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Beckenried Viaduct: Foundation Problems in a Creeping Slope

By

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Summary

The 3150 m long Beckenried viaduct traverses an unstable creeping slope, the surface of the sound rock lying at depths from 10 to 60 m. Altogether 44 of the total 58 piers of the bridge founded on the sound rock had to be protected by shafts against the creeping soil and loosened rock layers. Based on geological explorations and previous monitoring of the slope displacements the clearance between the rectangular concrete piers and the elliptical shafts was specified to be 1.5 m in the dip direction of the slope and up to ±1.0 m in lateral direction. In order to avoid failure of the shafts due to the unusually large slope movements likely to occur during the service life of the viaduct, an entirely new design concept was applied. It involves flexible shafts consisting of four main parts, namely a rigid shaft collar, articulated ring elements, a rigid trapezoidal cylinder and basal displacement rings resting on the pier footing. Fundamentally, this design permits both: sliding of the shaft as a whole along particular slip surfaces in the loosened rock at depth but also differential displacements due to the creep of the soil strata above it. The sliding at the base contributes to 50% and more to the total surface movements. The shafts also serve as a permanent drainage system for the highly water sensitive slope material. The paper describes the criteria upon which the novel design of the individual shafts were made and also particular constructional details. After a construction period of only four years the bridge opened to traffic in December 1980. Observations made on the completed structure indicate a behaviour of the slope and the shafts which is well in accordance with the predictions.

1. Introduction

With a length of 3150 m the Beckenried Viaduct is one of the longest bridges of the Swiss Federal Highway System. It belongs to the main North-South Route connecting the cities Basle and Chiasso. The bridge is located in a scenic area (Fig. 1) on the shore of the Lake of Lucerne in Central Switzerland. It traverses a slope striking in east-west direction and having dip angles between 10^0 and 40^0. The superstructure of the viaduct rests on 58 hollow piers of rectangular shape founded on the sound bedrock. In the western part of approx. 2100 m length the rock consists of Flysch-schists and in the remaining 1000 m long eastern part it is made up of cretaceous limestones. The rock surface, which runs at extremely variable depths (10 to 60 m), was formed by glacial erosion in the different periods of glaciation. Local rivulet erosions contributed to its uneven shape. In both rock types the sound rock is covered by a structurally disturbed rock zone which is referred to as "loosened rock". It has a variable thickness ranging from 1 to 10 metres. The structure of the soil cover resting on the rock is extremely

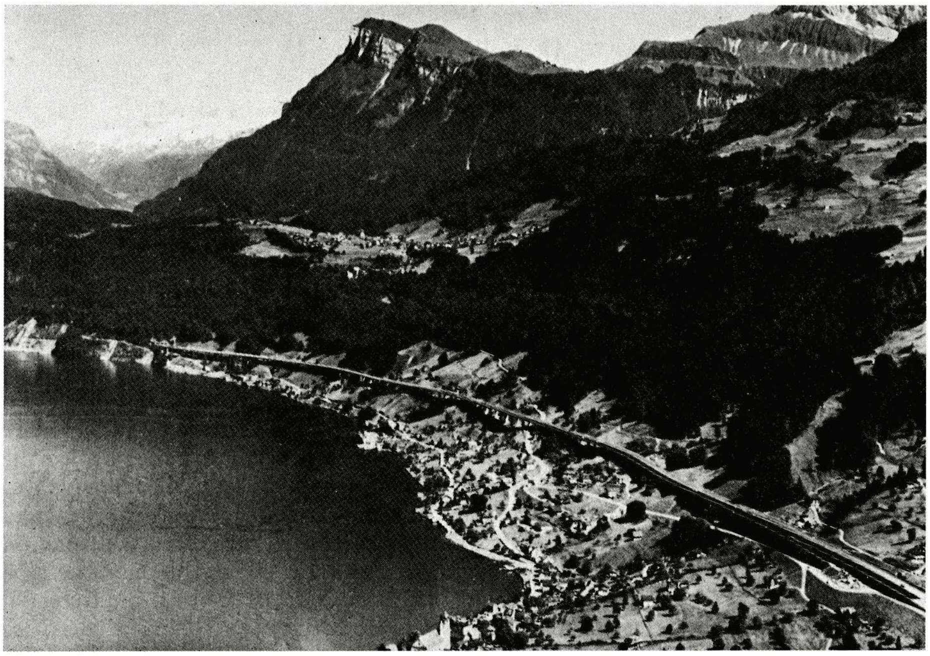


Fig. 1. Aerial view of Beckenried viaduct

heterogeneous. This is well illustrated by the west part of the longitudinal geological section (Fig. 2). The soils are morainic consisting of clayey/silty/ sandy gravels with a highly varying content of stones and boulders. The moraines are interspersed more or less erratically by colluvial deposits originating from weathered products of the underlying Flysch-schists. They consist of sandy gravels with very high clay and silt content. The alluvial zones exhibit a composition similar to that of the colluvials but they generally display a lower bulk density coupled with higher gravel content. In the east section the soil cover is relatively thin, rarely exceeding 10 m. It consists mainly of moraines. The geological conditions were discussed in detail elsewhere (Schneider, 1977). In the following only the 2100 m long west section of the viaduct resting on 44 piers will be dealt with. Three main geological factors were decisive for the design of the bridge foundation:

i. the risk of near surface slips in all areas of the hill with steep slopes,
ii. the evidence of large scale creep of the soil cover without the formation of a continuous slip surface,
iii. the evidence of additional creep displacements taking place on distinct sliding surfaces ("basal displacement") at the interface between soil and rock.

