VSL

SOIL AND ROCK ANCHORS

EXAMPLES FROM PRACTICE

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VSL INTERNATIONAL LTD.
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Preface

In the recent past there has been a considerable upsurge in the use of soil and rock anchors and in many countries they have now established a permanent place in civil engineering practice. Nevertheless, there are many clients, contractors and engineers who are still not or only little familiar with modern anchoring technology. It is hoped that the present publication will prove useful in providing information on this subject.
This booklet contains a collection of descriptions of works, in which soil and rock anchors have been used. It is divided into seven chapters, each of which covers a typical of application and contains several examples. The aim of The aim of these examples is to show what has already been achieved in the field of soil and rock anchors and also to provide an overall picture ofthewiderangeofpossibleapplications. Various examples are described in detail and some of them contain theoretical considerations. They may be of help to the reader in the solving of his own soil mechanics problems. At the end of the booklet there is also a bibliography which provides a guide for further study.
The projects described here are naturally associated with the use of VSL anchors since the VSL Organisations have a wide range of experience in this specialised field. This experience extends to all types of anchors, whether for temporary or permanent purposes, of large or small bearing capacity and whether installed in simple or difficult ground conditions. The VSL Organisations are therefore in a position to advise and assist you at any time. The local VSL Representative or VSL INTERNATIONAL LTD, Berne, will be glad to receive your enquiry and to send you on request the special prospectus giving detailed information about the VSL strand anchors and the types and units which are avaible.

1. Securing of slopes

1.1. General

Slopes, rock faces and embankments frequently lose their stability as a result of natural phenomena such as penetration of water, icing and thawing or erosion; in most cases, the cause however is to be found in a modification to the form of the ground or the loading condition by human intervention. It is therefore not surprising that equilibrium disturbances of this kind occur predominantly in conjunction with excavated slopes in the construction of new roads and railways or extensions to existing systems. Such excavated slopes are frequently of considerable extent and in certain circumstances can even affect the stability of adjacent zones and in particular of slopes and rock faces situated above them.
The use of prestressed soil and rock anchors today provides an economical and suitable means for securing slopes or rock faces where the stability is not guaranteed. The actual amount of material used is only quite small but it enables the equilibrium to be re-established or maintained in accordance with simple basic rules and to the desired safety criteria.
In the case of fractured rock, the method consists of applying a prestressing force onto the unstable layers at the surface by means of the anchors, so that the friction in the fracture planes is increased and slip is prevented. In this way the upper layers are secured in the deeper, sound mass of rock, the load-bearing capacity of which has remained intact.
For excavated slopes the same procedure can be used for securing relatively steep rock faces, thus enabling the volume of excavation to be considerably reduced. In loose rock and soil, on the other hand, it is generally necessary to build supporting structures on the surface (see Section 2), which can also with advantage be anchored back. In both cases, the anchors enable the excavation work to be carried out in successive steps and eliminate the risk of rock falls and earth slips.
Securing of slopes often needs to be carried out in conjunction with hydroelectric power plants, for example to secure the abutments of dams. The principle of securing slopes with prestressed anchors can also be successfully applied in open-cast mining. Normally, the inclination of the slope is chosen to avoid slips, in other words it is equal to the angle of friction of the soil or rock. If anchors are used to secure the sides, however, these can be made much steeper and the useful volume thereby increased.
The prestressing force is usually transmitted to the underlying ground through foundations, against which the stressing anchorages of the ground anchors bear. The form and dimensions of these foundations depend predominantly upon the type and nature of the soil, but also upon the distribution of the anchors and the magnitude of theforcesto beapplied. The foundation may consist of isolated concrete blocks or, where the rock face is fractured or steep, of vertical and horizontal concrete beams. Where the ground is excavated by stages, prefabricated or insitu concrete tie-beams may also be used.

1.2.	**Consolidation of the rock projection of Baji-Krachen, Switzerland**
Client	Public Works Department, Canton Valais
Engineer	Dr. G. Lombardi, Locarno
Contractor	0. Caldart, Naters, in the name of the Consortium Figinen
Drilling	SIF-Groutbor SA, Renens
Anchors	VSL INTERNATIONALSA (formerly Precontrainte SA, Lausanne)
Rock investigations	Terrexpert AG, Berne
Years of construction	1971-1972

Introduction

The consolidation of the rock projection of Baji-Krachen near Gondo on the southern slope of the Simplon Pass was carried out in conjunction with the widening of this highway to three lanes.
The rock face rose almost vertically for 40 to 50 m and then continued its ascent at about 45°. This meant that large cuts into the rock were necessary. For obvious reasons, the excavation and securing work had to be carried out without interrupting the traffic on the existin road, with the exception of a few short closures.

Brief description of the problem

The geogical report showed that the report showed that the rock consisted of very hard

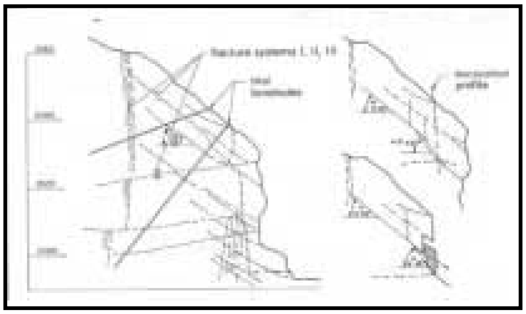


Fig. 1: Fracture systems of the rock projection

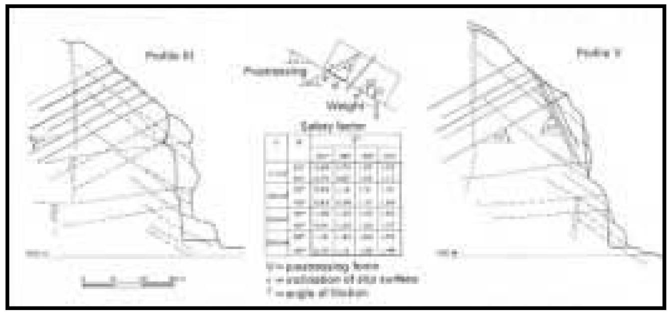


Fig. 3: Anchor arrangement and stability calculations

gneisses, which however were divided by various fracture systems into blocks. The position and orientation of some of these systems would jeopardise the stability of the rock mass. The investigation had revealed three main fracture systems, which are shown in fig. 1:

-A first system, sloping down towards the valley at 35 to 45° (I); this could clearly be the origin of slips.

-An almost vertical system (II), which was particularly pronounced in one area of the rearward slope. There the lower half of the rock projection had indeed separated from the remaining mass.

-An almost horizontal system (III), with a noticeable downward incline of 0 to 10° towards the mountain; this did not represent any direct risk, but mightlead to some overbreak along the slope.

The inclination of the entire first fracture system was, as already mentioned, from 35 to 45° (towards the valley). This situation is shown diagrammatically on the right side of fig. 1 for a specific fracture, the slope of which varied from 38 to 45° If the entirety of the upper block of the rock mass is considered, it becomes clear that the slope of 38°wascritical for its stability.

The steeper part of the fracture, which had an inclination of 45°, had a tendency to open if movement occurred towards the valley. In fact, many open cracks exhibiting this tendency could be observed, while the less steeply inclined parts of the fracturing were closed and provided the entire support for the weight of the rock. It therefore had to be assumed that, when excavating along the rock face, there would be some blocks that would rest upon the more steeply inclined part of the fracture and therefore would quite clearly be in unstable conditions (see fig. 1, bottom right).

The problem was thus established: the stability of the rock face had to be guaranteed during and after the removal work. Various possible ways of carrying out the work could now be investigated. The first solution provided for a working procedure in four steps, as follows:

Step 1 Anchoring of the upper part of the , rock mass,

Step 2 Removal of a fairly large zone of the rock, in order to reduce the height of the sheer face adjacent to the road,

Step 3 Anchoring of the lower part,

Step 4 Removal of the lower part of the slope.

This solution was rejected, on account of the considerable volume of rock that would have had to be removed, the difficulty of finding a dump for the spoil, the considerable occupation of the road during blasting work and the fairly long time required for successive operations of different types.

The second solution, which was finally adopted provided for two main operations, namely:

Step 1 Anchoring of the entire rock mass by prestressed anchors,

Step 2 Removal of the rock face and local consolidation of same. This variant permitted a clear separation between removal and anchoring work, at least in regard to the main anchors in the upper part of the rock mass.

Rock investigations

In order to carry out a stability computation, it was necessary to obtain sufficient information about the angles of friction in the main fractures. From a study of the natural conditions and the fact that the rock face in spite of everything was in equilibrium, it was certainly possible to conclude that the angle of friction in fracture system I must be of the order of 40°; nevertheless, direct verification by tests was imperative. The firm Terrexpert AG, of Berne, was therefore entrusted with the task of carrying out a number of shear tests on specimens, which had been core drilled transversely to the natural fractures, as they then existed. Fig. 2 shows the results of one such test. This was a multiple shear test, which was carried out on the same specimen in both directions, but with different axial forces. The diagram indicates the shear force and the corresponding stress as a function of the axial force and axial stress.

From the curves for the various movements, it is possible to recognise a thresh-

Fig. 2: Results of a shear test

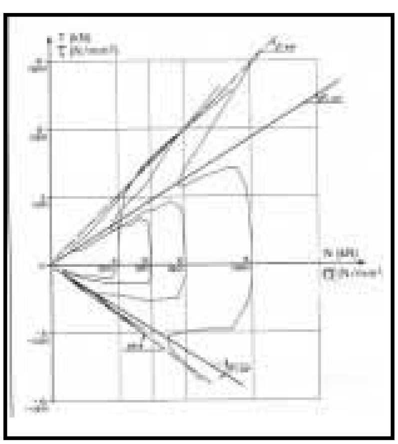
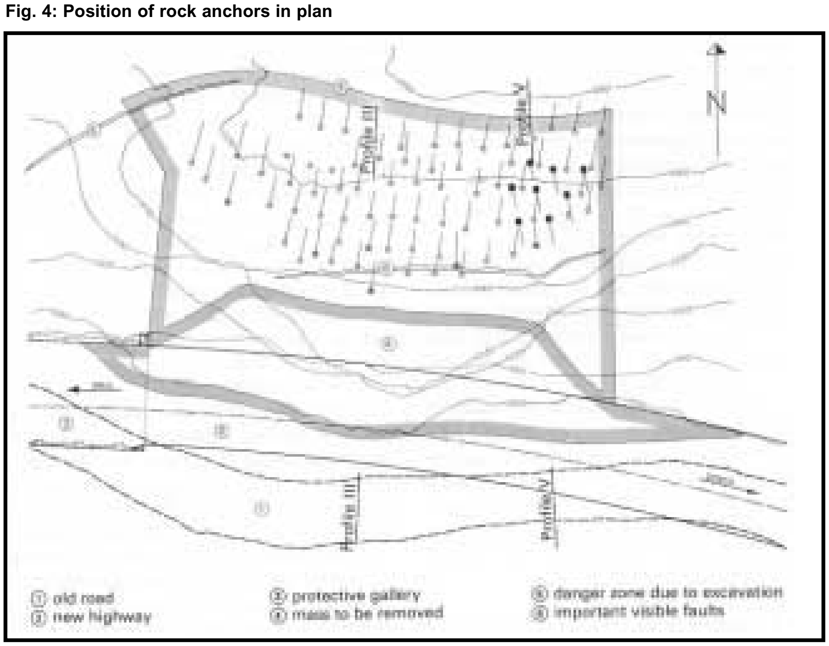


Fig. 4: Position of rock anchors in plan



old corresponding to a very pronounced angle of friction of about 32°; this occurs in both directions of movement. After this threshold has been passed, an increase in the frictional resistance up to an angle of 44° in the one direction and 36° in the other occurs. This indicates the presence of a phenomenon which could be termed «hardening». This means that the high angles of friction are not reached until a displacement of a certain magnitude has occurred between the blocks. A deformation of the solid rock was therefore necessary in order to mobilise this additional resistance. This fact was confirmed by surveillance of the rock mass. Observations indicated that the rock mass was in equilibrium, but only after fairly large deformations had occurred and it was these deformations which had caused the fractures with the steepest inclination to open.
In the present case, therefore, it was possible to confirm a very interesting and satisfying agreement between the observations made on the spot and the experiments carried out on the samples.
As might have been expected for a rock of this type with such pronounced fracturing, no cohesion was observed in the slip plane during the shear test.

Stability calculations

On this basis, simple calculations of the stability against sliding were carried out. Fig. 3 shows the results of a series of calculations. An optimisation study had previously shown that the most favourable angle for the rock anchors was 27° to the horizontal. This inclination was therefore adopted for all the main anchors. By investigating the various fracture planes, it was possible to determine the anchor length. It was also possible to eliminate a deep slip joint, which would have passed beneath the existing road. The small table in Fig. 3 gives the results of the calculations for prestressing forces of 0, 1000, 2000 and 2800 kN per metre of rock face, forfracture inclinations of 35° and 40°and forangles of friction of 32, 36, 40 and 44°. It was first of all confirmed that without prestressing equilibrium was evidently only possible if the angles of friction were at least as large as the slopes of the fractures.

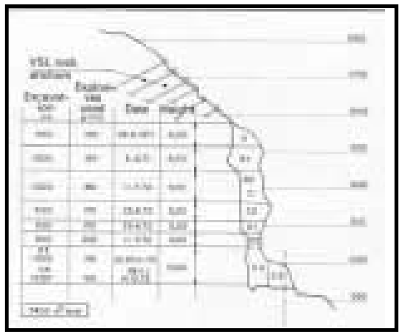


Fig. 5: Phases of rock removal

As the prestressing force increased, the factor of safety increased, but it can be seen that even for a force of 2800 kN /m, the factor of safety for a slope of 35° and a friction angle of 32° is only 1.16. From this it could be concluded that such a force was an absolute necessity, in order to prevent even small movements of the rock spur. If this is compared with a failure state, which would have presupposed larger movements, the factor of safety would have increased, for example to the order of 1.3, since it would be expected that larger angles of friction would come into play.
This conclusion therefore led, forthis type of rock, to a distinction between a first safety limiting value for small movements and a second, higher safety factor against failure, i. e. for larger movements. In this connection it should also be pointed out that in the calculations no hydraulic uplift was allowed for, since the permeability of the quite severely displaced rock could be expected to provide sufficient drainage effect.
For the consolidated rock mass, therefore, a larger safety coefficient had to be obtained than existed for the natural state before the works were carried out. If it had been necessary to increase it appreciably, then very large anchor forces would immediately have become necessary. In actual fact, the use of a force of 2800 kN per metre of road in this case increased the safety coefficient only by 0.3. If it is assumed, that the rock mass in the natural state was in a condition of limiting equilibrium, then the safety factor after anchoring is therefore about 1.3.

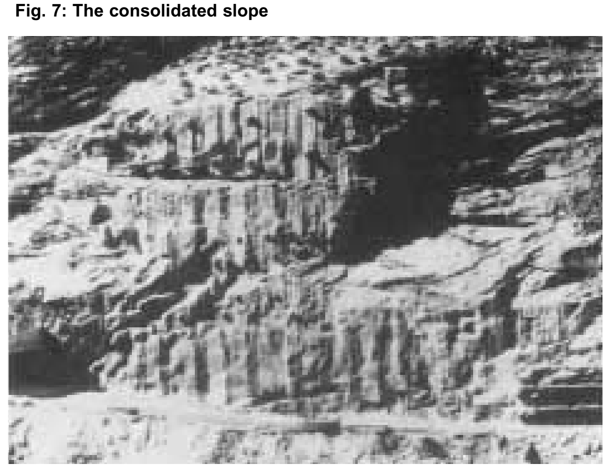
The left portion of Fig. 3 shows the normal arrangement of the anchors, for example for profile III, while the right portion shows the arrangement for profile V. In addition to the main anchors, in this case it was also necessary to provide anchors oriented almost perpendicularly to the first ones; the function of these is to retain blocks in the anchored zone which even in the natural state threatened to slip.
Fig. 4 shows the layout of the anchoring system in plan, the black squares denotirigthe 8 anchorswhich run orthogonally to the main system. The anchors denoted by a black circle were placed in site investigation boreholes.

The site works

On account of the difficulty of access, it was necessary to erect a cableway crane for servicing the site. In the Spring of 1971 placing of the anchors was commenced. The rock removal work was carried out in steps from September 1971 to the end of 1972, with a break during Winter. Fig. 5 shows the sub-divisions of the removal work. Six stages of blasting were carried out at successive terraces with a height of about 6 m and eight blastings with a height of only about 3 m.
All the prestressed rock anchors are VSL anchors. The series in the upper part of the rock slope comprise 69 anchors of 1400kN and 6anchorsof 800kN working force, placed in the investigation boreholes. During the excavation operations, it was also necessary to secure the front of the rock wall with 6 anchors of 800 kN, 3 of 1 100 kN and 1 anchor of 1400 kN; in addition, the already mentioned 8 anchors of 1 100 kN each were installed for anchoring the rock blocks in the consolidated zone. The stressing anchor head of the rock anchors were mounted on isolated foundation blocks of concrete (Fig. 6) and subsequently enclosed in concrete. Whereas the anchor work proceeded without difficulty, the removal of rock was a difficult and dangerous task, since the blasting operations had to proceed cyclically with the erection of scaffolding, positioning of the anchors, the bolting and the guniting of the exposed surface. The final state of the slope is shown in Fig. 7.

Fig. 6: Anchorage blocks of 1400 kN-anchors

Fig. 7: The consolidated slope



1.3. Securing of abutment at Libby Dam, Montana, USA

Client U.S. Army Corps of Engineers, Seattle, Washington
Engineer U.S. Army Corps of Engineers, Seattle, Washington
Contractor Joint Venture Libby Dam Builders, Libby, Montana
Anchors VSL Corporation, Los Gatos, California
Year of construction 1971

The Libby Dam is a gravity concrete dam, 128 m high and 885 m long. During construction, in January 1971, a wedgeshaped piece of the rock slope at the left abutment failed, 300,000 m^3 of material coming loose.
Detailed rock mechanics investigations showed that stabilising of the rock slope could be achieved with a prestressing force of about 160 MN. For this purpose, 90 VSL rock anchors type 5-16 with a working force of 1,800 kN (= 60% of ultimate strength) each and lengths ranging from 20to 45 m were installed in the boreholes of diameter 127 mm. The bond length is 6 m in all cases. Each anchor was tested with a load of 2,400 kN. Each anchorage bears on the rock face with a 600 X 600 mm reinforced concrete foundation

The stability of the slope was continually monitored with 5 measuring anchors. These differ from the standard cementgrouted anchors in that the strands are greased and sheathed by plastic sleeves in the free portion and the movable anchorage is equipped with a VSL load cell type G 200. Since the measuring anchors cannot be reached in winter, the load cells are connected up to a central reading station.

1.4. Securing the slope at the south portal of the Schallberg Tunnel, Switzerland

Client Highways Construction Department, Canton Valais
Engineer Ingenieurburo Walder AG, Brig
Contractor 0. Caldart, Naters
Drilling SIF-Groutbor SA, Renens
Anchors VSL INTERNATIONAL SA (formerly Precontrainte SA, Lausanne)
Stability investigations Terrexpert AG, Berne
Year of construction 1973

Introduction

The new line of National Highway N 9 over the Simplon pass meant that the south portal of the Schallberg Tunnel had to be protected against snowdrifts, rock falls, soil slips and rock slips by means of a gallery and that a safe passage for the road had to be found across the slope. To judge from a preliminary investigationthe foundation conditions for the gallerywere not, however, favourable, since a geologically difficult zone existed. Seismic velocity measurements indicated that even in fairly deep strata the conditions were no better, and they indicated that a continuous loose cohesion could be expected. The safety of the structure and the road therefore could only be guaranteed if the slope was thoroughly stabilised.

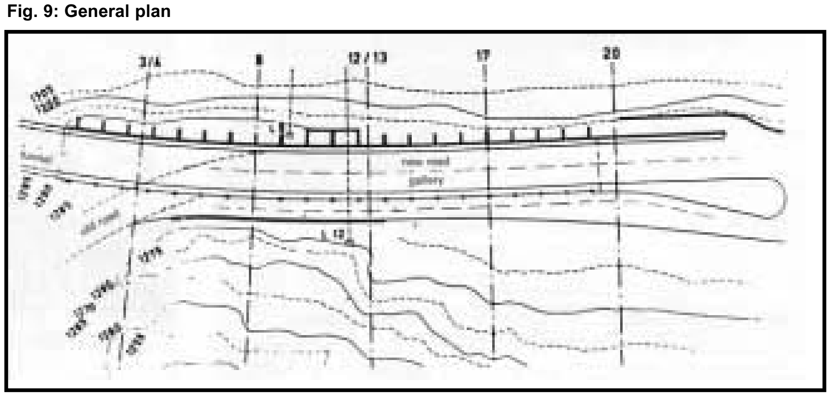
Nature of rock

The Schallberg region is located in an area of highly stratified, mica-rich calcareous schistswitha strong tendancyto mobility. The rock tends to disintegrate, especially if disturbed on face, leading to slips, the material breaking down into very small particles and tending to flow. Slips of this type occurred frequently right from the start of the work (fig. 8). They not only made construction difficult but posed considerable risk for the workforce.
After a new large slip in November 1972, the following measures were therefore proposed and carried out:

1. A concentrated, temporary anchorage above the main zone of incipient cracks, perpendicular to the slope;
2. A check of the stability of the rock slopes;
3. Determination of the geotechnical characteristics;
4. The setting up of a monitoring system, to obtain a better unders tanding of the behaviour of the rock.

Fig. 8: The site in Autumn 1972 after a slip

Fig. 9: General plan



The temporary anchorage was intended to reduce the danger in the working area and enable work to continue. It proved to be successful, since no further slips occurred in this zone.

The following mean values which were the result of extensive laboratory tests were used as a basis for further, theoretical stability investigations:

g =2.7 t/m^3
F = 28°
c_O =0.07 N/mm^2

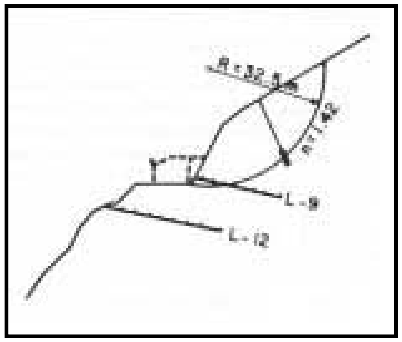


Fig. 10: Slip circle with minimum safety (upper part of slope)

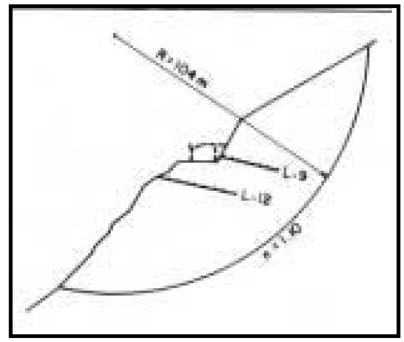


Fig. 11: Stability of entire slope at crosssection 12/13

Stability investigation

The stability investigation was carried out by the method of Fellenius, using a computer programme prepared by Terrexpert AG, of Berne. In view of the length/ height ratio of about 3 for the portion of the slope above the road and the low shear transmission capacity, no allowance was made for lateral support of the individual segments. Since the material when in the slip condition is practically cohesionless and behaves more like a kind of soil, soil mechanics slip concepts were adopted for the computer model.

