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Bochum Ring Road: Planning and Choice of Construction Method

By

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Abstract

As part of the West Tangent of the Ring road in Bochum it was necessary to construct, in difficult geological formations, a tunnel with two tubes, each of section 100 m^2. Severe limitations were placed on ground subsidence. Since the foundations of residential buildings were immediately adjacent to the tunnel tubes the question of environmental disturbance had a major influence on the choice of construction method. After the roof collapsed a few times the excavation work with a centrally positioned pilot tunnel had to be stopped. As a result of the change to top heading method the encountered geological difficulties could be controlled.

Introduction

The highway system of the city of Bochum was rendered unserviceable as a result of war damage and in the post war restoration period a completely new system was planned and designed. The sharp rise in private transport necessitated a traffic network that could be expanded. Up till now the necessary adjustments to it could be accommodated.

The historical north-south and east-west axes were more or less retained. These routes, however, have been substantially relieved of traffic by constructing a ring road system in the inner city area. The outer ring on the fringe of the inner city is almost 75% finished. With the construction of the West Tangent this work will be completed. The West Tangent, as a main traffic artery, is being constructed as a motorway with four lanes and no crossroads. It is built generally at a low level, running in cuttings. To negotiate a dense business and residential area with 3 to 5 storey buildings a tunnel with two separate tubes each of length 560 m was required. The depth of the tunnel is determined partly by the foundations of the buildings and partly by a planned metropolitan railway system crossing the tunnel at approximately mid-length. The depth of cover to the roof is between 2 and 10 m. The tunnel tubes are separated by a central pillar of width varying from 6 to 16 m. Each tube has 2 lanes 4.00 m wide and 2 hard shoulders 1.00 m wide. The cross-section is shown in Fig. 1.

Geology

The tunnel tubes lie predominantly in rock of the carboniferous period with typical layer sequences of shale, sandy shale, and sandstone interspersed with seams of coal from a few centimetres to 2.0 m thick. The rock is heavily jointed. Above it lie in general relatively shallow diluvial deposits, which consist mainly of the remains of post-glacial loess material.

To investigate the geotechnical site conditions along the route of the tunnel a total of 79 boreholes were drilled. Their location was influenced by the buildings in the vicinity. In all boreholes groundwater levels were measured. The fluctuation in the measured results for an individual borehole during the course of the investigation was not very great. Along the complete stretch of the tunnel, however, quite a large variation in water levels was observed: sometimes no water was encountered down to the full depth of the borehole, sometimes even within the uppermost layer a continuous water surface was observed, and sometimes water was observed in the joint system of the exposed layer. Due to the presence of sulphate and carbon dioxide the groundwater was weakly aggressive towards concrete.

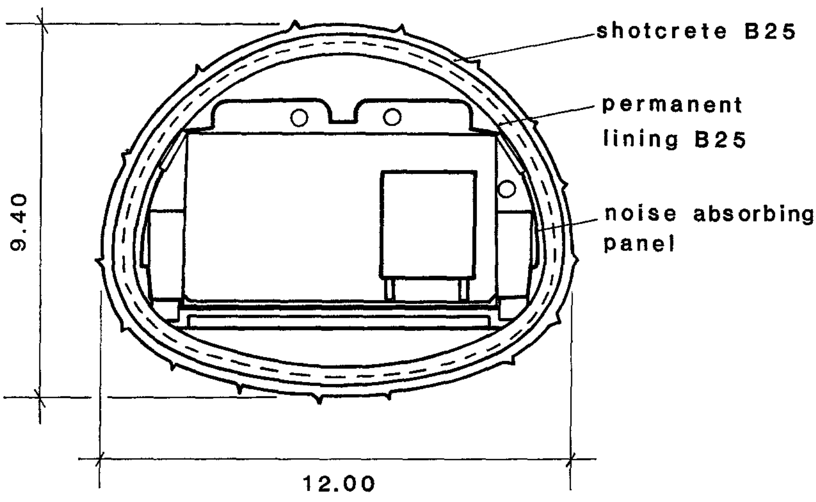

Fig. 1. Tunnel cross-section

The following geological profiles was revealed by the boreholes: At the ground surface there was a 0.5—1.5 m layer of fill material, consisting of building rubbish, slag, sand and fine silt.

The natural ground consisted at the surface of a coarse silty layer of max. thickness of 3.0 m, its underside lying at a depth of 1.0 to 5.9 m.