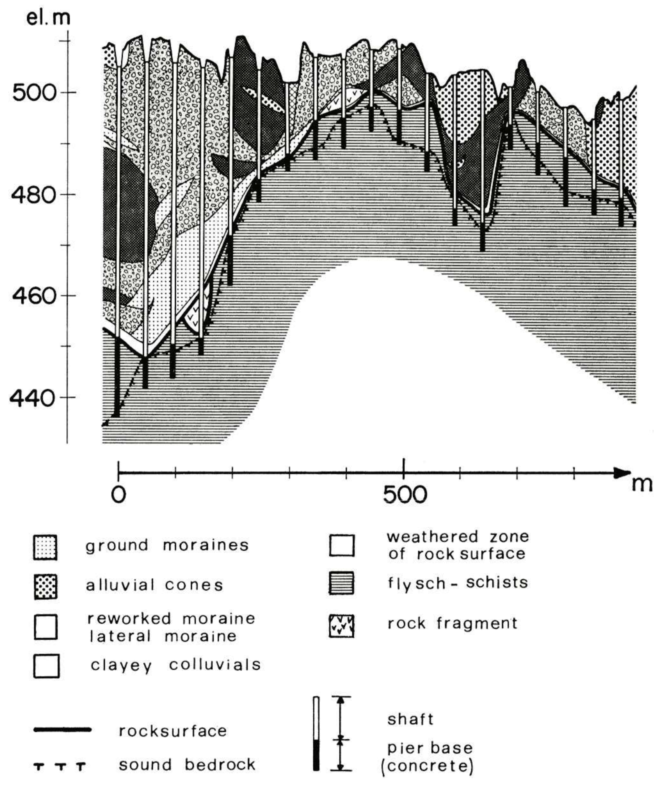


Fig. 2. Geological profile — western part of the construction site

This situation is illustrated in Fig. 3 with a simplified linear displacement distribution in the soil. Many recent slope failures have been observed in areas of steep slopes irrespective of the nature of the local soil. The depth of potential failure surfaces affecting the bridge foundation was assumed to be between 2 and 6 m. The large scale creep of the soil cover and the basal displacements have been revealed both by surveying and by borehole inclinometer measurements. Such observations were performed well in advance of construction. The displacements measured on the slope surface showed rates of movement between 10 and 40 mm per year. For example from the inclinometer measurements (Fig. 4) it becomes obvious that the basal displacements account for 40 to 50 % of the movements observed on the surface of the slope. It had to be assumed that in some areas only basal displacements occur. The main reason for these basal displacements is the presence of products of weathering from the loosened Flysch-schists.

Generally it was not possible to identify the exact position of the slip surface merely from the results of geological explorations without carrying out inclinometer measurements. The uncertainty with respect to the thickness of the zones of basal displacements varied between 0.5 and 4.0 m. The

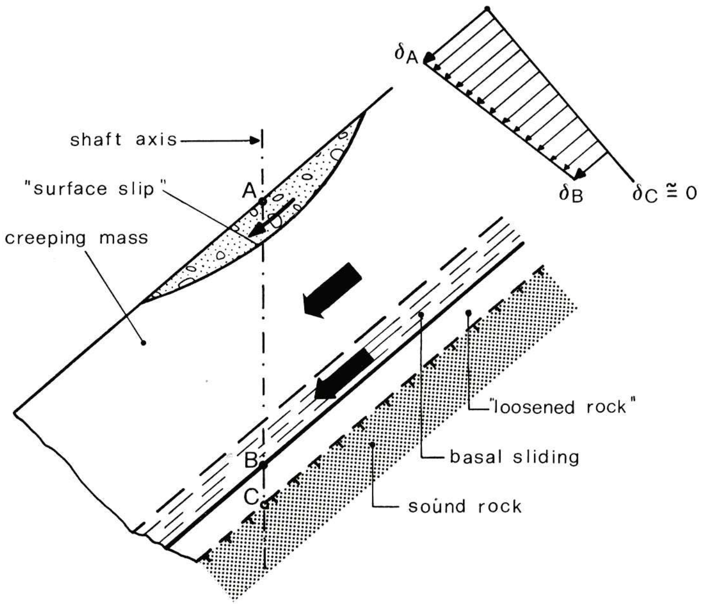


Fig. 3. Typical geological cross-section

simultaneous occurrence of mass creep and creep on a distinct slip surface at depth is clearly demonstrated by the results of the inclinometer measurements shown in Fig. 4.

From these findings it became clear that the bridge had to be founded on the sound rock through lined shafts, resulting in piers of up to 60 m length. The fundamental question then arose as to how to design the shafts considering the severe conditions discussed above. The idea of rigid shafts had to be abandoned at a very early stage of the investigations. Due to the large depth of a great number of shafts and the anticipated creep pressures rigid shafts would inevitably have failed in an uncontrolled manner. Therefore, a solution with flexible or "yielding" shaft construction was sought for. Such a construction would be rigid only in the near surface area in order to withstand possible local slope failures or "surface slips". The section below would, however, be allowed to deform with the surrounding ground without failure of the concrete segments. In the loosened rock zone exhibiting no movements the shaft would simply be filled by concrete to transfer the pier-load to the sound rock.

