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Construction of a Pile Wall in a Rockfill Dam

Construction d'une Paroi de Pieux dans un Barrage en Remblais Rocheux

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SYNOPSIS In conjunction with the expansion of the Harsprångets power station in northern Sweden, a large part of a 20 m high rock-fill dam had to be excavated and removed while, at the same time, the level of water upstream of the dam could not be lowered. After a number of alternative plans had been considered, it was decided that the excavation work would be carried out within pile walls, consisting of drilled-in, reinforced tubular steel piles. The gaps between the piles were sealed with shotcrete and the walls were anchored in the rock by prestressed tiebacks. A comprehensive control programme was prescribed. Only very small movements in the dam could be permitted. The work was commissioned by the Swedish State Power Board, the main contractor was Stabilator AB and the construction work was carried out by Geoberäkningar AB.

INTRODUCTION

The capacity of the Harsprånget hydroelectric power station on the Lule Älv river in northern Sweden has recently been extended to 940 MW. The last stage of the conversion included the installation of a new 460 MW generator. The intake for this was constructed immediately downstream of the power station's dam. A description of the extension work has been published in an earlier report (Mårtensson, 1979). The dam, which has a maximum height of 50 m, has a blast stone fill and a core of reinforced concrete (see figure 1). The dam was constructed in 1949, although the method of construction, i.e. with a concrete core and rock-fill embankments, is not the technique generally employed for such dams today.

The dam stands on hard granite. The intake to the new turbine consists of a vertical rock shaft, the top end of which emerges near the downstream dam embankment. Work on the site was carried out in three phases:

1) The construction of temporary drilled-in piles, from the downstream embankment as far as the existing concrete core, and the excavation and removal of the rock fill between the piles.

2) The construction of permanent concrete retaining walls between the pile walls.

3) The excavation and removal of the rock fill upstream or the core. Demolition of the core. These operations were carried out under water.

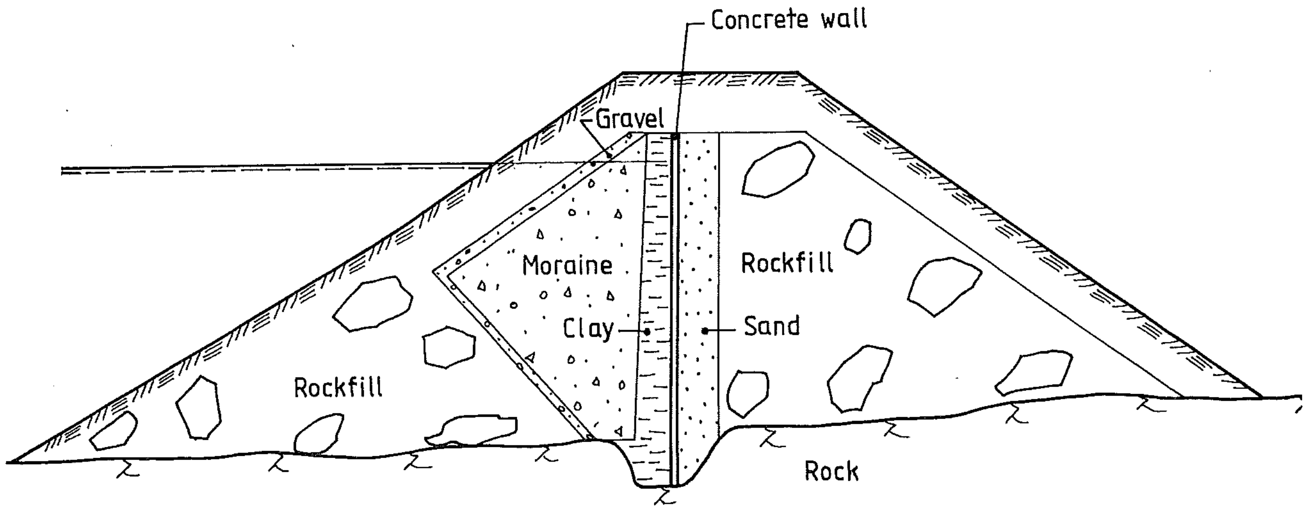

Fig. 1. Cross-section of the dam.

DESIGN OF THE PILE WALLS

Since each of the two retaining walls were to be constructed in an area excavated between pile walls, four pile walls were necessary. One of the pile walls is illustrated in figure 2.

