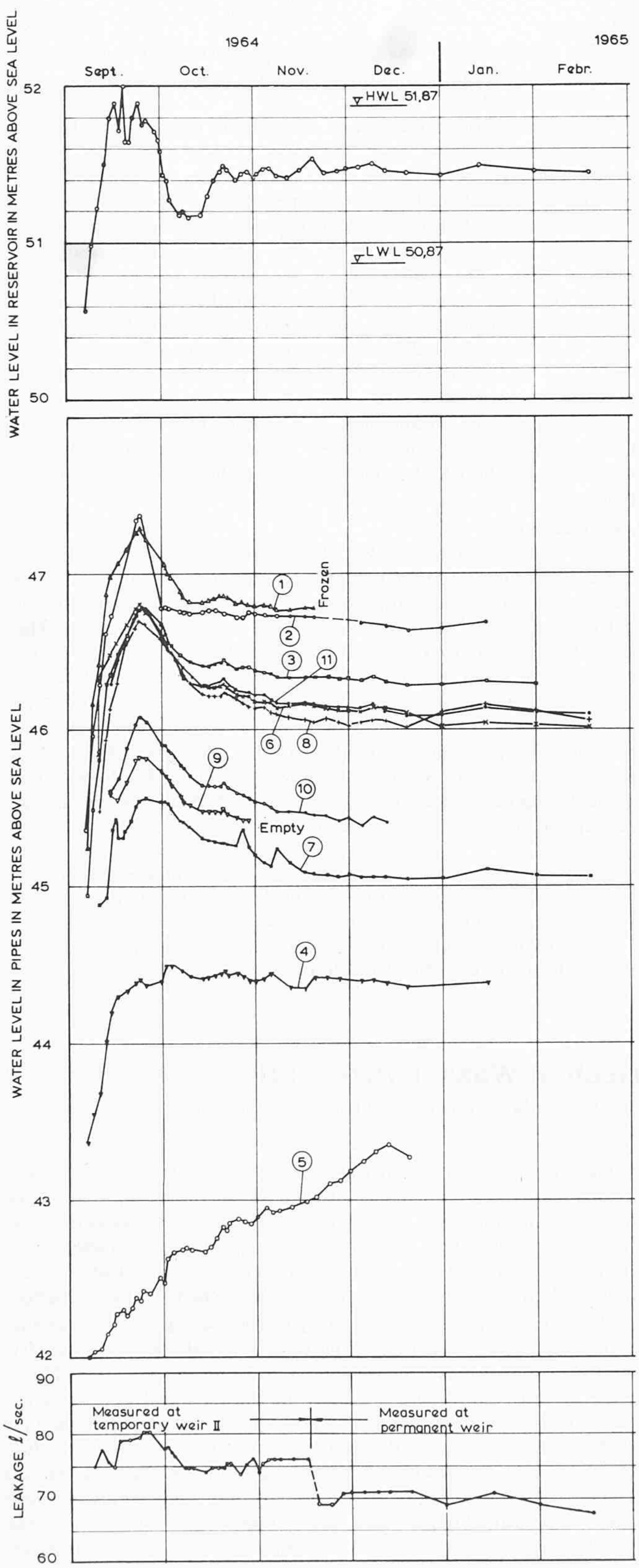

Fig. 7. Water Level in Reservoir, Standpipe Level and Leakage versus Time

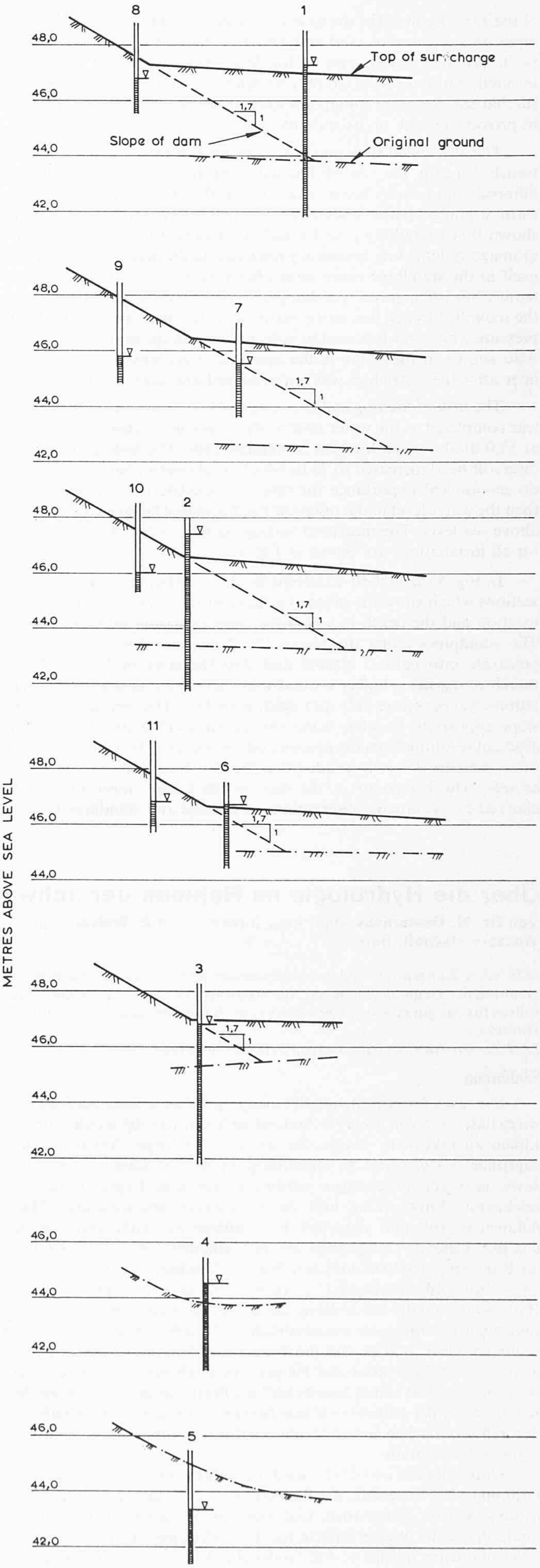

Fig. 8. Highest Standpipe Levels recorded

started along the toe of the dam and along the left-hand side of the bank into the tail water pool and against the right-hand side of the previous river bed. As the fill material was being placed new boils developed continuously in front of the surcharge. The loading was extended until eventually all the boils were covered. The area which was covered by the surcharge is shown in Fig. 5 and in Fig. 4 the thickness of the gravel load is shown in the cross section of the dam. During the placement of the surcharge the overflow weir I was demolished partly because it was felt necessary to reduce the pore pressure developed at standpipes 1, 2 and 3 which could be harmful to the downstream shell of the dam. Removal of the weir also made it possible to place drainage pipes on a cushion of sand and gravel at the bottom of the tail water pool. As anticipated, new boils developed as the tail water was lowered, but these occurred only within the area that was subsequently surcharged. Six more standpipes were installed in the slope of the dam to provide a check on its stability.

The stockpiled rock was placed as an elongation of the drainage trench beneath the toe of the dam. As the gravel surcharge was dumped, stones were handpicked or raked out to be placed so as to form drainage paths underneath the fill. Previous experience had shown that everything possible had to be done to obtain an effective drainage system. Any temporary reduction in drainage would manifest itself in the standpipe water level observations; for example, on September 8th when gravel was dumped in such a way that drainage from the rockfilled trench was partly restricted this caused an increased pore pressure which was followed by a decreased leakage as shown in Fig. 6. Also shown in the figure is the opposite effect which was produced later after the restriction was removed and drainage re-established.

The task of loading and draining the area downstream of the dam was completed as the water level in the reservoir reached its maximum at 52,0 m above sea level on September 18th. The leakage under full reservoir head increased to 78 l/s which is, of course, fairly large but of no economical importance for this hydroelectric development. Since then the water level in the reservoir has fluctuated between 51 and 52 m above sea level. The measured leakage as well as the standpipe levels for all installations are shown in Fig. 7.