The five computed cross-sections are shown in fig. 9. From the pattern of the contours the position of a number of furrows can be seen, giving an indication of the disturbed zones.

The result of the stability computation for the part of the slope above the road is given in fig. 10 for the cross-section 12/ 13. The calculation was made for circular segments, which passed through the corner of the foundation on the uphill side.To determine the least favourable profile, the radii of the circles were varied in the usual way.

This gave a minimum safety factor of F=1.42,for the case where full allowance was made for cohesion. If, however, such cohesion were to become ineffective for any reason, for example due to disintegration phenomena, then adequate stability would no longer exist. Measures therefore were necessary to ensure maintenance of cohesion and to prevent disintegration phenomena from occurring. In the present case rock anchoring was chosen for this purpose.

After the investigation of the slope stability the question of the stability of the entire slope was studied. It was pointless to construct structure with improved safety upon a foundation which did not possess equivalent safety.

For the entire slope, a similar investigation was carried out to that for the upper part of the slope using the same criteria and the same method. In this investigation, the circular arc profiles were determined by a lower tangent limit. Fig. 11 shows the result, again for the crosssection 12/ 13. The circular profile shown represents in this cross-section the most unfavourable case taking full account of cohesion. Under the same assumptions, it became apparent that the stability of the entire slope was lower than that for the upper portion of the slope alone. It is however to be expected that the cohesion at considerable depths in the rock would be less subject to deterioration and therefore would be higher. Moreover, with the present dimensional relationships, a partial lateral support for the entire slope could be assumed.

The rock anchors used

The evaluation of the stability investigations led to the following conceptual design:

In order to secure the portion of the slope above the road, a total of 174 prestressed rock anchors of 650, 900, 1300 and 1600 kN working force was necessary, providing a total force of 220 MN. The rock anchors used were of the system SIFTMD with VSL anchorages of types 5-6, 5-8, 5-11 and 6-10 with ultimate strengths of 985, 1313, 2029 and 2577 kN respectively. The anchor lengths vary between 17 and 26.5 m. The bond lengths (all 5 m long) were located in those zones where, without allowing for cohesion, there was still a factor of safety against slip of F= 1.0.

The factor of safety against slip of the most unfavourable circle through the base point of the upper portion of the slope after the remedial work and without allowing for cohesion is now at least 1.2. The effect of applying the anchor forces is, however, to counteract the loss of cohesion, so that the effective factors of safety should lie between the limiting values determined on the basis of cohesion and no cohesion.

Below the road 50 rock anchors of 900 and 1300 kN working force providing about 50 M N total force secure the slope. The lengths of the anchors in this part (24 to 30 m) were determined by the same considerations as for the upper part of the slope.

All the anchors were stressed in at least 2 steps. By carefully staged application of the forces, it was possible for the reaction of the slope to be anticipated and controlled.

Surveillance system and results of readings

In order to monitor the slope and control the stabilisation work carried out, a number of further arrangements were provided:

1. Individual rock anchors were designed with a free tendon length remaining elastic during working life to enable the behaviour in regard to applied force to be monitored,
2. Two extensometers of 25 and 34 m length respectively were installed

Fig. 12: Overall view of the slope during drilling operations in the middle section

Fig. 13: View of the site at a later stage

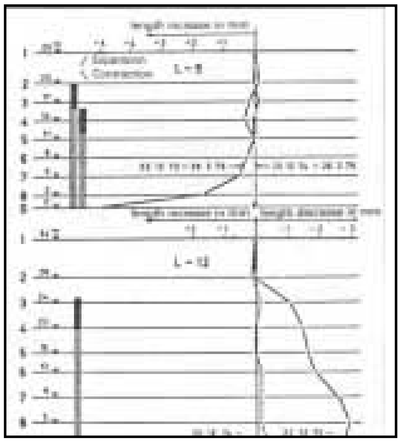


Fig. 14: Movement curves for the two extensometers L9 above and L12 below the road

3. above (L9) and below (L12) the road, in order to detect movements of the slope and to check the reactions of the rock slope to construction procedures, For two test anchors with extremely short bond lengths, the effective bond stresses were ascertained by a pull-out test.

The boreholes were drilled as rotary core boreholes (0 116 to 125 mm) which enabled conclusions to be drawn about thequalityof rock throughout the monitored region.

The results of the movement measurements from the extensometers were plotted both as movement curves and as time curves. Fig. 14 shows the movement curves for the two measuring positions. The anchor lengths in the region of the measuring positions are shown diagrammatically at the left. The movements are referred to the deepest measuring point of the extensometers, which was assumed as stationary for the evaluation. The movements in the axial direction of the borehole are plotted perpendicularly to the relevant measuring point and these points are then connected together. With a curve of this type in the arrangement shown, the parts of the curve inclined downwards and to the left indicate an expansion.

The full line represents the movement from the start of the measurements as a kind of summation line. The broken line, however, indicates the change of movement which occurred between the last two readings.

Fig. 15 shows the movement of the rock surface referred to the assumed stationary points within the extensometers for the two measuring positions, plotted against time on the horizontal axis, from the start of the measurements in December 1972. Expansion movements are plotted downwards and contraction movements upwards. The movement curve for the rock surface at the lower measuring point (full line) shows the initial, uniform expansion movement until the application of the prestressing in the vicinity of this extensometer (point 1). The application of these forces resulted in a considerable contraction,

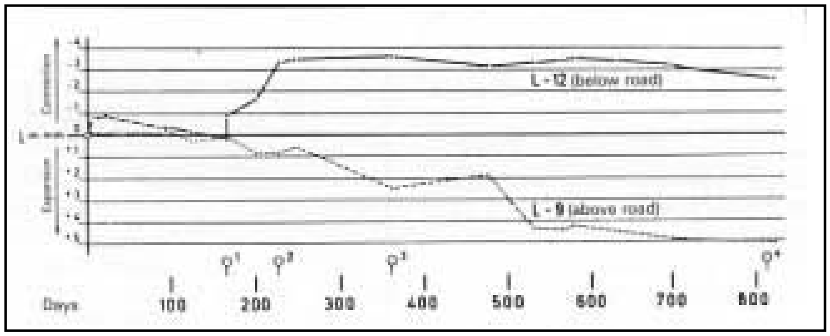


Fig. 15: Movement of rock surface plotted against time

was almost completely retained until the penultimate reading. The slight expansion, detected at the last measurement at the end of March 1975, might be associated with the preceeding heavy falls of snow and the resultant saturation (point 4).

The behaviour of the upper measuring point was somewhat different, as is shown by the broken line. The initially stable pattern could be the result of the temporary stabilisation of the slip in the vicinity of the measuring point. The superimposed expansion developed in parallel with the progress of the construction operations and accelerated notably as a consequence of the blasting for the foundation excavation on the uphill side (points 2 and 3). After 530 days, it was possible to observe an increased stabilisation of the movements, which can be explained by the new equilibrium introduced by the anchoring.

It can be concluded from the results of these readings that the stabilisation of the slope which was the objective of the anchoring operations was achieved. From the fact that the movements, although they did die away slowly, had not completely stopped at the end of the measurement period, it can be concluded also that the project was not overdesigned.

1.5. Securing of portal of Engiberg Tunnel, Arth-Goldau, Switzerland

Client Construction Department of Canton Schwyz
Engineer Th. Ulm, Schwyz
Contractor Joint Venture
Losinger AG, Lucerne
Locher & Co. AG, Zurich
Woest AG, Lucerne
Drilling Injectobohr AG, Zurich
Anchors VSL INTERNATIONALAG (formerly Spannbeton AG)
Year ofconstruction
1974

The bypassing of Arth by National Highway N4, which connects Zurich and Altdorf, was made possible by the construction of two parallel tunnels beneath the Engiberg. The north portals of the tunnels are situated in an almost vertical slope, which necessitated securing of the rock at the portal nearest to the mountain. Before the tunnel was driven, the rock slope was anchored to the limestone by 25 VSL rock anchors distributed in five layers. 7 rock anchors are of type 5-15 with a working force of 2,000 kN, the others are of type 5-18with a working force of 2,4000. All the anchors are 25 m long, including a bond length of 6 m.

1.6. Anchorage of scour prevention walls at Tarbela Dam, Pakistan

Client Pakistan Water and Power Development Authority (WAPDA), Lahore
Engineer Tippetts-Abbett-McCarthy-Stratton, New York
ContractorDrilling Tarbela Joint Venture
contractor Johann Keller GmbH, Frankfurt
Anchors VSL INTERNATIONAL LTD, Berne
Years of construction 1973-1974

The Tarbela Dam, about 100 km northwest of Rawalpindi and with a height of 148 m and a crest length of 2743 m the largest earth dam in the world, serves predominantly for regulating the water flow of the Indus and for irrigating the Lower Indus Valley. An important part of the plant consists of the two spillways on the left side of the valley, over which enormous quantities of water must pass during the rainy season. Since the rock at the lower end of both the structures is very bad and consequently there was a risk of heavy erosion of the ground due to water turbulence, which actually took place after a few years, it was necessary to take measures to prevent scour beneath the spillway channels. The measures consisted in the construction of a concrete wall descending from the flip bucket at 45° slope deep into the rock, thus preventing erosion at this point. This wall was secured by rockanchors, firstlyto prevent sliding and secondly to counteract increased water pressure behind the wall (Fig. 16).
The wall is 1.50 m thick, is of unreinforced mass concrete of 21 N/mm^2 strength and was constructed below ground in layers of 2.40 m depth.
The engineer specified rock anchors of 740 kN ultimate strength, arranged in chequerboard pattern atvertical and horizontal spacings of 2.40 m and anchored in the drainage galleries (fig. 17). VSL rock anchors of ultimate strength 738 kN comprising 4 strands 0 0.5" were chosen. The bond length of 4.27 m is situated partly in the wall and partly in the rock. The free length of the anchor is housed in a plastic sheath and thus remains elastic during working life. The average length of the 2,OOO anchors required in total is 17 m (fig. 18).

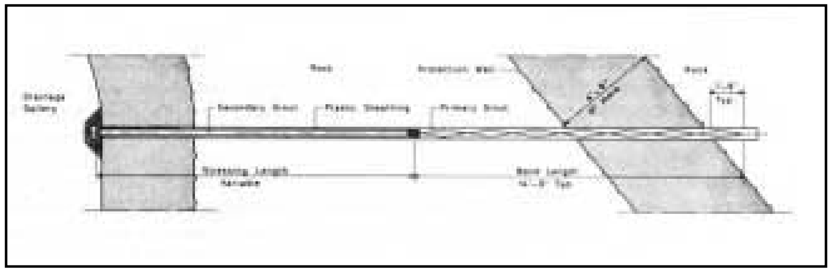


Fig. 18: VSL rock anchor

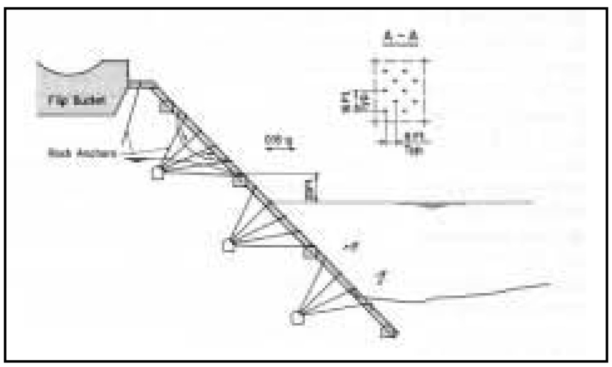


Fig. 16: Situation after erosion

Fig. 17: Drainage tunnel with anchorage blocks

2 Anchoring of retaining owalls

2.1. General

The cutting into a steep slope in soil or highly fractured rock normally results in a correspondingly large excavation of material in order to prevent slips. This excess removal can be considerably reduced with advantage by building an anchored retaining wall. A structure of this type can either be formed as a continuous wall, a procedure required particularly with soil, or it can be broken up into slabs, a method which can be used if local instabilities do not occur.
In general, retaining walls are constructed more or less vertical. The procedure usually adopted is to underpin in horizontal strips of 1.5 to 3 m height, depending on the stability of the material. Since the stability of the wall is assured by the stressed anchors and the function of the wall is mainly to act as a cover and to distribute forces, its thickness can be less than that of an unanchored wall and in addition no footing is required.
Where a retaining wall is constructed by underpinning, a different method from that of horizontal strips is frequently employed; this consists of progressively concreting and anchoring vertical ribs or columns and securing the rock between them by gunned concrete or filter concrete. A cladding orfacing isthen applied, which has both an aesthetic and a structural function. The height of the sections is usually 3 m. Each step ensures the safety of the next by stressing of the anchors before the excavation of the next step Prefabricated elements can also be used for the columns.
An anchored retaining wall can also be a rational solution for a generally stable rock face, since it provides an effective protection against loosening and crumbling of the rock due to weathering by rain, snow or frost.

One interesting variant of a retaining wall is the piled wall. The main advantage of this is that the boring and construction of the piles can be carried out along the entire length of the face before the excavation. The piled wall is a permanent structure, and requires no further work during the excavation except for the drilling of the boreholesand installation of the anchors.

2.2. Upper-retaining wall at Delli and slope stabilisation at Hauetli, Alpnachstad, Switzerland

Client Public Works Department, Canton Obwalden
Engineer Werffeli & Winkler, Samen
Drilling DelII:SIF-GroutborSA, Renens
Hauetli: Fehlmann Grundwasserbauten AG, Berne
Anchors VSL INTERNATIONALAG (formerly Spannbeton AG)
Years of construction
Delli:1976-1977
Hauetli: 1974-1976

Introduction

The building of Swiss National High-way N8 along the central section of Lake Alpnach (Lake Lucerne) raised the question of how to provide sufficient area for the three traffic routes, namely the N8, the railway and a secondary highway. Of the many possible variants, a combination of placing fill in the lake and cutting into the rock face was chosen. This required extensive slope stabilisation work, including the slope stabilisation at Hauetli and the upper retaining wall at Delli.

The problem

A description of the upper retaining wall of Delli should start with that of the stabilisation of the slope at Hauetli (a few hundred metres from Delli), since results of extensive investigations exist about the conditions at the latter and these also applytothe Delli zone. The rockface in the region of Hauetli was excavated in 1970/ 71 to slopes of 2:3 in the soil and of 1:1 in the rock, without any movement being observed. Shortly after completion of the work, however, cracks began to appear in the grass turf above the excavated slope. Ayearand a half later, in November 1972, the portion of the slope bounded by one of these cracks slipped and further cracks appeared, providing evidence of an extensive movement of the ground and the possibility of a deep slip surface. The slope was then further monitored by means of boreholes, extensometers, piezometers and geodetic systems.

The geological conditions can be seen in section in fig. 19. The slip surface lies in the transition zone between weathered and sound rock (marl shale) at a depth of up to 12 m below the rock surface. It is 200 to 400 mm thick and consists of clayey silt with sand and gravel. The stability conditions of the slope are dependent almost entirely upon the shear strengths of this soil stratum. The slip could not, however, be explained solely in terms of the ascertained residual shear strengths. It only appeared in conjunction with pore water pressures, which had built up as a consequence of the damming of the hill water.

Slope stabilisation at Hauetli

In view of the endangered stability of the slope, stabilisation with rock anchors was carried out, 17 anchors of type VSL 5-12 (ultimate strength 2097 kN) being installed in a first phase (1974), followed by 277 of the same type of anchor. Their stressing anchorages were mounted on pad foundations, between which planting of grass was possible. The anchors were tested to 75% of the ultimate load and locked off at 65%, i.e., at 1363 kN. It was specified that the anchors should be capable of being checked for load at any time and, if required, restressed or detensioned; their length was 17 to 36 m, including 5 m bond length.

An important question in this connection was the optimum inclination of the anchors, for which the specified stabilising action could be obtained for minimum installation costs. The optimum angle for the anchors was determined on the following assumptions:

- the slip surface is plane in the region where the anchors pass through it;
- the shear strength in the slip surface is t = s tan F , (c=0);
- the anchors in any one profile are arranged parallel, for simplicity in the drilling operations.

For an average anchor, with its anchor head situated at point X, the free anchor length Lf from fig. 20 is:

$$L_f = t(\sin d)^{-1} \qquad (1)$$

where t is the distance from point X to the plane defining the upper limit of the anchorage region, and rY is the angle between the axis of the anchor and the slip surface.

Forthepurposesofcostcomparison, the braking force B exerted by the anchors on the slip surface is assumed constant. From fig. 21 the anchor force A is given by:

$$A = B(\cos d + \sin d \tan F)^{-1} \qquad (2)$$

If we put $\frac{dA}{d\delta} = 0$, we obtain the solution $\delta = \Phi$, for which A has a minimum value. This solution is of interest if, for any reasons, it is the anchor force and not the cost which is to be minimised.

The construction costs K_1 for the free anchor length can be obtained from the following equation:

$$K_1 = EAL_f; \qquad (3)$$

where E is the price for the free anchor length per length and force unit (including drilling, steel tendon, corrosion protection). If equations (1) and (2) are substituted in (3) we obtain:

$$K_1 = EBt(\sin\delta\cos\delta + \sin^2\delta\tan\Phi)^{-1}$$

If we put $\frac{dK_1}{d\delta} = 0$, we obtain the solution $\delta_1 = \pi/4 + \Phi/2$, for which K_1 is a minimum. To this must be added the costs K_2 for the anchor head, bearing plate, bond length and stressing. These are proportional to the number of anchors, that is to say to the anchor force and can be obtained from the following function:

$$K_2 = DA$$

where D is the price for the aforementioned components and operations

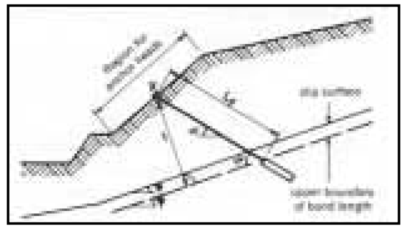


Fig. 20: Geometry of a profile (diagrammatic)

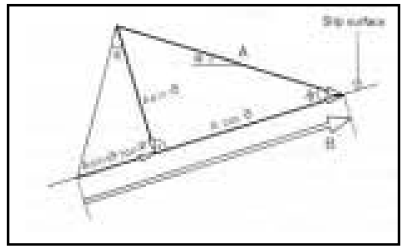


Fig. 21: Braking force B on slip surface

Fig. 22: Construction costs plotted against angle d

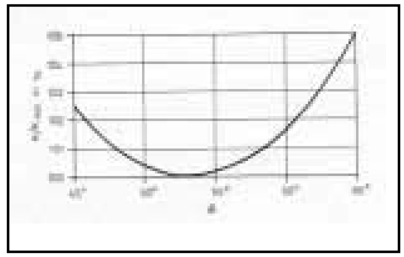
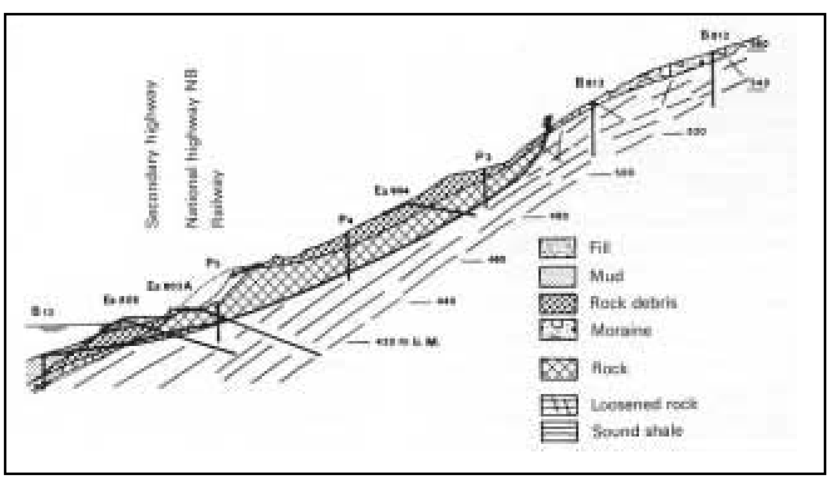


Fig. 19: Geological profile of slope



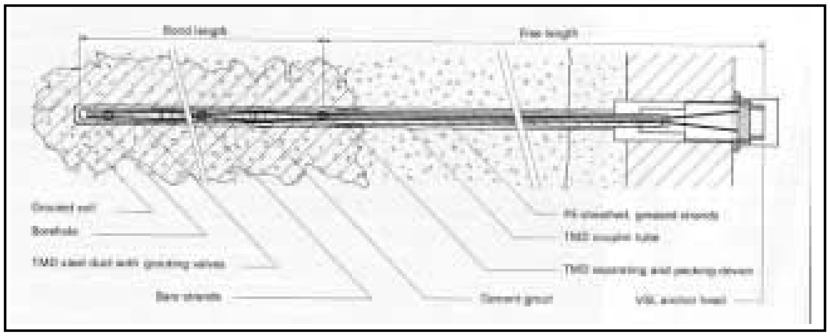


Fig. 23: Permanent soil anchor TMD-VSL 5-9

per force unit. K_2 has a minimum for $\frac{dA}{d\partial}=0$, that is $\partial_2=\Phi$.

The total construction cost K is thus given by: $K=K_I+K_2$

Forth is function the solution d_{opt} is to be found, for which K is a minimum. The ascertained values d_1 and d_2 represent the upper and lower limits for d_{opt}; they are independent of the geometry and unit costs and provide useful starting points in a very simple manner. The solution 6opt, on the other hand is dependent upon the geometry, the type of anchor and the unit costs. Fig. 22 shows the results of a cost comparison for one of the profiles; this gives d_{opt}=53°.

The retaining wall at Delli

The upper retaining wall at Delli with a total length of about 300 m is divided into three sections:

Section A l= 86 m	anchorage in the rock
Section B l= 31 m	natural slope without anchorage
Section C l=185 m	anchorage in soil (large rock debris)

The height of cutting varies from 7 to 10 m. The retaining wall, at an inclination of 60°, is broken up, i. e., it consists of 5.OOX 6.50 m concrete slabs at a centre spacing of 9 m, each anchored by 5 TM D VSL-anchors (fig. 23). This dividing up of the retaining wall proved to be an economical solution and was also aesthetically very satisfactory.

Fig. 24: View of divided retaining wall

The rock anchors in Section A (58 No.) are of type 5-12 (ultimate strength 2097 kN, working force 1225 kN, test force 1529 kN) and are 20to 42 m long including a bond length of 5 m. The soil anchors, all of 18 m length (bond length 6 m), comprise 9 or 7 strands. The ultimate strength of the 45anchors 5-9isthus 1573kN,the working force 885 kN and the test force 1238 kN. For the 60 No. of type 5-7, the corresponding values are 1223, 705 and 1028 kN. The requirement for all 163 anchors, as for the slope stabilisation at Hauetli, was that it must be possible to check the load at any time and to restress or detension them. This required a high standard for the durability of the anchors, both in regard to the quality of the bond length and also of the corrosion protection for the entire length of the anchor. The selected anchor type and the materials used fully satisfied these requirements.