This was then followed by a completely weathered zone of the carboniferous rock of thickness between 0 and 3.0 m, composed of sandy, stoney silt or stoney sand material. Under this relatively loose material is a layer of brittle, strongly jointed to clumpy material. The thickness of the weathered material depends on the hardness of the original rock.

The top of the unweathered carboniferous rock lies at a depth of 5—8 m, whereby the upper 2—4 m are heavily jointed with layer thicknesses ranging from 10 cm to 1 m (Fig. 2). The layer interfaces were mostly closed.

Fig. 2. Rock bedding

Beside these layers, joints were observed with openings and in the form of solution surfaces. Two distinct joint sets may be distinguished, which run parallel and normal to the strike of the layers. The continuity of the various joints could not be determined by the boreholes alone, so that it was necessary in 5 boreholes near the portals to investigate the structure, spacing and size of the joints by means of an optical device. The joint spacing depends on the rock type. As a rule the following average values can be assumed:

Rock type	Spacing in m	
	Main joints	Secondary joints
Claystone/shale	0.2—1.0	0.5—2.0
Sandstone	0.5—2.0	1.0—3.0

Open joints were to be expected above all in the sandstone. It could be assumed that, between the individual layers and at the joint surfaces, thin seams of silty or clayey material was present, which could act as slip surfaces thus reducing the shear strength of the rock.

Bochum is located in the most important mining area of West Germany, however deep colliery workings to be found in this area were closed down in 1960. Future mining activity is not to be expected under the present coal board, so that no influences from past and future mining operations needed to be taken into account.

To provide support for the structure against collapse it was necessary to drill holes to known underground cavities before the beginning of excavation and to fill them up. In addition, boreholes were driven into the seams on a grid of 5 m spacing. These also served the purpose of locating the openings. In some areas of loosened rock a cement grouting treatment was applied. The programme for borehole installation, filling openings and grouting was accompanied by close observations as the work was carried out and adjusted and extended accordingly.

If during the tunnel excavation other openings were to be encountered they were to be handled in the same way, but from the tunnel itself. However, it was found that all the seam workings had been successfully located and filled up with the methods employed.

Feasibility Studies

With the West Tangent project as with many other building projects in urban development areas in recent years a petition was raised against it. The protests were directed against the spoiling of nature and the infringement upon residential areas, and not against the tunnel itself. The people of the city of Bochum have learnt to live with mining activity. Further, they were not concerned with the safety of the buildings, as became evident from discussions. The main wish was to make the tunnel part as long as possible to save encroachment upon the residential areas. Thus the following studies were conducted:

1. to design a tunnel cross-section with minimum height taking into account the given clearance for traffic using a static optimisation programme, and then
2. to select a single pass construction method, which was appropriate to the geological and geotechnical conditions for the given tunnel route and gradient. Several alternative methods were investigated to find an economic solution giving the maximum sectional size.

In particular, the distance of the tunnel roof to the most critical buildings was investigated. A difficulty here, was the fact that the tunnel excavation was not symmetric with respect to the building foundations, so that differential settlements were especially critical.

To investigate the suitability of four possible constructional methods the most unfavourable geological section was taken. The aim was to find a method ensuring safety and restriction of settlements, and which could be controlled at any time regarding accident risk. In summary, the following constraints were given:

1. The two tubes with cross-sections of 100 m^2 each had to be constructed in the immediate vicinity of the building foundations (least cover 1.7 m, cf. Fig. 3).