In Fig. 8 the highest standpipe levels are indicated in several cross sections which show the original ground surface, the surcharge at that location and the depth to which the pipes penetrate into the ground. The standpipes along the toe of the dam, numbers 1, 2, 3 and 6, penetrate into natural ground and into the layer of fine sand and therefore register a higher ground-water level than at the other installations that penetrate only into sandy gravel fill. The seepage line in the slope appears to be safely below the surface and the upward hydraulic gradient resulting from the pressure difference is safely below 1,0. From these sketches it was concluded that the structure could be considered as safe. The behaviour of the dam in the future, however, will be checked by systematic observations of leakage and standpipe levels.

Conclusions

The Mänika Dam was designed and constructed in accordance with an observational procedure or perhaps more correctly by a trial and error method.

The dam is built on a deposit of sand and gravel imbedded with layers and lenses of fine sand. The major problem, therefore, was to prevent a dangerous reduction of effective stresses downstream of the dam and to prevent washing out of fine material. The most adequate means of accomplishing these objectives was found to be by draining and loading the area downstream of the dam with a gravel surcharge and drainage pipes. The realization of this proved to be complicated because the large amount of seepage water that had to be drained required coarse material which was available only in limited quantities.

It would have been difficult to prepare a design of the dam which would have taken into consideration all the problems involved and the necessary comprehensive field explorations would have increased the cost of the structure considerably. It is believed that the observational procedure which was followed led to an adequate and relatively inexpensive solution that would have been difficult to reach in any other way.

Acknowledgements

The Skogfoss Hydro-Power Plant is owned by Sameiet Skogfoss Kraftlag, a joint venture of A/S Sydvaranger and Varanger Kraftlag. Their consultant is Rådg. ingeniør B. A. Berdal, and the main contractor for the work is Pasvikkraft A/S, a joint venture of Ingenior Thor Furuholmen A/S and Astrup & Aubert A/S. The Norwegian Geotechnical Institute acted as advisor to the consultant on special matters concerning the Mänika Dam. The authors wish to express their gratitude for the permission to publish this paper and to everyone concerned for pleasant cooperation during all stages of the work. Further, the authors wish to acknowledge that Mr. T. Rønning, Resident Engineer, and his assistant, Mr. M. Andersen, have been in charge of the measurements at the site.

