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Design and Construction of the Furka Base Tunnel

By

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Summary

The Furka railway tunnel, with a length of 15.4 km, a maximum overburden of 1520 m and the portals lying at 1500 m above sea level, presented some special problems both in the design and construction stages, which required novel solutions. The cross section of this single track tunnel varied from 26 to 42 m^2. It is remarkable that the permanent support only consists of rock anchors and a shotcrete lining. In sections with high rock pressures it was necessary to excavate an elliptical or circular profile using steel ribs in addition to anchors and the shotcrete. All support measures were closely adapted to the geotechnical conditions encountered. As a basis for decisions, besides geological and rock mechanics investigations and construction experience, the results of systematic deformation measurements were used.

To make the rock in the vicinity of the opening impermeable a newly developed two component polyurethene grout was applied, to the author's knowledge the first time in a traffic tunnel. The grouting also improved substantially the strength of the jointed rock.

1. Introduction

The Furka-Oberalp railway line presents the only all-year-round east-west connection in the Swiss Alps. Besides providing for efficient local transport it has the important function of linking the holiday resorts in the Valais, Central Switzerland and the Grisons. The 15.4 km long single track Furka Base Tunnel at an elevation of 1500 m above sea level connects the localities Oberwald and Realp. It replaces the old 1.7 km long tunnel located 2100 m above sea level. This former connection could not be operated during the winter due to heavy snow drifts and danger of avalanches.

Construction of the new tunnel started in 1972 and it was completed in 1982. The causes of the relatively long construction time are to be found in the considerable length of the tunnel coupled with a relatively small cross-section (transportation problems), in the high altitude of the portals and also in the difficult geological and hydrogeological conditions. The preliminary tunnel design underestimated the problems which were likely to occur during excavation. The adjustment of the project to the actual rock condi-

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tions and the introduction of novel methods for supporting the rock and for dealing with ground water considerably speeded up the tunnel advance. Major setbacks were experienced at some locations of the tunnel where, surprisingly, extremely bad rock conditions were encountered. Despite the difficulties the overall construction costs of the completed tunnel did not exceed sFr. 12000.— per tunnel metre based on 1972 prices.

1.1 Tunnel Location and Geology

The tunnel passes through the Gotthard Massif, which underwent one or more metamorphic phases in the course of the formation of the Alps. In one of the later phases they were highly stressed due to tectonic movements. This resulted in the development of joints, facture zones, faulting and local crushing of the rock structure (Keller and Schneider, 1982). From the horizontal section through the tunnel axis (Fig. 1) it may be seen that the strike of the rock series is generally SE-NE, i. e. more or less parallel to the line connecting the tunnel portals at Realp and Oberwald. The dip of the rock layers is mostly steep to vertical. Due to the poor mechanical properties of the chlorite-sericite phyllite of the permocarbonaceous series as well as the unfavourable position of its layers with respect to the straight line connecting the portals, it was decided to adopt the alignment route shown in Fig. 1 with a strong curvature towards the south. It was aimed to locate the tunnel axis as much as possible in the orthogneiss and granite and to cut through the paragneiss layers with as steep an angle as possible. This deviation from the shortest connection between the portals led to an extra 3.5 km length of tunnel, but at least avoided the difficult geological conditions in the sedimentary rocks. Further, it was also possible to construct an access in the middle section of the tunnel by means of the Bedretto adit. With the help of this adit it was possible to divide the tunnel into practically independent contract sections and thus save about 1 $^1/_2$ years construction time.

Originally it was intended to leave the option open of exploiting this adit in the future by providing an additional rail link with the south. During the construction of the tunnel this access adit proved extremely useful by enabling an intermediate attack, especially in view of the difficult geological conditions in the middle section but also during the construction of the rail substructure.

The geological predictions were based on observations at the ground surface. The steeply dipping layers of the various formations permitted a reasonably accurate extrapolation of the geology at the ground surface down to the depth of the tunnel axis. Vertical exploration boreholes would, under the given circumstances, hardly have brought any further information. Also, given the great overburden (Fig. 2) their cost would not have been justified. The real problem was how to interpret the known geology with respect to its effect on construction taking into account the size and shape of the cross-section and the great overburden. Besides tunnel sections with relatively little variation of rock properties, short zones of geological disturbance were also expected. It was therefore most important to select a construction method which could be easily adapted to the varying conditions.