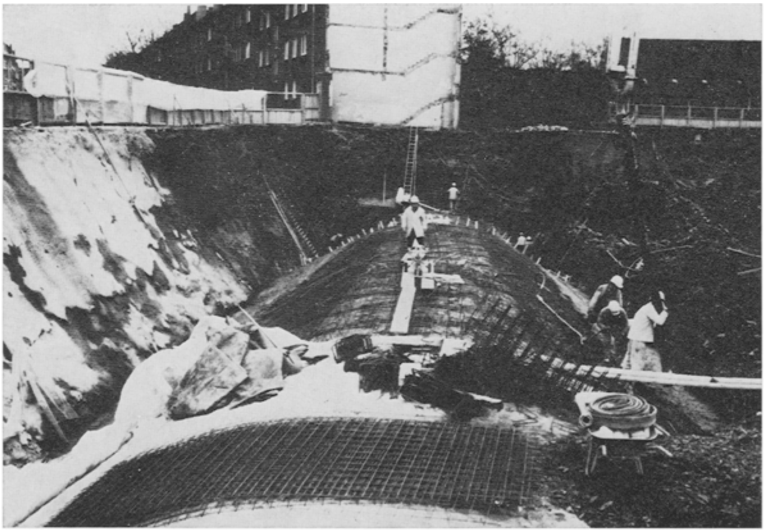

Fig. 3. Tunnel roof and soil cover

2. In the places where the overburden was least, the tunnel roof came to lie in the fill material.
3. Over large stretches the very compact Ruhr sandstone was present, requiring blasting even in zones near building foundations.
4. In addition to the above, there were the difficulties as a result of near surface mine workings, a very variable geology and geological disturbances.

The relatively large cross-section, obviously, had to be worked in several partial sections.

The following methods were investigated:

A. Stoping (Heading and Bench) Method

It is assumed that this method is well-known (cf. Fig. 4). This method of partial, step-like excavation of the face had a good chance of being adopted, because of the possibility of accomodating it to encountered geological conditions. In Fig. 5 three different subdivisions of the working face are shown, which can be readily adjusted to the geotechnical conditions and also offer a high degree of safety. The partial sections can be supported with steel ribs, anchors or shotcrete, whereby in the roof section forepoling with steel reinforcing rods or plates may also be necessary. Measures to provide support at the base of the partial sections, if needed, were also provided.

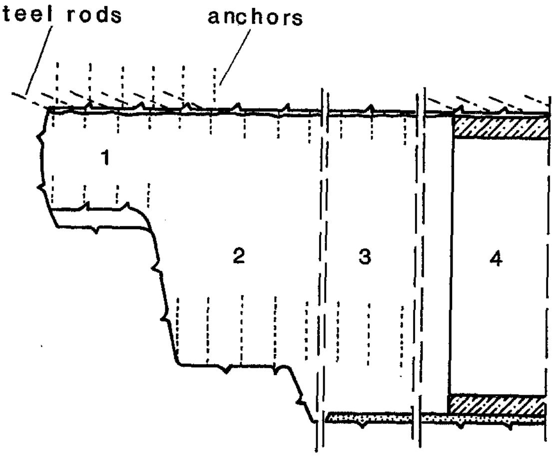

Fig. 4. Stoping: longitudinal section

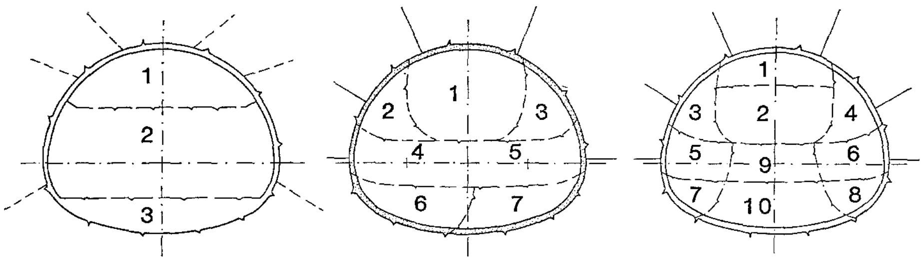

Fig. 5. Stoping: different subdivisions of cross-section

B. Stoping with Extended Roof Feet

An alternative to the above method is that shown in Fig. 6. The temporary support of the roof is achieved by means of extended feet, so that the headings can be excavated similar to the underpinning technique. The abutments should be reinforced at the level of the roof feet to provide greater support longitudinally.

C. Excavation with Leading Side Drifts

Immediately after removing the rock the individual excavated areas are supported by steel ribs, anchors and shotcrete.

As an alternative, a method with an advanced partial execution of the permanent lining in the side drifts was also considered (Fig. 7).

In the roof area the excavation should be carried out by mechanical means, and only in special cases after previous consultations with all concerned should blasting be used and then smooth blasting.

