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Subway Construction in Munich, Developments in Tunnelling with Shotcrete Support

By

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Summary

The planned network for the Munich subway system has a total length of about 100 km with 106 stations. At present 46 km of subway and 50 stations are in operation. A further 17 km and 18 stations are under construction. The annual volume of construction work has reached a total of approximately 300 million DM.

A stretch of 21 km has been constructed using shotcrete support. A part of this work still has to be completed.

In the paper, the method of tunnelling using shotcrete is presented in its total context, whereby the important aspects are discussed under the following themes:

Overview of the construction project as a whole with reference to finance, permitting procedures and realisation. Cut-and-cover and mining techniques.

The construction of single-track tunnel and the excavation of large cross-sections, switches and sidings, different geotechnical procedures in dealing with water, especially compressed air operations.

Experiences with contract practices, construction, questions of safety, quality and cost control, damage analysis and comparison of costs.

The work is based on 13 years of application and development.

1. Introduction

In the last few decades, with the greatly increased motorisation of all sectors of the population, the volume of traffic in the large centres of population has grown to such an extent that it is becoming more and more difficult to maintain the flow of traffic in these areas. The chronic overloading of the old city streets, especially in the historic city centre, has led to a lessening of the attractiveness and efficiency of the classical means of public transport and in many places to a permanent collapse of the traffic system. This dilemma can be finally resolved only by taking the public-transport system away from the congested roads, in the case of trams by constructing high-level or underground rail systems.

1.1 The Conceiving of Munich's Rapid-transit Network

In Munich, already at the turn of the century, initiatives were launched to construct an underground rail system. The work on the construction of a high-speed rail tunnel running north-south was started in 1938, but because of the war, only a short stretch was completed.

After the post-war reconstruction of the city in the sixties, there was such a flood of cars that the city was on the verge of a traffic breakdown to be likened to a heart attack. Thus in January 1964, the city council made the fundamental decision to go ahead with the construction of an underground rail system. The construction work was started on 1st February, 1965 at the northern outskirts of the city. Construction has proceeded now a good 20 years. Looking back, it can be safely said that the city council's decision at that time to build an underground system together with the high-speed rail system has made a decisive impact on the traffic conditions and thus has greatly improved the living quality of a large part of the population of Munich.

By creating a second traffic level underground, a permanent solution has been introduced, whose benefits are evident to all. Much progress has been made in easing the traffic problems, while the mobility of members of the public has increased. Today there are an annual 200 million passengers on our modern underground system, i. e. 700000 per working day. This is convincing proof of its efficiency and acceptance by the public.

As in many other large cities, the problem had to be faced, on the one hand, of building in heavily populated districts of the city, and on the other, of catering for the surrounding areas. Thus a concept was adopted which included both an underground and a high-speed rail system (Hochmuth, 1979).

The conurbation centred on Munich, whose present population is 2.3 million and covers an area of 5000 km^2, is served by a high-speed rail system with seven lines of total length of 412 km and 135 stations.

By connecting, by means of a 4-km-long stretch of tunnel, the largely existing railway lines to the west and east of the river Isar to five new high-speed rail stations in the centre of the city, it was possible to link up the whole region to the city centre. Thus this stretch of tunnel had the same function as a section of subway.

The development of the rail-transport system within the city itself is accomplished by means of the subway system of planned total length of over 100 km. It serves a population of 1.3 million. The network (Fig. 1) consists of three main lines and eleven branch lines. The possibility of changing from one line to another is given at nine stations. The subway joins up with the high-speed rail system at a further 12 points, making changing lines attractive. The main character of the subway system is its radial network with several lines running through the centre. In this way it is recognized that the main flow of traffic in Munich has always been into its attractive city centre.

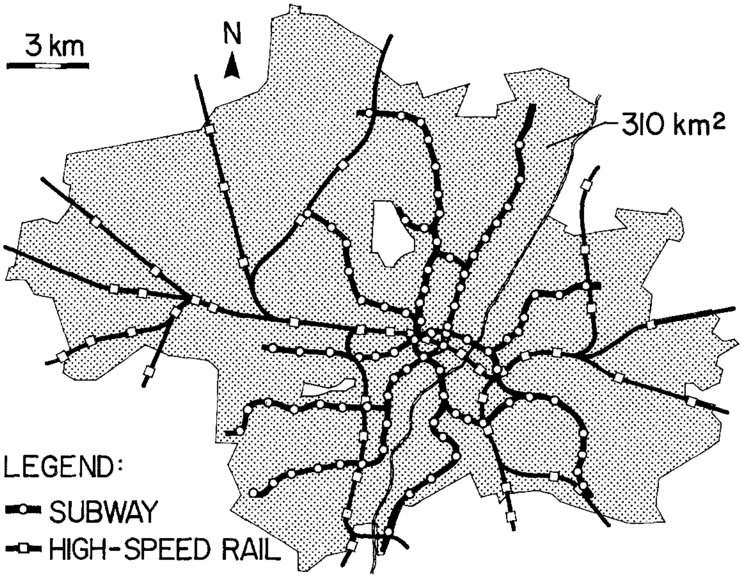


Fig. 1. The subway system of Munich when completed
60 % was by the end of 1985 in operation or in construction

The first 12 km of the subway were opened in 1971 after a period of construction of about 7 years. The opening of the high-speed rail system in 1972 together with six further stretches of subway already provide an efficient local rail system. At the present time 46 km of underground (including service connections) with 50 stations are in operation. A further 17 km of underground and 18 stations are under construction.

1.2 The Framework Conditions

The construction of underground systems in densely populated cities involves, from the planning stage right through construction to the putting into operation, a series of conflicts with private and public interests. Thus underground construction also requires extensive administrative work. Basically all technical, administrative, legal, financial and economic problems are taken care of under the sole direction of the "subway department" of the central administrative authority of the city. The success achieved in the execution and volume of construction work is due largely to the wide responsibilities and competence afforded to this special department.