An additional purpose of the shafts involved the effective drainage of the slope during the entire service life of the bridge to increase safety and diminish creep. In the following the design criteria, the structural details and the construction of such flexible shafts will be discussed in detail.

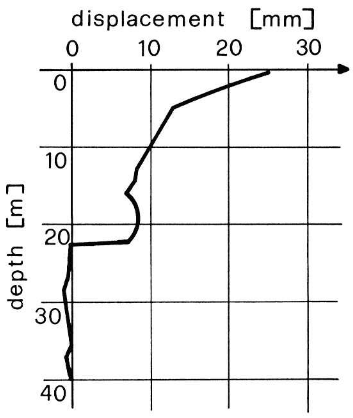


Fig. 4. Typical inclinometer displacement measurements in a 40 m long vertical borehole (observation period 4 months)

2. Shaft Concept

Considering the great number of shafts to be constructed in the creeping ground, having different lengths and situated in various geological environments, the design criteria had to be formulated very clearly and made easily adaptable to the individual shafts. The maximum permissible displacements of a shaft with respect to the hollow rectangular piers were stipulated

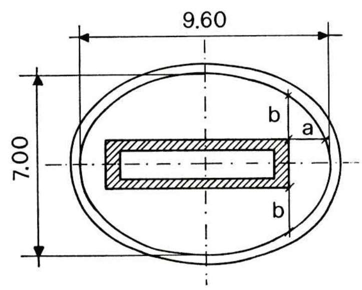


Fig. 5. Horizontal cross-section of the elliptically shaped shaft and the rectangular pier (a = 1.50 m, b = 1.0 m)

according to Fig. 5. The movements were assumed to occur either as a superposition of mass creep and basal displacement with equal contributions or as the result of basal displacement alone. The other design criteria came from the different types of earth pressure acting on the shaft lining. The kinematical, statical and constructional requirements lead to the adoption of the elliptical cross-section shown in Fig. 5. The main axis in the dip direction was selected on the basis of the permissible shaft deformation (a = max. 1.50 m) and the required free working space during construction. The minor axis was chosen to obtain a statically favourable shape of the cross-section. In view of the formwork used for the concrete lining the internal shape and size of the shaft cross-section was the same along the whole length of the bridge.

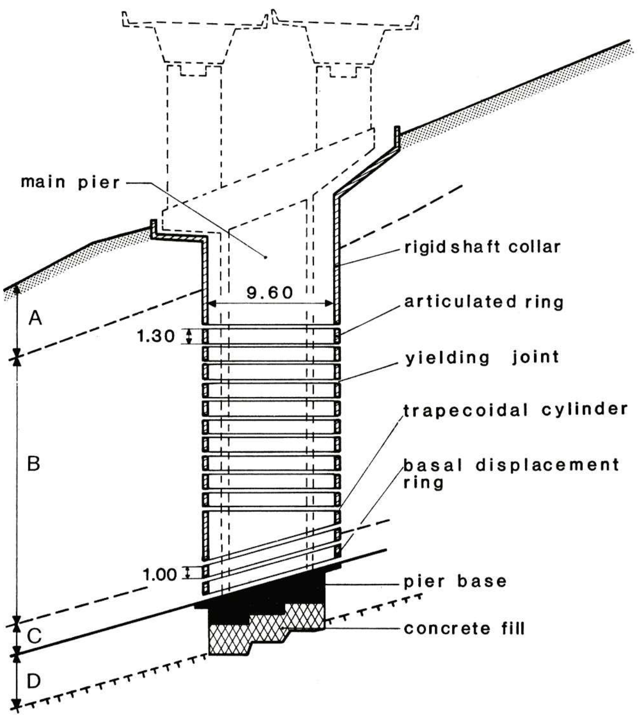


Fig. 6. The selected shaft concept
A — Zone of potential surface slip; B — Zone of creeping soil; C — Zone of basal sliding; D — Zone of loosened rock

To meet the different requirements the shaft concept shown in Fig. 6 was selected. Ideally there are four structural elements which perform different functions:

a) rigid shaft collar,
b) articulated ring section,
c) trapezoidal cylinder,
d) basal displacement ring section.

From the 44 completed shafts only about half of them contained all four basic elements. For statical or for kinematic reasons it was often necessary to omit one of the elements. Even if almost every shaft had a somewhat different appearance, the fundamental considerations have always been observed.

3. Shaft Design

The earth pressure acting on the shafts is highly dependent upon the selected shaft system. The rule generally applies that shaft parts which resist the creep movement of the slope are to be designed for creep pressure, while for those which adapt more or less to the slope deformation earth pressure