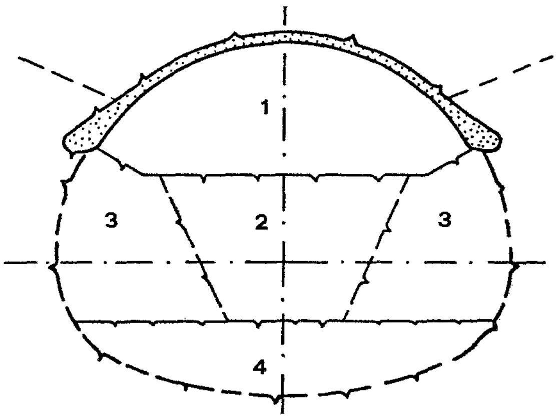

Fig. 6. Stoping: showing extended feet of roof lining

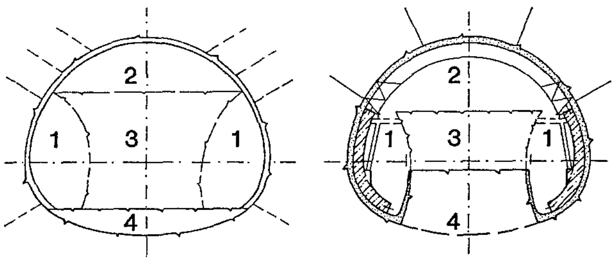

Fig. 7. Stoping: with advance side drifts

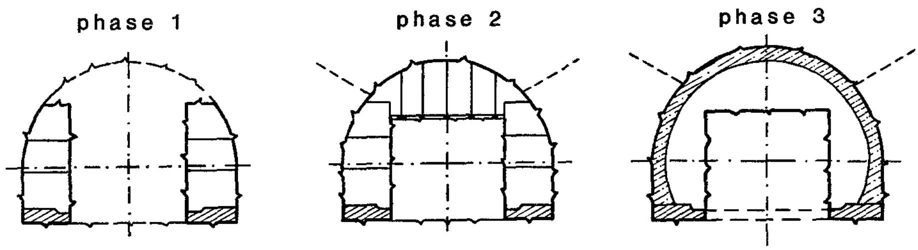

Fig. 8. Modified classical stoping method

D. Modified Classical Stoping Method

As in the previous case this method involves constructing the side drifts first, but these have a rectangular shape (Fig. 8). The steel ribs are braced so that anchors are not required for the side drifts. Good performance with respect to settlements was obtained with this method in the construction of a section of the metropolitan railway line in Muelheim.

The result of these investigations showed that the classical stoping method would be the cheapest one, and the best from the point of view of construction time. However, in areas with poor geological conditions a further subdivision of the cross-section would be required. These preliminary considerations lead to a shortening of the open cut section at the tunnel approaches compared to the original design.

Tender Requirements

Based on this first investigation the proposed stoping method A modified for a further subdivision of the roof into smaller excavation sections was recommended for the tendering process as the next step within the framework of the structural planning and design.

Beginning in the roof area a section of 30 m^2 should be excavated with as little disturbance to the rock mass as possible using a road header. The same applies for the subsequent extension of the roof excavation. The rest of the bench should also be excavated in several steps, where necessary with careful blasting, as only a minimum loosening in the rock mass should be allowed.

The above discussed methods of rock support should be used, whereby for partial excavation surfaces shotcrete support was to be used. Due to the short stand-up time of the rock a quick closure of the ring should be aimed at. Systematic anchoring should be employed where the rock conditions and the method required it. The temporary support of the working face, even of the partial headings, varies from case to case depending on the stand-up times as the construction method requires. In zones of crumbling rock and in the roof area in cohesionless material forepoling is planned.

The small distance between the tunnel and the building foundations required special measures. For example, the tunnelling technique required a continuous operation contrary to the wishes of the people living above the tunnel for no noise at night. Thus mucking at nighttime was not allowed. The residents were assured that unavoidable blasting operations were to be limited and replaced by cutting. The admissible vibrations due to blasting were set at a maximum of 8 mm/sec depending on the state of the buildings.

Towards the end of the excavation work it was discovered that most residents could put up with the blasting operations better than with the perpetual grinding of the cutting machine.

Special Proposals

Bids were received from 10 firms, which were supplemented by a total of 27 special proposals. In particular, two proposals, which on account of the construction technique and the price, appeared interesting to the client, are briefly described here.

In the first case, two side drifts should precede the excavation of the enlargement to full face. These drifts would be in the form of two tunnels excavated with full-face machines of 4.00 m diameter. Subsequently the rest of the tunnel cross-section would be mechanically ripped and scraped.