The rock fill in the dam consisted largely of 2 to 3 m^3 blast stone boulders, and it was therefore impossible to use conventional steel piles. After a number of alternatives had been studied, it was decided to construct the pile walls of drilled-in tubular steel piles with a diameter of 114 mm and a wall thickness of 7 mm. To increase their resistance to bending, each pile was reinforced internally by six 16 mm dia. reinforcement steel rods and filled with grout. The distance between the piles was 500 mm and the gaps were sealed with shotcrete (see figure 3).

The construction work was not to interfere with the normal operation of the power station and, because of this, the water level could not be lowered during the construction period. The height of the dam where the construction work was carried out was about 18 metres. This paper deals with the design and construction of the temporary pile walls.

The method of drilling used was Stabilator's Alvik-J method, in which the casing follows the bit during drilling. When the drilling steel was withdrawn, one section of the bit was left in the hole (see figure 4). It should be emphasised that drilling through 18 m of rock fill, consisting of large boulders of varying composition, is a highly qualified operation, requiring an extremely skilful crew.

In this operation, the deviation between the calculated drilling direction and the actual position of the bit never varied more than about 10 cm in 15 m of drilling, which, under the conditions existing at the site, must be seen as a very satisfactory result.

Since the permanent concrete structure was to be constructed in the excavated area between the pile walls, pile tiebacks were prescribed. This implied that the site itself would be unobstructed by struts and braces. The anchorage consisted of prestressed wire rope tiebacks. Each tieback comprised 7 Dyform ropes, with a diameter of 13 mm and of St 160/180 grade steel. Accordingly, the ultimate tensile strength of each tieback was 1.6 MN. The tiebacks were spaced about 3 m apart.

Anchorage of the tiebacks to the piles was effected by means of channel beams, erected on brackets welded to the piles (see figure 3). The design of the anchor fittings and the locking of the wedges is patented (VSL system).

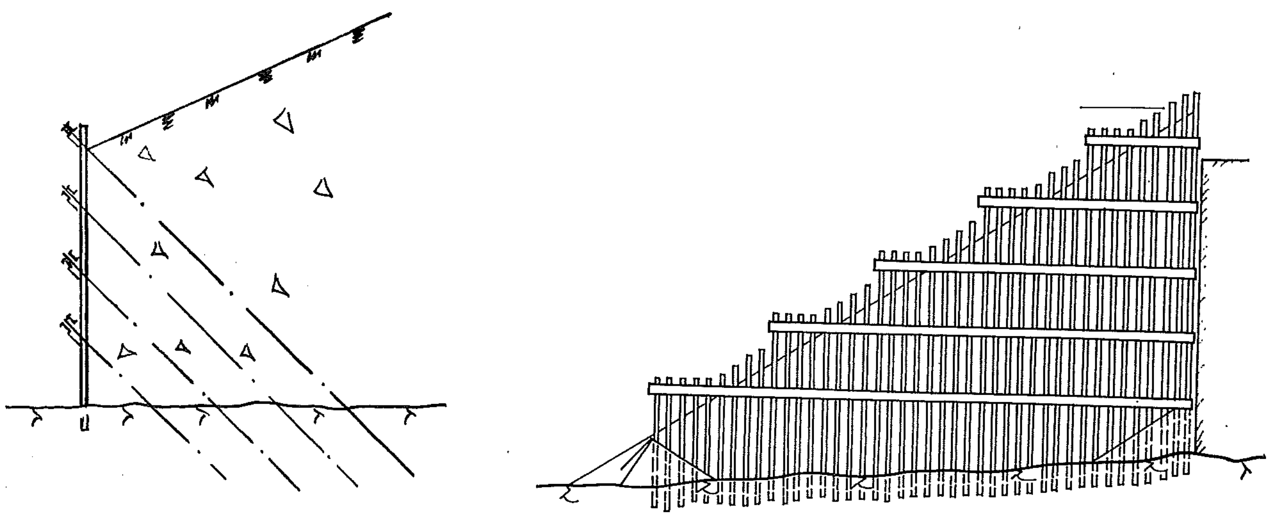

Figure 2. Elevation of pile wall.