A detailed prediction of the hydrogeological conditions was much more difficult. However, it could be assumed that the tunnel would cause a considerable drainage effect and correspondingly large quantities of water

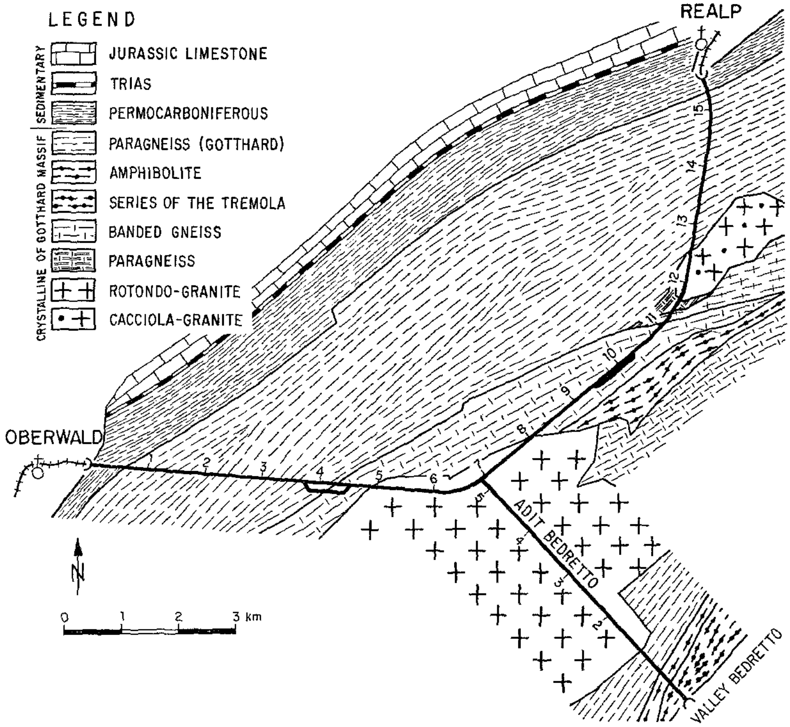

Fig. 1. The geology and the route of the tunnel

would be encountered. Up to the point of break-through in the contract section Realp the water inflow rose to 180 l/s, while at the portal in Oberwald a single measurement carried out in 1981 registered 240 l/s. In the section Bedretto quantities of up to 100 l/s were recorded during short periods (Keller and Schneider, 1982).

2. Design Concept and Construction

The assessment of the mechanical properties of the various rock series led to the choice of a horseshoe profile with straight sides. The tunnel cross-section show in Fig. 3a encloses, with very close tolerances, the required railway clearance and results in the minimum volume of excavated material. The Furka tunnel was constructed for 80% of its length with this cross-section either with full face excavation or by heading and benching. In the zones with difficult geological conditions elliptical and circular cross-sections were used (Fig. 3b). As may be seen from Fig. 2 these cross-sections