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Address of the authors: *L. Bjerrum* and *B. Kjaernsli*, Norges Geotekniske Institute, Oslo-Blindern, Norway.

Über die Hydrologie im Rahmen der schweizerischen Wasserwirtschaft

DK 551.49:627.8.09

Von Dr. **M. Oesterhaus**, dipl. Ing., Direktor, und **E. Walser**, dipl. Ing., Chef der Unterabteilung Landeshydrographie des Eidg. Amtes für Wasserwirtschaft, Bern

«Oft eilen Konzeption und Lösungsversuch einer konkreten Aufgabe der Kenntnis der Grundlagen voraus, die eigentlich die Voraussetzungen bilden sollten für das gute Gelingen des Werkes in technischer und wirtschaftlicher Hinsicht.»

Prof. *G. Schnitter*, Vortrag Generalversammlung GEP 1964

Einleitung

Die rasch fortschreitende Entwicklung auf dem Gebiet der Wasserwirtschaft muss von unserem Amt aufmerksam verfolgt werden, um die Linien zu erkennen, welche der heute so wichtigen Vorausplanung zugrunde zu legen sind. In verwaltungsinternen Studien und Berichten sowie in Veröffentlichungen werden die jeweilige Lage, die sich abzeichnende Entwicklung und die erforderlich erscheinenden Massnahmen geprüft und abgeklärt. So benützen wir auch gerne die sich uns hier bietende Gelegenheit, einige Gedanken über die Hydrologie im Rahmen der schweizerischen Wasserwirtschaft zu äussern, die wir als wichtig und grundlegend für die notwendige weitere Förderung der Hydrologie und die Behandlung der sich in stark zunehmendem Masse stellenden gesamtwasserwirtschaftlichen Aufgaben erachten. Wir tun es um so lieber, weil es sich um Fragen handelt, die Professor *Gerold Schnitter* beruflich sowie aus Neigung beschäftigen und interessieren, besonders auch in seiner Eigenschaft als Präsident der Hydrologischen Kommission der Schweizerischen Naturforschenden Gesellschaft und des schweizerischen Landeskomitees für das internationale hydrologische Dezennium.

Unter *Wasserwirtschaft* wird im allgemeinen die zielbewusste Ordnung aller menschlichen Einwirkungen auf das ober- und unterirdische Wasser verstanden, und zwar im Hinblick auf eine umfassende Pflege der Wasserschätze für die zahlreichen Bedürfnisse kultureller und wirtschaftlicher Art. In der Bundesverwaltung befassen sich in mehr oder weniger grossem Umfang und zum Teil auf ganz speziellen Gebieten sechs Amtsstellen mit wasserwirtschaftlichen Aufgaben, nämlich die Ämter für Wasserwirtschaft, für Gewässerschutz und für Strassen- und Flussbau, das Meliorationsamt, die Inspektion für Forstwesen, Jagd und Fischerei sowie das Gesundheitsamt. Die folgenden Ausführungen werden sich weitgehend im Rahmen der unserem Amt übertragenen wasserwirtschaftlichen Aufgaben halten.

Eine der wichtigsten, vielleicht gar die wichtigste aller Voraussetzungen für eine moderne, erfolgreiche und rationelle Wasserwirtschaft ist die *Hydrologie*. Sie wird unter anderem gemäss Vorschlägen, welche im Zusammenhang mit dem Internationalen Hydrologischen Dezennium gemacht worden sind, als das weite Forschungsgebiet und die Wissenschaft definiert, die sich mit dem ober- und unterirdischen Wasser, seiner Entstehung, Erscheinung und Verbreitung, seinen chemischen, physikalischen und biologischen Eigenschaften sowie seinen Wechselwirkungen mit der Umgebung, insbesondere der menschlichen Tätigkeit, befasst. Charakteristisch für die Hydrologie ist unter anderem ihre ausgesprochen starke Verbundenheit mit andern Naturwissenschaften und das grosse Interesse, das ihr von der Praxis entgegengebracht wird.

Wenn wir nun von verschiedenen Forschungsarten sprechen, so möchten wir vorweg darauf hinweisen, dass in dem hier betrachteten Rahmen die Forschung mit kommerziellen Zwecken keine wesentliche Rolle spielt. Die Hydrologie bietet ausser der angewandten auch der *reinen Forschung*, dem für unsere Kultur so bedeutungsvollen, von praktischen Zwecken freien Streben nach Erkenntnis, lockende und interessante Ziele. Durch *langfristige Grundlagenforschung*, ob es nun bereits feststehe, dass deren Ergebnisse praktisch angewendet werden können oder nicht, dient die Hydrologie verschiedenen Zielen, so besonders auch einer weitblickenden Wasserwirtschaft. Nur auf derart gewonnenen Grundlagen wird einerseits im erforderlichen Ausmasse