2.3. Anchored wall at Flachau, Austria

Client Tauernautobahn AG (Tauern Motorway Ltd)

Engineer Dr. Heinz Brandl, Dr. Hermann Brandecker and Consulting Civil Engineers Vilas/Westhausser

Contractor Joint venture Flachau (Lang & Menhofer/ Fischer)

Drilling and Anchor contractor Sonderbau GesmbH, Vienna

Years of construction 1974-1976

Introduction

The Tauern Motorway runs generally north-south from Salzburg to Villach in Carinthia, traversing the alps. The main part of this connection which is kept open during winter comprises the 75 km long top section including the Tauern tunnel. In the region of Flachau in the Ennsvalley, the motorway crosses a pronounced geological faultzone. The outcropping rock is a deeply weathered graywacke zone, in which the cohesive shale decomposition products in particular have a low shear strength, which can progressively decrease still further when fairly large shear deformations occur. When air and water gain access, these shales very rapidly soften. At the valley floor, silty sands to organically contaminated silty clays of very high compressibility are found.

The cutting of the slope

Because of these unfavourable ground conditions, the already existing constructions in the valley floor and on account of the desire to retain the line of the road unchanged, a cutting of the slope of several hundred metres length and almost 40 m high was necessary in the section described here. In its central portion, the building of a rigid gravity retaining wall was too risky, since the slope was in an unstable condition of equilibrium. In addition, the scatter of the soil and rock properties was so great, even within a short distance, that to design the supporting structure simply by calculation would have been completely inadequate. Therefore, an elastic, anchored retaining wall was chosen for securing the slope. This permitted stage-by-stage removal of the rock, and moreover, due to the flexibility of the structure varying deformations would be more easily accepted than with a rigid wall. In addition, this method afforded the possibility of adapting by stages and in the optimum engineering and economic mannerto local differences in rock pressure, slope movements and foundation conditions, using as a basis extensive and accurate readings taken during the entire construction period. By continuously monitoring the deformations of the wall and the slope and also the anchor forces, an effective substantiation of safety was obtained «in situ», to an extent that would never have been possible by theory alone.

Bases of design

With steep slopes of such great height and the presence of hill water, soil mechanics calculations can, of course, give only broad guidelines; they are useful predominantly for deciding upon theoretical or hypothetical limiting values. Socalled refined computation procedures usually provide a deceptively high degree of accuracy, which does not exist in practice. To examine the possibilities of slope failure, both non-laminar and laminar methods (the Swedish method according to Fellenius) were used and comparative calculations were carried out by earth pressure theory. All the investigations were based upon highly idealised assumptions, due to the wide scatter of the soil parameters. The extraordinarily large influence of the soil parameters upon the result of the stability calculations can be seen from the following: By changing the angle of internal friction by only 1°, the anchor force necessary for obtaining a calculated safety coefficient

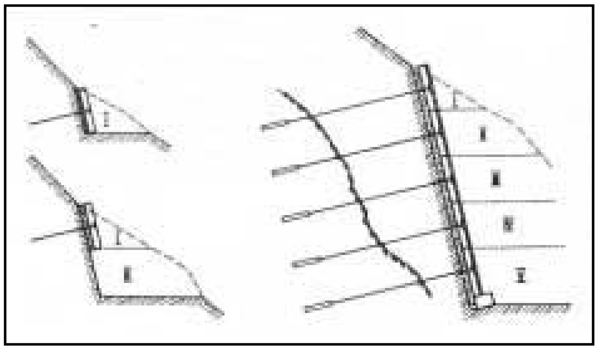


Fig. 25: Construction procedure

Fig. 26: General view of wall during consiruction

of F=1 changes by about 1 MN/m. For a variation in cohesion of 1 N /mm2, values differing by 1 to 1.5 MN/m were also obtained for the required anchor forces, and for F= 1. A recheck of the soil parameters after completion of the anchored wall showed that the angle of friction differed byonly 1 to 1.5'from the original assumptions; this small difference however had led to an increase of the necessary anchor forces by a factor of almost 2.

Construction of the anchored wall

The removal of soil and construction of the anchored wall was carried out by steps working downwards as shown diagrammatically in fig. 25. The material in the uppermost excavation step I was removed in a single run along the entire length; the filter concrete was then placed, the anchor holes drilled, the anchors placed, the reinforced concrete slabs constructed and finally the anchors were stressed after 7 days. In the excavation steps I I to V below, the procedure was as follows: the excavation was carried out along the entire length sufficiently far for a natural angle of repose to remain up to the already anchored slabs of the row next above. The remaining excavation for the slabs could then be carried out in a chequerboard pattern. It was only after each alternate slab had been completed beneath an upper row of slabs that the intervening material was removed. In order to assess the outcropping types of soil and the lengths of anchor necessary, about 5% of the boreholes were rotary drilled with core recovery. All the other boreholeswhich had a diameter of 90mm were formed by a more rapid and economical drilling procedure using down the-hole-hammers in conjunction with a casing.In a first construction stage, the wall was brought to completion with an acceptable minimum of anchor forces. It was still possible at anytime to provide for additional forces, if the surveillance measurements indicated the need for this. This did indeed prove necessary, since as a result of catastrophically high precipitation during winter and spring, movements of the slope had commenced. It was therefore necessary to place additional anchors, the lengths and capacities of which were designed according to the results of the continuing surveillance measurements.

The entire wall required the installation of 800 anchors in total. Of these 402 were VSL-anchors, because due to the tight construction schedule the project had been divided into two parts. 291 VSLanchors were of type 5-6 (ultimate strength 1059 kN), had a working force of 600 kN and lengths of 20 to 40 m, while 1 1 1 anchors were of type 5-10 (ultimate strength 1765 kN), of 1,000 kN capacity and 40to 70m long. The bond length in all cases was 10 m. The retaining wall was constructed during the period November 1974 to February 1976.

To provide continuing surveillance of the stability of the slope and of the supporting structure and also to assist in detailed dimensioning, load cells were used and checks carried out on the stressing forces during and after construction, in addition to the extensometer and geodetic measurements already mentioned.

2.4. Retaining wall on the N2, Eptingen, Switzerland

Client Construction Department of Canton Baselland
Drilling GreuterAG,Zurich
Anchors VSL INTERNATIONALAG (formerly Spannbeton AG)

Year of construction 1970

A three-part retaining wall without footings, anchored with 175 VSL soil anchors of type 5-7, each of 700 kN working force.

2.5. Retaining walls on the N5 on Lake Biel, Switzerland

Client Highway Construction Department, Canton Berne
Engineer Engineering joint venture Suiselectra AG, Basle Schaffner & Dr Mathys AG, Biel Steiner & Grimm AG, Berne
Contractor Walo Bertschinger AG, P Andrey & Cie / H. R. Schmalz SA
Drilling Fehlmann Grundwasser-bauten AG, Berne
Anchors VSL INTERNATIONALAG (formerly Spannbeton AG)
Years of construction 1973-1974

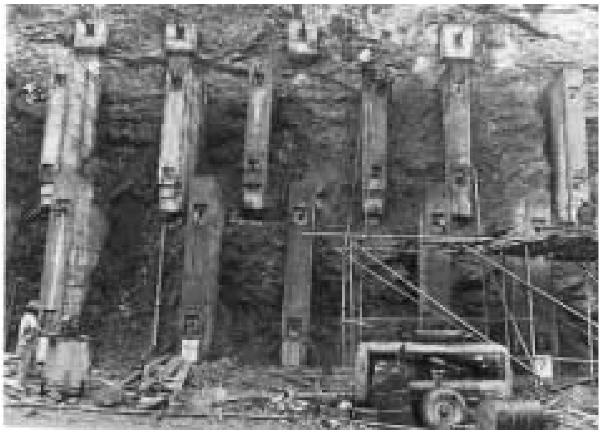


Fig. 27: Anchored concrete tie-beams

Fig. 29: Construction of staggered wall

The widening of the left Lake Biel highway necessitated the removal of portions of rockof 10to 20m height, especially in the sections Vingelz and Wingreis. At both locations, the excavated surfaces were consolidated in the same manner. In the upper part of the face, individual prestressed rock anchors were installed and anchored in concrete foundations, while for the lower part of the face anchored retaining walls along the new road were chosen. In principle, the retaining walls at the two locations are almost identical, but the methods by which they were constructed differ.

At Vingelz, where the rock was relatively stable at the surface, the supporting elements of the wall consist of vertical concrete tie-beams of 3 to 4 m height, anchored with 2, occasionally 3 rock anchors (Fig. 27). These tie-beams were constructed of in-situ concrete, to enable thebearing face to be adapted to the excavated profile. They were connected with a concrete cladding (Fig. 28), which not only serves for ensuring thestabilityof the rock wall, but was also intended to prevent destruction and weathering of the rock due to the action of ice and melted snow. At places where the cuttings were very high, two staggered walls, one above another, were built, the upper wall being set back about 2 m (Fig. 29). In total, 496 VSL rock anchors type 5-3 to 5-8 of 300 to 960 kN working force, 493 to 1314 kN ultimate strength and 8 to 40 m long were placed for the retaining wall at Vingelz.

At Wingreis, where the rock surface was less firm, the excavation steps were limited to 1.50 m. The tie-beams, spaced at 4 m centres, were made from prefabricated elements also of 1.50 m height. Each element was anchored with one rock anchor. The tie-beams thus consist of disconnected components, which could move freely when the anchors were stressed and therefore did not induce secondary forces due to deformations. Between the tie-beams, a drainage layer in the form of filter concrete was placed and finally the whole assembly was covered with concrete cladding (Figs. 30 and 31). The securing of this section required 200 VSL rock anchors type 5-4 to 5-18 of 500to 2500 kN working force, 738 to 3320 kN ultimate strength and 15 to 40 m long and also 60 VSL soil anchors type 7-1 of 250 kN working force and 28 m long.

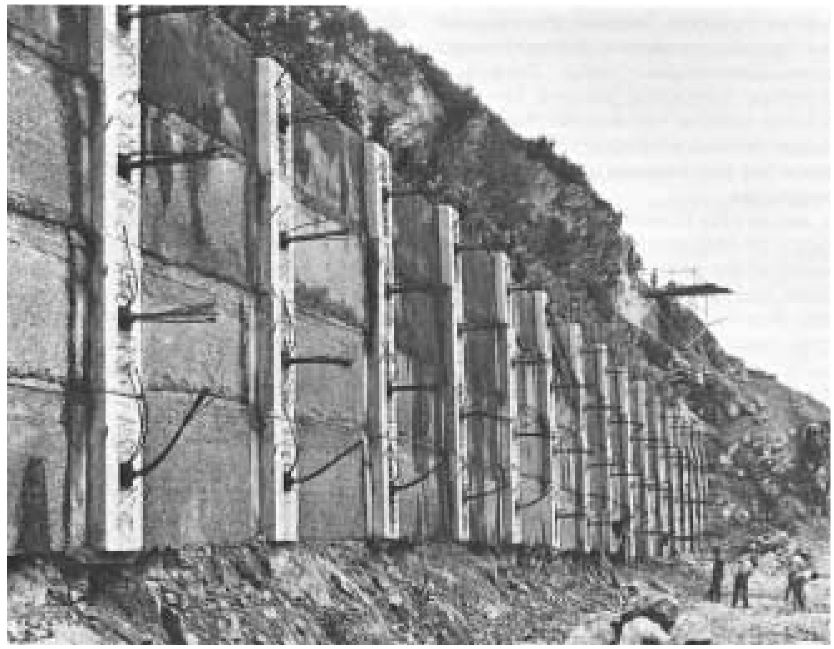


Fig. 31: The wall at Wingreis

Fig. 28: Cladded wall

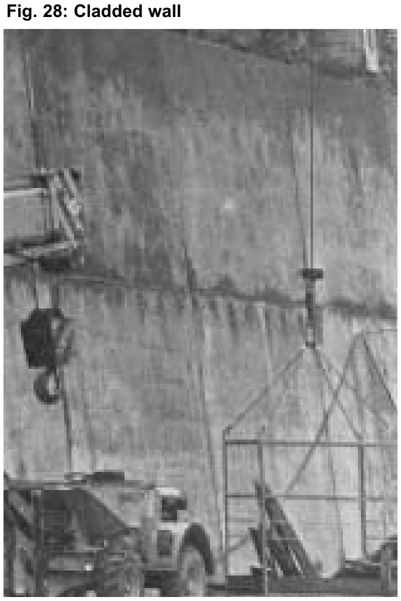
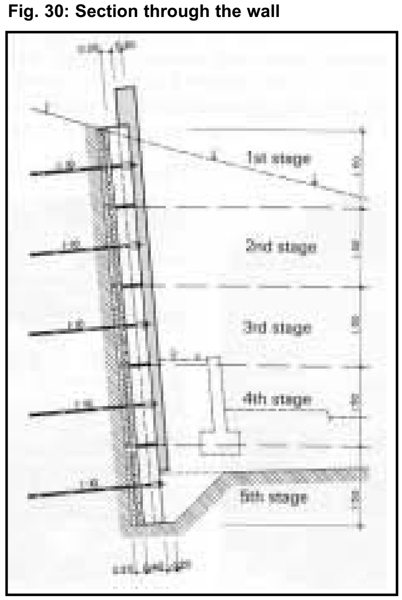


Fig. 30: Section through the wall



2.6. Pile wall at the south portal of the Naxberg Tunnel, Switzerland

Client Construction Department of Canton Uri
Engineer Cantonal Construction Department of Uri and Ingenieurbüro Th. Külin, Goschenen and Schwyz
Contractor Joint venture Naxberg
Ed. Züblin & Cie AG
LVG Bauunternehmung AG
Bonetti AG
Drilling Injectobohr AG, Zurich
Anchors VSL INTERNATIONA LAG (formerly Spannbeton AG)
Years of construction
1972-1973

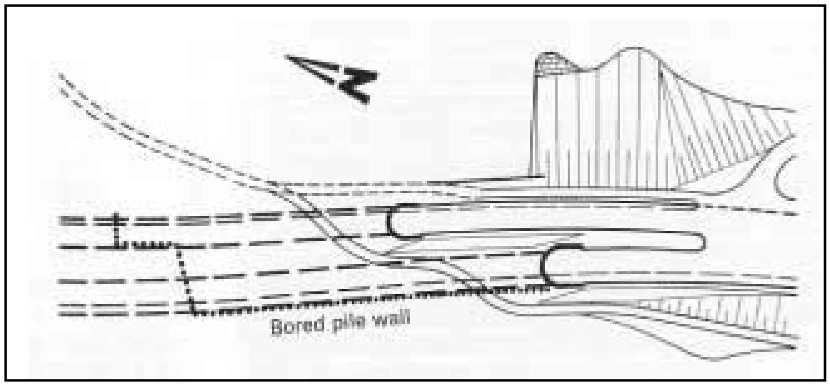


Fig. 32: Site plan

In the region of the Naxberg tunnel on the north ramp of the Gotthard, the valley of the Reuss forms a ravine with very steep, sometimes undercut sides. There are therefore compelling reasons for conducting national highway N2 through a tunnel, net only on account of the limited space but also because of the danger of avalanches.

Il was already known from general geological investigations that about half the length of the tunnel would lead through loose material, which would prove extraordinarily time-consuming te penetrate. Additional trial bores enabled the boundary between the loose material and the rock te be more accurately determined. Il was intended te construct a short length behind the south tunnel portal in open cul and the rest by tunneling through the loose material. Experience showed, however, that work of this kind must be expected te give rise te unpleasant surprises and relatively high risks. In addition, il is becoming increasingly difficult te find the necessary skilled and semi-skilled labour for the tedious manual work. Rational working and the meeting of the contract dates could be jeopardised, quite apart from the high costs which would result from these difficult procedures.

For the above reasons, the joint venture decided te offer a special proposal. This proved satisfactory not only from the engineering but also from the cost aspects. It is described in more detail below. From the geological investigations it could be deduced that in the south tunnel region, where the two tunnel tubes pass through the loose material, the cliff face descends relatively steeply and is net very far back from the tunnel profile. There was therefore a possibility of excavating the slopefora length of about 130m in the protection of a pile wall, te be constructed and anchored in the rock and extending down te the level of the future road; the tunnel tubes would then be constructed without hindrance in the open trench (Fig. 32). The entire trench could subsequently be backfilled with the excavated material, so thatfinallythe original condition would be reinstated. This procedure did require a very large amount of earth moving, but il is clear that il could be highly mechanised, permitting economical operations te be carried out.

This work was carried out in steps; first of all a level work surface was prepared, from which the piles, 900mm in diameter and at 2 m spacing, could be bored te the desired depth. The slope was then excavated te a depth of 4to 6 m, exposing the piles on the valley side. Simultaneously, the space between the piles wasfilled with filter concrete and thus a closed wall was completed. The filter concrete prevented water from building up behind the wall. From this formation level, the first row of anchorswasthen installed, anchoring the piles back into the rock. The entire operation - removal of slope and exposure of piles, concreting of intervening spaces between piles and construction of the anchors - was repeated until the motorway level had been reached. A maximum of 5 rows of anchors was required for the greatest wall height of about 20 m. The wall, which was adapted te the shape of the rock, was just 170 m long and contained a total of 83 piles.

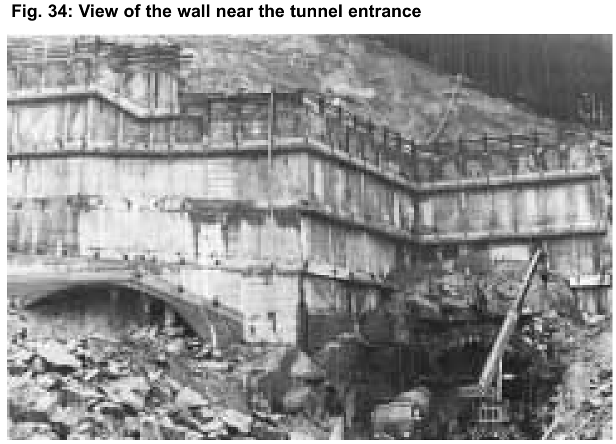
The detailed design of the pile wall required extensive structural calculations, on account of the numerous construction states, the differing inclinations of the face and the differing bearing conditions for the pile bases (some fixed, some freely supported and some cantilevered). The piles, which in the majority of cases rest on the rock, were connected together at the level of the anchorages by concrete tie-beams. The holes for the anchors were dri lled through the piles and through the loose material behind them into the rock. The following VSL rock anchors were placed:

84 No. of type 5-6	(Working force 734 kN, ultimate strength 1048 kN) of lengths 10 to 26 m, including 4 m for bond
95 No. of type 5-9	(Working force 1100 kN, ultimate strength 1572 kN) with lengths of 13 te 21 m whereof 5 m bond length
22 No. of type 5-11	(Working force 1345 kN, ultimate strength 1922 kN) with lengths of 13 te 21 m, including 6 m bond length

On completion, the pile wall was back-filled together with the tunnel tubes, but it still retained its function subsequently as a retaining wall until the filled material had properly settled.

Fig. 33: View of the anchored pile wall

Fig. 34: View of the wall near the tunnel entrance



3. Securing of excavations

3.1. General

Excavations are now seldom constructed with battered sides, since their size and depth makes this almost impossible particularly in towns, where there is little or no distance between the buildings. Vertical excavation walls are therefore constructed and anchored simply and economically by prestressed anchors. It thus becomes possible even to construct very high walls and to avoid cour plicated struts and cross-bracing, which form a considerable obstruction to work. By the use of anchors, an obstruction-free excavation, suitable for the use of mechanical equipment, is obtained.

Various methods are available for the construction of the excavation walls; each of these is by its nature suitable for the requirements of a particular site.

A sheet pile wall is particularly suitable where groundwater is present in sandy soil or gravel, containing no rock fragments. If little or no water ingress is to be prevented, a more economical method, the Berlin Wall may be used. In this system steel sections are driven into the ground ait 1.5 to 3 m intervals and timber planks or concrete are incorporated between them as the excavation proceeds. At the saure time, prestressed anchors are installed. Their stressing anchorages are mounted on horizontal walings (of steel or concrete) or on steel seatings, fixed to the soldier beams.

Ever-increasing use is also being made of the anchored diaphragm wall. In this method, before the main excavation is carried out, a trench is excavated to the desired depth and to a width corresponding to the wall thickness. During its excavation, this trench is filled with a thixotropic liquid to prevent it from collapsing. The wall is then concreted in sections of 3 to 6 m length and throughout the full height and the liquid is pumped out. The advantage of the diaphragm wall is that it forms a final load-bearing element of the structure, can accept appreciable vertical loads and can be constructed to very large heights. Depending upon its height, the wall may be anchored by one or several rows of anchors. Instead of concreting it in-situ, it can also be prefabricated; this has the advantage that the surfaces are smooth and clean and thus do not normally require any additional finishing.

The capacity of the anchors required for securing the sides of excavations is generally relatively small. Depending upon the distance between the anchors, it ranges from 200 to 1000 kN. It is possible at any time during the excavation work to adapt the arrangement of the anchors and to obtain a balanced application of force, so that normally no wall movements or settlements of adjacent buildings occur.

In most cases the anchors are required and are in use only for a limited period. If they extend into neighbouring plots of ground and remain in the ground after destressing, they can form obstructions to later building operations in these zones. The owners of such adjacent plots and also the authorities are therefore increasingly demanding the removal of the entire anchor including the bond length after use. In order to satisfy these requirements, a VSL anchor suitable for such cases has been developed.

3.2. Centre Beaubourg, Paris, France

Client Etablissement Public du Centre Beaubourg, Paris
Engineer Ove Arup and Partners, Paris
Contractor Joint venture H. Coutant/ E.T.F./Intrafor-Cofor, Paris
Drilling and Anchor contractor Intrafor-Cofor, Paris
Years of construction 1972-1973

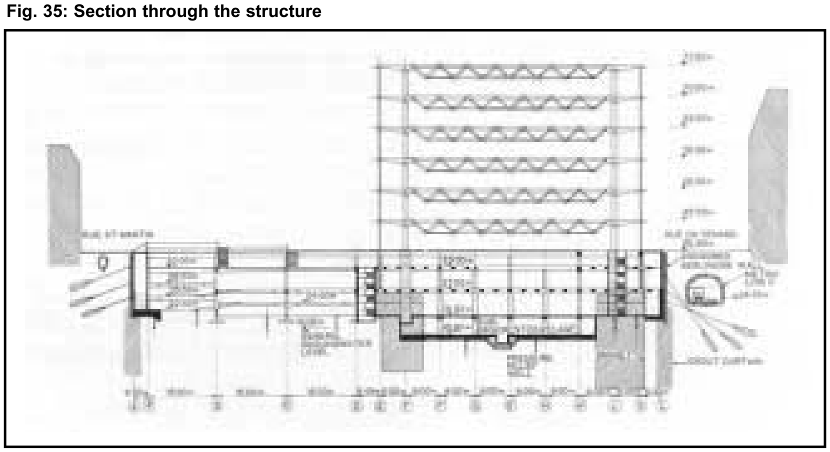
Introduction

In 1970 the French government held an international competition, with the aim of receiving designs from architects and engineers for creating the «Centre Culturel Beaubourg», which was to be constructed in the heartof Paris in the vicinity of Les Halles. The design put forward by the architecte R. Piano & Rogers with the co-operation of Ove Arup & Partners was successful and was adopted for construction. The Centre combines under a single roof a large information centre with library, a cinema and a museum of modern art; it is also known today by the name «Culture Centre Georges Pompidou».