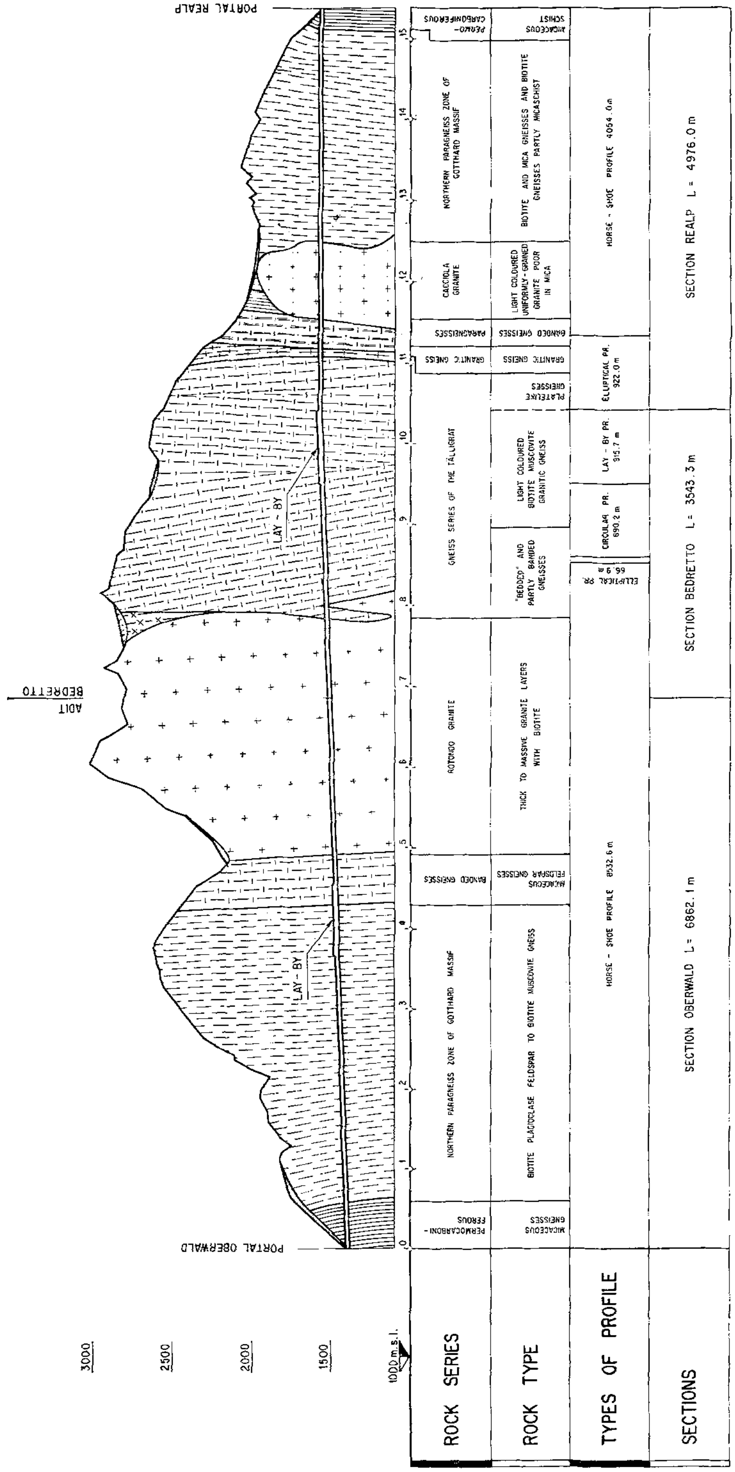

Fig. 2. Geological profile

were concentrated in the western end of the contract section Realp and on the eastern side of the Bedretto section. The two track stretch of the Bedretto section is also situated in a zone of highly disturbed rock and it was con-

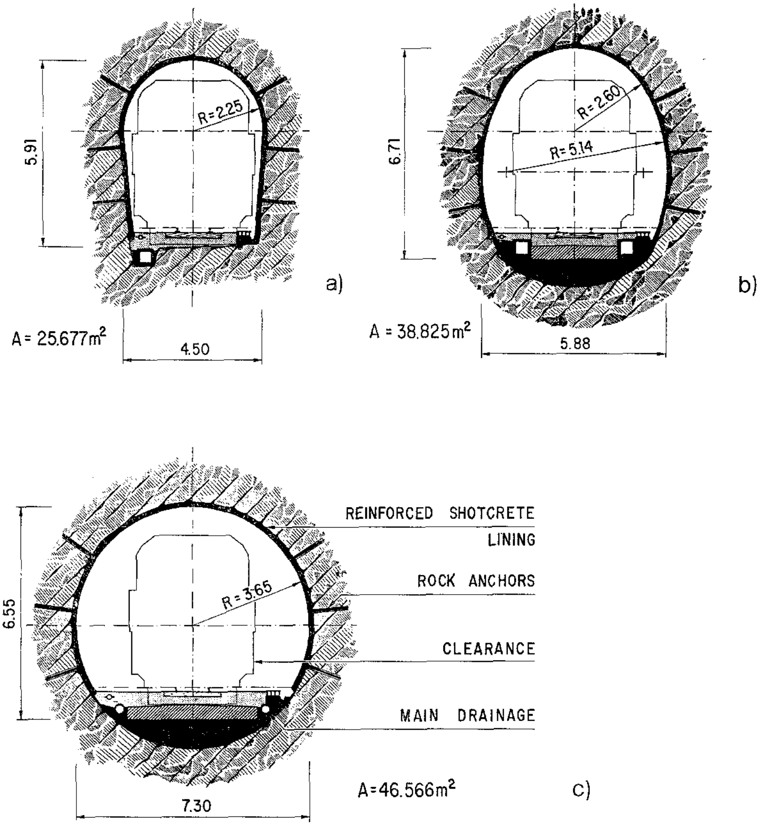

Fig. 3. Main types of tunnel cross-section
a) horseshoe, b) elliptical, c) circular

structed with a circular cross-section. Given the special problems in this section, it will be discussed in more detail later on. The following support measures were applied:

(i) shotcrete (with and without reinforcement)
(ii) rock anchors
(iii) steel arch, and
(iv) rock stabilization with polyurethene.