With three hundred staff this "subway department" has an ideal size. From the beginning it was organized like a private firm in the sense of a municipal-management project. Whereas the sections for general management, legal and building administration as well as finance dealt with the complicated administrative and legal problems, the technical sections for planning, design, construction, operating equipment and architecture, together with the specialist group for geotechnology, dealt with the engineering problems involved in the subway construction.

The annual volume of construction work on the Munich subway reached the 100-million-DM mark for the first time in 1967, then increased to 200 million DM in 1974 and since 1980 lies around 300 million DM. All together, almost 4000 million DM have been spent on the construction up to now (Fig. 2). Approximately 62 % of this amount is for the tunnel excavation and the station structures. Thus this work and the construction methods used are of considerable significance if only because of their financial importance.

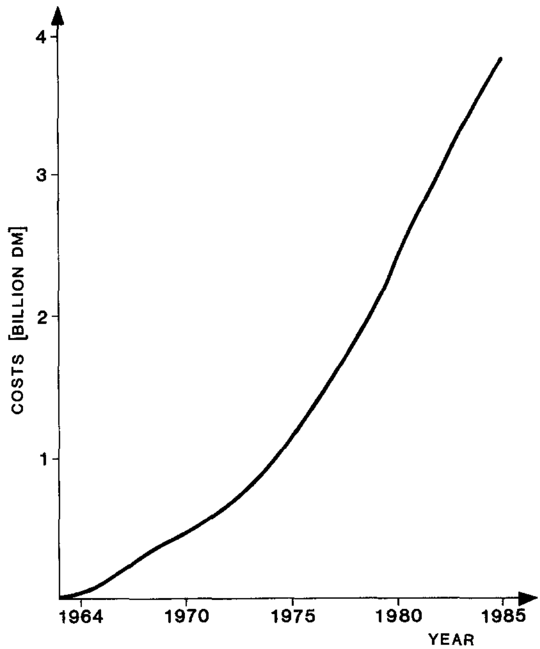


Fig. 2. Expenditure for subway construction in the period 1965—1985

If one considers the importance of a subway project in relation to traffic, economic, social and safety factors as well as the effects on the interests not only of relatively few close-lying properties but of whole residential areas and parts of a city, then it is clear that the planning process and the plan finalisation procedure is considerably more complex and difficult than is the case for a building.

The planning and planning permission are essentially divided into three parts, namely for the stretch, the (detailed) constructional plans and the operation. The finalising of the plans is the main aspect in this multistep process.

From the first stroke of the pen to the opening of a subway line there is a gap in general of 13 years. However, neither the period of time for producing the plans nor the success in having them accepted can be predicted with certainty in advance even today. Delays in this process always result in retarded progress in planning and construction. Thus, for example, in evaluating the advantages and disadvantages of different routes in one section of the diametral east-west line, twelve route variants were investigated; eight variants with four different station locations in the Lehel area of the city were put on the short list. At the same station 18 locations for a lift suitable for handicapped people were also investigated. In this section complaints amongst others by the church authority, which feared settlement damage to the church building, caused a delay of about three years to the completion of the project.

Last but not least, it is precisely for the reasons given above that the further development of subway-construction methods has been pushed ahead by the plan-finalisation procedures that had become more difficult, since a plan-finalisation decision in many cases would be impossible of realisation without this further development. Fig. 3 shows the percentage-wise distribution of the various construction methods in the 63 km of stretch hitherto completed and under construction.

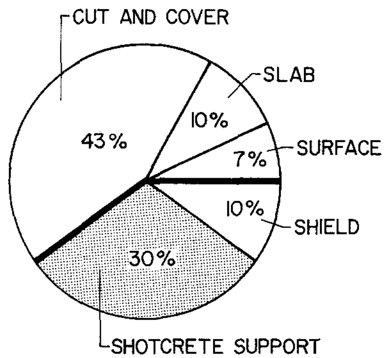


Fig. 3. The share of various construction methods for a length of 63 km lines

Of the 68 stations involved, 6 % are situated above ground. 55 % were constructed by the cut and cover method, 29 % under top cover, and 10 % by the mining method, either by the shotcrete and/or by the shield-driving method.

If one compares the construction methods employed over the course of time (Fig. 4) on the various subway lines, the sharply increased use of the top-cover and mining methods becomes immediately apparent. On the north-south subway line, constructed from 1965 to 1974, only 28 % of the stretch was built by the shield-driving method, and out of 19 stations, only one by the enclosed method and one by the top-cover and mining method.

In the case of the east-west subway, which was started in 1977 and is due for completion in shell in 1987, the share of mining method construction is by contrast already 58 %. Four of the 16 stations have been built by the shotcrete method, and seven by the top-cover method.

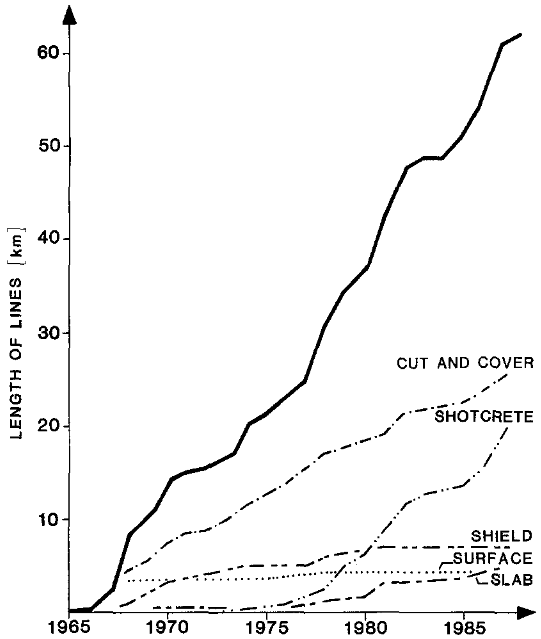


Fig. 4. The share of various construction methods in the period of 1965—1985 involving totally 68 stations and 126 km single tubes (63 km length of lines)

The trend to these construction methods can only partially be explained in connection with the construction of the subway in the inner-city area. It is the increasingly difficult legal problems involved in building a subway that have been a highly significant factor in the increased use of these environmentally less objectionable methods, and thereby in the acceleration of further technical development.