The baisements of this Centre, which are as much as 20.50 m below street level, have plan dimensions of 122x 154 m. The main building, a steel structure, covers approximately one half of the ground area and is carried below the first basement storey by reinforced concrete panels (Fig. 35).

In view of the depth of the excavation, difficult problems were encountered, in particular along the Rue du Renard, since Metro line no. 1 1 also passes along there.

Fig. 35: Section through the structure



Choice of support system at the Rue du Renard

At a first glance, it appeared that a large part of the foundation work could be carried out in excavations without supports. This idea had however to be abandoned for the following reasons:

-The extent of the slopes along the Rue du Renard would have become too great, as sufficient space had to be provided for the construction plant such as cranes, excavators etc.

-The construction of the substructure had to be capable of rapidly providing as large a working platform as possible for the erection of the main building.

These requirements necessitated the construction of a vertical, temporary retaining wall through the entire depth of the excavation, without any internat supports.

The retaining wall

Several methods of building this wall were investigated:

-Construction by sections in strutted trenches. This method is very labour intensive and was too slow for the project.

-Diaphragm wall: This is in essence a very suitable method for this type of work and it would have been possible te, incorporate it into the substructure of the building. In the case of the excavation for the Centre Beaubourg, however, there was far too much preparatory work in the removing of the walls and foundations of the old buildings demolished on this site, se, that this method also could not be considered.

-Berlin Wall construction: This method was adopted. It was constructed here by placing steel soldier beams (Ibeams) at regular intervals in prebored holes and casting concrete around their bases. During the excavation, horizontal timber planks were progressively laid between the sections. The stability of the wall was ensured by anchoring each steel section individually with soil anchors.

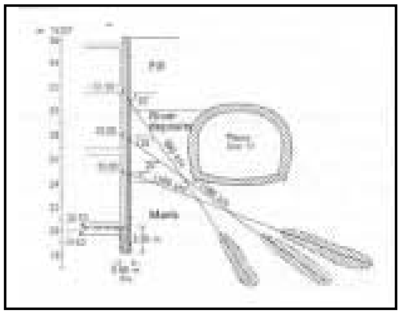


Fig. 36: Section through the wall

The Soil Conditions

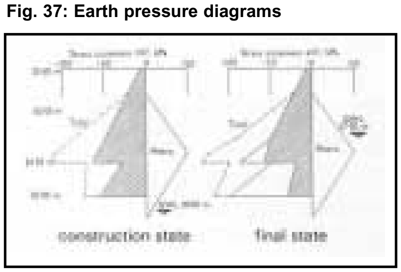
The geological conditions are shown in Fig. 36. In thé région of thé retaining wall, thé ground consists of three différent formations. The upper layer of fill was produced in thé démolition of thé old building and is a mixture of rubble and sand. Its lower level is between 28 and 32 m NGF (NGF = Niveau Général de France = Référence datum). The layer below this, extending down to +24 m NGF, consists of old and new river deposits; itcomprises in particular dense gritty sands. Belowtheseriverdepositsaretobe found pure and siliceous marls, in which thé anchors are bonded. The thickness of this stratum ranges from 5 to 17 m. The upper part consists of marly chalk of firm to hard consistency, which is highly disturbed by leaching and weathering. The lower part is laminar and is composed alternately of layers of mud limes and highly fractured limestone with open joints. The hardness of thé limestone increases with its depth. Below thé maris is thé so-called coarse limestone, a fossiliferous limestone of medium hardness.

The groundwater level is at about + 18 m NGF, that is 18 m below thé natural ground surface. For thé design of thé Centre Beaubourg, thé maximum groundwater level was assumed to be + 32 m NGF, which is thé level that can be reached by floods.

Design and method of construction

The earth pressure diagrams used for thé design of thé excavation wall are given in fig. 37. They are based upon thé conventional Coulomb theory. It was assumed that thé wall moves sufficiently for thé entire active earth pressure and one half of thé passive earth pressure to come into action, and that no friction occurs at thé wall. This, because thé wall in thé constructed state had a tendency to move down under thé vertical components of the anchorforcesand because, in thefinal state, a bitumen layer would be located

Fig. 37: Earth pressure diagrams

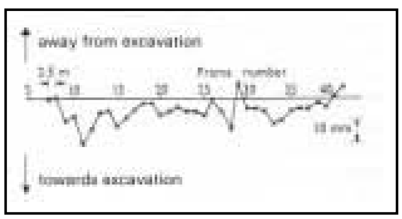


between the reinforced concrete wall and the inner face of the Berlin wall.

The vault of the underground railway tunnel was considered to be stiff and the horizontal force at the springing of the arch was calculated as 389 kN /m. It was distributed on the wall as shown in fig. 38. The I-beams were located at intervals of 2.5 m and anchored with three layers of VSL anchors. The inclination of the anchors was chosen to avoid the underground rail tunnel. More information about the anchors is given in table I.

The main excavation, starting in Summer 1972, was taken down from the level + 36 m NGF to +26 min one operation, with the exception of a working platform along the Rue du Renard, which was used for the drilling and installing of the vertical steel members of the Berlin wall. Only two weeks after the pile boring machine entered the site, half of these members had already been positioned. The platform was then taken down in steps of 2 to 3 m depth. As soon as it had reached the appropriate level, the holes for the anchors were drilled, the anchors were positioned and their bond lengths grouted. The level 19.5 m NGF was reached in February 1973. During stressing, each anchor was subjected to a test load and then released to the working force. Some anchors were fitted with a measuring system. The changes in force read once a month proved insignificant however. Fig. 40 shows the horizontal movements of the tops of the steel members. They ranged from -6 to +20 mm and on average were +5.8 mm.

Fig. 40: Horizontal movements of the tops of the steel members



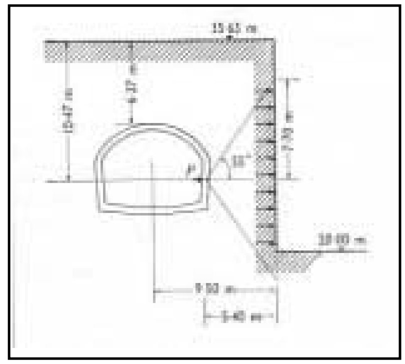


Fig. 38: Distribution of the horizontal force at the springing of the arch

The permanent wall

The method of construction chosen for the permanent wall consisted of casting a reinforced concrete wall against the Berlin wall. As soon as the reinforced concrete wall had reached the level of the anchors, these were destressed.

Concluding comment

The use of the Berlin wall construction method proved to be a rapid and economical solution. It enabled the excavation to be kept clear of obstructions so that the works could be carried out without interference.

Fig. 41: Anchor work

Table I: Type and number of VSL anchors used

Anchor type VSL	5-6	5-7	5-8	5-9	5-11	5-12
Working force (kN)	492	689	788	886	1083	1182
Ultimate strength (kN)	820	1148	1312	1476	1804	1968
Anchor lengths (m)	15–16	14–18	12	18–22	14–18	20–24
Whereof bond length (m)	6–7	7	6	7–8	7	8
Total number	71	68	58	33	22	122

Fig. 39: View of the wall

3.3. Underground railway station Lok Fu, Hong Kong

Client Hong Kong Mass Transit Railway Corporation

Engineer Freeman Fox & Partners (Far East), Hong Kong

Contractor Metro Joint Venture (MJV) consisting of: Hochtief AG (FR Germany), Dragages (France), Gammon Ltd. (Hong Kong), Sentab (Sweden)

Drilling and Anchor contractor VSL Engineers (HK) Ltd., Hong Kong

Years of construction 1975-1976

Introduction

I n view of the enormous increase in traffic resulting from the high population density, the government of the British Crown Colony of Hong Kong decided in 1972 after extensive studies to construct an underground railway network. When completed, this will comprise four independent lines with a total of 53 km of track, 48 stations and two underwater tunnels (see fig. 42).

The first section to be constructed is the line between the stations of Chater on Hong Kong island and Kwun Tong in Kowloon, which has a length of 15.6 km (including 12.8 km underground) and includes one of the two underwater tunnels(1.4km long). Workwascommenced on this length, known as ((Modified initial system» in August 1975; the commissioning date is 1980. The basic network was divided into a number of construction sections, for which international tenders were invited. One of the first sections to be awarded was No. 201, which comprises the railway station at Lok Fu and the tunnel tubes between the stations of Lok Fu and Wong Tai Sin and also between Wong Tai Sin and Diamond Hill.

Building of station at Lok Fu

The station at Lok Fu lies right in the middle of the densely populated urban area. It has plan dimensions of about 230x26 m and at the access points is about 26 m below the ground surface. The station is equipped at each end with air inlet and discharge shafts (fig. 43).

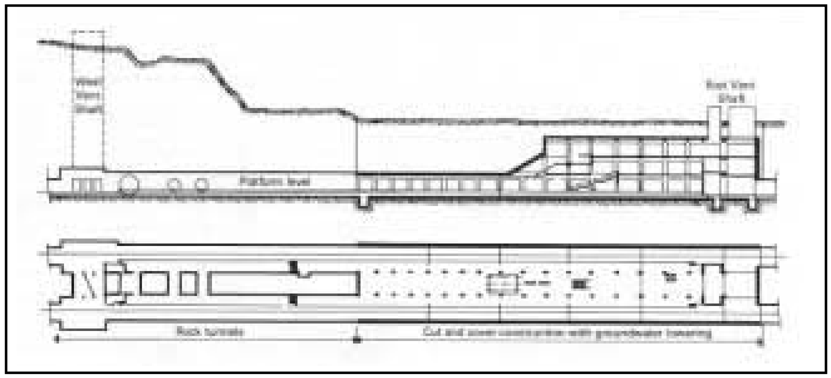


Fig. 43: Longitudinal section and plan view of the station

Construction was carried out partly in a 130 m long open excavation and partly, for the remaining 100 m, by tunnelling. Since the ground water level is considerably above the lowest excavated level, well points were sunk around the excavation to draw down the water table. The open excavation was completely surrounded with a Berlin wall, tied back by VSL soil anchors (fig. 44).

The Berlin Wall, which was designed conceptually by Hochtief in Germanyand developed to the last detail in Hong Kong, consisted of steel sections placed at intervals of 2 m and of concrete lining between them. The steel sections were placed in previously excavated, cased holes and were driven to the sound rock. The

Fig. 44: View of the excavation

Fig. 42: Underground railway system, Hong Kong (when completed)

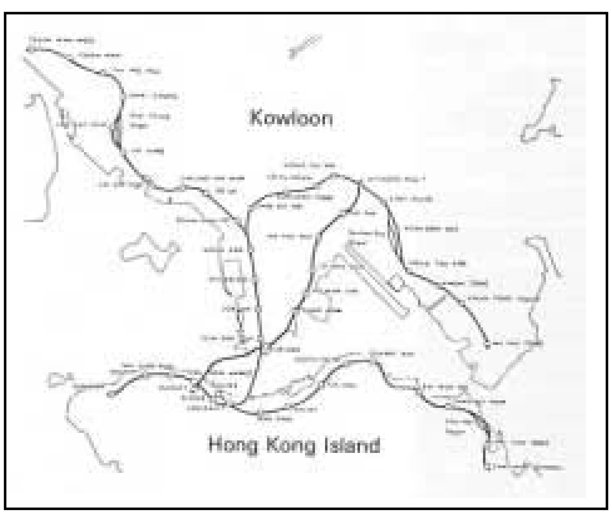


Fig. 45: Construction of Berlin wall

200 mm thick concrete intermediate wall was constructed in vertical sections of 1.5 to 2 m depth.

The VSL soil anchors

The detailed design of the soil anchors and the drilling of the anchor holes and the anchor work itself were carried out by VSL Engineers (HK) Ltd. The anchors used consisted of VSL types 5-2, 5-3 and 5-4 with working forces of 215, 322 and 430 kN respectively (equal to 58% of ultimate strength). The drilling of the 89 mm diameter holes was carried out in the soft material by wash-boring and the holes were cased. In the harder ground and in the rock, the percussion boring method was used. Where casings were necessary, the anchors were installed in them and the casings then withdrawn during the grouting of the bond length. The grout, with a water-cement ratio of 0.45, was injected through a tube passing along the centre of the anchor and kept under pressure until it set. For the 215 kN anchors, the bond length was 5.20 m, for the 322 kN anchors 7.60 m and and for the 430 kN anchors 10.00 m. The stressing of the anchors, the strands of which were individually sheathed in polyethylene and greased in the stressing length, wascarried outasfollows: theanchorwas first stressed sufficiently far to produce the working force when it was locked off. Then it was subjected to a test force, equal to 1.33 times the working force (or 80% of ultimate load). Finally, it was released again to the working force. The anchors were designed for a maximum life of three years. Extensive test and monitoring systems were used for measuring the wall deformations and the anchor forces.

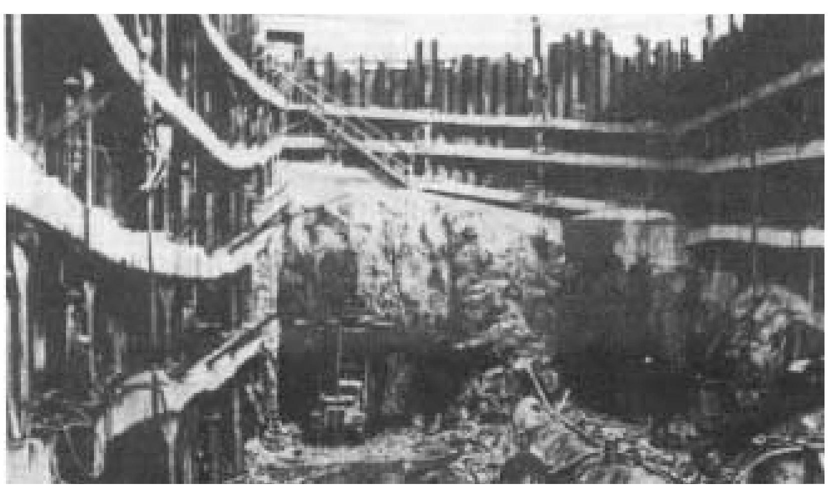
Concluding comment

The successful use of soil anchors in the excavation for the railway station at Lok Fu has ledtoan increasing useofthisform of construction in Hong Kong, and VSL soil anchors havealso been employed in a fairly large number of other sections of the first underground railway line.

3.4. Underground railway station, Stockholm, Sweden

Client Stockholms Gatukontor
Engineer AB Skanska Cement-gjuteriet, Stockholm
Contractor AB Skanska Cement-gjuteriet, Stockholm
Stabilator AB, Stockholm
Drilling contractor
Stabilator AB, Stockholm
Anchors Internordisk Spannarmering AB, Stockholm
Yea r of construction
1974

The rock formation underlying Stockholm is always posing new problems for contractors, since it is very unevenly undulating, sometimes outcropping at the surface and sometimes descending abruptly to great depth. These difficulties were frequently encountered in the construction of sections of the underground railway in open excavations. At one of the sites, the problem was solved in the following way: I-sections were driven at intervals of 7 m down to the rock (granitic gneiss). Sheet piles were then driven behind them to from a sheet pile wall. Due to the variation in the rock formation, the depth of penetration of the I-beams and piles varied considerably. The driving of the sheet pile wall was carried out to suit the progress of the excavation, as this enabled friction to be reduced. As soon as a certain excavation depth had been reached, a continuous transverse tie-beam of reinforced concrete was constructed, supported by the I-sections.
On each side of the I-beams, steel tubes were incorporated in the tie-beams, to serve later as guide tubes when drilling the anchor holes. The anchor holes were driven through the sheet pile wall at least 8 m into the rock. Every 2 to 3 m further down, a further horizontal tie-beam had to be concreted, until the entire excavated depth of 20to 25 m was reached. 619 VS L rock anchors of types 5-3 to 5-12 (ultimate strength 627 to 2, 506 kN) were used for anchoring the wall.

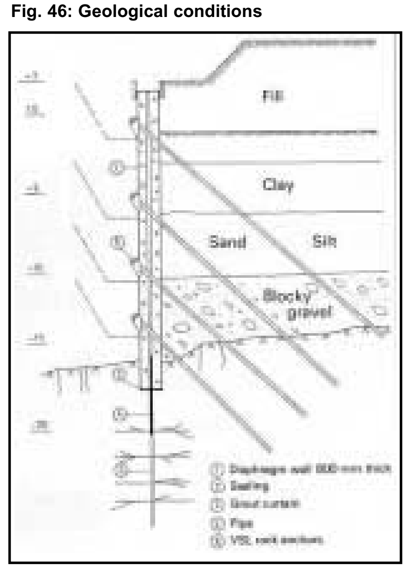


3.5. Building for Swedish Credit Bank, Stockholm, Sweden

Client Swedish Credit Bank, Stockholm
Engineer Hans Hansson & Co. AB, Stockholm
Contractor Samuelsson & Bonnier AB, Stockholm
Drilling contractor
Stabilator AB, Stockholm
Anchors Internordisk Spannarmering AB, Stockholm
Years of construction
1970-1972

I n the construction of the new building for the Credit Bank in the centre of Stockholm, a diaphragm wall was used for the first time in Sweden for retaining the excavation. This wall made it possible to construct the three to five basement storeys, extending down to 22 m depth, without pumping. In earlier projects, the work had seldom descended to below groundwater level, since on account of the high permeability of the soil considerable difficulty was encountered in lowering the groundwater. The Credit Bank wished, however, to construct at least two basement storeys below the groundwater level. A sheet pile wall could not be considered, because of the difficulty of driving sheet piles into the blocky gravel and the problem of sealing, so a diaphragm wall remained the only solution.
The wall surrounded the entire excavation along the site perimeter and was sealed at the contactjoint between wall and rock. In addition, a grout curtain was formed below the base of the wall. This complete sealing also relieved the bottom slab of the building from uplift. During the construction state, the diaphragm wall served as an excavation retainment, and in the final state it is the load-bearing external wall. To ensure that it would resist the horizontal pressure when the excavation was open, it was temporarily anchored with rock anchors. In the final state, after the anchors had been destressed and removed and the holes in the diaphragm wall plugged, the basement slabs resist

Fig. 46: Geological conditions



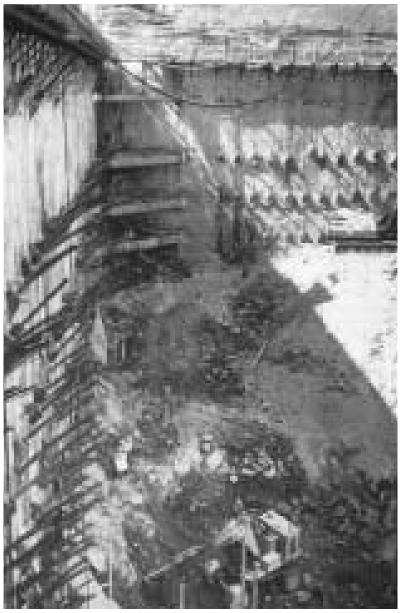


Fig. 47: View of the excavation

Fig. 48: Execution of the anchor works

the forces due to earth and water pressure. A total area of 5,500 mz diaphragm wall of 0.80 m thickness, divided into 59 panels, was built. The total volume of material removed during the excavation was 125,000 m3.

The VSL rock anchors used for anchoring the diaphragm wall were of type 5-12, 519 and 5-21 and had a working force of 1,000 to 2,400 kN. Their inclination was 45° and their total length up to 45 m, the bond length being 5to 7 m. In each rowof anchors, a line load of up to 1200 kN /m had to be supported. The drilling of the holes proved very troublesome, on account of the blocks and the groundwater. Special precautions had to be adopted to seal them and many holes had to be grouted after drilling.

Since, as already mentioned, this was the first occasion on which an anchored diaphragm wall had been constructed in Sweden and there were not yet any standards relating to this type of structure, detailed measurements were taken during construction at the diaphragm wall itself, on the neighbouring buildings and on the individual soil layers, in order to keep a proper control on the movements. The anchor forces also were checked.

3.6. Children's Clinic of the «Inseb» Hospital, Berne, Switzerland

Client Construction Directorate for Canton Berne

Engineer Dr. Staudacher & Siegenthaler AG, Berne

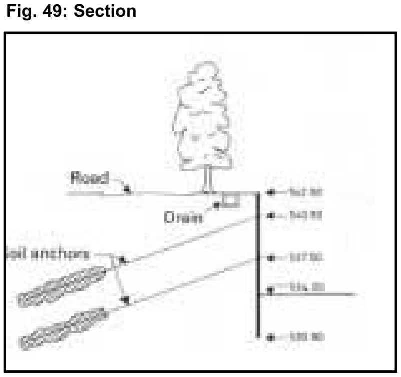
Contractor Losinger AG, Berne (Diaphragm wall)
(Prefabrication: Igeco AG, Lyssach)

Drilling Fehlmann Grundwasser-bauten AG, Berne

Anchors VSL INTERNATIONALAG (formerly Spannbeton AG)

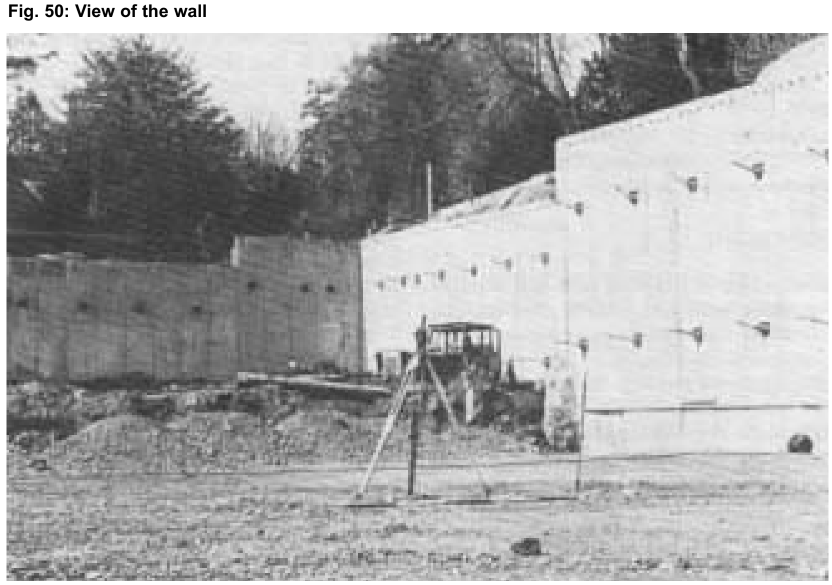
Years of construction 1972-1973

Fig. 49: Section



Excavation retainment in the form of a prefabricated diaphragm wall, which was anchored in two layers by 86 temporary regroutable VSL soil anchors type 5-4 (working force 341 kN, test force 492 kN, ultimate strength 657 kN) of 13 to 25 m length (including 6 m bond length). The 600mm thick and 104m long diaphragm wall was necessary, since the strata to be cutthrough (claytofinesand)werewaterbearing and since the excavation was situated a few metres behind the 16storey main building of the hospital, which meant that any reduction in stability must be prevented.