They were applied to a greater or lesser extent and in different combinations depending on the rock quality encountered (Fig. 4). These measures formed the elements both for the initial support and the permanent lining. The construction of an additional inner lining was considered unnecessary even in zones of high rock pressure. The construction concept proved to be particularly flexible, time-saving and economical. Immediately after blasting the necessary measures to guarantee safety during construction were taken, i. e. by providing initial supports. The permanent shotcrete lining was constructed later when the tunnel had advanced some distance away. This distance was based on theoretical considerations and the evaluated results of extensometer and Distometer measurements (Kovári et al., 1974). Sixteen standard lining types with a maximum admissible amount of shotcrete and a certain number of anchors were used

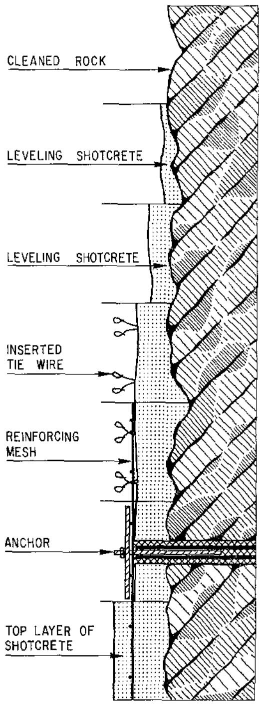

Fig. 4. Schematic composition of the permanent lining

(Amberg and Sala, 1984). The normal lining type for the horseshoe cross-section ranged from the minimum (type 1) lining with only 0.8 m^3 dry mix per tunnel metre up to the maximum (type 4) with at most 5.6 m^3 dry mix, 16 m^2 of reinforcing mesh and 6 anchors per tunnel metre. Depending upon the requirements the shotcrete was reinforced with a single or double layer of steel mesh. The mesh types used ($100 \times 100 \times 6$ mm and $100 \times 100 \times 8$ mm respectively) could be easily adjusted to the excavated surface. In the selection of the lining types for a given stretch of tunnel the engineers relied on visual observations and the results of deformation measurements. In this way an economically optimum adaptation of the permanent lining to the effective rock conditions could be achieved. A similar procedure was applied at the Arlberg toad tunnel for the design of the temporary support (John, 1977).

In the Bedretto and Oberwald contract sections the greatest length of tunnel with the same type of lining was around 300 m and the shortest only 2 m. This lining concept also had considerable operational advantages. During the tunnel advance it was still possible to operate several independent working groups in succession on the construction of the final lining. Due to the confined working conditions the trailing installation of an inner lining would have caused serious transport problems, thus substantially increasing the construction time.

Rock Anchors: The newly developed "SIGBOLT" tube anchors were applied for the first time in the Furka base tunnel. The "SIGBOLT" anchor consists of a completely grouted polyesterfibre glass tube with alcali resistant fibre glass. A two-component epoxy resin served as the grout material (Hagedorn, 1983). By using steel instead of fibre glass the anchors can be mechanically tensioned and then later completely grouted. These anchors have the advantage that immediately after installation they can carry load; at a later stage, they can also be grouted. This type of anchor was used in the two track section in the contract section Oberwald. The anchor lenghts installed in the Furka tunnel varied between 2.0 and 4.5 m.

Polyurethene Injections: This injection system has been developed in mining (Meyer, 1981), and has been used successfully in this field for some time. The polyurethene is injected in two components. The development of a foam only occurs when the grout comes into contact with water in the rock. The penetration of the grout into the rock mass can be controlled by means of a suitable choice of reaction time, thereby preventing any uncontrolled loss of material in areas of greater permeability. The process is accompanied by the development of pressures of up to 5 bar. The combined effect of expansion and pressure pushes away the water from the joints and openings. In this way the polyurethene penetrates the smallest cracks, and upon hardening the rock mass is also strengthened. This could be proved without doubt by the deformation measurements. The dual function of the grout, namely sealing and stabilizing the rock mass, was one of the important factors allowing the shotcrete lining to act as a permanent lining even in stretches with difficult rock conditions.

Investigations have shown that in winter the freezing point in the tunnel may be as far as 1 km from the portal. The outflow of water would, under such circumstances, lead to considerable ice formation, which would cause hindrance to rail traffic. Due to the polyurethene injections this problem was totally eliminated. In sections where considerable water discharge occurred at seams and diffusely, a dry tunnel was obtained with this treatment.