If one looks at the different construction methods under a summation curve, one recognises that the annual increase of the use of the cut and cover method has remained roughly constant. Only in earlier years did above-ground stretches attain significance worthy of mention, as did, however, the shield-driving method also. Top-cover methods, and above all the shotcrete method, have been, on the contrary, continuously on the increase. 1600000 m³ of tunnels have thus been excavated up to now by this method, and been stabilised by means of 140000 m³ of shotcrete.

What was also decisive for this development was the the fact that the construction of the subway that was consistently pushed for two decades in Munich provided the opportunity to optimise the construction methods and adapt them better and better to the hydrogeological conditions, and to risk the step into the technical frontier again and again by accepting alternative proposals.

Thus it was possible not only to master the enormous difficulties of building a subway in the centre of a city, but also to counter the rise in construction prices. In particular, the costs of underground excavations which were initially double those of cut and cover, were able to be reduced (Fig. 5). The trend shows clearly that in the near future, cut and cover methods will be roughly equal in cost to shotcrete methods.

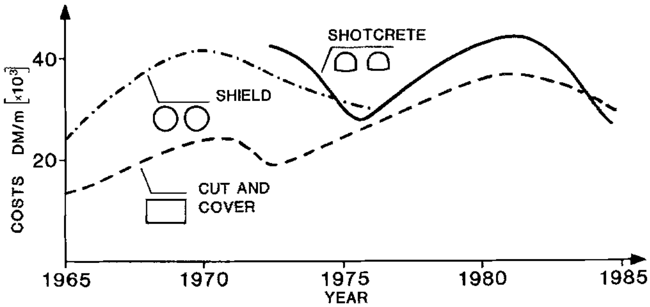


Fig. 5. Cost developments of two track lines (rough construction costs)

Consequently the cost considerations that have otherwise overshadowed other factors in the choice of construction method are now receding somewhat into the background. Events have shown that further development of cut and cover methods alone would never have made it possible to overcome the problems of the construction of the inner-city subway.

1.3 The Subsoil Conditions and the Significance of the Shotcrete Construction Method

For a better understanding of the following remarks, the geological conditions as reported by Soos (1966) and Gebhardt (1977) are briefly summarised. The subsoil in Munich has the following geological and hydrogeological character (Fig. 6):

Quaternary sediments are uppermost; these are composed of thick to very thick layers of interglacial brash of various ages. Their thickness in general lies between 4 and 7 m; in the zone of superimposed lower and upper terraces, it lies between 8 and 25 m. Beneath lie the deposits of soft late-Tertiary sandstone that are known as "Flinz". They constitute a layer package of highly variable composition, made of aquiferous thick to very thick layers of micaceous fine-to-medium sand and of almost impermeable often marly clays of highly variable monaxial compressibility ranging from about 50 kN/m^2 for highly plastic lime-free clays up to 6000 kN/m^2 and more for marl. The mean compressibility lies around 500 kN/m^2. In the multilayered Tertiary there is often a regular sandwich system to be found in the soil layers. The layers have a wavy profile and sudden indentations can show up in erosion trenches. The excavation therefore passes mostly through zones of differing hydrogeological formation, for the dewatering of which it is necessary to take different measures.

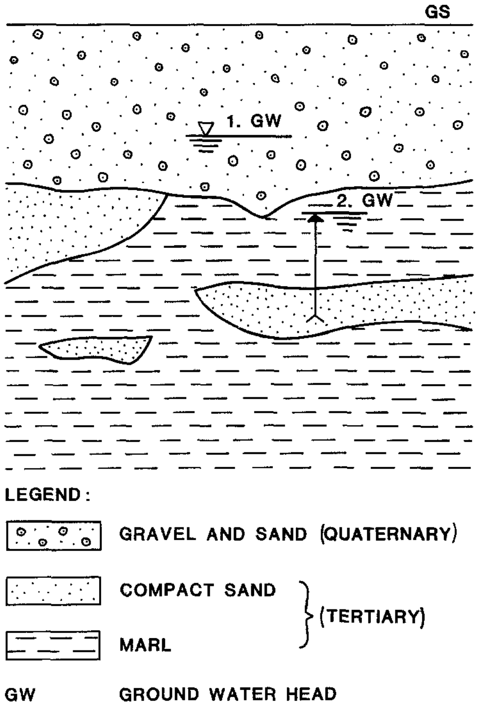


Fig. 6. Schematical representation of the geological situation in Munich

To begin with, for the tunnel stretch, fully mechanised shield driving was used as tunnelling method, with an open rotary cutter. This method had proven itself in a test stretch. At the outset, a double-layer lining was used for the tunnel, with block or spiral tubbing, an inner shell, and in between a lagging of bituminous waterproof sheeting; after the experience of two fires, the lagging was omitted and an inner lining of waterproof concrete was built in.

It was, however, not possible to solve all the problems of the construction of a traffic artery in the midtown area by using the shield-driving methods. With its inflexible cross-section, predicated by mechanical and technical considerations, shield driving is capable of immense forward progress in uniform soil, but it hardly permits additional measures such as dewatering, grouting or freezing of the face, and it is almost impossible with the shield method to excavate complicated traffic structures such as switches, sidings, stations, connecting and stairway tunnels, in short, the basic components of a viable subway network.

Thus it is not surprising that the shotcrete method that was first used in two sections in 1973 and since then has been further developed in Munich with a view to the soils and groundwater conditions prevailing there, has in the meantime become one of the construction methods used most.