Fig. 50: View of the wall



4. Anchoring against hydrostatic uplift

4.1. General

In the vicinity of bodies of water such as rivers, lakes, or the sea, where the groundwater level is in general relatively near to the surface, buildings must often be secured against uplift. One solution to the problem is to give the building sufficient weight; this is, however, not always possible and in many cases it is also not economical. The anchoring of the structure into the deeper, load-bearing ground by prestressed anchors results however in considerable savings, both in the quantity of the spoil to be excavated and the materials to be used.
A provision for resisting uplift may be of a temporary nature, for example for the foundation slabs of buildings, in that it only needs to remain effective up to the time at which the structure possesses sufficient weight, or it may fulfil a permanent function,forexampleinthetanks of sewage treatment plants, swimming pools, dry docks etc., which are not sufficiently heavy when empty to resist uplift. The anchors used for uplift prevention may therefore be formed as eithertemporary or permanent anchors.
Another way of using prestressed anchors is in conjunction with piles. If buoyancy or horizontal forces act in addition to the vertical loads, the piles must also be capable of accepting a tensile load. Generally such piles are therefore prestressed. If, however, it is not possible to drive them sufficiently into the ground, they cannot transmit the tensile forces by surface friction. They then are provided with anchors which pass through them and anchor them in the deeper strata.

4.2. Rainwater overflow tank, Ellwangen, Federal Republic of Germany

Client The town of Ellwangen (J agst)
Engineer Ingenieurburo M. Brandolini, Ulm
Contractor
Drilling Klee KG, Ellwangen
contractor Dr. Ing. Kurt Waschek, Gunzburg/Donau
Anchors VSL GmbH, Langenfeld
Year of construction 1976

The rainwater overflow tank is a circular structure with an internal diameter of 60 m. At the maximum water level, the depth is 2.63 to 4.02 m (the tank floor sloping down towards the centre). Since the groundwater level can rise to 0.62 m below the rim of the tank, uplift prevention must be provided for the empty state. The tank bottom was therefore anchored with a total of 111 VSL permanent rock anchors. These anchors were arranged in three concentric rings. The innermost ring (diameter 15 m) contains 20, and the central ring

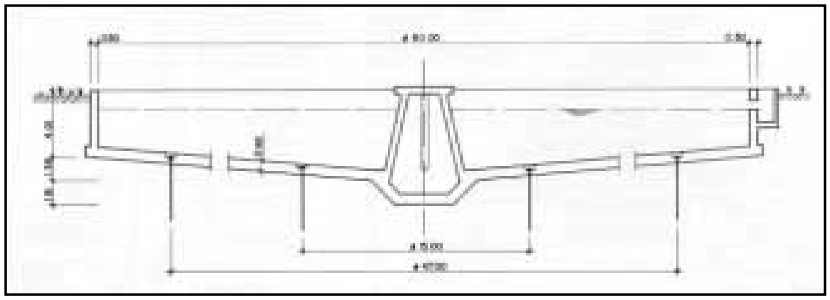


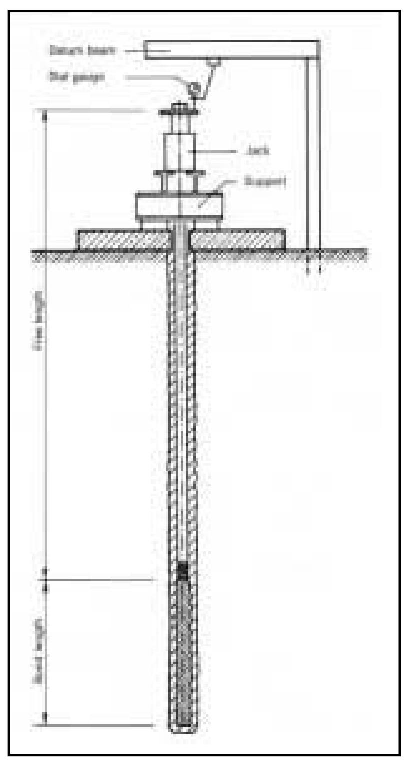
Fig. 51 Section through the anchored tank

Fig. 52. View of the site with installer anchors

central ring (diameter 27 m) 36 VSL rock anchors of type 5-6, each 13 m long and of 500 kN working force (= 1 /2 ultimate load). The 55 anchors of the outermost ring (diameter 47 m) are of type 5-7 (working force 583 kN) and also 13 m long. All anchors are vertical and provided with permanent corrosion protection and a 4 m bond length.
The actual anchor work, which was carried out in late Autumn 1976, was preceded by a suitability test on 3 anchors. At the time these tests were carried out, the excavation had reached the foundation level. At this level, hard sandstone outcropped. In the anchorage bond zones trial bores indicated the presence of fractured, broken siltstone. Three anchors of the outer ring were chosen for the suitability test. Holes of diameter 101 and 102 mm were drilled with core drills and rotary percussion drills, the anchors were placed, the bond length grouted and the test carried out one week after grouting. The test arrangement is indicated diagrammatically in Fig. 53. The tests were based upon DIN standard 4125, Part 2 and the draft of the authorisation guidelines for cement mortar-grouted rock anchors. Force-displacement curves were recorded and the elastic and permanent deformations, friction loss and creep determined. All the tensile tests satisfied the requirements of the standards.

The remaining anchors were then executed within four weeks.

Fig. 53: Test device (diagrammatic)



4.3. Stilling basin No. 3 at Tarbela Dam, Pakistan

Client Pakistan Water and Power Development Authority (WAPDA), Lahore
Engineer Tippetts-Abbett-McCarthy-Stratton, New York
Contractor Tarbela Joint Venture
Drilling contractor Rodio S.p.A., Milan
Anchors VSL INTERNATIONAL LTD, Berne
Years of construction 1976-1977

Introduction

The Tarbela Dam, which has already been referred to in Section 1.6,comprisesapart from the main dam and various auxiliary plant four bypass tunnels, two of which are equipped with stilling basins,

In August 1974, that is two years before the planned commissioning of the installation, the lake had to be drained as an emergency measure through tunnels no. Sand 4, due to damage in tunnels nos. 1 and 2. Due to the asymmetrical outflow, stones and broken rock and concrete were washed from downstream into the stilling basins, and erosion and cavitation damage occurred to the bottom slabs of these basins. Since both tunnels had nevertheless to be kept in use for irrigation purposes, the erosion increased, especially in stilling basin no. 3, to a dangerous extent. In the winter of 1975/ 76, the damage was repaired with underwater concrete. In April 1976, the basins were again brought into use butafteronly a few hours stilling basin no. 3 again had to be taken out of use, since large areas of the concrete base had been carried away. This new setback made comprehensive remedial work essential. The main part of the new work consisted in the anchoring of the bottom slab by rock anchors, and making good the slab and strengthening it. The main objective of anchoring the bottom slab was to secure it against static and dynamic uplift forces (vibrations) and also to make the underlying rock participate in the actions in the concrete.

Stilling basin No. 3is 186m long, 36.60m wide and is sub-divided into 12 sections (see Fig. 54). The rock conditions in the vicinity of the bottom slab are very heterogeneous. Chlorite shales, carbonate shales, limestone and gypsum outcrop. The thickness and depth of the individual strata vary considerably, and so does the quality of the rock. Moreover, in the 80 trial bores which were driven, a high sulphate content was observed.

Design of the anchors

The design of the rock anchors was based upon the following uplift forces;

- a hydrostatic pressure a_1 from the difference between the mean tail water level and the bot tom of the basin, which gave the value 2.96 bars for sections 8 to 1 1;
- a hydrodynamic uplift$a_2$of 1.02barsat slabs 3 to 9.

The maximum uplift was therefore 3.98 bars. The anchor force and anchor length were determined from:

-V_G=A
where V_G=calculated working force of anchor (=60% of ultimate strength). A=the uplift force associated with the anchor, calculated from a=a1+a2

- G'=VGS1
where G'=the average weight of underlying soil co-operating with the anchor
S_1 =safety factor= 1.25

The free anchor length was determined from G', the participating rock being assumed to be a pyramid shape from the centre of the bond length of the anchor and having a submerged weight of 1.6 t/ m^3.

When choosing the size of anchor, the governing factor was the possible arrangement between the existing drainage system and the upper limiting value of 2,500 kN per anchor for VG, which was adopted as a maximum in view of the difficult ground conditions. The anchors chosen were VSL rock anchors 6-14 and 6-16 and also 6-7 and 6-8 (Fig. 55). VG was 155.6 kN per strand, that is 2490 kN total for the largest anchors. The strands used were individually coated with corrosion protective grease and sheathed with polyethylene at works. The grease and sheathing of course had to be removed for the bond length and at the stressing anchorage. The bond length was 6 m for the production anchors throughout.

Test Anchors

Ten anchors were constructed as test anchors, in order to test the anchor itself, the installation methods and the transmission of force to the underlying rock and thus the design bases for the anchor and its components. For equivalent boreholeand bond length dimensions, the test anchors must be capable of transmitting

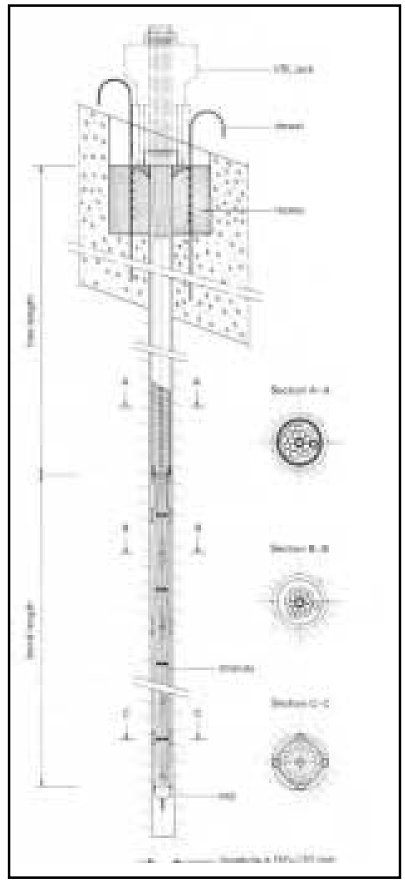


Fig. 55: VSL Rock anchor

twice the working load. These anchors therefore required 22 strands, to prevent the yield point of the steel from being exceeded.

Four test anchors were given a 9 m bond length, and the others a 6 m bond length. The test anchorswere distributed overthe bottom slab in zones of good, average and bad underlying soil. For carrying out the test programme, Losinger extensometers and electrical VSL load cells were used (Fig. 56).

Fig. 56: Device for testing the test anchors

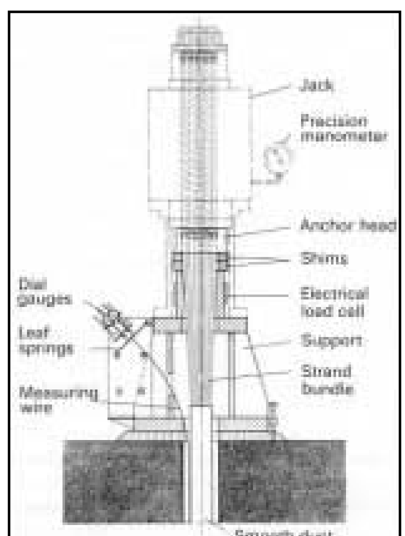
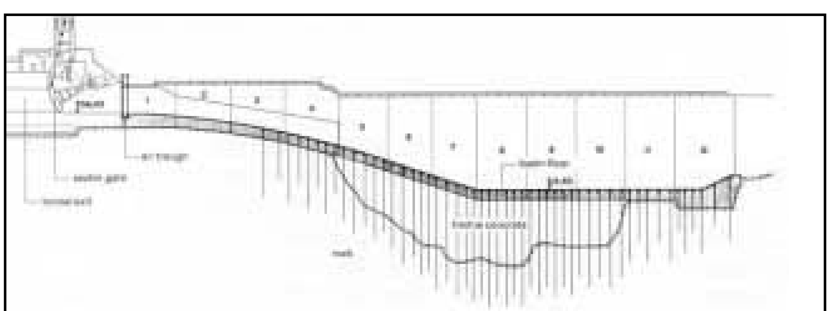


Fig. 54: Longitudinal section



Readings were taken of the anchor force, cable extension, deformations in the bond region and settlementatthe bearing plate. The tests showed that the anchors were capable of fulfilling the requirements specified; a further result obtained was that a bond length of 6 m was sufficient, since no advantages were obtained by using 9 m.

Execution of the work

Only six and a half months were available for the work on the 576 anchors from the date of awarding the contract to its completion. Day and night working was therefore necessary, using several sets of drilling equipment simultaneously.

Some of the holes were rotary drilled, but the majority percussion drilled using down-the-hole-hammers. The hole diameter was 152 to 171 mm. After drilling, the holes had to be grouted to stabilise them and then re-drilled. A hydraulic pressure test was then carried out.

The anchors were assembled and stored until required on the left side of stilling basin no. 4. Two tower cranes lifted the anchors from the storage area and fed them directly into the boreholes. These cranes were just high enough for placing the 16.80 to 38.10 m long anchors. A ulphate-resisting cement grout with a w/c ratio of 0.42 was used for grouting. Seven days after grouting, the stressing operations in accordance with the FIP recommendations on ground anchors were carried out. After the anchor force had been successfully tested, the protruding strands were cut off two to five days later, the anchors then being completed.

Fig. 57: The stilling basin during anchor work

5. Securing of caverns

5.1. General

The excavation of underground chambers in rock for tunnels, galleries and caverns inevitably leads to a change in the state of stress, which is reflected in strains and deformations and in the worst case can lead to a collapse. To prevent such an event, the cavity must be secured. This can be done by the installation of supports followed by the concreting of a rigid lining. The principal disadvantage of this method consists in the considerable reduction of the space available for carrying out the work by mechanical means.

Present-day knowledge of rock mechanics and modern rock anchor technology makes the securing of large caverns by prestressed anchors and gunned concrete possible. With this method, which was introduced in the sixties, the anchors have the function of creating in the rock mass around the cavity a loadbearing ring, which is intended to prevent destressing and appreciable displacement of the rock during the excavation. The method has proved very successful both as a permanent support system itself and also in relation to the execution of the rock removal work. It leads to improved efficiency of working as a result of the continuous working sequence and to a more accurate excavated profile with improved safety. As a result, shorter construction times and reduction of the tunnelling costs are achieved.

The supporting system using anchors can be adapted and corrected at any time during excavation. Fluctuations in the state of stress are monitored by measuring anchors, which areinstalled just like normal anchors immediately after excavation but differ from them in incorporating measuring equipment. They enable a permanent check to be carried out until completion of the stabilising phase, that is, if necessary, for several years.

5.2. Cavern Waldeck II, Federal Republic of Germany

Client Preussenelektra, Hanover

Engineer Siemens AG, Erlangen

Contractor

Joint venture Cavern Waldeck II
Beton- and Monierbau AG, Frankfurt
Baugesellschaft H. Rella & Co., Vienna
Allg. Bauges. A. Porr AG, Vienna
Dyckerhoff & Widmann KG, Wiesbaden

Drilling contractor

Terrasond Grundbau GmbH, Essen

Anchors VSL INTERNATIONAL LTD, Berne

Years of construction

1970-1972

The region of Waldeck in Northern Hesse has long been recognised as particularly suitable for the construction of a pumped storage scheme. After the building of the Edertal dam (a river power station) between 1910 and 1914, the pumped storage scheme at Waldeck I was built in its immediate vicinity from 1929 to 1931. Forty years later, namely between 1968 and 1975, the Waldeck I I plant was constructed. Whereas at Waldeck I all the equipment was above ground, at Waldeck I I only the upper

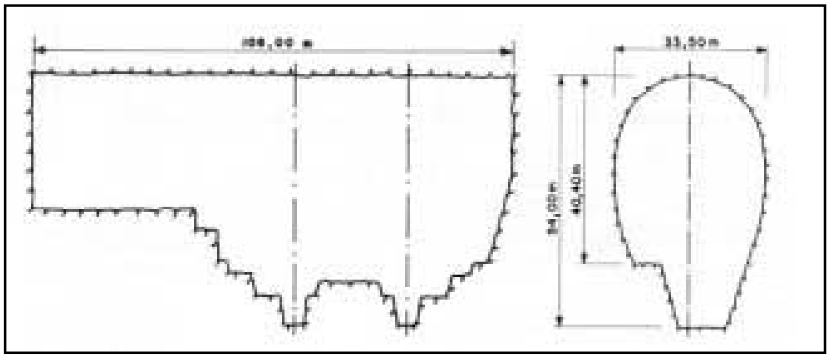


Fig. 58: Dimensions of cavern

upper basin and the distribution plant is above ground; the pressure shaft, power house, surge chamber and tail race tunnels were built into the rock. In the erection of the pumped storage work at Waldeck II, provision was also made for the requirements of a third extension stage. The Waldeck II plant has an output of 440 MW and utilises an average gross head of 329 m between the upper basin on the Ermerod (of 4.3 million m3 capacity) and the lower basin situated in the Eder valley (effective capacity 5.4 million m3).
The cavern for the underground power house at Waldeck I I is 106 m long, 54 m deep and 33.5 m wide and required the excavation of 106,000 m3 of rock (Fig. 58). In view of these dimensions, the only feasible way of securing the cavern was the production of a self-supporting vault by means of prestressed rock anchors, since concrete lining would have been far too costly. The stress conditions around the cavity were determined by computer calculations and photoelastic analyses and on this basis the necessary anchor forces and lengths were determined.
Before excavation was commenced, five different anchor systems were tested in a trial tunnel and on the basis of the results obtained it was decided that 85% of the rockanchors in thecavern would beofthe VSL system. In total, 716 VSL anchors of 1300 kN working force (=0.5 x ultimate strength) and 68 VSL anchors of 940 kN working force were used.

used. The actual securing of the cavern was carried out by means of the larger anchors, which were 20 to 28 m long (including 4.50 m bond length) and were inclined between 40° downwards and vertically upwards, while the smaller anchors were used for securing the crane girders. These anchors have lengths of 13 to 18 m (bond length 3 m) and inclinations from 45° to 75° upwards.
The anchors, which have a free length that remains elastic during working life, were made up on site and installed progressively to suit the rate of excavation. Anchor holes of 116 m diameter were drilled into the rock, which consisted of graywacke (grey sandstone) and clay shale; after a water test, these holes were grouted if necessary and re-drilled. The anchors and prefabricated foundation blocks of concrete were then positioned. A few days after grouting the bond length, the anchors were tested with 1.3 times the working force and anchored at the working force. One week later, a check on the stressing force was carried out; the secondary grouting was then completed and 1'/z to 3 months afterwards the stressing force again checked.
Ninety anchors were constructed as measuring anchors for long-term monitoring, the stressing lengths being injected with grease instead of grout and the anchors being fitted with VSL 2000 kN load cells, which could be read at any time from a central measuring station.

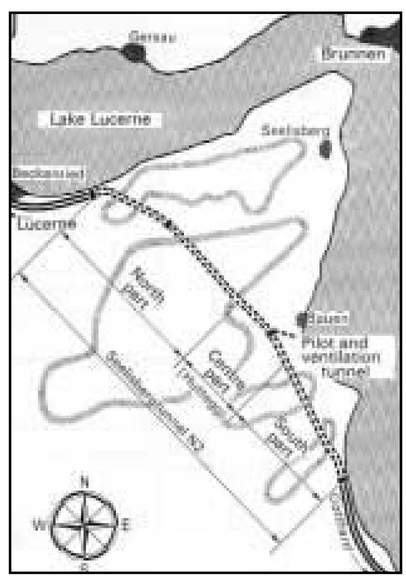
Fig. 59: The secured cavern

5.3. Ventilating station at Huttegg for the Seelisberg tunnel, Bauen, Switzerland

Client Highways Construction Department of Canton Uri
Engineer Elektrowatt Ingenieurunternehmung AG, Zurich
Contractor Joint venture Huttegg MurerAG, Erstfeld
Losinger AG, Berne
Emil Baumann AG, Altdorf
Drilling InjectobohrAG,Zurich
Anchors VSLINTERNATIONALAG (formerly Spannbeton AG)
Years of construction 1972-1975

Introduction
Apart from the Gotthard tunnel, the Seelisberg tunnel is the most important underground work on Swiss National Highway N2, which forms the main motorway connection in the North South direction between Basle and Chiasso and crosses the Alps. The tunnel is situated on the left bank of Lake Lucerne, between Beckenried and Seedorf. It consists of two parallel tubes, each 9.25 km long (Fig. 60).
For the contract, the tunnel was divided into three sections, the limits being defined by the geological conditions. The north and south section together with the two portals lead partly through very hard limestone formations. Its horseshoeshaped cross-section was driven using conventional blasting procedures. The central section, on the other hand, i. e. the Huttegg section, is located throughout its entire length of about 2 km in the Valanginien marl, which experience has shown to possess a tendency to compression phenomena under high overburden, its loadbearing strength then lasting only a few hours to a few days. It was therefore decided to adopt a circular tunnel section and to use shield driving with tubbings to line the tunnel.

Fig. 60: Site plan and line of Seelisberg tunnel



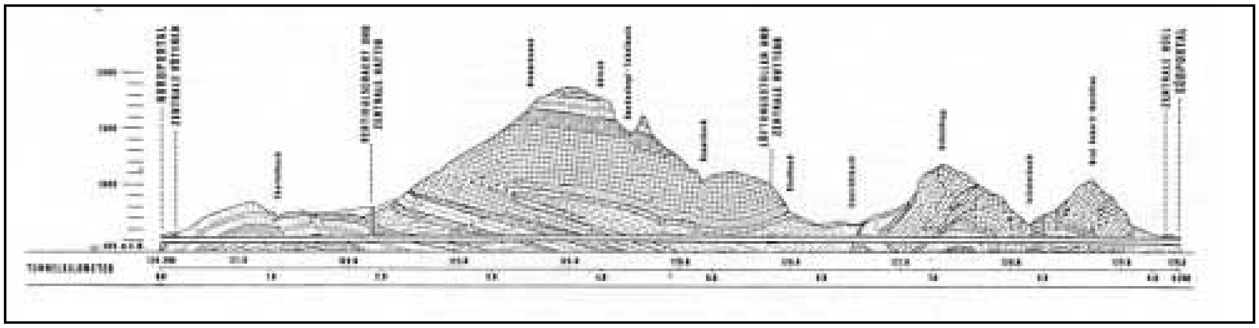


Fig. 61: Longitudinal section of the Seelisberg tunnel

The access tunnel

Since the Huttegg tunnel section is situated about 400 m inside the mountain, several pilot tunnels and a 640 m long access and ventilation tunnel, terminating in the underground ventilating station, had to be driven first (Fig. 62). The

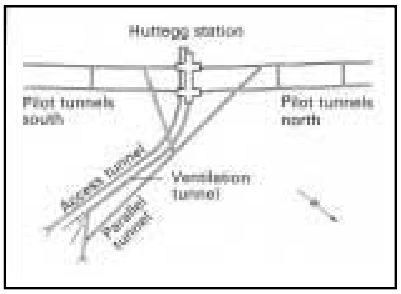


Fig. 62: Pilot tunnel layout for Huttegg section

behaviour of the marl, which is overlain by 200to 900 m of cover, was not particularly well-known, since only a few tunnels had been driven through this type of rock under the same conditions. On account of the very deep cover, high rock pressures must be expected, leading immediately after excavation to rapid breakdown of stability of the rock surface and to considerable deformations.