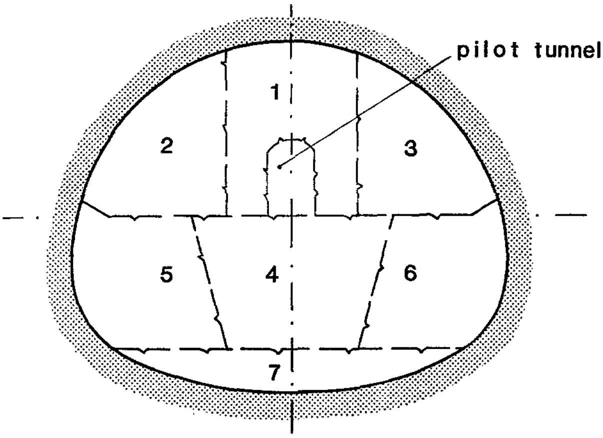

Fig. 9. Special proposal: pilot tunnel

In the second case, it was proposed to drive a pilot tunnel with a mini full-facer (Atlas Copco) along the whole length of the tunnel (Fig. 9). The pilot tunnel would be unlined in the more compact rock, otherwise, in less compact rock according to four different excavation classes, anchors and steel meshes sprayed with shotcrete and where necessary supported by liner plates.

The pilot tunnel proposal was based on the following considerations:

— accurate survey of the continually changing geological conditions
— dewatering of the rock
— rock stress relief for the subsequent careful blasting work with rock falling into the free opening of the pilot tunnel
— easier ventilation during main excavation.

This proposal was accepted due to its advantages including the increase of safety together with the economic benefits offered.

Construction

We would like to point out, that here the views of the owner are expressed, which are not necessarily shared by all the parties involved.

After successfully completing the construction works in the portal stretches, whereby the topsoil was cleared and a roof slab placed before excavating the tunnel section, the mini full-facer, was put into operation under the cover of the roof slab (Fig. 10). Initially, the roof of the pilot tunnel was 2.5 m below that of the final excavation, being located in a more compact rock zone. The first rock formation to be driven through was composed of alternating, relatively unstable layers of sandy to clayey shale. Then very brittle rock and soft zones of rock were encountered. These could be cut through easily, but the machine could not be fixed in place properly.

Fig. 10. Pilot tunnel

After about 9 m a part of the face failed during the cutting process followed by roof falls. Then in the region of a coal seam there was another fall of rock, comprising clayey shale and fill material, which was accompanied by substantial inflow of water. Due to the lack of working space and the percolating water attempts to stabilize the face with shotcrete were not successful. As a result there were further minor collapses, so that for the safety of the personnel the pilot tunnel excavation work was temporarily stopped.

After withdrawing the mini full-facer a certain distance the ground in the area not built upon was removed up to the point where there were rock falls and a large concrete slab 6×4 m was placed over the tunnel at a height of 2 m above the roof. The unstable roof behaviour continued as the excavation proceeded so that the concrete slab had to be extended in the direction of excavation.