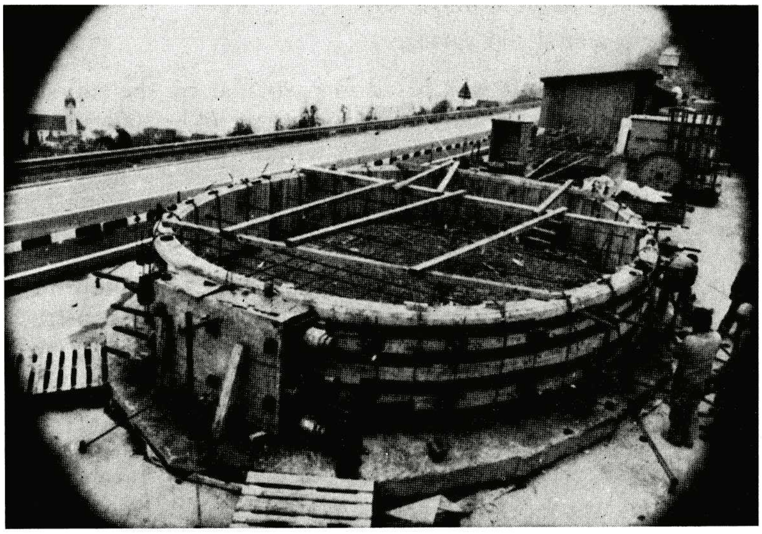


Fig. 7. Instrumented full scale shaft ring test

at rest has to be taken into account. Design for active earth pressure is only admissible where a continuous stress relief zone is verifiably present. Hence, it is generally true that in the zone of a potential slip surface and in the zone of possible basal displacement creep pressure is the more decisive factor. In the zone of differential displacement the design is based generally on the condition of earth pressure at rest, and in special cases on active earth pressure. However, it must be confessed that in the determination of the earth pressure there is a considerable degree of uncertainty. In some cases even the determination of plausible limit values is difficult. In these cases sound "engineering judgement" should at least partially take the place of strict mathematical criteria. With the special shaft concept, i. e. division into individual structural parts of the shaft acting as rings, the entire shaft analysis can virtually be based on the theory of an elastically bedded ring. After a thorough investigation it was established that the two criteria, i. e. instability (ring buckling) and limitation of deformation (inadmissible cracking), must be taken into account.

The theoretical critical loading conditions can only be determined by means of the theory of second order, taking the elastic-plastic stress-strain behaviour of the ring into account.

To check these theoretical solutions and for the actual determination of deformations and failure behaviour, a full scale ring test was performed (Fig. 7).

For every shaft the designer had a geological profile at his disposal containing data on:

- soil layers (identification and thickness),
- geotechnical parameters (γ, ϕ', c', E_s) for every layer,
- thickness of a possible shallow slide,
- depth of deep-seated slip surface.

The design of the shaft was based in particular on the soil stiffness E_s. Its value in the critical layers was approximately $(40—100) \cdot \gamma \cdot z$ depending on material type, where γ denotes the unit weight of the soil and z the depth.

3.1 Rigid Shaft Collar

To ensure overall stability with respect to shear failure during the construction stage and slope failure, a rigid shaft collar was constructed to a certain depth. The soil conditions at the ground surface also necessitated such a construction to maintain a stable shape. The depth of the rigid collar was governed by stability considerations. Depending on slope and soil conditions it ranges from 7 to 18 m. Depths exceeding 10 m are due primarily to high earth pressures resulting from possible surface slips.

The following assumptions were made in relation to the earth pressures:

- creep pressures in the region of surface slips,
- earth pressure at rest laterally and on the side facing up the slope,
- reduced passive earth pressure on the side facing down the slope.

Three dimensional effects were considered in the earth pressure calculations. The rigid shaft collar was designed as an elastically embedded structural unit like the ring elements. This simplification could be made based on the results of extensive model analysis on elastically embedded cylindrical shells. The decisive design criteria were buckling or bulging of the ring, whereby the following four load cases were considered (λ_x, λ_y = earth pressure coefficients in the x and y directions, λ_0 = earth pressure coefficient at rest, λ_a = active earth pressure coefficients, λ_k = creep pressure coefficients, y = direction of the slope).

Load cases LF without creep pressure

LF1: $\lambda_x = \lambda_0$ and $\lambda_y = \lambda_0$

LF2: $\lambda_x = 0.9\,\lambda_0$ and $\lambda_y = 1/2\,(\lambda_a + \lambda_0)$

LF3: $\lambda_x = 1/2\,(\lambda_a + \lambda_0)$ and $\lambda_y = 0.9\,\lambda_0$

Load cases with creep pressure

LF4: $\lambda_x = \lambda_0$ and $\lambda_y = \lambda_k$

For the assumed loads a global safety factor $F_g \geq 2.0$ for no creep and $F_g = 1.5$ with creep was required. The design of the wall thickness d and the amount of steel reinforcement μ was based on design charts. With these requirements it is ensured that the shafts exhibit adequate safety against bending even in the case of non-uniform earth pressures.

3.2 *Articulated Ring Section*

The middle segment of the foundation shafts is the articulated ring section. This segment consists of single, horizontally split shaft rings fitted with compressible joint packings which can shift and tilt to a certain extent and, at the same time, compress. With the articulated ring section it should be possible to accommodate differential creep movements, totalling approx. 50 to 75 cm (Fig. 8).