Without the use of the modern method, it would not have been possible to construct a number of the major traffic structures of the Munich subway network. To cite a few examples:

The Karlsplatz job of the subway was carried out between 1978 and 1981 beneath a heavily trafficked area working from a shaft in the median strip of the street, almost unnoticed by the public. The tunnels of the stretch lay below the existing traffic facility of Karlsplatz, which has a piled foundation and contains a pedestrian level, an underground garage, various stores and utility areas as well as a rapid transit station. Originally it was planned to integrate the station of the subway into this traffic facility. However, further development of the plans for the line predicated that, in the course of driving the tunnel for the station and stretch structures, it would be necessary to prop up various piles of the existing Stachus structure from the tunnel face. The propping of these piles as well as of the subterranean curtain and of an existing underground garage, and the driving of the tunnels of the stretch and station, and of the connecting passages with differing cross-sections were possible only by means of consistent use of the shotcrete method.

Beneath a subway station that was already in use (Odeonsplatz), a new station that could be excavated only underground was built between 1979 and 1983. In addition, for this job extremely strict requirements regarding settlements were imposed on the tunnel driving because of various historical buildings in poor structural condition; in particular, no additional ground settlement induced by dewatering measures in the upper groundwater level could be tolerated. The combination of compressed-air with the shotcrete method led to success in this case (Weber, 1983; Lessmann et al., 1986). The delicate structures were able to be tunnelled under without damage; the existing structure of the subway network could be propped up from the tunnel face without any reduction of the operation of the subway line and practically without any settlement, and complicated connecting structures such as inclined shafts for stairway tunnels could be built without lowering the groundwater level.

On the Lehel job, a station structure with the connecting tunnel stretches was also excavated by the method of shotcrete under compressed air. Not only the tunnelling under the Isar River could be carried out here without safety problems, in spite of the river structures that had piles driven projecting into the tunnel face, but also that under historical buildings (Fig. 7), as e. g. two churches, including a renowned rococo church, was carried out without any settlement worthy of mention. This is the church that was the starting point of the trouble mentioned above in connection with the final planning stages. Measurements carried out during the excavation work showed settlements of less than 5 mm. The church suffered no damage whatsoever.

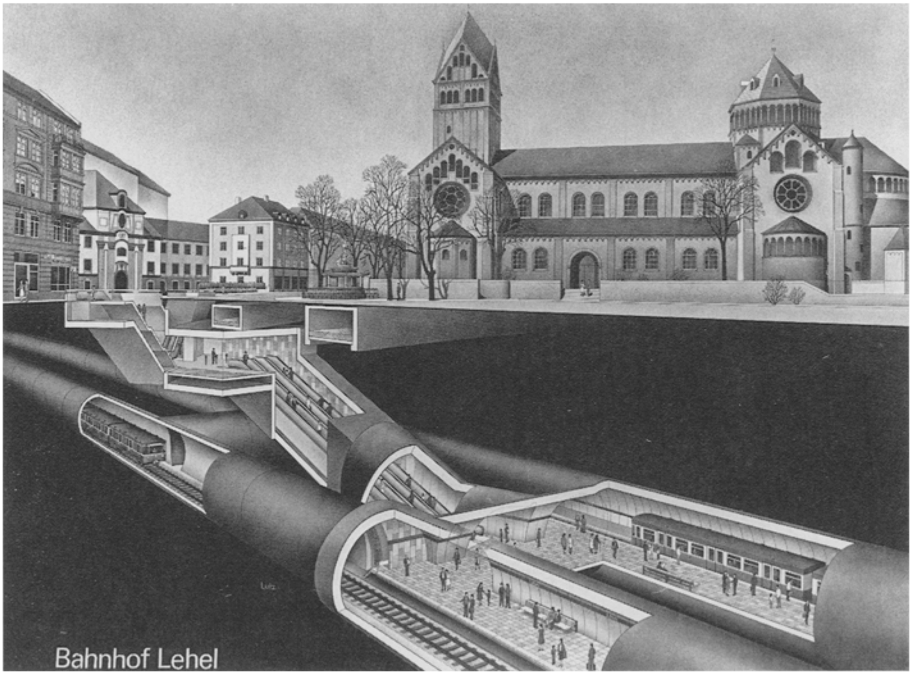


Fig. 7. The church of St. Anne close to the subway station Lehel (shotcrete support using compressed air)

2. The Shotcrete Methods

Why has the shield been replaced by shotcrete in Munich? The first answer to the question is that the lengths of the stretches between the stations are too short for economical use of the shield-driving method, and that, in particular, many different cross-sections have to be excavated, including variable ones.

A glance at the significant tunnel types (Fig. 8) shows the variety of cross-sections that are used for the subway network: for tunnels in the stretches between stations and for the stations themselves, as well as for sidings, turntables, switches and branching.

These variable cross-sections can be excavated with shotcrete, by corresponding widening of the normal cross-section; this is the principal advantage over shield driving. Up to now, all double- and triple-track tunnels not over 150 m² cross-sectional area for switches and sidings have been executed in shotcrete.

Of the 8 platform tunnels built by the mining method, the first was excavated by sheet-pile proofing and multiple addit. The next two, which were costly and difficult, were built in connection with the shield-driving method, and all the rest in shotcrete.

Even for single-track tunnels, shotcrete is more advantageous than the shield, particularly when short lengths are involved, thanks to the lower costs for mechanical equipment.