Deformation measurements with electronics registration unit: In order to check the effectiveness of the chosen support measures an unusually large number of measurement sections were necessary. Due to shortage of time and the limited accessability of many measuring points traditional methods of taking readings with mechanical dial gauges were not feasible. Thus, for the borehole extensometers an electronic registering unit allowing programmable reading intervals and the storage of measurement values was developed (Amberg and Hertelendy, 1983). The storage capacity of the registration unit amounts to 1300 separate readings. For long-term registration the capacity may be increased by means of a data logger (tape). After acquisition, the data can be evaluated and plotted using a desk computer (Fig. 5). The rapid and simple evaluation of the measurements permitted detection of any trend in the structure's behaviour after relatively short measurement periods thus support adaptation measures, if necessary, could be arranged in good time. Even the design of the permanent lining was based on a knowledge of the time-deformation response and the influence

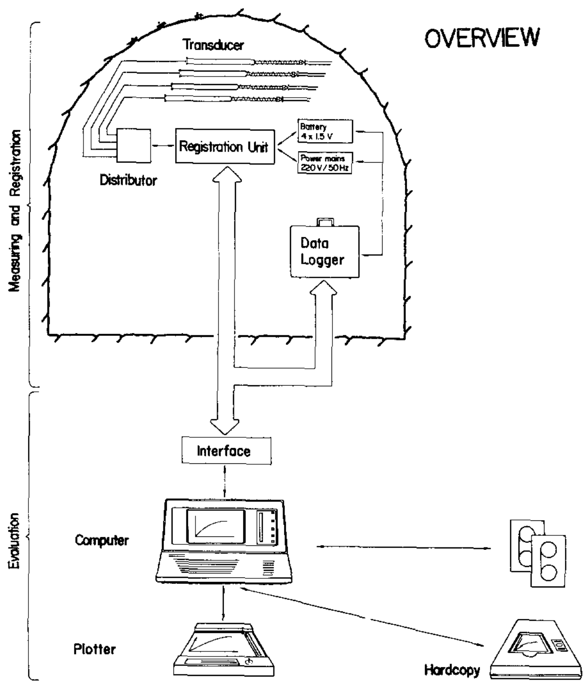

Fig. 5. The function of the electronic extensometer

of the different support types on the deformation behaviour. In this way the scope for subjective decisions was kept within reasonable bounds. The automatic reading and storage of data also proved to be useful after the end of the construction. Long-term monitoring could be carried out in critical tunnel stretches, whereby the stored data was evaluated at intervals of 1—2 months.

3. Problems During Construction

The numerous rock mechanics and construction problems which arose during the construction of the Furka tunnel, may be divided into two groups. On the one hand, the difficulties must be mentioned which occurred at the working face itself, and, on the other hand, those which had to be overcome in the existing temporarily supported tunnel sections.

3.1 Problems at the Tunnel Face

The tunnel was cut through numerous zones of geological disturbance, whose existence or exact position and extent was not known beforehand.

Fig. 6. Flow of crushed material and water at TM 3385

As a result, several collapses occurred, the cause of which was usually a combination of the following geological factors (Keller and Schneider, 1982):

(i) lamprophyres
(ii) recent disturbances
(iii) disintegrated rocks
(iv) water inrush.

An example of a geology related problem during excavation is the roof collapse at tunnel metre TM 11,996 in the Realp contract section. It occurred in a recent tectonically disturbed zone of the Caociola Granite (Fig. 1) with gneiss-like lamprophyre, disintegrated rocks and water inflow. The collapse destroyed three steel arches that had already been put in place and developed an 8 m high chimney-like opening stretching back 9 m. The originally dry material was increasingly soaked with water and a discharge of up to 12 l/s was measured. The caved-in material began to flow (Fig. 6) and had to be stabilized by means of grouting. In this 40 m stretch of geological disturbance the excavation had to be carried out by heading and benching method, dewatering the rock by means of drainage boreholes. It took six months to

Fig. 7. Buckling of vertical layered rock

complete this short stretch. To avoid this happening again exploratory core drilling to lengths of as much as 97 m was performed, at certain points of the tunnel, to provide information on the rock conditions to be expected during excavation. Other difficulties encountered, which were of geological origin, have been discussed in detail by Keller and Schneider (1982).