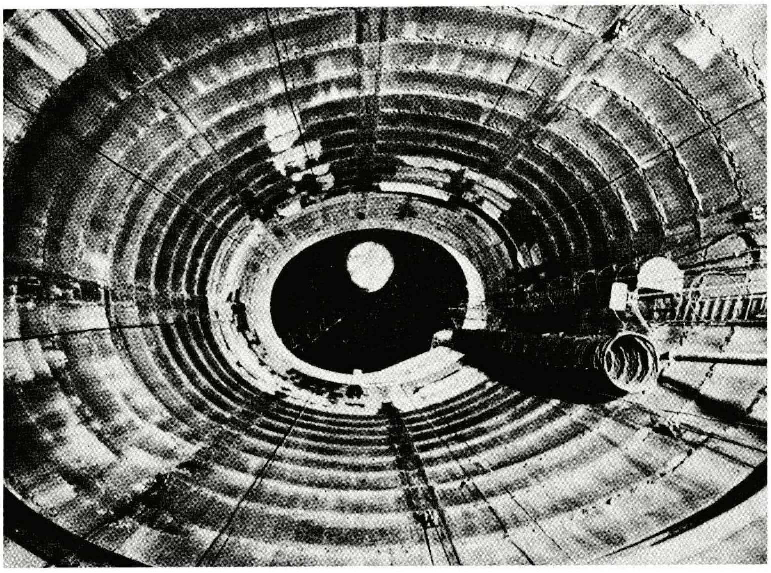


Fig. 8. View from shaft bottom with articulated ring section and yielding joints

From working considerations the ring height was set at 1.30 m. For purely kinematic reasons slanting ring elements, i. e. oriented in the direction of the creep movement, would be preferable to horizontal ones. However, from construction considerations, and also from a lack of knowledge of the exact vectors of creep movement, a slanting ring configuration for the ring elements was foregone and the less efficient statical solution was accepted.

The ring elements were designed for earth pressure at rest conditions assuming elastic embedment. The earth pressure was assumed to act all around the ring. With the aid of design charts it had to be shown that a safety factor with respect to buckling $F_k \geq 2.0$ was achieved. Additional stressing due to non-uniform ring loading was accounted for by the reinforcement ($\mu' = \mu = 0.3$ plus an additional amount at the corners $\mu' = \mu = 0.1$). These measures were taken as a result of the computer analysis of the elastically embedded ring under conditions of non-uniform loading.

3.3 Trapezoidal Cylinder

The trapezoidal cylinder serves as a transition piece between the horizontal ring elements and the slanted basal displacement rings. The static function corresponds quite well to the ring elements described above. The height of the trapezoidal cylinder is governed primarily by the local geological conditions. However, minimum dimensions must be adhered to for static reasons. Generally the wall thickness d of the shaft is selected such that the cross sectional stiffness is the same in all directions. This requires a greater wall thickness on the side facing the slope than on the downslope side.

3.4 Basal Displacement Rings

The basal displacement rings can accommodate the anticipated basal displacement of max. 1.00—1.50 m. Since the location of the basal displacement usually encompasses not only a single plane, but often an entire area, it may be necessary to install several such elements. The basal displacement rings are separated from the trapezoid cylinder and the bedrock base by a compressible joint packing with a thickness of 20—40 cm. The thickness of the basal displacement rings is up to 1.0 m because they must be capable of accepting the locally acting creep pressure. The basal displacement rings were also treated as elastically embedded. The design was based on conditions of earth pressure at rest with $\lambda_x = \lambda_0$, $\lambda_y = \lambda_0$ and a safety factor with respect to buckling of $F_s \geq 2.0$. For the case of creep pressures the values $\lambda_x = \lambda_0$, $\lambda_y = \lambda_k$ and $F_s \geq 1.0$ were adopted. The creep pressure coefficient λ_k was found from a three dimensional failure mechanism. Depending on the soil conditions and the inclination of the sliding surface values of λ_k were estimated to be in the range $(1.5 \text{ to } 2.5) \cdot \lambda_0$.

3.5 Yielding Joints

The joints are a crucial element of the novel shaft design. The ring joints must meet two conditions. Firstly, a limited horizontal, differential ring displacement must be allowed by the arrangement of the joints, and secondly an excessive vertical stressing of the shaft elements should be avoided by compression of the joints. Without joints the shaft, especially deep shafts, would be overstressed. This could lead to a critical condition, i. e. to failure of the shaft.

Thus all shaft elements, i. e. shaft collar, ring elements, trapezoid cylinders and basal displacement rings are separated from one another by joint packings. The joint thickness was established on the basis of kinematic criteria adapted to the shaft configuration, the shaft depth, the extent of creep and the movement mechanism. The thickness is 5 to 10 cm in the

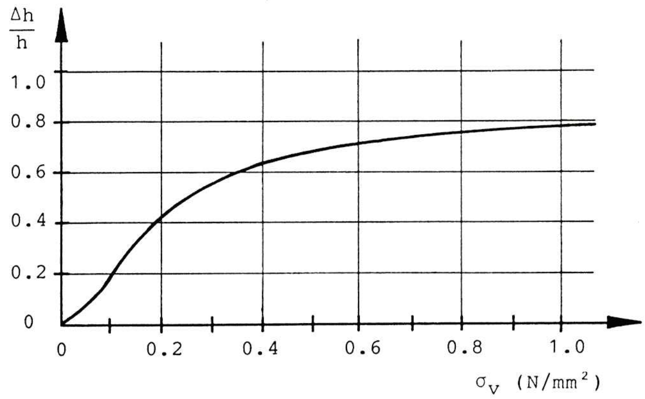


Fig. 9. Joint packing material (Flumroc). Compression strain $\Delta h/h$ as a function of normal stress σ_v