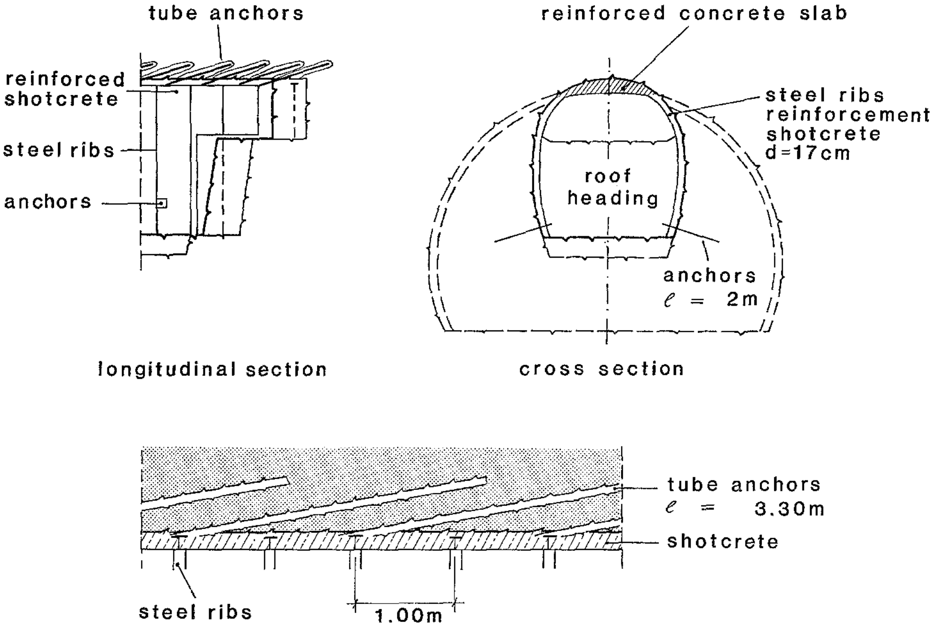

Fig. 11. Method of excavation finally adopted

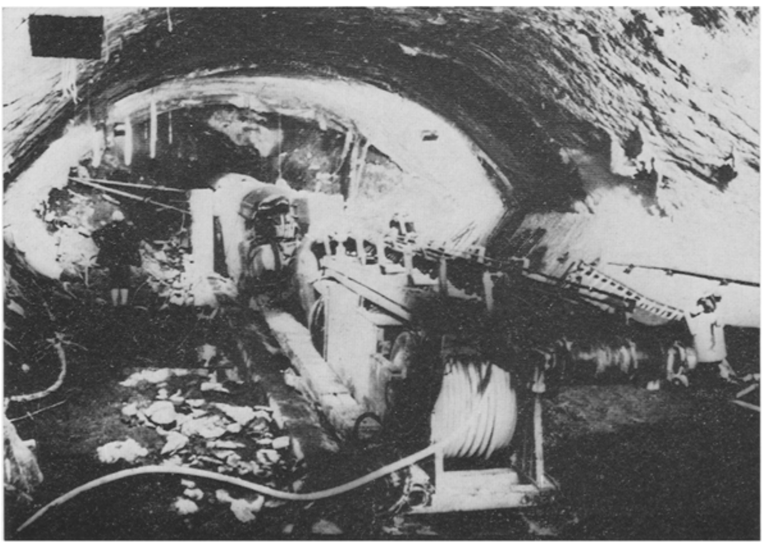

Fig. 12. Mini full-facer in operation

For the purpose of geological investigation a further 18 m were excavated by hand in 0.8 m intervals using timber, struts and liner plates. In addition, the working face had to be repeatedly supported in zones of geological disturbance and rock layering changes. Despite all these countermeasures minor rock falls were experienced again and again. In the meantime the built-up area was being approached and thus the method of excavation had to be discontinued.

The contractor responsible for the work then developed a method involving an advance top heading, which was based on a reinforced shotcrete beam located above the final cross-section (Fig. 11). The top heading was advanced a length of 30 m and supported with a shotcrete lining. In the course of the subsequent work this lining was lost (Fig. 12).

The advance top heading technique represented a return to the original design (A), namely by providing flexibility and support in the direction of the tunnel axis. The size of cross-section of the top heading was sufficient to allow use of a road header, a drilling machine, a hydraulic excavator and a wheel-mounted loading shovel. Most important of all, however, was the possibility of providing support directly at the working face, whereby under unfavourable geological conditions the working of the face could be subdivided into more steps. This method of excavation proved to be most successful (Fig. 13). In the meantime the tunnel excavation work has been completed without any damaging settlements or roof collapses.