3.2 Problems in Sections with Existing Initial Lining

Difficulties were frequently encountered in sections with an initial support when some time had already elapsed and the excavation had progressed to some distance from the affected spot. Although they were not as serious as those described in the previous section they still demanded a significant expenditure of time and materials. The most important phenomena observed were the buckling of steep layers of gneiss (Fig. 7) and spalling in the granite zones (Fig. 8). In view of the great depth of overburden

Fig. 8. Spalling of granite

the occurrence of such phenomena in stretches with only shotcrete support is quite understandable. Consequently, the desired stability of the opening was maintained using a regular grid of anchors. Difficulties of another kind were given by large rock deformations, which led to an inadmissible size reduction of the cross-section area. Thus at various places in the tunnel it was necessary either to place additional support or to reexcavate. As a last possibility an already existing horseshoe profile had to be transformed to the statically more favourable elliptic shape. Such a case is illustrated in Fig. 9. The buckling of the arches may be seen as well as further back the concreted tunnel stretch. As was expected, the concreting was not as effective as providing the tunnel cross-section with the statically more efficient form of an ellipse to be seen in the foreground of the picture.

4. The Two-Track Section

As the Bedretto contract section was driven toward Realp increasingly difficult conditions were encountered at approx. TM 8000. Thus at TM 8532 it was decided to change from the horseshoe to an elliptical cross-section and then after a further 67 m even to a circular one. With the larger circular cross-section due to its more favourable statical properties, and the resulting improvement in working space conditions it was possible to achieve a substantially higher rate of tunnel advance. From TM 9489, where the

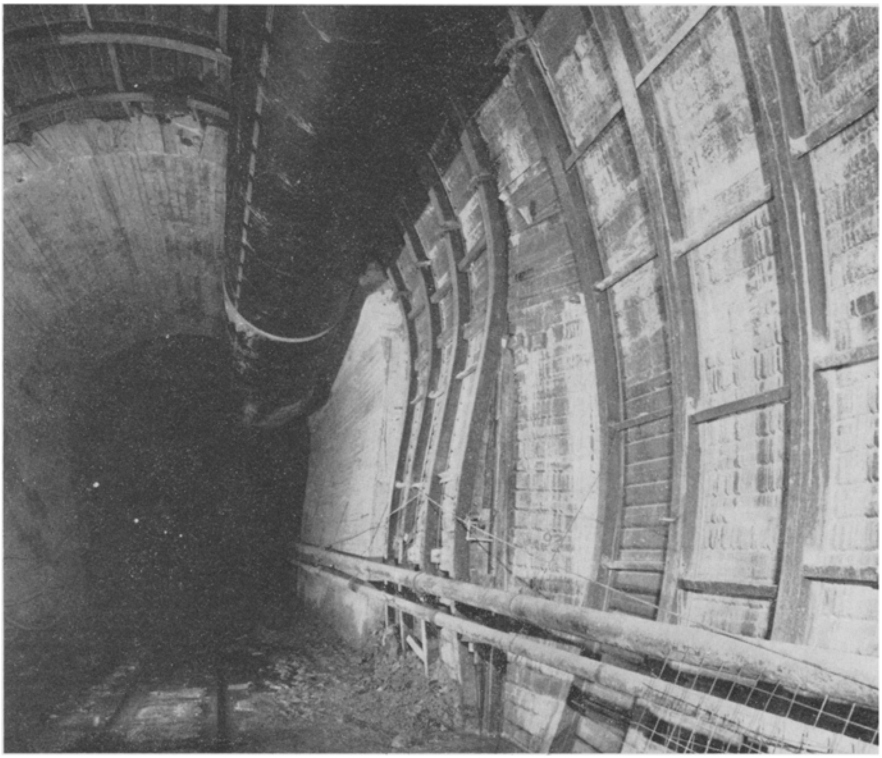

Fig. 9. Failure of steel ribs in the horseshoe cross section leading to elliptical profile (foreground)

overburden was about 1100 m, a cross-section to accommodate the two-track lay-by section was excavated by the heading and benching method. However, due to bad geological conditions the work was temporarily brought to a standstill at TM 10314. Likewise on the Realp side at TM 10489 the elliptical profile suffered such large deformations that extensive bracing of the profile had to be installed to prevent a collapse of the tunnel (Fig. 10). The material encountered in this zone had been completely disturbed by tectonic action. In some also high discharge of water occurred. In the foreground in Fig. 10 a stretch of shotcrete lining with steel ribs and regularly spaced anchors can be seen. This is followed firstly by a short stretch, in which only shotcrete was used as support, and then by another, having extensive support measures. The latter soon proved inadequate so that as an emergency measure the tunnel had to be braced.