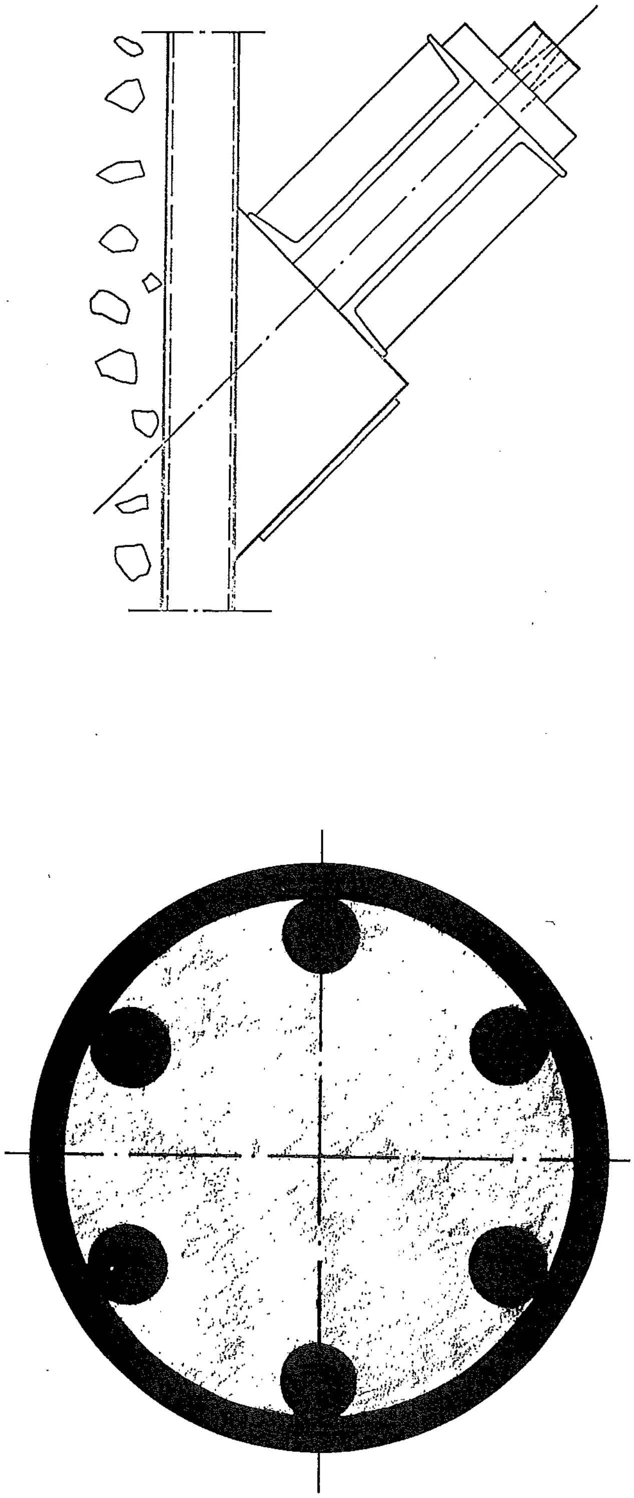

Figure 3. Details of construction.

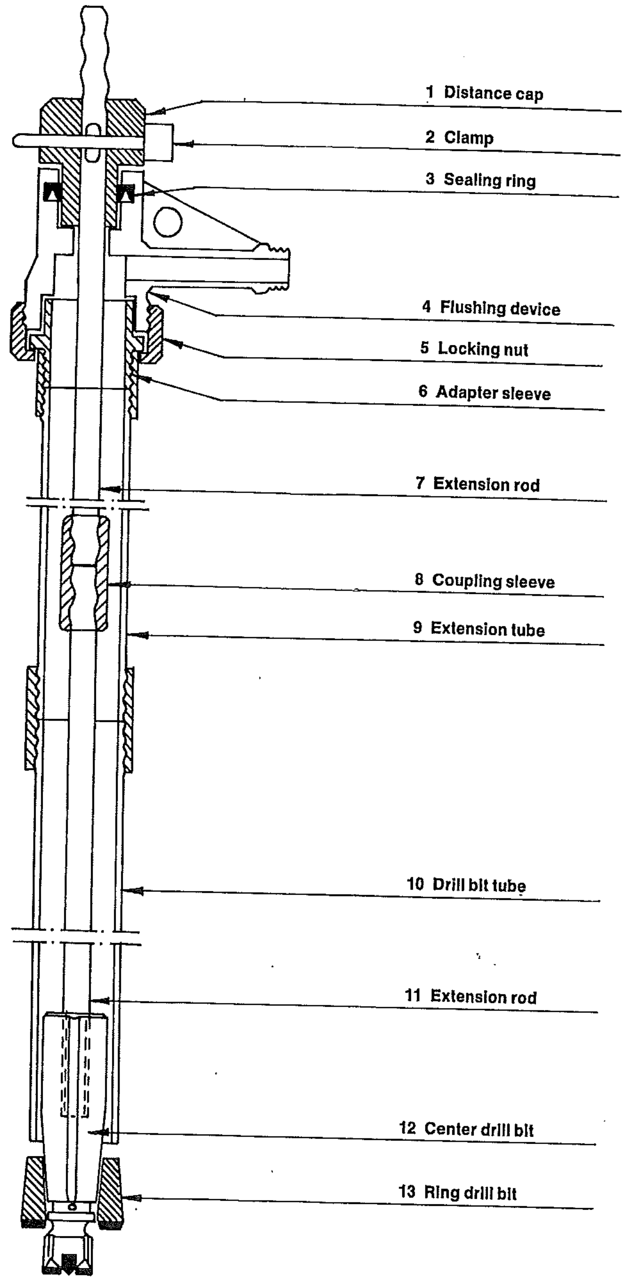

Figure 4. The Alvik J drilling method, with the ring bit left in the hole.