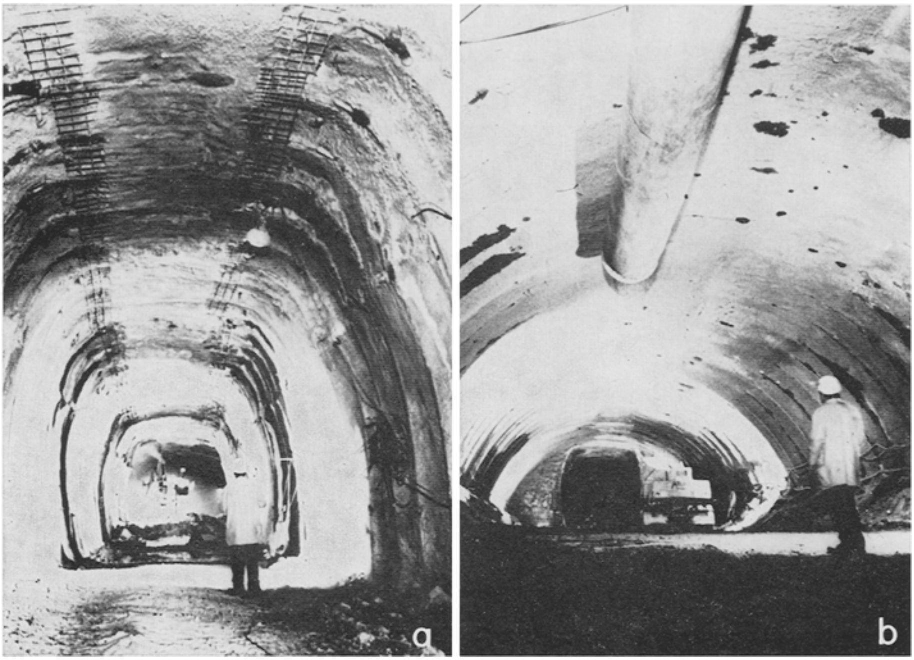

Fig. 13

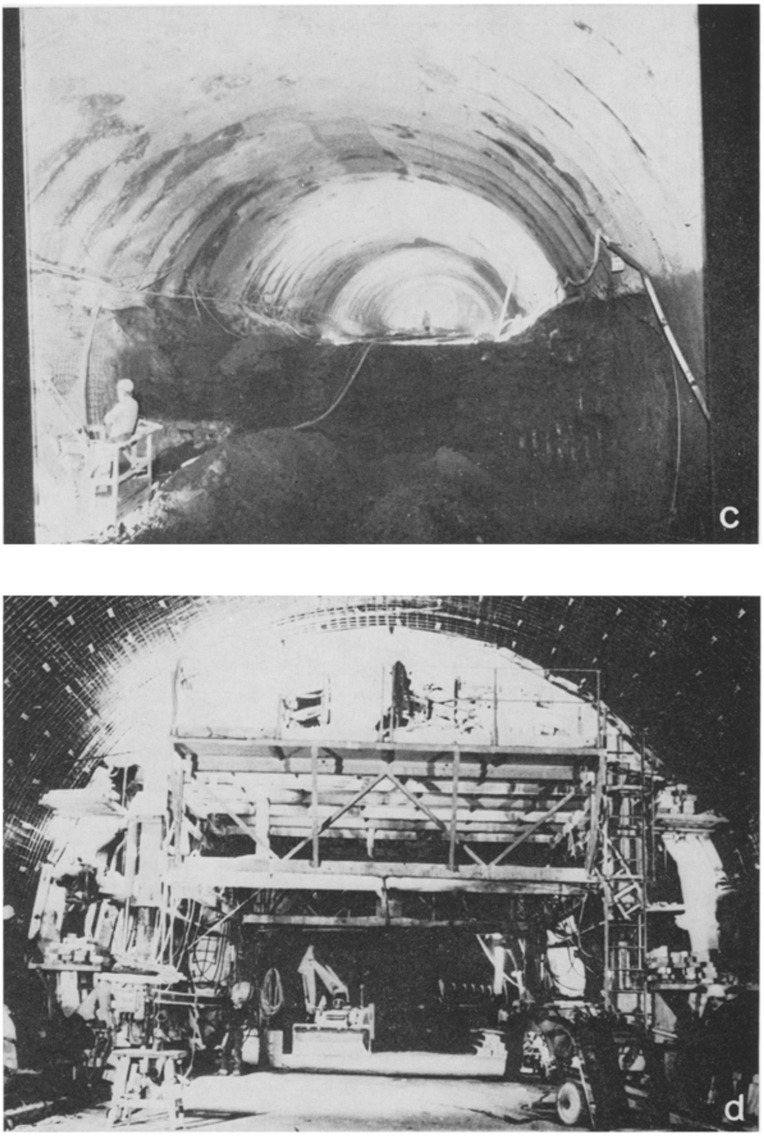

Fig. 13. Stages of construction

The question may be asked, why the excavation of the pilot tunnel with a tunnel boring machine proved unsuccessful? Our experience can be summed up as follows:

— The machine can only perform safely in homogeneous rock with adequate stand-up time. It is not possible to use it in changing rock conditions with the added complications of geological disturbances and inflowing groundwater.

— There was not sufficient rock strength to firmly fix the grippers of the machine.
— The signs of unstable, disturbed geological zones were not recognized or else falsely interpreted.
— In spite of the small cross-section of the working face and the careful excavation it was not possible to control every situation encountered in the rock mass. Thus it was first possible to fix anchors at a distance 5 m behind the cutting head, while the shotcrete could only be applied behind the machine, i. e. 12 m from the face. This requires a corresponding stand-up time of the rock mass.

This case history clearly shows how the stand-up time of a rock may influence the choice of the excavation method and the type of machines used, depending greatly on the possibility of short-term support measures.

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