Fig. 10. The elliptical profile in changing rock conditions showing bracing as an emergency measure

In addition, the spacing between the steel ribs (HEB 160 and 180) was reduced to 0.50 m. The rate of deformation decreased as a result by about 30—40%. In addition 18—20 fully-grouted 4.5 m long steel tube anchors per metre length of tunnel were placed. The numerous measurement sections in this zone showed a definite reduction of deformation rate. However, a complete stabilization of the rock movement was not achieved. Thus the decision was made to apply polyurethene injections. Its success was quite dramatic. The stretch of tunnel could be satisfactorily sealed against water inflow, while the deformation measurements indicated a complete stabilization of the rock mass. The shotcrete lining in this section was 20 cm thick and reinforced with mesh. Based on this experience it was decided to excavate the remaining short section of tunnel up to TM 10314 using a different method: A top drift was excavated first (Fig. 11).

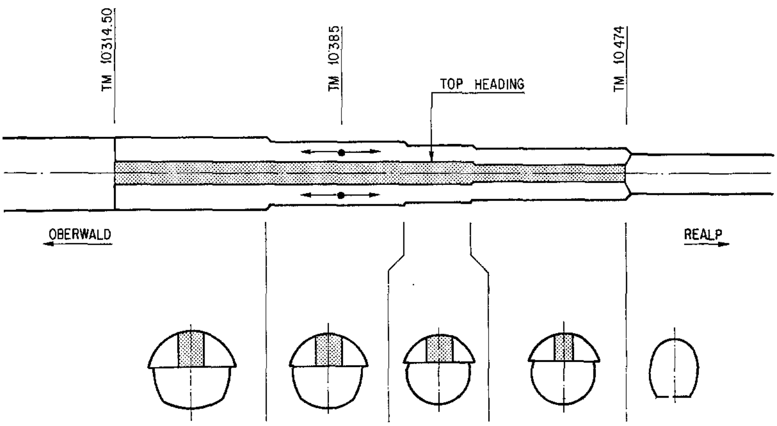

Fig. 11. The tunnel section with difficult rock conditions in the area of the two track cross-section

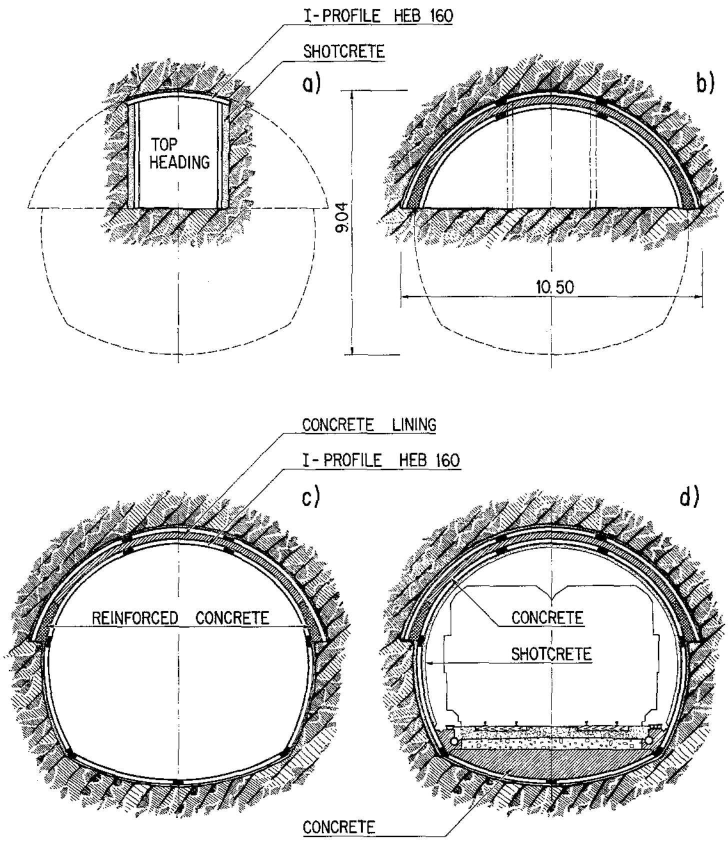

Fig. 12. The method of construction at the difficult two track section

Then it was first attempted to widen the top drift successively to both sides to the full width of the heading. In the Bedretto section the working space provided by the large circular profile made the widening work quite efficient such that a rate of advance of 3—4 m per day was achieved. The constricted space conditions in the single track elliptical cross-section on the Realp side, however, impeded the widening of the top drift so much that only a small rate of tunnel advance (0.5—1 m per day) was possible. As a