Anchorage at the other end of the tiebacks was effected by their being grouted-in to a depth of 3 - 5 m in the bedrock. As for the vertical piles, the Alvik-J method was employed. The tiebacks were set at a declination angle of 45°, which is accepted practice in Sweden for the anchorage in rock of pile tiebacks (Broms, 1968). The tieback components are made of cold drawn steel, which rapidly loses its strength on heating. Thus, the tieback components and walers were enclosed in fire insulation consisting of 5 cm mineral wool, as shown in the photographs in figures 5 - 7.

DESIGN CONDITIONS

In accordance with Swedish building standard SBN-75, it was assumed that an evenly distributed, horizontal, lateral pressure of 30 kPa would be applied to the retaining walls. The magnitude of the lateral pressure is governed not only by the height of the wall and the properties of the soil but also by the magnitude of the prestressed load (Stille, 1976) and the rigidity of the wall (Terzaghi, 1943).

In respect of the vertical, drilled-in piles, the calculated lateral pressure was increased by 40% to allow for the estimated deviations in the construction. A corresponding increase of 25% was made for the channel beams. The theory of plasticity was used in the sizing.

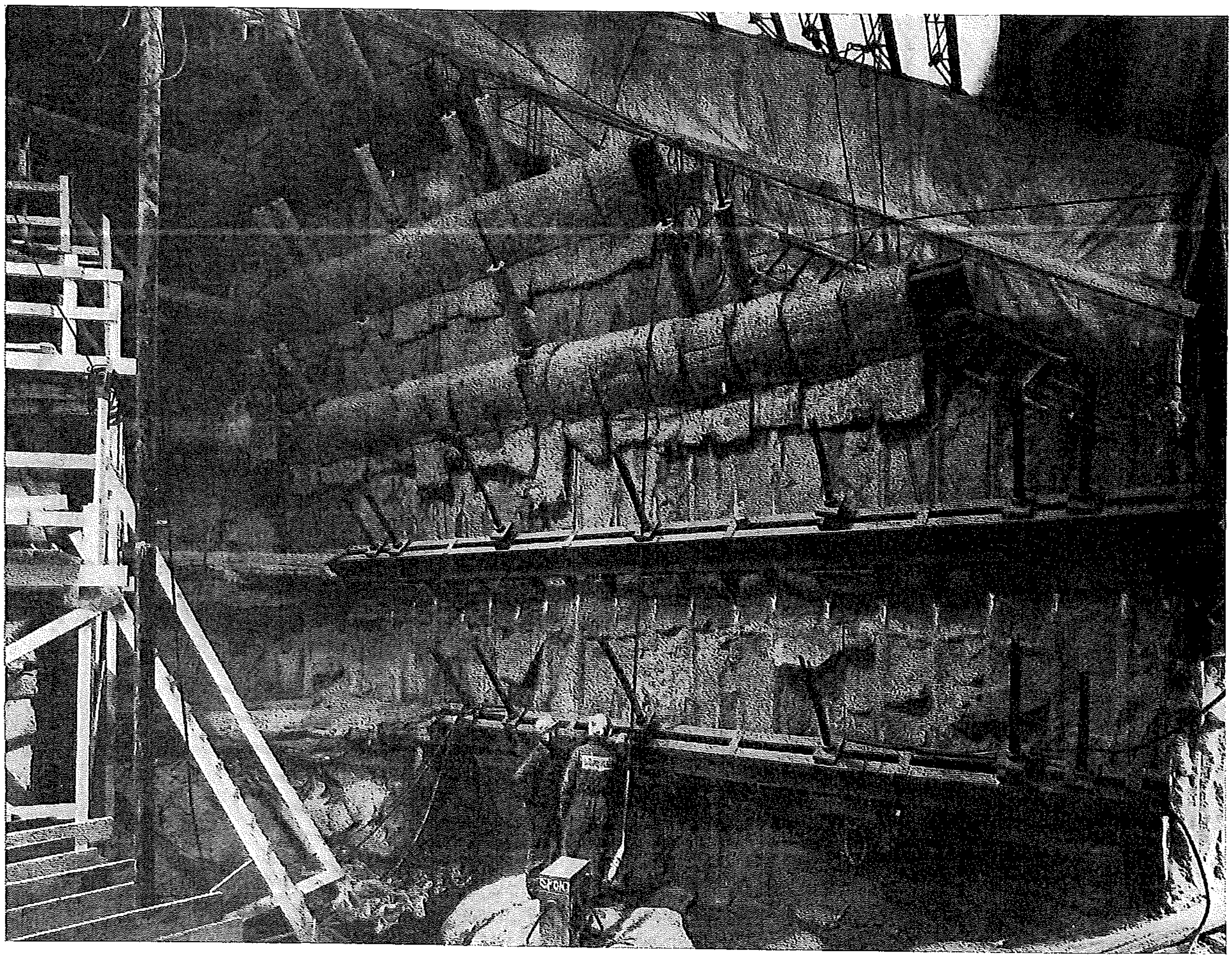

Figure 5 Elevation of the No. 1 east wall

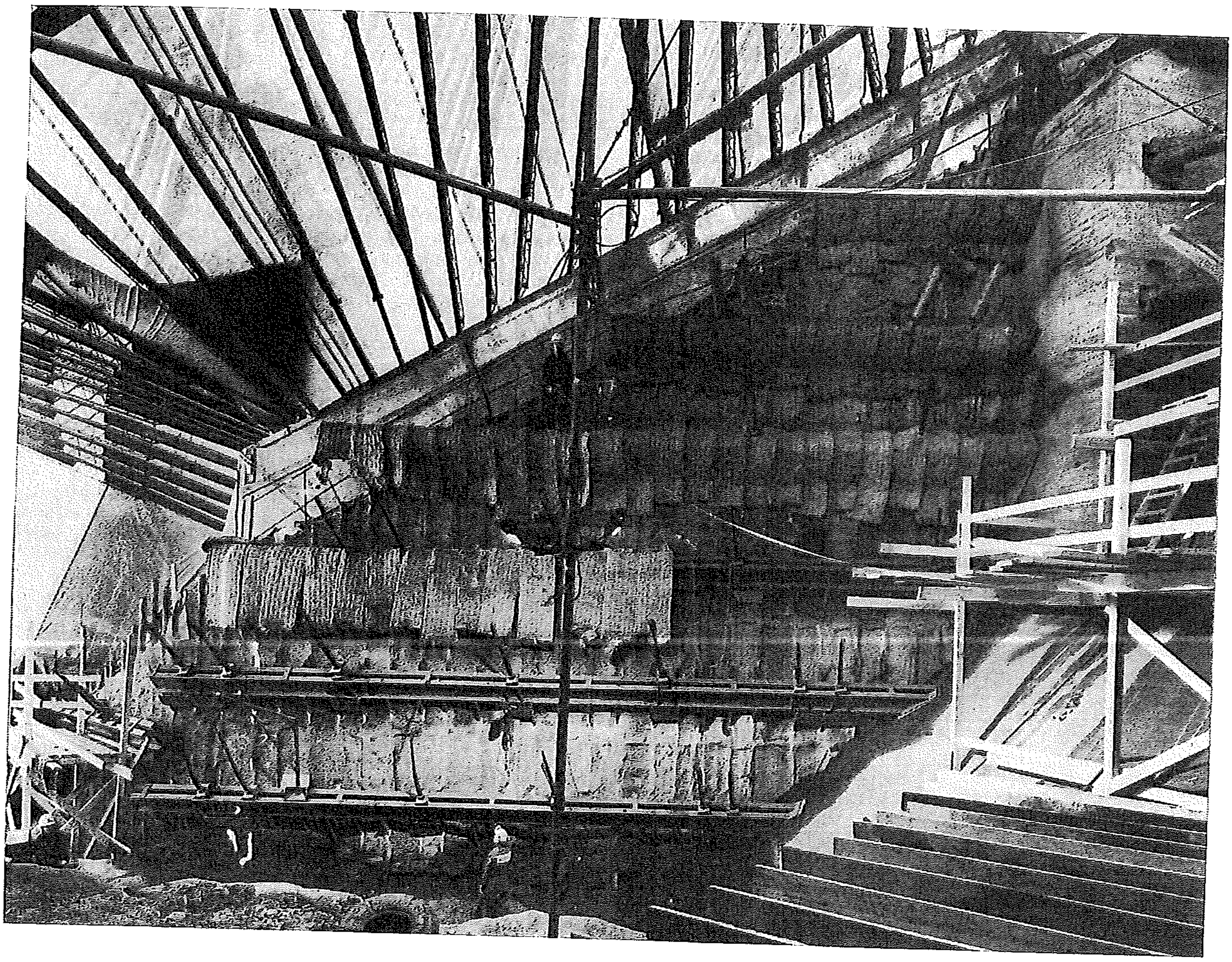

Figure 6 Elevation of the No. 1 west wall

The tiebacks were subjected to a test load of 700 kN, which represents twice the calculated working load. Because of the relatively high test load, the stresses in the steelwork were substantially higher during the tests than they would be in practice. In the calculation of the test loading, it was assumed that the piles would act as beams on an elastic foundation and as elastic supports for the walers (Bredenberg, 1978). The maximum recorded horizontal movement in the soil was about 1.5 cm, which corresponds to a modulus of subgrade reaction for the rock fill of about 40 MPa.