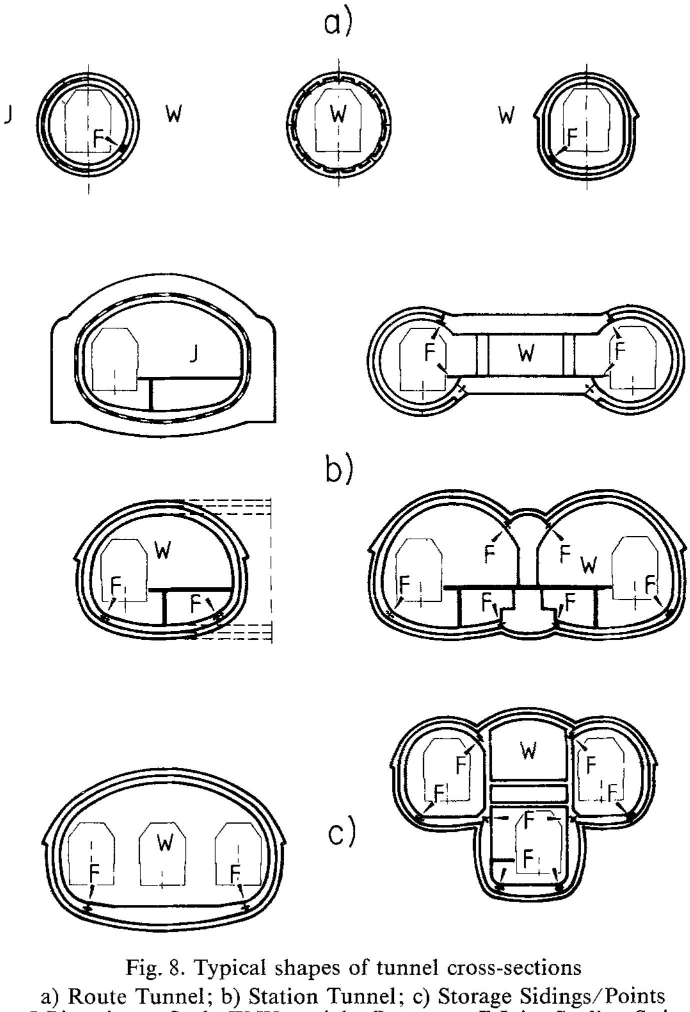


Fig. 8. Typical shapes of tunnel cross-sections
a) Route Tunnel; b) Station Tunnel; c) Storage Sidings/Points
J Bituminous Seal; *W* Watertight Concrete; *F* Joint Sealing Strip

The same is also evident from Table 1: for the 6 shield jobs the tunnelled length amounted to 12.8 km, i. e. an average of 2.1 km per job; for the 27 shotcrete jobs, the tunnelled length was 34 km, corresponding to a mean value of 1.3 km per job.

A further advantage of the shotcrete methods lies in the fact that they function in both Tertiary and Quaternary soils, and that it is possible to take additional measures at the face, primarily additional lowerings of the

Table 1

Type of Cross-section	Excavated section (m^2)	Length of tunnel (m)
One track shield tunnelling	38	12800
One track shotcrete support	37	34000
One track tunnel:		$\Sigma = 46800$
Two and three-track tunnels using shotcrete support	80—150	2590
Cross-sectional area of stations	150	210
	80	1150
	176	120
	130	270
Stations/tunnels with two or three tracks:		$\Sigma = 4340$

groundwater level. The dewatering is of the greatest importance, since the tunnels are located predominantly in groundwater-bearing soil (Fig. 6).

2.1 Measures Against Groundwater

The excavations have to pass through soil layers with various groundwater levels, for the dewatering of which different steps have to be taken (Weber, 1983).

As described in Section 1.3, the uppermost soil layers in Munich are Quaternary sediments (Fig. 6). These are composed of thick to very thick highly permeable limestone brash. As already mentioned, they have a thickness of 4 to 7 m, and in the southern periphery up to 25 m. The groundwater table therein is from 3 to 10 m below the surface, in the southern periphery up to 15 m. The permeability coefficient of the brash is not less than 0.001 m/sec.

The Tertiary sand and marl layers under the Quaternary (Fig. 6) form the floor of the upper groundwater level. The sand layers can be a few decimeters thin, or can attain thicknesses of up to 10 m. They have a low permeability of $\approx 10^{-5}$ m/sec. The permeability coefficient of the marl is $\leq 10^{-8}$ m/sec.

The sands that are enclosed by marl always contain confined groundwater, and thus constitute the lower groundwater levels.

Figs. 9—12 show 4 cases of dewatering measures taken in a tunnel location in the uppermost groundwater level (1st GW), viz. in two cases without and two cases with compressed air. Fig. 9a shows deep groundwater in the first level. Because of the large volumes and the great distance involved, lowering of the groundwater table would not be possible here. For this reason, water is kept out with waterproof walls that extend into the impermeable layers and form with the latter a trough that then gets pumped empty. The remaining water in the bottom area is pumped out of the tunnel with vacuum lances.

In Fig. 9b, the groundwater depth of the 1st GW is low in the Quaternary, and the tunnel lies in Tertiary sand; thus the lowering of the 1st GW can be carried out with exterior wells. It is most effective when the sand layer extends under the bottom of the tunnel. However, if the upper surface of the backed-up water lies in the tunnel zone (as shown in the illustration), then it is impossible to achieve sufficient lowering with outer wells on account of the irregular contours of the soil. A certain amount of water remains that has to be removed from the face by means of vacuum lances. The sand lens enclosed in marl underneath the tunnel profile ordinarily contains confined water having the piezometric height of the 2nd GW. This water is depressurised with the same wells.

Driving with compressed air (Weber, 1983) in the Quaternary gravel is possible only with additional bentonite-cement grouting to keep the air losses within bounds (Fig. 10a).

Fig. 10b shows the tunnel completely in Tertiary sand. Here the excavation can be carried out with compressed air without any additional grouting.

In both cases, the air pressure must be kept to that of the water surface of the first level (1st GW).

In the thick gravel of the city's southern periphery, the gradient, where possible, is laid so high that the tunnels do not enter into the groundwater, or only partially so. The earth cover of the tunnels is then often only a few metres.

When the tunnel is located under the first groundwater level in the Tertiary marl, then two solutions present themselves. In the first case (Fig. 11a), if the marl cover is thin, then the groundwater of the upper (1st GW) as well as of the lower (2nd GW) level is lowered.