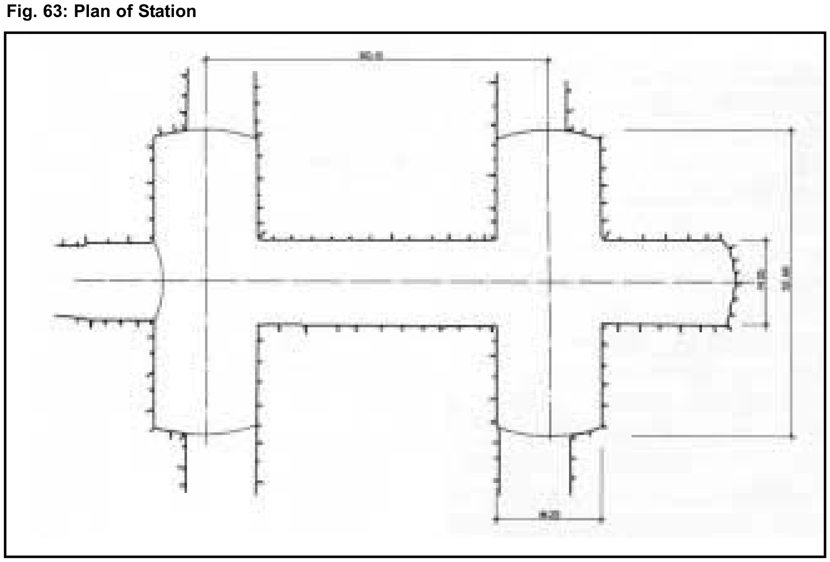
In laboratory investigations, this type of marl proved to be very heterogenous and exhibited highly variable geomechanical properties. In addition, when driving the pilot tunnel appreciable quantities of meth ane gas were encountered, which could well give rise to dangerous explosive mixtures. To eliminate,this risk, suitable precautions had to betaken for ventilation during the tunnel construction.

The ventilation station

The underground ventilation station at Huttegg consists of two widened out portions of the tunnel tubes in the form of caverns each 52.7 m long and 18.2 m wide, connected together by a transverse tunnel of 14 m width and 16 m height (Fig. 63). The entire volume of excavation for this central station was almost 49,000 m3. The caverns house the feed and extract blowers, while the electrical equipment such as transformers, switchgear etc. are situated in the transverse tunnel.

The method of working chosen for excavating the station (Fig. 64) is distinguished by its adaptability. After the roof tunnel has been broken out and widened on both sides to almost the entire width of the cavern, the profile was completed in a number of vertical steps of 3 to 4 m depth. In order to stabilise the rock surface during the work, a support system was constructed continuously and immediately after the excavation in the form of a 150 mm thick, reinforced gunite lining and rock bolts (one per 1.25 ml) and rock anchors (one per 20 mz). The rock anchors produced around the cavern an aactive support arch», which made the vault and side walls self-supporting. The lining

Fig. 63: Plan of Station



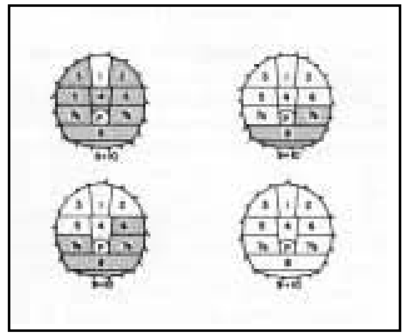


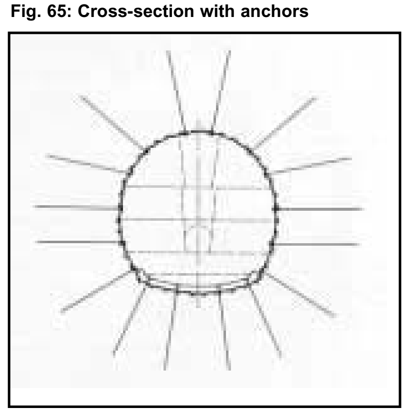
Fig. 64: Method of excavation

system was designed for a supporting force of 300 kN/mz. This force was determined by extensive geomechanical calculations based upon the results of readings which had been carried out previously in the pilot tunnel. Nevertheless, there was considerable uncertainty due to the highly variable rock mechanics properties. Control measurements were therefore carried out during the entire construction period to check the validity of the assumptions and to achieve the necessary safety. In cases where strengthening was necessary, the excavation work cou Id be interrupted.

The prestressed rock anchors

The entire ventilating station was secured with VSL rock anchors type 6-9, each of 2365 kN ultimate strength, arranged in a 4.5 m lattice. In the longitudinal tunnels, each anchored cross-section contains 16 anchors, and in the transverse tunnel there are 14 in each (fig. 65). The anchor lengthsare 16to 18m. Foralltheanchors, the bond length ist 4.5 m, this figure being

Fig. 65: Cross-section with anchors



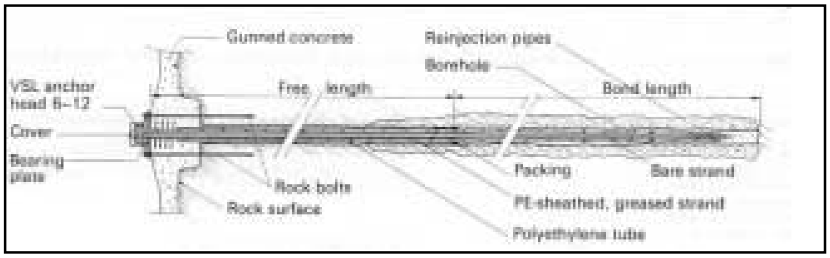


Fig. 66: VSL rock anchor 6-9 (diagrammatic)

based upon tests carried out in the pilot tunnel.
In addition, the anchors have the following features:
- the strands are plastic sheathed and greased in the free length, to enable the anchors to be restressed and the force to be checked at any time;
- a smooth polyethylene tube surrounds the bundled strands along the entire length as an additional corrosion protection and as a protection against mechanical damage during installation;
- the stressing head of each anchor is threaded, to enable a movable load cell to be attached for measuring the anchor force;
- the primary and secondary grouting were carried out in one single operation; the bond length can, when required, be regrouted;
- the bearing plate was of sufficient size to keep the bearing pressure on the rock behind the plate during the test load to 13.0 N/mm2.

All anchors were factory-assembled and transported to the site. Seven days after installation and grouting, theywere stressed to the test force of 1650 kN (70% of ultimate strength) and then anchored at 800 kN, i.e., 34% of ultimate strength. This low initial force was chosen in view of the rock deformations expected to occur. A total of 634 VSL rock anchors were installed in the central ventilation station; approximately 40% of them are ascending, the remainder descending.

Surveillance instrumentation

During the entire construction time, the deformations of the rock at the excavated face and within the rock mass were continually monitored, in order to ascertain whether the support system was adequate or not. Readings were taken for this purpose at three points as follows:

-A measuring cross-section at the centre of the transverse tunnel contained 3 extensometers (2 horizontally in the walls and 1 vertically in the roof) and 6 measuring anchors equipped with hydraulic VSL load cells (fig. 68).

-At the measuring points attheintersections of the axes of the transverse and longitudinal tunnels of the ventilating station, there were one vertical extensometer and 2 measuring anchors at each point.

In the central region of the transverse tunnel, very severe deformations were observed. The downward movement of the rock amounted in total to almost 80 mm, the principal deformation occurring during excavation phases 2 and 3 (fig. 64). The reason for this was a locally weakened zone between the two longitudinal tunnels. Movements as large as this are naturally undesirable, so at this point two additional rows of anchors had to be installed in the roof, to enable the movements to be kept under control. About 80% of the vertical movement occurred during the removal of the roof material. It was also established during the measurements that ther esilientrockwaslimitedto the first two metres behind the face. Fig. 69 shows clearly the difference in the deformations at depths of 2 and 6 m at extensometer 2.

At the point of intersection of the tunnel soffitts, the force determined by the load cells at the anchors increased from 800to 1000 kN; this could however be accepted, sincethe anchors had been designed for a working force of 1500 kN.

The already mentioned periodic force measurements were also carried out at the standard anchors, using a «travellling»

Fig. 67: Stressing operation

loadcell (fig. 70). Someanchorsdid indeed show increases of force, but these remained below the limiting values and therefore no special measures were required.

After the tunnel floor had been concreted, the deformations again decreased.

Concluding comment

The choice of a flexible method of construction for the excavation and securing of the tunnels, in conjunction with a method of systematically monitoring the rock deformations, enabled the underground ventilating station at Huttegg to be constructed economically and safely. It may be pointed out here that, in spite of conditions that were at times extremely trying, the work was brought to full completion without a single accident.

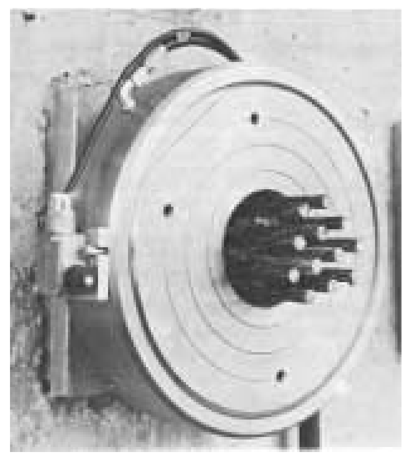


Fig. 70: VSL load cell type G 200

Fig. 68: Measuring cross-section at centre of transverse tunnel

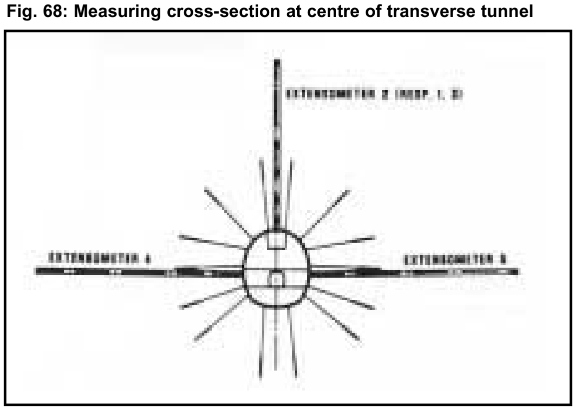
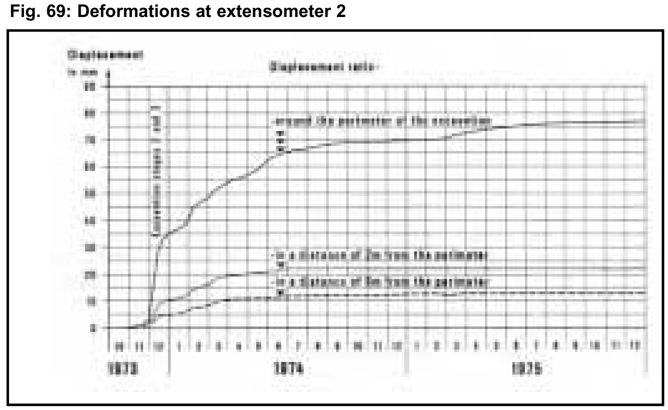


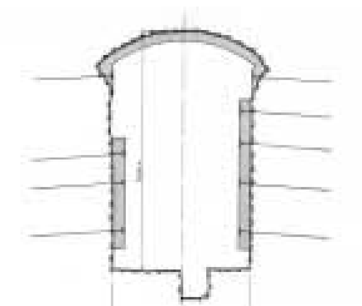
Fig. 69: Deformations at extensometer 2



5.4. Review of various other caverns

Cavern of the pumped storage scheme at Vianden, Luxemburg

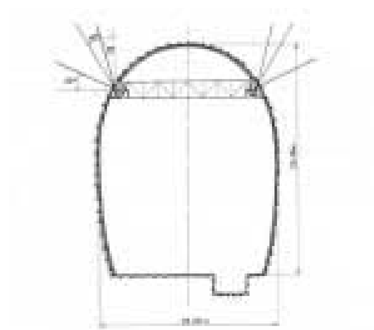
Client	Societe Electrique de LOur, Luxembourg
Engineer	Elektrizitats AG, formerly W. Lahmeyer & Co., Frankfurt a/Main
Contractor	Consortium «Centrale de Vianden» Hochtief AG, Koblenz/Wayss & Freytag AG, Cologne/Alfred Kunz & Co., Munich/Compagnie Beige de Chemins de Fer et d'Entreprise SA, Brussels/Entreprise E. Nerming, Luxembourg /Conrad Zschokke SA, Geneva/Losinger SA, Lausanne
Anchors	VSL INTERNATIONAL LTD, Berne
Years of construction	1961-1963
Dimensions of cavern	Height 29.3 m, Breadth 17.0 m, Length 330.0 m Excavation 150,000 m'
Type of rock	Coarse-bedded reddish clay shale



Securing of crane track:
Obtained by means of 164 VSL rock anchors of 1250 kN working force and 10.40 to 13.40 m length. Spacing: 4.0 m
Securing of the vertical tie beams:
Each tie beam was secured with two or three rock anchors. 270 VSL rock anchors of 900 kN working force and with lengths between 9.40 and 13.70 m were used for this purpose.

Underground powerhouse, Sackingen, Federal Republic of Germany

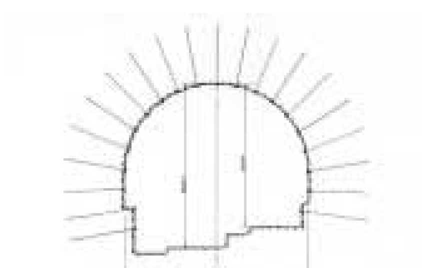
Client	Schluchsee Werk AG, Freiburg /Breisgau
Engineer	Elektrizitats AG (formerly W. Lahmeyer & Co.), Frankfurt a/Main
Contractor	Joint Ventrue , «Maschinenkaverne Sackingen» Hochtief AG/Dyckerhoff & Widmann KG/A. Kunz & Co/Sanger & Lanninger KG
Anchors	VSL INTERNATIONAL LTD, Berne
Years of construction	1964-1966
Dimensions of cavern	Height 29.55 m, Breadth 23.00 m, Length 161.60 m Excavation 100,000 m'
Type of rock	Paragneiss fractured in 2 directions



Securing of vault abutment:
145 VSL rock anchors of 1500 kN working fore and lengths between 12.0 and 15.0 m. Spacing 1.0 to 2.0 m.
Securing of crane track:
Obtained by means of 156 VSL rock anchors of 800 kN working force and 10 m length.

Underground power station, Veytaux, Switzerland

Client	Forces Motrices de l'Hongrin SA, Lausanne
Engineer	Compagnie d'Etudes de Travaux Publics SA, Lausanne
Contractor	Consortium «Centrale de Veytaux» Losinger SA, Lausanne /Deneriaz SA, Lausanne, Sateg SA, Lausanne/Oyex Chessex & Cie SA, Lausanne
Anchors	VSL INTERNATIONAL SA (formerly Precontrainte SA)
Years of construction	1965-1967
Dimensions of cavern	Height 23.0-26.5 m, Breadth 30.5 m, Length 137.5 m Excavation 90,000 m^3
Type of rock	Almost horizontally stratified limestone and marl, highly fractured in several directions



Securing of roof:
366 VSL rock anchors of 1350 kN working force; Length: 11.4-18.4 m. Plus 132 VSL rock anchors of 1150 kN working force; Length: 11.4-18.4 m 1150 kN or 1350 kN rock anchors were used depending upon the local conditions.

Spacing in longitudinal direction: 4.3 m Spacing in transverse direction: 3.0 to 4.0 m On average, there was one rock anchor to 14 m2 of developed vault surface.

Securing of walls and face sides:
155 VSL rock anchors of 1150 kN working force; Length: 1 1.4-18.4 m

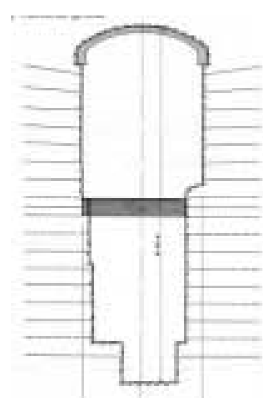
Additional securing between the large anchors:
1,729 VSL rock anchors type 6-1; length; 4.0 m. These rock anchors were anchored with synthetic mortar. The synthetic mortar enabled the rock anchors to be stressed after 7 to 24 hours.

Underground power station, Roncovalgrande, Italy

Client	ENEL, Ente Nazionale per l'Energia Elettrica, Rome
Engineer	ENEL, Progettazioni a Costruzioni idrauliche elettriche e civili, Department of Milan
Contractor	Consorzio Caldart a Astrid, Maccagno
Anchors	VSL INTERNATIONAL SA (formerly Precompresso SA)
Years of construction	1967-1970
Dimensions of cavern	Height 61.0 m, Breadth 21.0 m, Length 195.0 m

Securing of crane track:
35 VSL rock anchors type 5-5 (working force 542 kN)
70 VSL rock anchors type 5-7 (working force 758 kN)
Length: 21.0-27.0 m

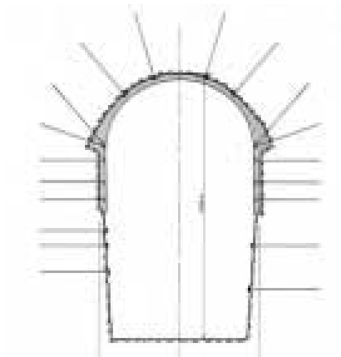
Securing of walls:
The high cavern required considerable securing of its walls.
This was done with:
270 VSL rock anchors type 5-2 (working force 217 kN)
270 VSL rock anchors type 5-3 (working force 325 kN)
185 VSL rock anchors type 5-4 (working force 433 kN)
11 VSL rock anchors type 5-5 (working force 542 kN)
260 VSL rock anchors type 5-7 (working force 758 kN)
45 VSL rock anchors type 5-9 (working force 975 kN)



Length: 16.0-17.0 m
The spacings between anchors both horizontally and vertically were 3 m.
In addition, 2,400 VSL mono rock anchors type 5-1 with a length of 5.0 m were installed at regular spacings. The mono rock anchors were anchored in synthetic mortar.

Underground power station, El Toro, Chile

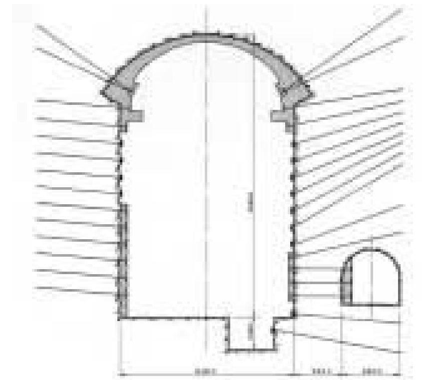
Client	Empresa Nacional de Electricidad SA, Santiago, Chile
Engineer	Empresa Nacional de Electricidad SA, Santiago, Chile
Consulting Engineer	Elektro-Watt AG, Zurich
Contractor	Empresa Nacional de Electricidad SA, Santiago, Chile
Anchors	VSL INTERNATIONAL LTD, Berne
Years of construction	1967-1970
Dimensions of cavern	Height 39.5 m, Breadth 24.5 m, Length 103.0 m Excavation 92,000 m'
Type of rock	Granodiorite, fractured in 3 directions



Rock anchors installed:
1 70 VSL rock anchors of 1200 kN working force Length: 15.0-17.0 m Spacing in longitudinal direction: 6.0 m Spacing in transverse direction: 3.0 to 5.0 m 660 VSL rock anchors type 6-1 Length: 4.0 m These rock anchors were installed to provide local stabilisation between the large anchors.

Cavern for pumped storage scheme, Taloro, Italy

Client	ENEL, Ente Nazionale per l'Energia Elettrica, Venice
Engineer	ENEL, Ente Nazionale per l'Energia Elettrica, Venice Dr. G. Lombardi, Locarno
Contractor	Caldart S.p.A., Belluno
Anchors	VSL Italia s.rl., Milan
Years of construction	1975-1976
Dimensions of cavern	Height 40.5 m, Breadth 21 .0 m, Length 121.9 m
Type of rock	Granite



Securing of vault abutment:
46 VSL rock anchors of 1000 kN working force and lengths between 28 and 37 m. Spacing 3 m.
Securing of crane track:
92 VSL rock anchors of 1000 kN working force and 22 to 25 m length.
Securing of walls:
Approx. 360 VSL rock anchors of 1000 kN and 1500 kN working force and lengths between 6.5 and 34 m. Spacings: horizontally 3 m, vertically 2 m.

6. Anchoring of concentrated forces

6.1. General

The use of prestressed anchors for the anchoring of concentrated forces represents a very suitable method of solving problems of this type. The function of the anchors is to anchor parts of a structure or entire structures to the ground.Theclassical use of anchoring concentrated forces is found in foundation blocks which are subjected to large tensile forces. Examples are the anchoring of cables for cable railways, cable cranes, suspension bridges and tension structures, of penstocks, crane beam brackets, galleries etc.

6.2. Rock fall gallery on the Axenstrasse, Switzerland

Client Construction Department, Canton Schwyz
Engineer Franz Pfister, Ingenieurburo, Schwyz
Contractor Leimbacher, Lachen
Drilling Injectoboh rSA, Locarno
Anchors VSL INTERNATIONAL AG (formerly Spannbeton AG)
Years of construction 1968-1970

Introduction
The Axenstrasse, which leads from Brunnen along Lake Lucerne towards Altdorf, is subjected to heavy rock falls. The effects of erosion are particularly unpleasant here and the mechanical destruction of the rock due to snow, ice, wind, rain and vegetable growth leads to a risk of rock falls onto the main road. Rock cleaning teams check the critical rock faces every spring and if necessary at other times of the year and use crowbars or explosives to remove the looser parts of the material. Rock falls still occur, promoted by warm winds in the spring, and it is impossible to prevent them completely on the 500,000 m2 rock face.The Axenstrasse is situated throughout at about 435 m above sea level. It is bounded on the lake side by almost vertical rock faces. The highly fractured limestone rock walls continue upwards into wooded and overgrown slopes and finally, at about 800 m above sea level, change to grassland.

The problem
Observations overa numberofyears have shown that in general the falling stones are not larger than a man's head, i. e., 2050 kg in weight. Occasionally, blocks of larger weight do occur. The most suitable way of counteracting the rock falls effectively was to construct a rock fall gallery. Apart from the size and frequency of the falling stones, the distance of drop was of great importance for the design of this protective gallery. The maximum difference in levels between the Axenstrasse and the uppermost band of rock is about 350 m. However, since the steep parts of the face alternate with larger and smaller steep slopes, the actual heights of free fall are considerably less. The probability of stones dropping from heights exceeding 50 m directly onto the road is extremely small. The free fall is interrupted by impacts

Fig. 71: Partial view of the finished rock fall gallery

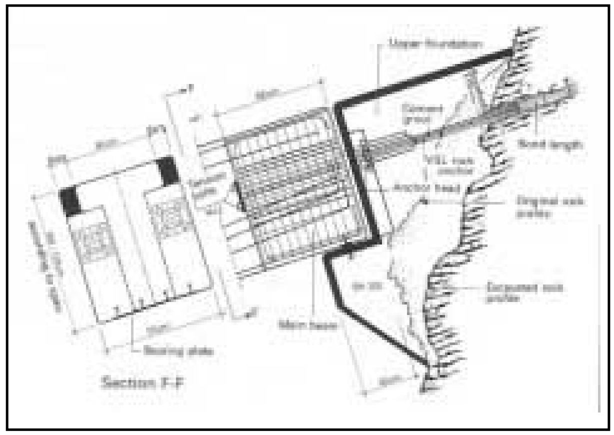


Fig 72: Upper anchorage for gallery

pacts on the way, and some of the disturbed stones plunge directly into the lake.