THE CONSTRUCTION WORK

The vertical piles were drilled at least 500 mm into solid rock. Rock control at points about 3 metres apart was performed during the drilling to a depth of 5 m below the surface of the rock.

After cleaning and checking of the straightness of each pile, the reinforcements were fitted and the piles filled with grout. The water/cement ratio of the grout was between 0.30 and 0.33. Standard cement containing an intrusion aid was used. Mixing was carried out in a colloid mixer and the grouting work employed a screw pump and hose. The nozzle of the hose was fed through to the bottom of the pile and was not withdrawn until a clean mix overflowed from the top of the drill casing.

Since some of the construction work was to be carried out during the winter at temperatures down to -40°C, a shelter was erected over the site to prevent the work from being hampered by snow and severe cold. The structure of the shelter is illustrated in figures 5 - 7.

The same equipment was used in drilling for the tiebacks as in drilling for the vertical piles, and the inner drilling steel was used for drilling down to a depth of 3 - 5 m in the rock. Once the drilling steel had been withdrawn (leaving the casing in the hole), the hole was flushed clean and leakage checks were performed on the rock (water-loss measurement). The temperature of the rock was also recorded. Where leakage through the rock was judged to be excessive, grouting was carried out, followed by redrilling. Pumps and hoses were used in the anchorage operation for the tiebacks, to a distance 3 m from the end of the wire rope. Thereafter, an elastic binding was fitted to the rope so that the free length would not be shorter than intended. The grout mix had a water/cement ratio of 0.45 - 0.50. Rapid hardening cement containing 1% by weight of intrusion aid was used.