In the second case, only the sands lying under the marl are drained and depressurised, and the upper groundwater level is tunnelled under without being lowered. This is the solution that is ordinarily sought after and carried out. The prerequisite for it is a sufficient groundwater-bearing marl covering layer having a thickness of 2 ½ m. If the marl cover is thinner (inset 11b), then it is brought up to the necessary thickness by grouting or freezing.

Driving at the boundary layer between Quaternary and Tertiary with thin marl cover has proven to be costly and risky. On the otherhand, if compressed air is used, these zones can be tunnelled through relatively free of problems. If only a thin marl layer exists above the tunnel roof, then the air pressure P_i is chosen corresponding to the water level 1st GW (Fig. 12a). If sufficiently thick marl covering is present, the interior pressure P_i is chosen to correspond to the piezometric pressure in the lower sand layers (2nd GW).

The case most frequently encountered in the use of compressed air is shown in Fig. 12b. Driving under compressed air is carried out with supplementary lowering of the groundwater in the lower level (2nd GW).

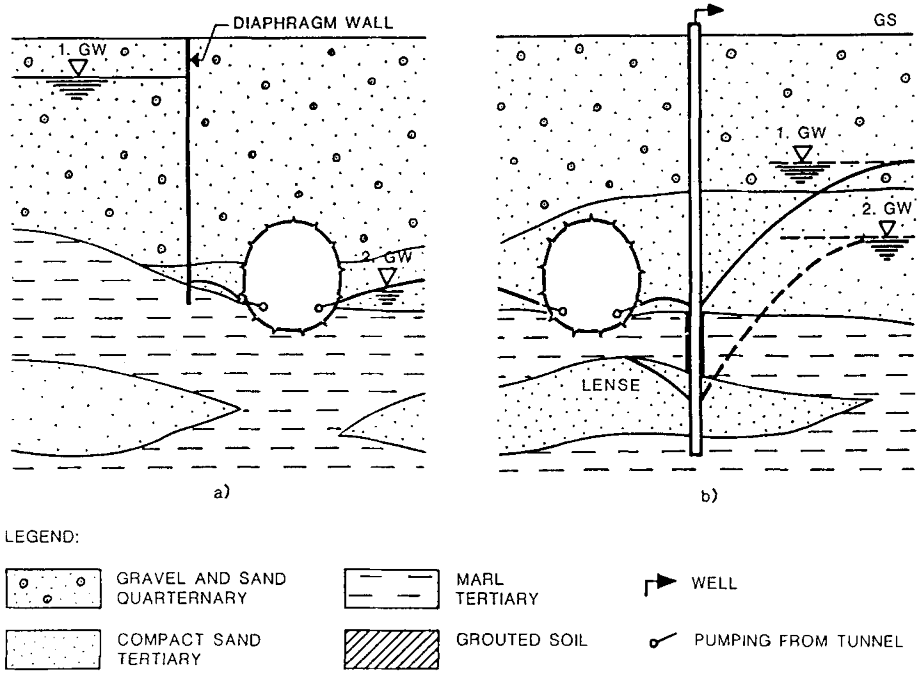


Fig. 9. Tunelling in first ground water regime (1. GW)
a) Dewatering of a trough between diaphragm walls; b) Lowering of 1. GW and destressing of the sand bars (2. GW) beneath the tunnel

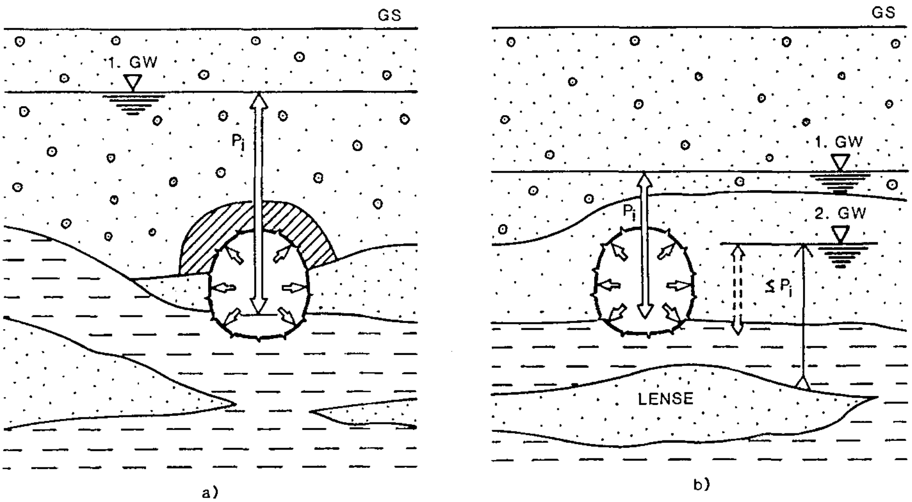


Fig. 10. Tunnelling in first ground water regime (1. GW), using compressed air
a) Grouting of the gravel against piping
b) Tunnelling without grouting of the dense sands

This is necessary in the lower tunnel locations and with increasing water pressure, because the overpressure of the air is kept as low as possible, generally under 1 bar (Weber, 1983). Moreover, the depressurising wells make possible in tightly enclosed sand lenses a flow process that is indispensible for pushing back the groundwater.

2.2 Support Measures

Certain standard designs have been developed for supporting the tunnels. The principal elements of the outer shell are:

- steel ribs
- reinforced shotcrete
- liner plates
- bars and system anchors

Double-flange beams or corresponding channel profiles or even lattice girders are used as steel ribs. The distance between successive steel ribs is 1 m as a rule, corresponding to the excavating cycle.

In the case of well dewatered Tertiary, one can dispense with the steel ribs in the stepped-face zone. They are always built into the roof section of a tunnel as immediate protection. Provision is made for connecting up with ribs at the springings.