The Axenstrasse has been repeatedly widened and extended in recent decades, both towards the lake and towards the rock face. On the hill side there is the outcropping rock face, mostly artificially excavated, while on the lake side there is a fill embankment of broken rock material. On the lake side, the rock surface is encountered between 3 and 10 m below the mean water level. It falls steeply down to the lake. A foundation for the rock fall gallery on the lake side could there fore only have been found at great depth and at enormous expense. The existing walls of the shore were not capable of being used as foundations. For these reasons, a solution had to be found, by which the conventional system of rock fall galleries with supports on the lake side could be abandoned. This of necessity implied a less expensive and aesthetically better solution.

Choice of the type of structure

Two basic solutions were proposed:

-a gallery with a sloping roof and protective covering: the rocks would fall into the lake,

-a gallery with an approximately horizontal roof and protective covering in the form of an earth blanket: the fallen rocks would remain on the roof.

These two basic types were investigated thoroughly in regard to their structural design, construction aspects and costs. Since the pile foundations made up a considerable part of the total cost, solutions without pile foundations were also included in the study. The design studies carried out therefore have a fairly general character and can serve as a basis for other projects of this type. The variant which was developed (fig. 71) proved to be the most suitable on the basis of the studies carried out here.

Description of the design

Since the Axenstrasse must always be kept open to traffic, a design solution utilising prefabrication was essential. This would enable the work in the actual road region to be kept to a minimum. Moreover, it makes for easy replacement of any elements damaged by excessive rock falls, a rock falls, a problem which had already arisen. Apartfrom the erection work itself, it was only necessary to occupy the road when constructing the rock foundations on the hill side. For this purpose, the Axenstrasse had to be temporarily closed. Importance was attached from the start to keeping the design as light as possible. Reinforced concrete was chosen for the main supporting structure. It was decided to carry out full-scale fall tests with the cooperation of the Swiss Federal Materials Testing Institute to obtain the necessary design data. A suitable test installation with a 45 m free drop could be constructed at Brunnen. The rock material for the fall tests came from the same region, so that fairly good agreement with the actual conditions could be attained.

The main supporting structure is cantilevered out from the rock face and consists of upper main girders and lower struts (see fig. 71). The covering of this main support structure is of concrete panels with corrugated steel sheets on the lower side (lower reinforcement) and an upper layer of bitumen concrete. The theoretical spacing of the girders is 5.00 m.

The rock foundations are the only parts of the works which had to be constructed on site. The concrete was applied directly to the cleaned rock and, for the upper foundations, was pressed directly against the rock face by prestressed VSL rock anchors 5-12 of stressing force 1500 kN (see figs. 72, 73). The lower foundations were connected to the rock by dowels constructed from reinforcing steel.

After the rock foundations had been built, virtually all the remaining work consisted of erecting prefabricated elements.

The length of the rock fall gallery is more than 1300 m and the costs were of the order of 8 million SFr.

Fig. 73 Anchor work on an upper foundation

6.3. Spherical valve anchoring, under ground power station Waldeck II, Federal Republic of Germany

Client	Preussenelektra, Hanover
Engineer	Siemens AG, Erlangen
Contractor	Joint venture Cavern Waldeck I I Beton- and Monierbau AG, Frankfurt Baugesellschaft H. Rella & Co., Vienna Allg. Bauges. A. PorrAG, Vienna Dyckerhoff & Widmann KG, Wiesbaden
Drilling contractor	Terrasond Grundbau GmbH, Essen
Anchors	VSL INTERNATIONAL LTD, Berne
Year of construction	1972

As explained in Chaper 5.2, the cavern for the pumped storage scheme at Waldeck II was secured by rock anchors. This meant, however, that it was not possible to anchor the high thrust forces from the supply water line directly to the

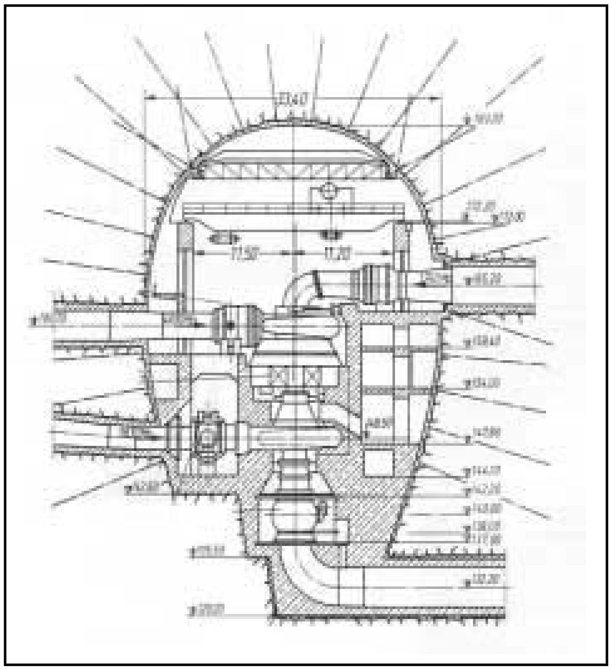


Fig. 74: Section through cavern

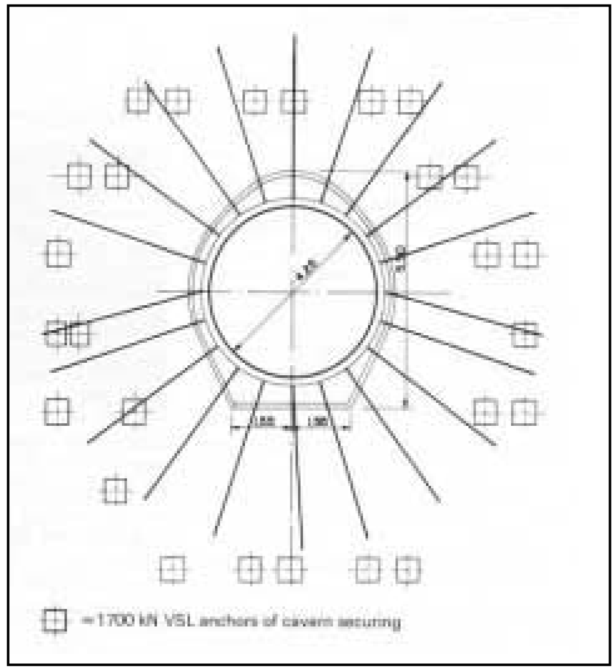


Fig. 75: Elevation

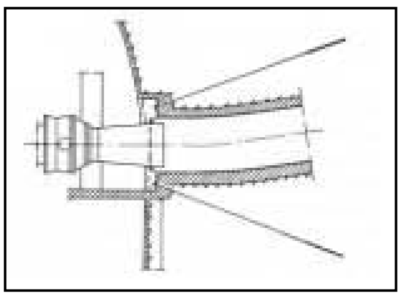


Fig. 76: Detailed section

- from friction of spherical valve on its seating 0.4 MN
- from stuffing box friction 0.6 MN

A 50% addition for dynamic forces had to be made, so the total force to be anchored was 17.4 MN (Fig. 74).

Each anchoragewas composed of 20VSL rock anchors of 960 kN working force for each spherical valve. (Fig. 75). These anchors are 12.5to 15m long (including a bond length of 3 m) and press the pipe collar against the rock face (fig. 76).

When fixing the position of the bores, account had to be taken of the pattern of thejoint planes in the rock and the already existing anchors for securing the cavern. After the pipe and collar had been installed, the 116 mm diameter bores were marked out and rotary drilled. All the holes were pressurised with water, but had to be grouted because the permeability was too high and then drilled out again. The programme of drilling and anchoring work lasted about six weeks without interruption to the continuing erection of plant.

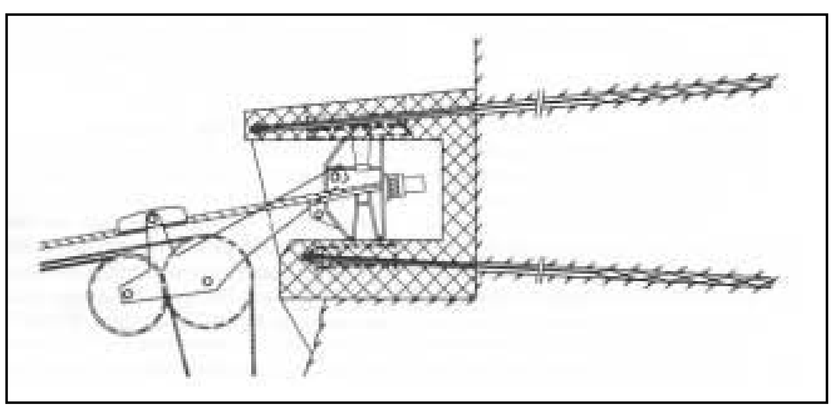
The anchors were stressed to 0.65 x ultimate load and after ten minutes were released to the permanent load of 0.5 x ultimate load. In the free length, the anchors were filled with a corrosion preventative grease, so that itwill be possible to adjust the stressing force at any time. Four anchors of each anchorage were equipped with a VSL load cell for later monitoring by the power station staff. These measurements can be obtained by hydraulic actuation from an instrument cabinet situated adjacent to each turbine discharge.

6.4. Cable crane anchorage at Jiroft Dam, Iran

Client Kerman Water and Power Authority
Engineer Office of Prof. Stucky, Lausanne
Contractor Partnership Lozan-Porr-Losinger, Tehran
Drilling contractor Solperse, Tehran
Anchors VSL INTERNATIONAL LTD, Berne
Year of construction 1977

In the building of the 133.5 m high arch dam at Jiroft (approx. 200 km north-east of Bandar Abbas), which has a crest length of 210 m and is situated in a deep narrow gorge, two cable cranes each of 200 kN capacity and 520 m length were used. The suspension cables were secured on the left side of the valley in a fixed point anchored in the limestone,while on the opposite side of the valley was the track on which the cranes could be moved horizontally.

The fixed point, a concrete structure, in which the two pivot assemblies were installed, had to be capable of withstanding a total tensile force of 6,400 kN. This force was anchored back to the rock with 8 VS L rock anchors type 5-12 of 17 to 22 m length (including 5 m bond length). The anchors, which had upward and downward inclinations respectively of 10%, had a working force of 1265 kN (60% of ultimate load).



7.Stability against overturning

7.1. General

The problem of preventing overturning arises in a great variety of structures, the common factor of all being that they are subjected to horizontal forces such as wind, water pressure, earth pressure, waves, ice pressure, earthquake forces and the like of sufficient intensity for their stability against overturning to be jeopardised unless special additional measures are taken. One possible way of solving this problem is to increase the weight and dimensions of the structure. This procedure is, however, frequently impossible or undesirable and moreover is not often the most economical. The stability can be provided at less cost by means of anchors, enabling material and construction time to be saved.

The structures where this type of problem arises include in particular slender, light structures such as towers, masts, pylons and also quay walls, which in many cases are vertically anchored, instead of being provided with broad and heavy footings. On the other hand, a large number of old dams or spillway structures, which were built in the first decades of this century, need to be strengthened. Most of them are indeed still in a generally satisfactory state, but they no longer satisfy modern safety requirements in regard to stability at exceptionally high water levels and earthquake movements.

An ideal means of strengthening such structures is the prestressed anchor. It requires little space, is adaptable, can be installed at short notice and rapidly, and no complicated site equipment is required. Moreover, the number of anchors required can be kept small because of the large forces which can be applied. In earthquake regions, the anchors increase the resistance to shearing, sliding and overturning.

7.2. Lighthouse at Kullagrund,Sweden

Client Swedish Navigation Department
Engineer Vattenbyggnadbyran (VBB), Stockholm
Drilling contractor Stabilator AB, Bromma
Anchors Internordisk Spannarmering AB, Stockholm
Year of construction 1975

Introduction

At the end of the fifties the Swedish Navigation Authorities drew up a comprehensive programme for the replacement of the majority of the lightships and buoys situated alwig the east coast of Sweden by stationary equipment. Since the weather conditions in this region are often bad, it was more economical to carry out the building and installation workasfaras possible on land and then to tow out the completed structure to the appointed position.

Fig. 77: The lighthouse at Kullagrund

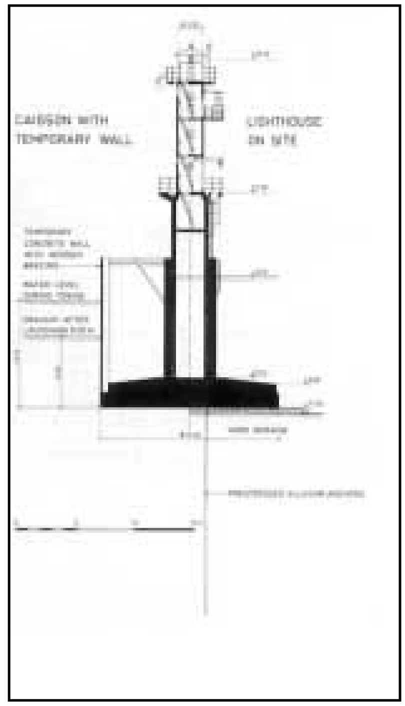


Fig. 78: Section through the lighthouse

The stability of the structure during use can be assured either by a suitably large and heavy foundation slab or by anchoring by means of soil or rock anchors. The method using anchors enables the design to be simplified and material to be saved by the reduction in size of the foundation slab. This will be illustrated below by the example of the lighthouse at Kullagrund. This lighthouse of medium size was built in winter in a protected bay in north Sweden and then towed for a distance of almost 800 nautical miles to its intended location about 10 km south-west of Trelleborg in the south of the country.

Asshown in Fig. 78, the structure consists of a circular foundation slab of 15 m diameter, on which a concrete cylinder of 2.40 m internal diameter stands. A cylindrical steel tower, 12 min height, is built on this concrete cylinder. A wall, erected around the perimeter of the foundation, was temporarily constructed to provide buoyancy during towing. At its permanent site, the tower stands in 11 m of water on a flat ballast bed grouted with cement mortar, which had been laid on the hard moraine of the sea bed.

The Stability of the Structure

The equilibrium of the lighthouse is a function of thetilting and sliding stability. In the case of the Kullagrund Tower, the critical force is that produced by the waves; for a wave height of 9 to 10 m, the horizontal force reaches 3,600 kN, and the lifting force 800 kN. The design was based upon the principle that the selfweight of the lighthouse (830 tonnes) affords stability with a safety factor of about 1.0 in the normal state, while extreme conditions an additional safety would be provided bysoil ancho--s. As will be seen below, this method resulted in considerable savings in the building costs and also complied extremelywell with the requirements laid down. In this case, the increase in tilting safety achieved with the anchors was relatively small by comparison with that which would have been possible using gravity (approximately equal to 3.0), but was quite sufficient, since a safety factor of 1.55 was obtained. By contrast, the anchors play a decisive role in stability against sliding, as can be seen from the following:

Coefficient of friction between concrete and ballast (estimated) m=0.45

Sliding stability without anchors:

$$m = \frac{H = 3600}{N = 8300 - 800} = 0.48 \rightarrow F = 0.94$$

Sliding stability with anchors:

Prestressing force after all losses > 4000 kN

$$m = \frac{3600}{8300 - 800 + 4000} = 0.48 \rightarrow F = 0.94$$

This value would have been difficult to achieve by gravity alone and without a considerable increase in the weight of the structure.

As a result of the use of anchors, the lighthouse is also capable of withstanding a force of 550 kN /m from ice pressure at the water surface, which is twice the value which in general would be expected.

The anchors

The lighthouse is anchored with 6 VSL permanent soil anchors. These are vertical and are uniformly distributed in the concrete cylinder wall. Each anchor has an ultimate strength of 1463 kN and consists of 7 strands of 0.5" dyform. To protect

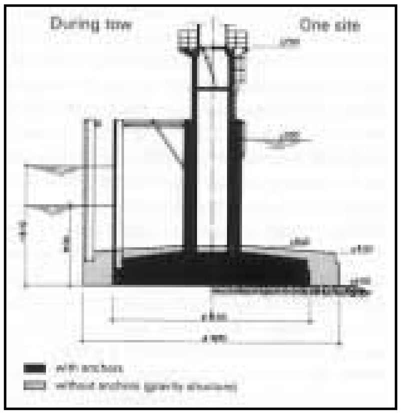


Fig. 79: Comparison of dimensions

against corrosion, the strands are greased and individually sheathed in a polyethylene duct. In the bond length, the grease and duct were removed and the strands were encased instead with epoxy resin, a corrugated polyethylene tube being used as «formwork» and as an additional protection.

Cost comparison

The construction costs for the lighthouse of Kullagrund amounted to 2.0 million Swedish crowns, of which 600,000.crowns were for the electrical equipment (including the connecting cable to the mainland). A cost comparison showed that a gravity structure to give a factor of safety of 1.3 against sliding under the action of 10 m high waves would have required a thicker foundation slab with its diameter increased to 19.50 m (Fig. 79), resulting in additional costs of approx. Swedish crowns 200,000.-.

By comparison with the as-built construction costs, which without equipment amounted to 1.4 million SKr, this would have meant a 14% addition.

Conclusion

A structure, for which the stability is attained by gravity alone, exerts a lower pressure on the soil and the resulting settlements are smaller. In addition, the stability when floating is better due to the larger diameter of foundation, and therefore higher towers could be built. The following points, however, are against the foregoing and in favour of anchoring:

- the size of the forces acting upon the structure depends to a great extent upon its geometry, so that if the dimensions are increased in orderto increase the weight, the force due to the waves also increases. The increase in weight then leads to larger loadings and these in turn lead to an increase in the dimensions.
- the overall stability of a lighthouse is determined predominantly by the friction of the foundation slab on the ground. Even if it is accepted that prestressing does not provide as high a factor of safety against overturning as gravity, nevertheless a structure secured by anchors has a considerably increased factor of safety against sliding.

7.3. Lalla Takerkoust Dam, Morocco

Client Ministry for Public Works and Communications of the Kingdom of Morocco
Engineer Elektro-Watt AG, Zurich/ Rabat
Contractor (incl. drilling and anchors) Joint venture Societe Generale des Travaux du Maroc/Intrafor-Cofor SA, Paris/VSL INTERNATIONAL LTD, Berne
Rock investigations Terrexpert AG, Berne
Years of construction 1978-1979

The Lalla Takerkoust Dam, situated 35 km south-west from Marrakesh, was built in the years 1929 to 1935. It is a gravity dam of concrete, originally 52 m high and 357 m long along the crest. Due to heavy silting of the reservoir, the stored volume decreased over the years from 53 to 34 million m3. In order to increase the capacity of the lake, therefore, it was decided to raise the dam by 9 m and to prestress and anchor the central region of the structure by means of rock anchors (Figs. 80, 81). The 54 vertical anchors required are of VSL types 6-40 to 6-48 with lengths varying between 63 and 114m, including 10to 12 m bond length. They have ultimate capacities of 10.59 to 12.71 MN and working forces of 5.65 to 6.78 MN, i. e. the anchors used here are amongst the largest ever employed for such a task. The spacing between anchors variesfrom 2to 4 m, and is usually 3 m. To permit surveillance of the behaviour of the anchors over a fairly long period, 5 of them are equipped with VSL load cells type G 850.

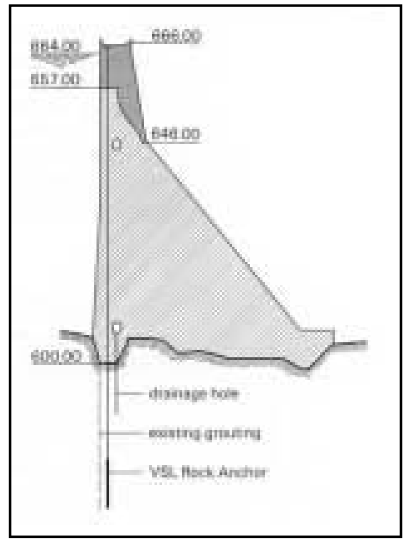


Fig. 81: Section through the raised dam

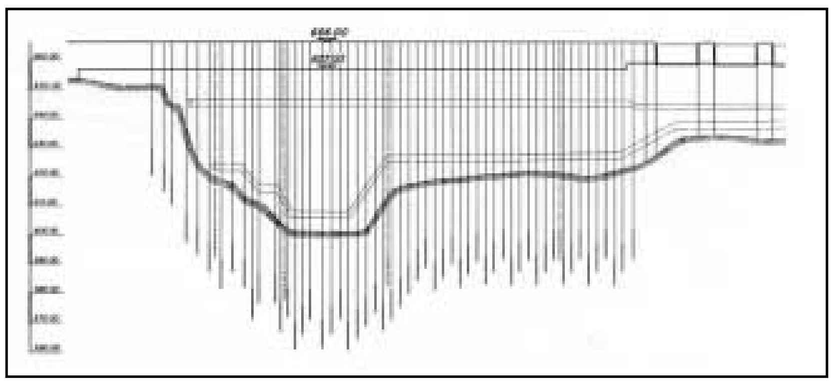


Fig. 80: Arrangement of rock anchors

7.4. Milton Lake Dam, Ohio, USA

Client The town of Youngstown, Ohio
Engineer E. D'Appolonia, Consulting Engineers, Pennsylvania
Contractor Mergentime Corporation, Flemington, New Jersey
Anchors VSL Corporation, Springfield, Viriginia
Year of construction 1975

Introduction

Milton Lake Dam is a 15.2 m high earth dam, which dams the Mahning River, Ohio; it was built in 1913 to 1917. Like many other small and medium-sized dams, which were constructed at the beginning of the twentieth century, it was rapidly approaching the end of its useful life after more than fifty years service. This became clearly apparent when inspectors from the state of Ohio and the Federal Administration discovered near the West abutment a depression on the upstream side of the earth dam and settlement of the fill behind the wing walls.

Shortly after this discovery, the lake was lowered by 4.6 m, and the town of You ngstown commissioned E. D'Appolonia Consulting Engineers of Pennsylvania to carry out a careful investigation of the safety of the dam and to make proposals to the town for improvement works.

D'Appolonia established that the spillway was just stable in the best case at the normal height of the lake, but would be in an almost critical condition if 1 m of water flowed over the discharge. It was therefore a vital matter to repair the spillway without delay.

The repair measures

Due to financial restrictions, the work was divided into four stages (one per year). The most critical part of the repair was considered to be the stabilising of the spillway. D'Appolonia recommended for this purpose the use of large rock anchors, which would be anchored in the sound rock 12.20 m below the foot of the dam (Fig. 82).

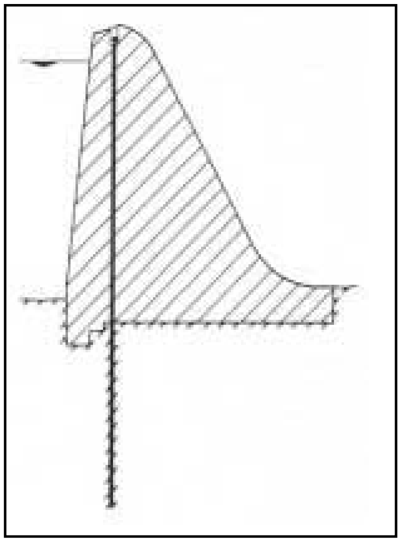


Fig. 82: Section through the spillway

Although in earlier years large rock anchors had frequently been installed, difficult and unusual problems arose with the Milton Dam.