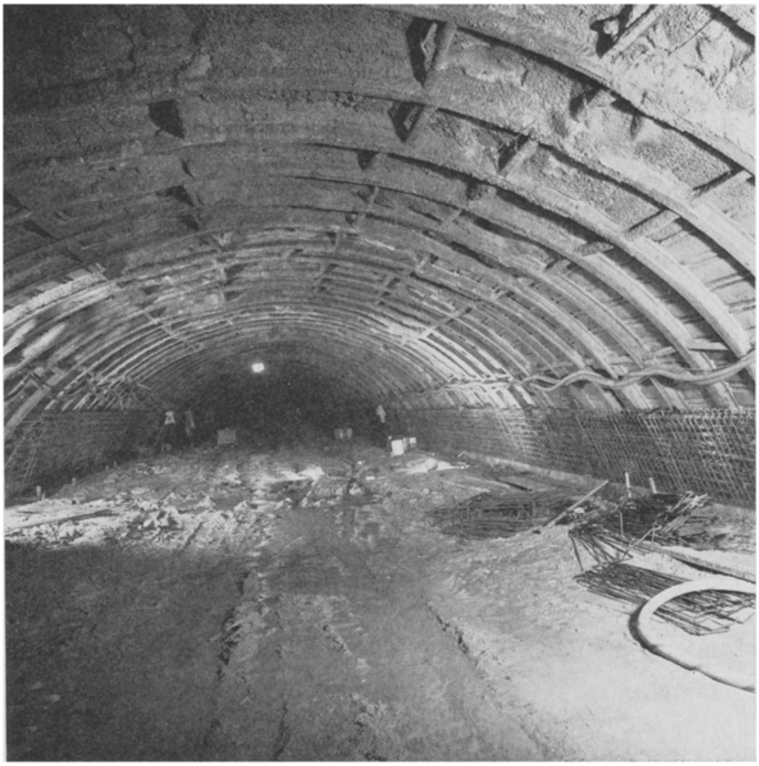

Fig. 13. The top heading with the outer steel ribs and the reinforcement for the longitudinal concrete beam

result, the deformations at the working face produced substantial loosening of the rock mass, which manifested itself in high rock pressures. It was observed how rock blocks, which possessed relatively high strength properties, were crushed by this action caused by the poor advance of the face. This was all the more remarkable, since the elliptical profile had much smaller dimensions than that of the two track section. Based on this observation the top drift widening work was discontinued on the Realp side, and all the extension work was carried out from the middle section (TM 10385) with simultaneous advance in two directions. The individual steps are illustrated in Fig. 12 a. Both an inner and an outer series of steel ribs at intervals of 1.0 m were installed. A reinforced longitudinal concrete beam (Fig. 12 b) was constructed at the bottom of the ribs. This beam had a high load capacity and was intended to bridge longer excavation stages during lowering the bench. The reinforcement of the beam is shown in Fig. 13. The picture shows the state of construction before installation of in Fig. 13. The picture shows the state of construction before installation of the inner series of steel ribs. The latter also served as a support for the formwork during the concreting of the calotte arch and as additional reinforcement. However, the main function of the ribs was to provide a strong connection with the rib elements in the lower part of the tunnel. Thus during the bench excavation the steel structure could be closed to form a statically favourable ring (Fig. 12 c) after each round. Fig. 14 shows

Fig. 14. Heading and bench method of excavation with inner row of steel ribs underpinning upper half

completion of excavation of the lower half with the inner row of steel ribs underpinning the upper half. Using this procedure a rate of excavation of 4—6 m per day was easily achieved in each direction. The extensometer and convergence measurements showed that the tunnel section remained stable. The whole section was provided with a 20 cm thick shotcrete permanent lining.

5. Conclusions

The construction of the Furka Road Tunnel offered a unique opportunity to assess the present state of tunnelling technology for long tunnels with high overburden under changing geological conditions. In retrospection, after the completion of the tunnel one can easily point out where improvements could have been made. A greater width of the horseshoe profile would have simplified the transport of labour and material during construction. With a greater tolerance of the excavation cross-section in relation to the specified clearance, it might have been possible to dispense with time-consuming reprofiling work. It would also have been better to apply the elliptical section more frequently. However, the overall design concept with the adaptation of the cross-sectional shape, the thickness of shotcrete and its reinforcement, and of the number of rock anchors and steel ribs to the encountered geological conditions proved to be safe and economical. Using shotcrete as a permanent lining is considered to be the optimum solution for cases similar to the Furka tunnel. Improvements of present tunnelling technique resulted in a new type of rock anchor using alkali-resistant glass-fibres fully grouted with synthetic epoxy resin, and a two-component polyurethene injection of the rock mass which improved the rock strength and made it impermeable. Borehole extensometers with electronic data recording equipment also represent an improvement. Some of the results of problem solution at the Furka site can be applied generally in tunnelling.

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