Lattice beams (Baumann and Betzle, 1984) have been in use for only a few years. They are preferred when driving with compressed air. Their advantages are lower weight, smaller spray shadows and thereby a better bond with the shotcrete, and also an improved waterproofing of the shotcrete lining. This is particularly important for the time between the lowering of the air pressure and the installation of the inner lining. Up to now triangulated girders have been employed exclusively.

The thickness of the shotcrete lining in the single-track tunnels is at least 15 cm, in the large cross-sections 20 to 30 cm. Concrete quality B 25 is required, having an early strength after 6 hours of 5 N/mm^2. The reinforcement is carried out with reinforced-concrete mats: generally single-layer and on the outside in the case of single-track cross-sections, and on the inside and outside in the case of larger cross-sections. The amount of reinforcement lies between 20 and 30 kg/m^3.

These support measures are the result of a further development (Lessmann, 1978) that is based on continuous experience and a program of selected measurements. Thus the first single-track tunnels (Fig. 13), for example, were executed with an outer shell 20 cm thick and with much stronger steel ribs, which were placed in the tunnel-roof and the springings. In addition, there were 7 system anchors per metre. Since then, material savings of from 30 % to 70 % have been achieved with the individual support elements. In particular, it turned out that the time-consuming anchors could be dispensed with. Nowadays anchors are no longer used even on the large cross-sections, albeit still in special cases, e. g. in groins.

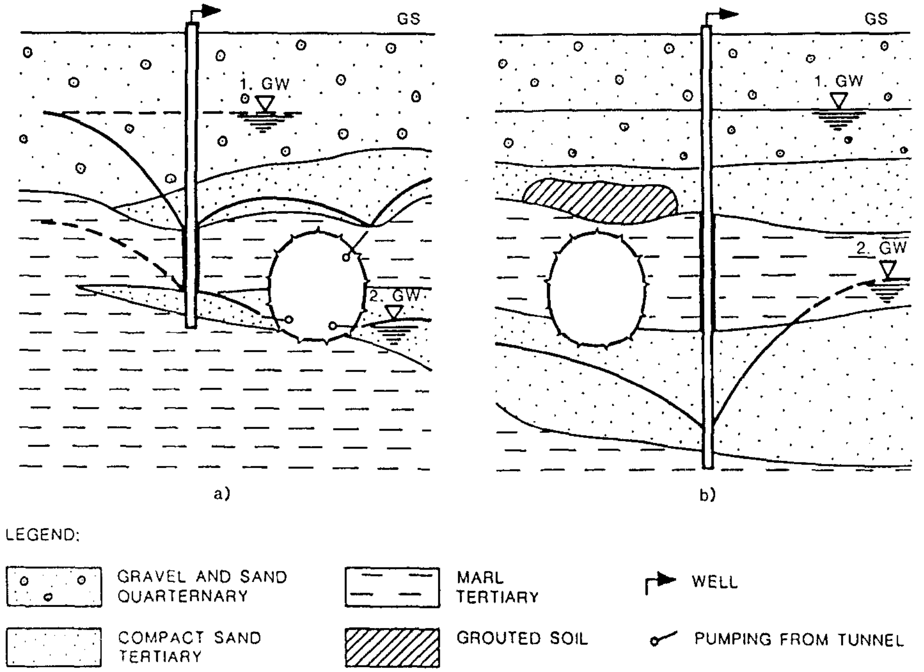


Fig. 11. Tunnelling in the second ground water regime (2. GW) having a thin layer of marl over the roof

a) Lowering of both ground water levels

b) Grout cover or freezing in the roof, decompression of artesian water in the lower sands (2. GW)

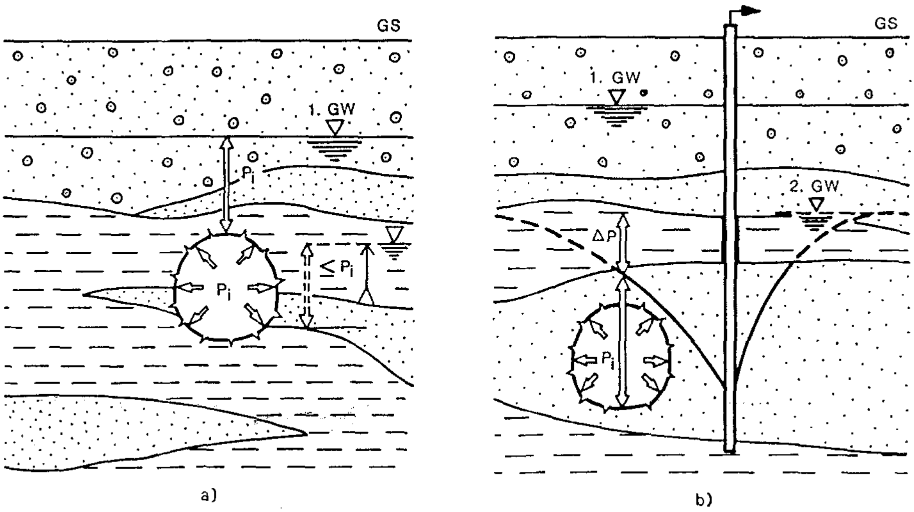
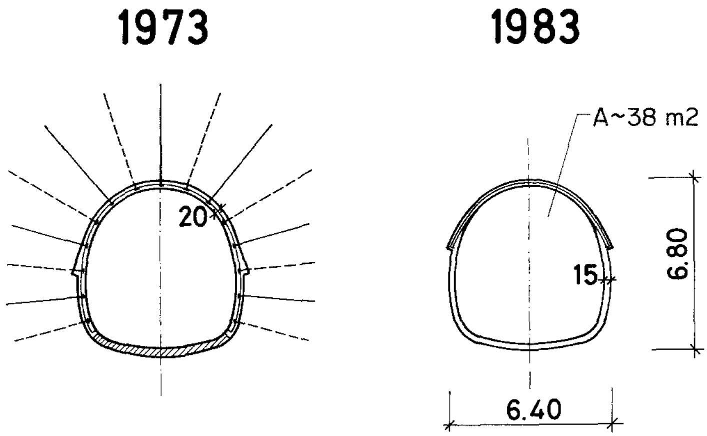