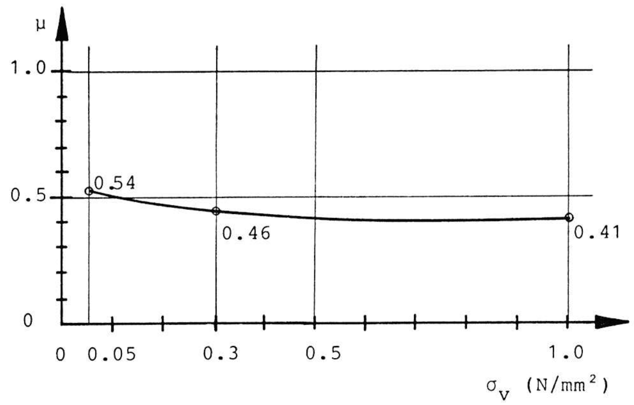


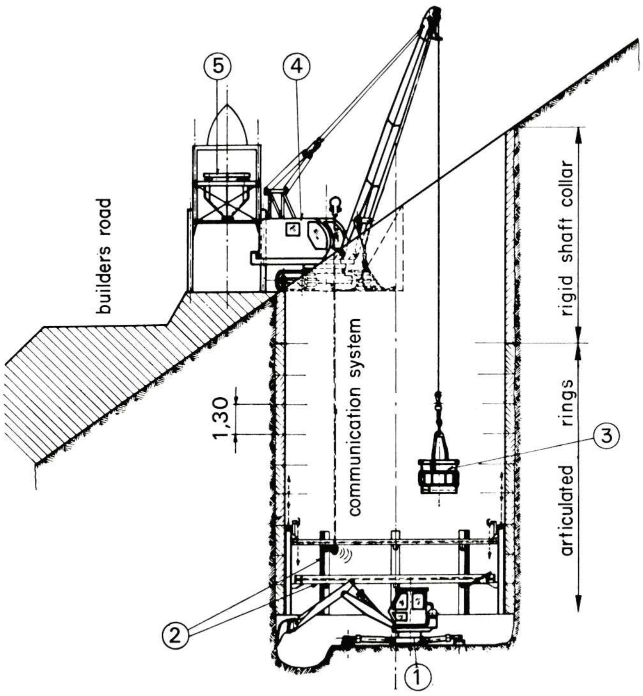
Fig. 10. Joint packing material (Flumroc). Internal friction resistance μ as a function of normal stress σ_v

area of the articulated ring elements and 20 to 40 cm in the area of the basal displacement rings. As joint packing material Flumroc (Flums rock wool) was selected (Fig. 9) because of its high compressibility (approx. 50% compression under a stress of 0.25 N/mm²), its high resistance to ageing and above all its price. Flumroc has a certain disadvantage by virtue of its relatively high internal friction resistance of approx. $\mu = 0.50$ (Fig. 10). Technically more suitable materials, for example products based on caoutchouc, bitumen or special plastics were ruled out for economic reasons.

3.6 Selection of Cross Section

For the shaft cross section an elliptical form was selected. For this section the following criteria were generally applicable:

- Inside diameter profile, calculated from pier dimension and free movement clearance of 1.50 m in the slope line, and 1.0 m transverse to the slope.
- Required free working space for optimum use of machines.
- Statically optimum cross-section with regard to earth and creep pressure.



LEGEND

(1) excavator, type Kamo 5.5 t

(2) shaft formwork, type Aeberli

(3) hoist bucket, capacity 2 m^3

(4) conveyer, cable excavator lifting capacity 5 t

(5) silo for material handling, capacity 8 m^3

Fig. 11. Construction concept, shaft sinking method

The inside diameter profile and the free working space dictated the selection of the major main axis and the static criteria the selection of the minor main axis.

As a consequence of the earth and creep pressure, an elliptical shaft is preferred to a round one. It should also be noted that with the elliptical shaft the material and work expenditures can be kept at a minimum. In view of the mechanical formwork used the axis dimensions were kept constant and for all shafts they amounted to 9.60 m in the direction of the slope and 7.00 m transverse to the slope (Fig. 5).

4. Shaft Construction

The problems encountered in the construction of the shafts (Fig. 11) were very numerous. The main problems from the engineering point of view are only briefly discussed here. The difficulties are concentrated in particular on the shaft collar and the shaft toe, while construction of the central articulated section was relatively unproblematical.

4.1 Shaft Collar

The shaft collar is usually 6—10 m high, in extreme cases 7—18 m. The wall thickness is around 30—40 cm. With a rigid shaft collar difficulties are encountered with construction work on steep slopes.

In a slope with potential slip hazard the provision of a construction pit as a primary prerequisite for appropriate basic working conditions has already proved to be quite problematic, even with small open cuts. To overcome these difficulties basically two methods are conceivable, and they were also applied in this case for the shaft construction:

— Construction of the first section of the shaft collar following installation of an anchored sheet-pile wall or braced cut.

— Step or segment-wise construction of the first shaft ring using minimum open cuts.

The first method is substantially simpler and also more reliable but it involves a high material and time expenditure. With individual shafts this method was prescribed in view of the high slip hazard and the buildings located in the affected area.

The second method, shown in Fig. 12, was used very successfully with the majority of the 44 shafts. Even though the work proceeded in minimum excavation steps, larger open cuts of 4—5 m excavation height can scarcely be avoided. As experience showed, such cuts can already lead to difficulties. To achieve a sufficient stability margin regarding local slope failure a downhill sheet pile wall may become necessary to increase the passive earth resistance.

With both procedures the parts of the rigid shaft collar following the first ring are constructed in stages. The depth of the step-wise shaft construction is governed by considerations of local slope stability.