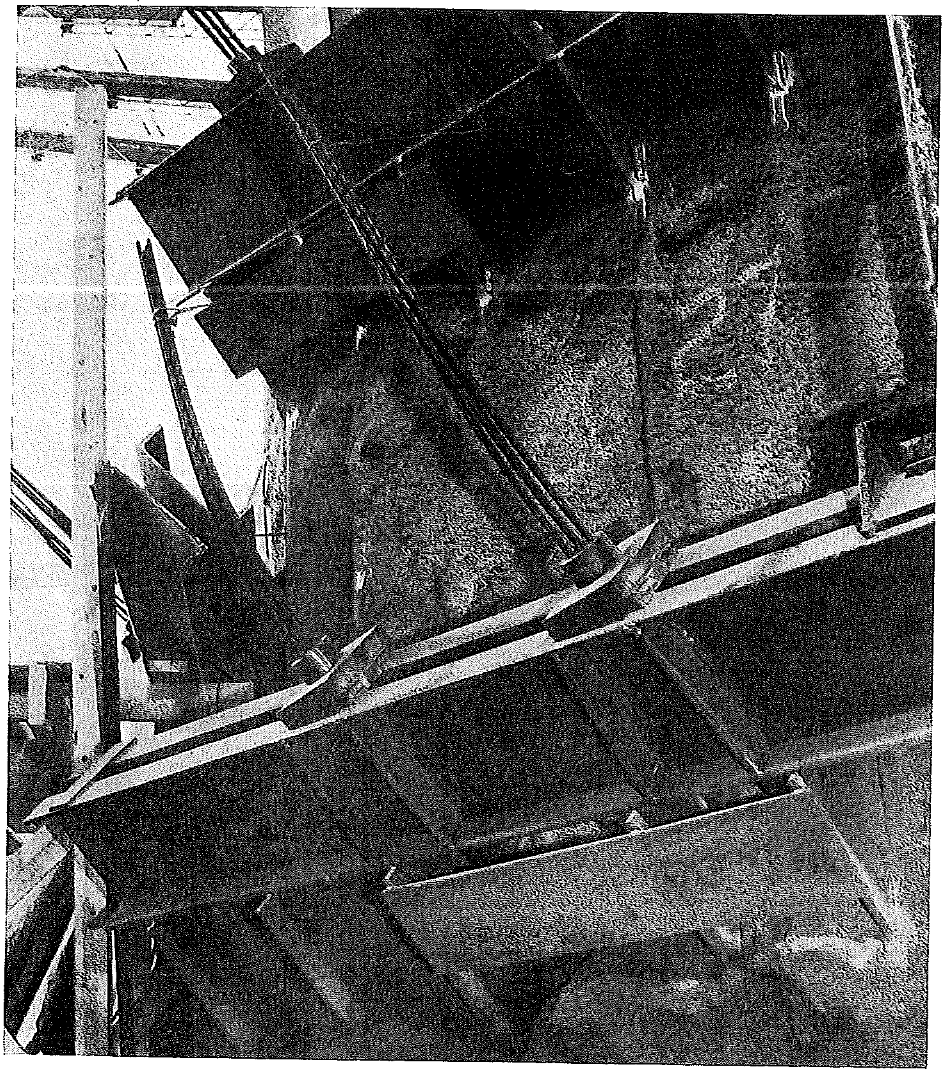

Figure 7 Welding details

A special schedule was followed in the test loading, the objectives of which were as follows:

1) Checking of the anchorage in the rock
2) Pressing of the locking wedges into the wire ropes
3) Checking of the anchor components
4) Checking the free length of the tieback, in other words, that the grouting had been carried out in accordance with the specification.

A test load of 700 kN was applied for 10 minutes, during which time creepage should gradually decline but should never exceed the prescribed maximum value of 2 mm. The tieback anchorage was then adjusted to a remaining prestressed value of 350 kN. Test loading and tensioning were carried out between 5 and 7 days after erection.

The tiebacks and walers were erected in stages, in pace with the excavation work. The gaps between the piles were sealed successively with shotcrete. The maximum permissible wall area that could be exposed in each phase before shotcrete had to be applied was 1 x 2 m (height x width). The shotcrete was applied in arch form with a rise of 5 cm.

CHECKING PROCEDURES

Excessive movement or failure in the piling could result in damage to the dam core which, throughout the construction period, was subjected to water pressure on one side.

In view of the extensive damage that could result from this, a comprehensive control programme was prescribed.

Throughout the construction period, monthly checks were made at three points close to the core of the horizontal movement of the dam. The measuring instruments used were a theodolite and an inclinometer (Kallstenius and Bergau, 1960).