Whereas the state of the dam had gradually deteriorated over many years, only a very short period was available for the installing of the anchors, on account of the climatic conditions. In addition, the access facilities for the spillway were limited and the crest of the dam, only 2.42.7 m wide, made the erection of drilling and installation equipment on the dam impossible. Howthen should the 59 holes of diameter 165 mm be drilled through theconcreteofthespillwayintothesound grey sandstone below the dam? The contractor decided to erect the drilling equipment on a pontoon, while various possibilities for the installation and stressing of the anchors were investigated. After all possible alternatives had been considered, it was decided to use a narrow gauge rail track running the entire length of the spillway for handling the stressing equipment. A helicopter was chosen for bringing up the rock anchors from the adjacent assembly area and lowering them into the boreholes in the spillway.

The execution of the anchor work

AT-shaped barge, equipped with a downthe-hole drilling unit, was brought to the positions where the holes had to be bored. The drilling work was carried out almost continuously, working day and night in two 12-hour shifts. 6 holes were drilled on average per shift. All the holes had to be subjected to a water pressure test in the anchorage zone. The maximum permissible water loss was specified as 0.005 l / min per 10mm diameter and perm depth, in a period of 10 minutes at a pressure of 0.345 bar. The effect of this very stringent condition was that all 59 holes had to be grouted and then redrilled. For 11 holes indeed, this operation had to be repeated, in order to meet the specified conditions. After the boreholes had been drilled, holes had to be drilled at the top of the spillway as recesses for the anchorages. During the last week of drilling, the 59 VSL rock anchors were assembled in the vicinity of the spillway. Each anchor (ultimate strength 4783 kN) contains 26 strands of 13 mm dia, with lengths of 29.30 to 34.80 m and was fabricated to the exact borehole length as ascertained by plumbing and was marked to identify it. One week afterwards, all the anchors were transported by the helicopter from the place of assembly to the dam and lowered into the holes.

At the day of installing the anchors, the helicopter hovered at 09.00 hours above the assembly area in order to hook on the first rock anchor. A choker was attached to the lead line of the suspension cable on the underside of the helicopter and a short time afterwards the first rock anchor was in the air and commenced its 300 m travel to the team waiting on the dam. Two

Fig. 83: Positioning of the anchors

operatives guided the anchor into the hole while the helicopter descended (fig. 83). While the helicopter continued to hover, the choker was unhitched by the team. Four minutes later, the helicopter returned with the second anchor. In spite of a bad thunderstorm lasting two hours and causing a corresponding period of interruption to the work, all the rock anchors had been positioned by 14.30 hours; the operation was therefore completed 5% hours after arrival of the helicopter. If a derrick or a crane in the stilling basin had been used, the work would have taken a month or more. After all the rock anchors had been installed, grout was introduced from the lower end of the borehole through a tube inserted down the centre of the anchor and injected to a height of 8.80 m. The specification required that this grout should reach a cube strength of 27.6 N/ mm z after 28 days. By using a suitable mix and a high-speed mixer, the 28 day strength was reached after only 4 to 5 days.

One week after the grouting of the bond length, each anchor was stressed to an initial force of 3273 kN, i. e. to 70% of the ultimate strength. Two weeks afterwards, a lift-off test was carried out. If the difference between this force and the initial force was less than 5% (neglecting relaxation losses), the anchor was secondary grouted with mortar within 48 hours. Six anchors exhibited a loss of force in excess of 5%. These were restressed to the initial force, again tested after two weeks and then also grouted.

Five months after the award of the contract by the City of Youngstown, the last anchor was installed, i.e., three months earlier than planned.

7.5. Laing Dam, South Africa

Client	East London Municipality, Department of Water Affairs
Engineer	Ninham Shand and Partners Inc., Cape Town
Contractor	LTA Construction (E. Cape) Ltd., Port Elizabeth
Drilling and Anchor contractor	Ground Engineering Ltd., Johannesburg
Years of construction	1975-1977

Introduction

The Laing Dam is situated on the Buffalo River approximately 40 km west of East London and 15 km south-east of King Williams Town and is used to supply the King Williams Town region with water. In August 1970, over a period of six days, more rain fell in the King Williams Town area than the annual average, turning the Buffalo River into a torrent. At the Laing Dam, the water flowed over the spillway to a height of 5,18 metres (exceeding the original design discharge by 80%). There was some fear forthestructure's stability, and it was felt that after the flood had subsided, the structure

might not be completely safe.
Thedam isa mass concrete gravity dam of slender section, designed before, and built just after, the Second World War using current parameters and design criteria for the probable maximum flood of 3,6 m (1750 cumecs) over the spillway. Afterthe 1970flood, extensive investigations of the hydrology of the Buffalo River catchment area with respect to the Laing Dam were carried out using the present day flood assessment techniques and these indicated that the Probable Maximum Flood (PMF) was of the order of 8200 cumecs (6,6 m over the spillway). While the Standard Project Flood was 4100 cumecs. Clearly the dam had to be modified.

Design concept

Taking into account such factors as overturning stability, resistance to sliding etc. for the revised PMF the Engineers conducted feasibility studies on various alternative proposals. Taking into account the problems and costs associated with each alternative it was decided that the vertical post-stressing technique would be most suitable. This involved inserting poststressed anchors vertically through the wall into the foundation rock. A total number of 131 anchors were required, which varied in initial working capacity from 4800 to 6000 kN. The anchors were located 1 m from the upstream face and at centres ranging from 1,0 m to 5,4 m depending on the height of the dam wall. The depth of the anchors varied from 14to 63 m and had to penetrate at least 12 m into the underlying dolerite bedrock. In order to ensure satisfactory spacing of the anchors, accurate drilling was specified.
The top anchorages were to be placed on a newly constructed reinforced concrete distribution beam.

Anchor design

The criteria for the anchors which governed the successful tender's thinking were:

- Maximum initial working force of 6000 kN
- A «multi-pull» single jacking operation
- Size of drill-hole limited
- Reasonable amount of flexibility for installation purposes
- A large hollow central core to be left down the entire length to improve the efficiency of grouting.

With this in mind it was decided that to cater for the above an anchor consisting of 36 0 15.2 mm strands (ultimate strength 9000 kN) was required. For the top anchorage the VSL system was chosen, with a specially designed anchor head to cater for the geometry and also the hollow core through which flushjointed fluted pipes could be inserted for grouting.

Initial work

The specification called for five test anchors to be installed close to the dam wall in order to assess:

- The efficiency of the drilling
- Grouting and stressing techniques
- The bond length
- Prestressing losses

Pull-out tests were carried out on two of these anchors using bond lengths of 2.8 m and 3.3 metres. Based on these results it was decided to use an 8 m bond length on all anchors in order to achieve a minimum factor of safety of at least 2.5. The remaining three anchors were used to monitor the relaxation loss in the steel and the creep of the bedrock. From this data the long-term losses of the prestressing were found to be approximately 12% (normal relaxation strand) for a cable stressed to 70% of its ultimate capacity.

Drilling

For drilling the 175 mm diameter boreholes, two custom built drill rigs of small size required to fit on the dam wall and sufficient power to reach the depths were constructed. The drills using down-thehole hammers were compact and set up on bogies to run on track fixed in position on the wall (fig. 84).
The tolerance on verticality of holes specified was 1:150. Over 131 holes, the average verticality measured was 1:500 from plumb, and three holes were found to be absolutely vertical. To obtain this accuracy, very great care was exercised when starting holes, especially over the top 6 metres.
After drillholes had been pressuregrouted they were redrilled and then scrubbed using a rotary brush, after which they were tested for watertightness. Test criteria were:
With an applied pressure of 5 bars the hole should allow outflow of not more than 0,05 litres/metre of hole/minute. (This specification was tightened up from the original allowable of 0,1 litre/m/ minute.)
If a hole failed, it was regrouted, redrilled and rebrushed until it passed. Passed holes were then flushed three times with a solution of caustic soda and water and blown out to remove dirt and oil traces, then left full of water.
Drilling operations continued for 3 months working 24 hours per day, 7 days per week.

Fig. 84: Drilling operation

Fig. 85: Anchor installation

Anchor assembly and installation

The anchors were assembled in a weatherproof enclosure using a system of hollow ferrule spacers and steel binders. Over the lower 8 m bond length the anchor was seized at 1,5 m centres and a special steel shoe was fixed to the end of the anchor. Just prior to the installation of the anchor into the borehole the lower portion, which formed the bond length, was thoroughly scrubbed with detergent to remove the soluble oil applied as a temporary corrosion protection to ensure that proper bond would be developed between the anchor and the anchorage grout.
The anchors were removed from the assembly shed by means of an overhead blondin and inserted directly into the boreholes (Fig. 85). After allowing the anchor to hang freely for 24 hours a 25 mm grout pipe was lowered through the ferrules down to the bottom of the hole. The anchors were then bonded to the bedrock by injecting grout under water. On completion of this grouting operation the grout pipe was withdrawn using a vibrator.
Extensive tests were carried out in order to design grouts which had optimum strength, density, shrinkage and bleed properties. An anchorage grout with a minimum 9-day strength of 25 N /mmz and a maximum bleed of 1 % was used in the bond length of the holes while a second stage protection grout with a minimum 28-day strength of 20 N /mm' and virtually no bleed was used to grout the remainder of the anchor once the stressing operations had been completed.

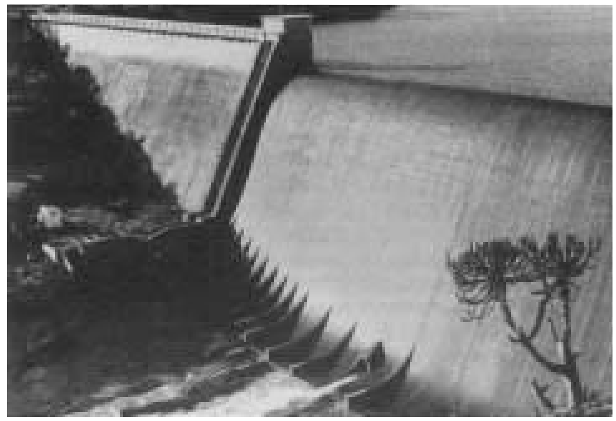


Fig. 87: View of anchor tops

Stressing
After the bond grout had attained the required strength the anchors were

Fig. 86: Stressing operation

type stressed to 70% of ultimate load (Fig. 86). The initial stressing operation was carried out in three phases in order to determine the friction losses down the hole, the wedge draw-ins, and to confirm the « lock off» loads. On completion of the initial post-tensioning, the residual load in each anchor was checked at 24 hours in order to determine whether the loss of posttensioning was within the margins specified and whether the anchorages were acceptable. The post-tensioning loads were then adjusted to give a final stress of 60% of ultimate under working loads. A further check was carried out 3 days later when the residual loads had to be within 2% of the corresponding value on the master relaxation curve. Allowance had been made for additional residual checks to be carried out at 6, 9 and 12 days in the event of the earlier readings falling outside the specified limits. All the anchors installed satisfied the 3-day acceptance criteria and the final stage grouting operations were normally undertaken 4 days after initially stressing. Before the final stage grouting was undertaken the boreholes were flushed with detergent to remove the soluble oil from the free length portion of the cables. After the anchors had been successfully stressed and grouted, the anchor blocks and distribution beam were incorporated into the reconstructed profile of the spillway and the non-overflow sections.
The installation of the prestressed rock anchors was carried out over a period of 7 months.
In terms of raising and strengthening of dams, the Laing Dam contract was by no means unique by way of theories and techniques used. What was unusual was the loading used in the stressing and thus the size of anchors required and the difficulties of handling such large material and equipment in the limited space available.

7.6. Center Hill Dam, Tennessee, USA

Client U.S. Army Corps of Engineers
Engineer U.S. Army Corps of Engineers
Anchors VSL Corporation, Springfield, Virginia
Year of construction 1973

The Center Hill Dam is situated on the Caney Fork River in the vicinity of Nash ville, Tennessee. It is a combination o earth dam and concrete gravity dam and has a total length of 660m. The maximum height is 73 m. The reservoir has a capaci ty of 2. 58 x 109 m 3 .
The problem which had arisen at the Center Hill Dam was not that of the stability against overturning, but a con structional one. During construction o the concrete portion, a number of horizontal concrete joints had been neces sary. One of these joints had been the cause of leakage problems right from the start. To rectify this, the Corps, of En gineers decided to install VSLanchors, by which the open joint would be closed by compression. Twelve rock anchors of type 5-23 (working force 2960 kN, ultimate strength 4228 kN) with an average anchor length of 16.76 m were used for this purpose.

8. References and Bibliography

8.1. References

8.1.1. Securing of slopes
Buro, M. «Rock anchoring at Libby Dam» Western Construction, March 1972, pp 42, 48 & 66
Lombardi, G. ‚Consolidation de l'eperon rocheux de Baji-Krachen». Publication de la Societe Suisse des Sols et des Roches No. 91, session de printemps 1975.
Schyder, R. & Brugman B.J. «Hangsicherung and Südportal des Schallbergtunnels». Mitteilung der Schweizerischen Gesellschaft für Boden- und Felsmechanik Nr. 91. Fruhjahrstagung 1975.
«Tarbela Dam, Pakistan». VSLJ ob Report S 7, August 1974.

8.1.2. Anchoring of retaining walls
Betschen, G. «Autoroute du Leman N9». Route et Trafic No. 7, juillet 1972, pp. 350-356.
«Nationalstrasse N8 in Obwalden, Bauabschnitt Alpnachstad-Kantonsgrenze Nidwalden» Mitteilung des Institutes für Grundbau and Bodenmechanik Nr. 106, ETH Zurich, Sept. 1976.
«Obere Stutzmauer Delk, Obwalden» Baustellenbericht der Firma Spannbeton AG, Lyssach, Schweiz.
Roud, M. « La construction de la double vole le long du lac de Bienne». Bulletin technique de la Suisse romance No. 3, fevrier 1976, p. 54-59.
Schmid, P., Ardüser, H.P. & Hugentobler, 0. «Naxbergtunnel mit Bohrpfahlwand fur Tunnel im Tagbau». Strasse and Verkehr Nr. 3, März 1974, Seiten 119- 124.
Seltenhammer U. «Ankermauer an der BrennerAutobahn». Österreichische Ingenieur-Zeitschrift, Heft 6, 1968.

8.1.3. Securing of excavations
«Concrete walings speed Swedish Metro project». World Construction, New York, Dec. 1974.
Corbett, B.O. & Stroud M.A. -Temporary retaining wall constructed by Berlinoise system at Centre Beaubourg, Paris». Institution of Civil Engineers, Proceedings of the Conference on Diaphragm Walls and Anchorages, London, 1975, Paper No. 13.
EderP. &Rummeli H. «Die erste vorfabrizierte Schlitzwand in der Schweiz, Schweizerische Bauzeitung, Heft 28, Juli 1973.
Lundahl, B. & Sjokvist, K. «Djup grundlaggning med komplikationer». Vag-och vattenbyggaren nr. 5, 1972, Stockholm.
O'Neill, M. « Prestressed Rock and Soils Anchors - A new tool». Asian Building & Construction, January 1977.

8.1.4. Anchoring against hydrostatic uplift
Institut fur Grundbau and Bodenmechanik, Technische Universitdt Munchen, «Eignungsprufung an drei Dauerankern für die Auftriebssicherung des Regenuberlaufbeckens in Ellwangen». Prufbericht Nr. 7582 B/3 vom 14. 12. 1976.
Sommer, P. & Graber, F. «Felsanker zur Sicherung des Tosbeckens N r. 3 in Tarbela (Pakistan) ». Schweizerische Bauzeitung, Vorabdruck 1978.

8.1.5. Securing of caverns
Abraham, K.H. &Porzig. R. « Die Felsankerdes Pumpspeicherwerkes Waldeck II». Baumaschine + Bautechnik, Heft 6 and 7, Juni 1973, Seiten 209-220 und 273-285.
Abraham, K.H. & Pahl, A. ‚Bauwerksbeobachtung der grossen Untertageraume des Pumpspeicherwerks werks Waldeck II». Die Bautechnik 5/1976, Seiten 145-155.
Aeschlimann, U., Herrenknecht M. & Bauholzer, H. «Das Baulos Huttegg des Seelisbergtunnels». Schweizerische Bauzeitung, Heft 6, Februar 1977.
Buro. M. « Prestressed rock anchors and shotcrete for large underground powerhouse». American Society of Civil Engineers, Civil Engineering, May 1970.
Letsch U. «Seelisberg Tunnel: Huttegg Ventilation Chamber» Proceeding of the International Symposium on Field Measurements in Rock Mechanics, Federal Institute of Technology, Zurich 1977, pp.577-586.
Lombardi, G. « Der Einfluss der Felseigenschaften auf die Stabilitatvon Hohlraumen ». Schweizerische Bauzeitung, Heft 3, Januar 1969.
Moeschler, E. & Matt, P. «Felsanker and Kraftmessanlage in der Kaverne Waldeck II». Schweizerische Bauzeitung, Heft 31, August 1972.
«Pumped storage scheme Taloro, Sardinia, Italy». VSL Job Report S 16, August 1977.
Rescher, O.J. «Amenagement Hongrin-Leman, Soutenement de la centrale en caverne de Veytaux par tirantsen rocheretbeton projete». Bulletin technique de la Suisse romande No. 18, sept. 1968.

8.1.6. Anchoring of concentrated forces
Abraham, K.H., Gloggler, W., Pahl A. & Sprado, K-H. «Die EinfiOhrungen der Triebwasserleitung in die Maschinenkaverne des Pumpspeicherwerks Waldeck II». Die Bautechnik 4/1976, Seiten 131-138.
Abraham, K. H. «Construction progress at Waldeck II plant», Water Power, December 1973, pp. 464-466.
Gasser, J. «Steinschlag-Galerie an derAxenstrasse». Planen and Bauen, Zurich, Juli 1969.

8.1.7. Stability against overturning
Emstsons, E. ‚Recent Lighthouse Construction in Sweden». Proceedings of the IALA Lighthouse Congress, Ottawa, Canada, 1975.
Friedrich, R. -Extending the life of an old dam using rock anchors». International Water Power & Dam Construction, February 1976.
Thompson, C.J. «Laing Dam, East London». Concrete, Journal of the Concrete Society of Southern Africa No. 9, March 1978, pp. 20-22.

8.2. Bibliography
Bishop, A. W. « Fhe use of slip circle in the stability analysis of slopes». Geotechnique Vol V (1955).
Bureau Securitas. ‚Recommandationsconcernantla conception, le calcul, ('execution et le controle des ti rants d'ancrage». Editions Eyrolles, Paris-Ve, 1972.
Cambefort, H. «Parois de soutenement maintenues par une ligne d'ancrages». Annales de I'Institut technique du batiment et des travaux publics no. 333, nov. 1975, pp. 25-44.
Comte, Ch. ‚Technologie des tirants». Institute for Engineering Research, Foundation Kollbrunner/ Rodio, Verlag Leemann Zurich, 1971.
Deutsche Normen, DIN 4125, Blatt 1. «Verpressanker für vorbbergehende Zwecke inn Lockergestein». Judi 1972.
Deutsche Normen, DIN 4125, 4125,Teil 2. « Verpressanker für dauemde Verankerungen (Daueranker) inn Lokergestein». Februar 1976.
FIP «Prestressed Concrete Foundations and Ground Anchors» Papers presented to the 7th FI P Congress, New York, 1974.
FIP «Guide to Good Practice, Practical Construction, (1975) Chapter R4.
Huber, J. «Wirtschafthche Stutzwande im Grund-, Erd- and Strassenbau». Schweizerische Bauzeitung, Heft 30/31, August 1977.
Huder, J. «Sicherheitsfaktor fur dine geradlinige Boschung gegen Rutschen». Die Bautechnik 12/ 1977.
Institution of Civil Engineers. Proceedings of the Conference on Diaphragm Walls & Anchorages, London 1975.
Jaeger, C. «Assessing problems of underground structures». Water Power & Dam Construction, December 1975 pp 443-450, January 1976 pp 2936.
Keute, H. & Schusser V «Bemessungshilfen and Ankerlangenbestimmung für Baugrubensicherungen mit Hilfe von Diagrammen». Die Bautechnik 5/ 1974, Seiten 155-160.
Kovari, K. & FritzP. «Stability Analysis of Rock Slopes for Plane and Wedge Failure with the Aid of a Programmable Pocket Calculator». Proceedings of the 16th Symposium on Rock Mechanics, Sept. 1975, Minnesota, USA.
Kovari, K. «Design Methodsfor Underground Structures. Proceedings International Symposium on Underground Openings, Lucerne, September 1972.
Lambe T.W & Whitman R.V «Soilmechanics..John Wiley & Sons, New York, 1969.
Littlejohn, G.S. and *Bruce*, D.A. , «Rock Anchors State of the Art». Foundation Publications Ltd. 7, Ongar Road, Brentwood, Essex, CM 15 9AU, England, 1977.
Logeais, L. & Graux, D. ‚Les tirants d'ancrage Conception et contrdle». Annales de I'Institut technique du batiment et des travaux publics no. 312, decembre 1973, pp 125-146.
Lombardi, G. «Der Einfluss der Felseigenschaften auf die Stabilität von Hohlräumen». Schweizerische Bauzeitung, Heft 3, Januar 1969.
VSL INTERNATIONAL LTD «VSL Post-tensioning». Technical Brochure, Berne, Switzerland.
VSL INTERNATIONAL LTD reVSL Soil and Rock Anchors.. Technical Brochure, Berne, Switzerland.
Ostermayer, H. «Construction, carrying behaviour and creep characteristicsofgroundanchors». Institution of Civil Engineers, Proceedings of the Conference on Diaphragm Walls and Anchorages, London, 1975, Paper No. 18.
Österreichisches Normungsinstitut, ÖNORM B 4455, Entwurf 1977. «Erd- and Grundbau, Vorgespannte Ankerfur Lockergestein and Festgestein».
Portier, J.L. «Protection of Tiebacks Against Corrosion» . Proceedings Tech. Session on Prestressed Concrete Foundations and Ground Anchors, 7th HIP Congress, New York, 1974, pp 39-53.
Prestressed Concrete Institute (PCI) «Tentative Recommendations for Prestressed Rock and Soil Anchors». Chicago USA, 1974.
Proceedings of the International Symposium on Field Measurements in Rock Mechanics, Zurich 1977, Vol I and II, Federal Institute of Technology, Zürich.
Schweizerischer lngenieur- and Architekten Verein (SIA), Norm 191 (Ausgabe 1977). «Boden- and Felsanker»/«Tirants d'ancrage».
Schweizerische Gesellschaft für Boden- and Felsmechanik. ‚Aktuelle Berechnungsmethoden in der Praxis des Grundbaues/Methodes de calculs modernes en géotechnique». Mitteilung No. 94, ETH Zurich, November 1976.
Terzaghi K. «Theoreticalsoil mechanics». John Wiley & Sons, New York, 1943.
TerzaghiK. &Beck,R.B. «Soil mechanicsinengineering practice» John Wiley & Sons, New York 1967.

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