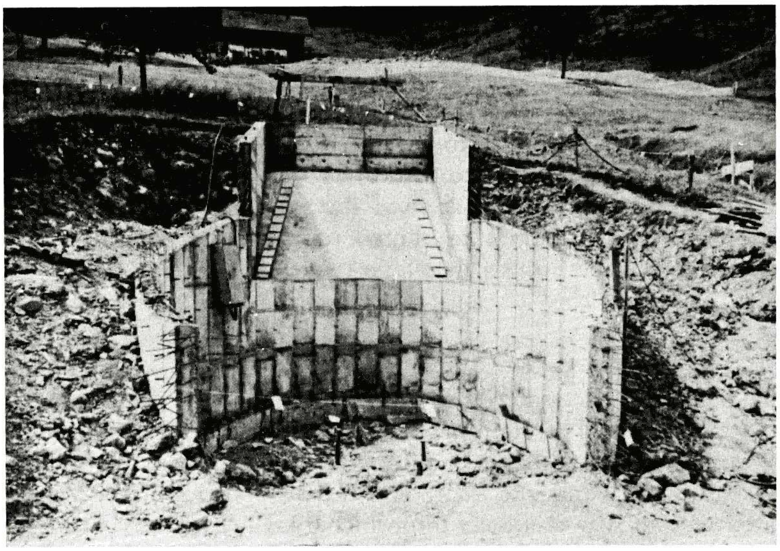


Fig. 12. Construction of rigid shaft collar in a slope

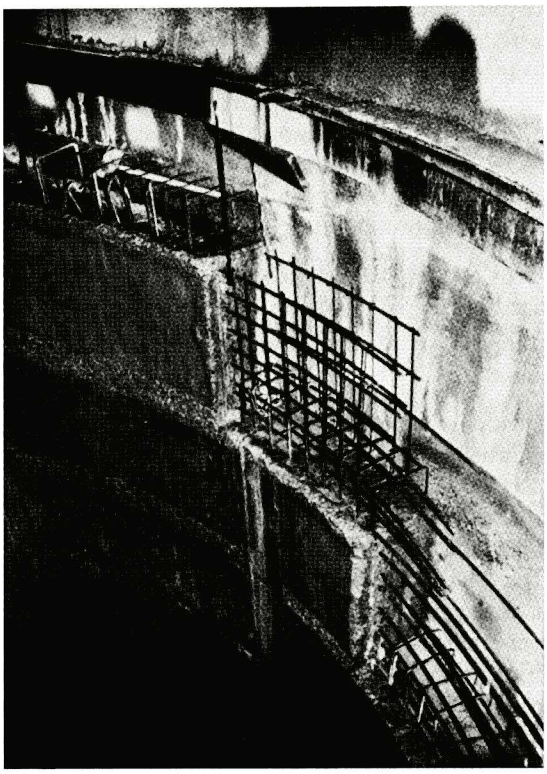


Fig. 13. Construction of shaft toe with trapezoidal cylinder. Transition stage with gunited shell

4.2 Ring Elements

As shown in Fig. 13 the entire shaft up to the shaft toe was constructed by the underpinning method. The ring elements of 1.30 height and 30—50 cm wall thickness were constructed in one process. Contrary to expectations the strength of the excavation material did not involve major difficulties, in spite

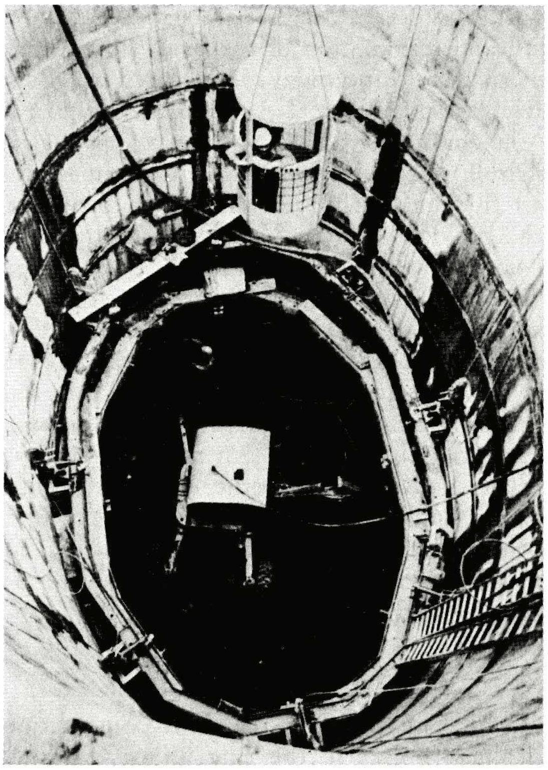


Fig. 14. View into the shaft with formwork, excavator and lifting equipment

of the adverse water conditions. It was possible to forego segment-wise construction of the ring elements in almost all cases. The initial fears that when excavating the material for the next ring element the previously concreted element could settle also proved to be unfounded. The amount of overprofile is prone to underestimation. In this case it is on average about 15—20 cm, which applies for gravelly-sandy soil, as well as highly cohesive morainic material.

To accomplish the required work effort mechanical equipment is used for excavation, material handling and formwork (Fig. 14). The excavation was performed with an electrically driven small excavator with a weight of approx. 5.5 t and an average capacity of approx. 10 m³ per hour. The material handling was performed partially by heavy rope excavators partially by crane systems with a bucket capacity of approx. 2 m^3. For the shaft wall construction a specially manufactured formwork was used, which also enabled step and segment-wise ring construction. The entire equipment usage proved to be good.