The maximum movement was 2 cm, measured in a downstream direction at the crest. The movement occurred at a fairly early juncture and was probably associated with the preliminary excavation of the downstream embankment, which reduced the resistance of the embankment to slide. Theoretical analysis of the movement found that movement of this magnitude was to be expected.

However, such calculations are entirely dependent on the assumed deformation parameters and are therefore fairly unreliable. Moreover, movements due to the shaking and vibration generated during compaction cannot be calculated in advance unless an analysis, employing the finite element method, for example, is limited to normal movements caused by static changes to the previous equilibrium during the operating stage.

During the drilling-in of the vertical piles, a rock temperature of around 0°C was recorded. This provided effective cooling of the drill bit, but raised the question about the suitability of conventional grouting methods. A series of tests, on the grouting of tiebacks, in which various grout mixes and grouting methods were used, were therefore carried out. During the testing period, the drill holes were heated by steam to a temperature of 4°C before the grout was introduced, although this was found to be a difficult job owing to the low outdoor temperatures and the enclosure of the site in the shelter.

The outcome of the tests was that the anchoring of the tiebacks was carried out in the conventional way, as described above. The tests established that no advantage could be obtained from the use of a standard cement mix with a water/cement ratio of 0.30. None of the 146 tiebacks parted from its anchorage.

A transducer was used to keep a continuous check on the forces in 26 tiebacks (see figure 8).

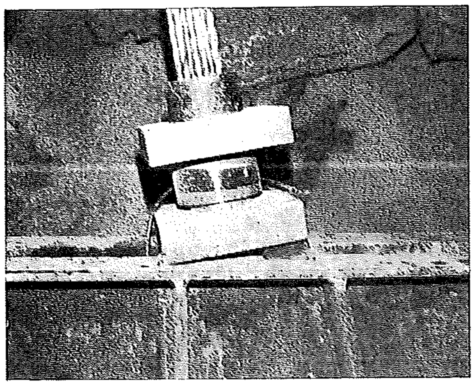

Figure 8. Tieback force transducer.