4.3 Shaft Toe

The greatest difficulties were encountered with the shaft toe. At the time the shaft work was started neither the exact location of the rock surface nor the depth and termination of the basal displacement were known. For the shaft construction (trapezoidal cylinder and basal displacement rings) exact knowledge of these geological conditions is required. To circumvent these difficulties the shaft was sunk from the trapezoidal cylinder depth to foundation depth following placement of a light, mesh-reinforced gunited shell with a wall thickness of 15—25 cm. After the boundary conditions following from the geology were established, the shaft was constructed conventionally from bottom to top. For reasons of sound design the trapezoidal cylinder was in the first stage only constructed to the extent to allow for unproblematic adaptation in the construction (Fig. 13).

The thickness of the basal displacement rings, as an essential component in this shaft depth, is 40—100 cm. Because of the ring slant, which varies continuously from shaft to shaft and which in the extreme case amounted up to 45^0, the conventional formwork can no longer be used for constructing the ring elements. As a substitute the use of a concrete formwork of bent concrete boards has proved to be the most expedient solution. The disadvantage of this solution is that these formwork boards can only be made cut to size, and because of the conditions which vary strongly from shaft to shaft they have only a very limited reusability.

4.4 Shaft Drainage

The slope drainage by virtue of the shaft construction is effected primarily through leakage points in the ring joints. Where it was necessary the shaft wall was additionally perforated. Conversely, horizontal drainage borings (fan arrays) to increase the effectiveness of the slope drainage were initially foregone for economic reasons. Such measures can be implemented at a later time from the terrain insofar as they are necessary at all. The shaft drainage is routed via a drilled longitudinal drainage system from shaft to shaft with intermittent discharge to the environment at a suitable natural waterway system. The discharge is effected by individual, permanently installed pump stations. Wherever possible, however, it is effected directly through a drilled perforation line.

5. Closing Remarks

The construction of the viaduct, whose piers are founded in sound rock and are protected by shafts from the existing shallow and deep-seated soil creep movements in the slope, may be justly described as a novel solution.

Fig. 15. Aerial view, middle section of bridge structure

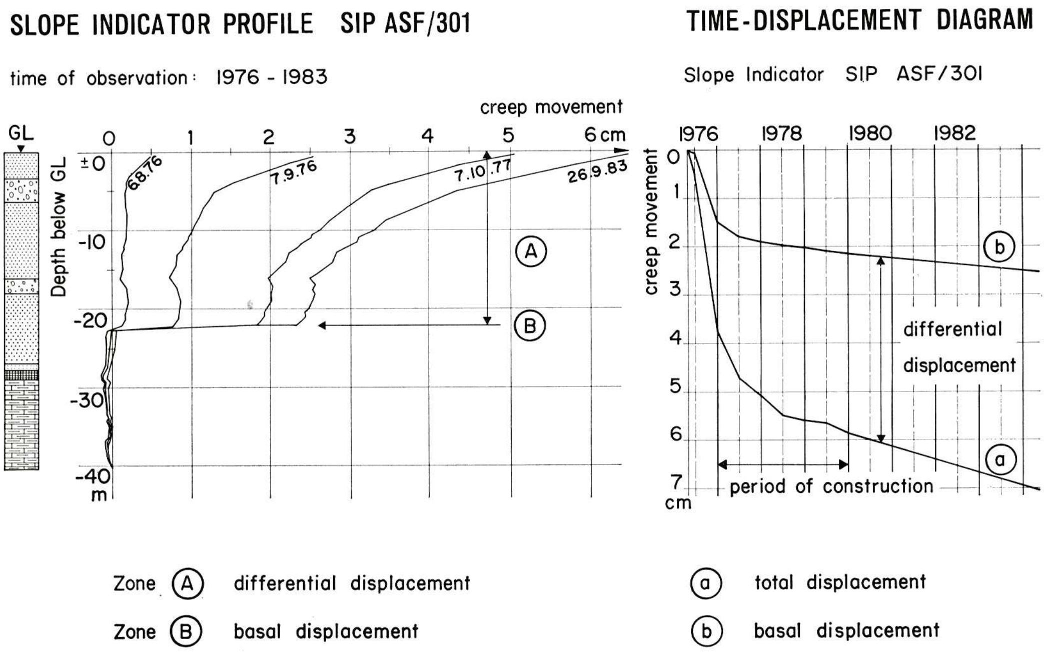


Fig. 16. Measurement of the horizontal displacement in the creeping slope by means of inclinometers

Compared to a shallow shaft foundation, which would involve fairly high maintenance costs due to necessary adjustment of the base of the piers and also a rather high risk factor, the solution adopted is much more economical. Examination of other foundation concepts, e. g. pile systems, spread footings or diaphragm walls down to the rock surface, revealed that they were not feasible for technical reasons.

Besides their protective function the shafts also serve to drain the surrounding slope and thus they contribute to its stabilization. The slope stabilization work carried out parallel to the construction of the shafts was an additional measure. This work comprised of stream correction and control of the ground water level by means of drainage boreholes and galleries.

A typical example of the development of creep movements is shown in Fig. 16, in which measurements of displacement versus time are presented graphically for the stages before, during and after shaft construction and the execution of slope stabilization work. It may be seen that the measure adopted could not fully stabilize the slope, but could, however, reduce the rate of creep considerably.

Acknowledgements

The geological documents originate from the works of Dr. T. R. Schneider, Geologist, Switzerland, who was responsible for the entire Beckenried Viaduct geological consulting. Individual photographs were taken from the book "Lehnenviadukt Beckenried" published by D. J. Baenziger, Zurich.

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