The transducers consisted of resistive strain gauges connected in a full bridge. The guage foil was attached to a specially treated section of a pipe. After 2 years in operation, a check was made on the functioning of some of the transducers but no loss of accuracy of practical importance was observed. This type of transducer is much more economical than pneumatic transducers. In addition, in contrast to transducers based on piezo electric crystals, they are quite suitable for long-term field use on construction sites.

The main concern, of course, was that the tieback forces would increase in time. The implication of this would be that the estimated values for soil pressure were unreliable. However, the readings showed that in most tiebacks the force decreased slightly after the tiebacks had been anchored at 350 kN (see figure 9).

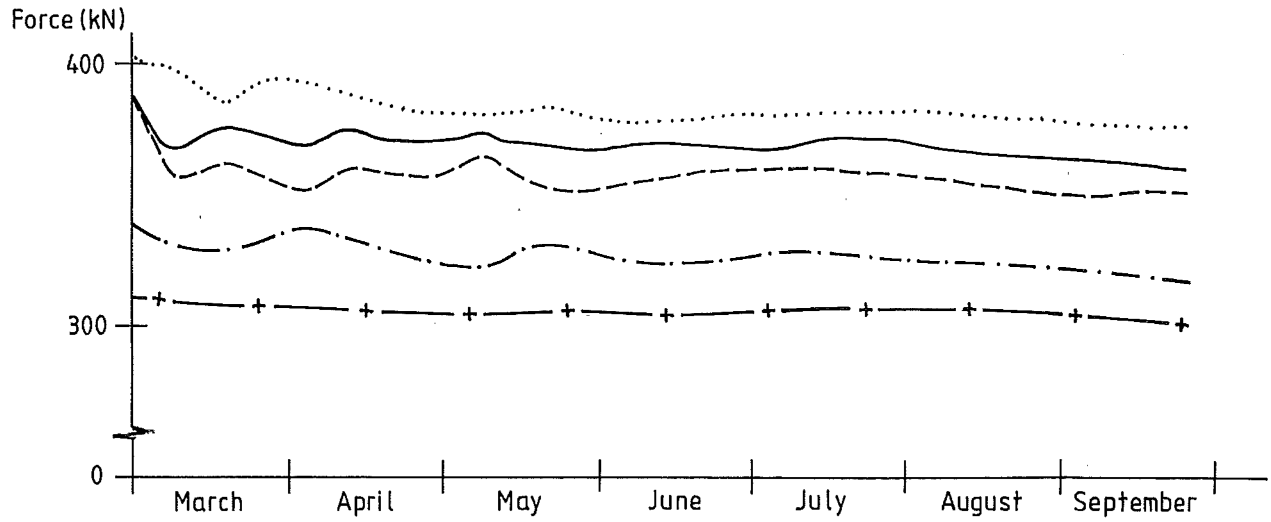

Figure 9. The variation in the tieback force with time.

At worst, this decrease could mean that the anchorage in the rock had started to creep, which could conceivably be associated with the low temperature at which the grouting work was carried out. Fortunately, however, there was a more straightforward explanation, namely, that the prescribed anchoring force that was applied was of such magnitude that the soil pressure at rest at the respective anchorage point was also of such magnitude that there was movement towards the soil. It was easy to show the theoretical possibility of this. As an additional check, the distance from the opposite pile wall was measured, which proved conclusively that there was movement inwards.

Control of all welding work was extremely thorough and included radiography, X-ray photography and magnetic particle crack detection. The control requirements were consistent with those applicable to nuclear power stations. In view of the sometimes very difficult conditions under which the welders had to work, the number of defective welds was quite small. One particular problem that demands attention in the welding of temporary structures of this type is that most of the welding takes place on sections which are subjected to high stresses. This reduces the scope for using preheating and fixtures.

SUMMARY

. A drilled-in pile wall has been constructed in an 18 m high rock-fill dam, partly consisting of large boulders. Only small movements of about 2 cm occurred in the dam during the construction period.

. Anchorage of the tiebacks in the rock at temperatures of as low as 0°C was carried out using rapid hardening cement with a water/cement ratio of 0.45 - 0.50. The tiebacks were subjected to a test loading of 700 kN after 5 or 7 days.

. When high factors of safety for the soil pressure calculations and the tieback forces are prescribed, the permanent tensile forces in the tiebacks will be of such a magnitude that the pile wall will creep towards the soil during the period of operation, As a result of this movement the tieback length will decrease slightly and the tieback tension decreases correspondingly.

. The use of resistive transducers, which were glued to specially treated steel pipe sections allowed economical and reliable readings of the tieback forces to be made over a long time.

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