Fig. 12. Tunnelling in the second ground water regime (2. GW) using compressed air

a) in shallow tunnels P_i according to 1. GW head

b) in deep tunnels decompression of 2. GW combined with air pressure up to $P_i = 1$ bar

In the large cross-sections, the form of the partial drifts was improved, and for tunnels up to about 90 m² cross-section, so-called two-part driving was developed in place of the original three-part drive



YEAR	STEEL RIBS	SHOTCRETE	REINFORCEMENT	ANCHORS
1973	300 kg/m	d=20 cm	122 kg/m	7 p.m
1983	175 kg/m	d=15 cm	80 kg/m	—

Fig. 13. Support measures in 1-track tunnels as applied in 1973 and in 1983

(Fig. 14). Liner plates are necessary as additional shoring when the standup time of the soil is too short to apply shotcrete shoring and let it become effective.

This is the case:

- in principle in the Quaternary brash
- fully or partially in the roof zones in Tertiary sands and in fissured and dying-out marl layers, which are encountered mostly in the transition zone between Quaternary and Tertiary
- in the vicinity of artificially disturbed zones, e. g. near wells or exploratory drill holes

The plates are installed in advance in the Quaternary, whereas in the Tertiary, on account of the high ram resistance, it is usually possible to bring along the plates only step by step. For the support of the roof in Tertiary deposits, often "bars" (or pipes) are also used instead of liner plates.

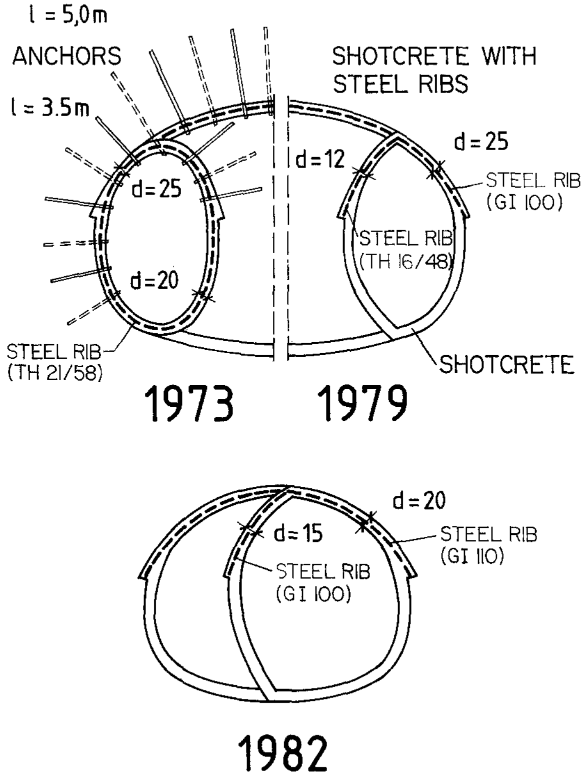


Fig. 14. Methods of excavation and the associated support measures in large tunnel sections. Evolution since 1973

2.3 The Excavation Procedure

2.3.1 Single-Track Tunnel in Tertiary Soil

In the Tertiary marls and sands, two different excavation procedures are used. Fig. 15 shows full-section tunnelling with the stepped-face method. The roof section precedes the excavation of the bench and the bottom by only a small margin (max. 4.0 m). Thus the time for closing the ring is short. Mucking out is done with an hydraulic digger or even with a milling loader.

While the roof section is always supported at one-metre intervals, an interval of 2 metres can be employed for the bench and the bottom when the soil conditions are good. The distance *P* (for pier) between the individual tunnels is generally kept low (*P* not greater than 6 m), in order to remain in the street area and not place demands on privately owned land. During excavation of the tunnel, the influence of the driving of the neighbouring tunnel is always kept in mind, and rules regarding the sequence of tunnelling operations have to be laid down.

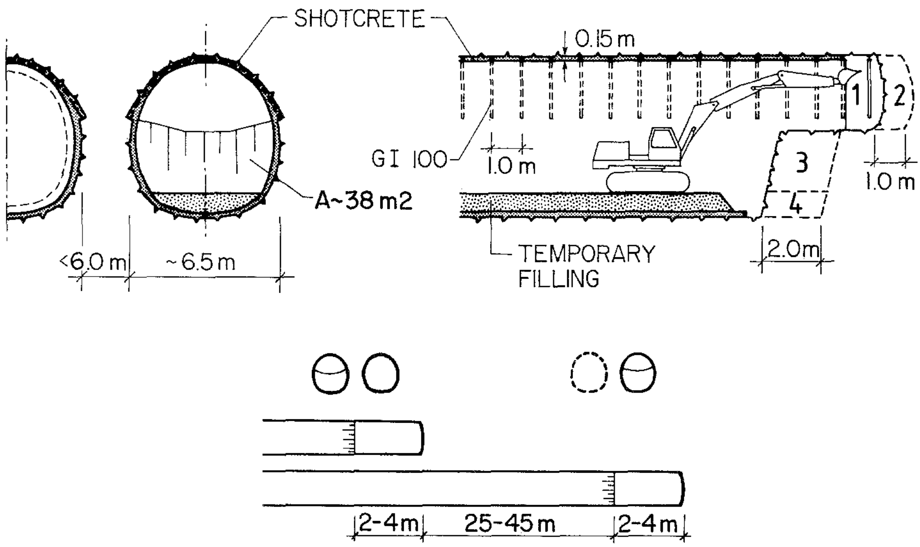


Fig. 15. Method of excavation in the Tertiary with a stepped face

For example, it has been shown that when the distance between the tunnels is small, the settlements increase markedly, but that staggered simultaneous tunnelling leads to less settlement than parallel simultaneous tunnelling does (Weber, 1979).

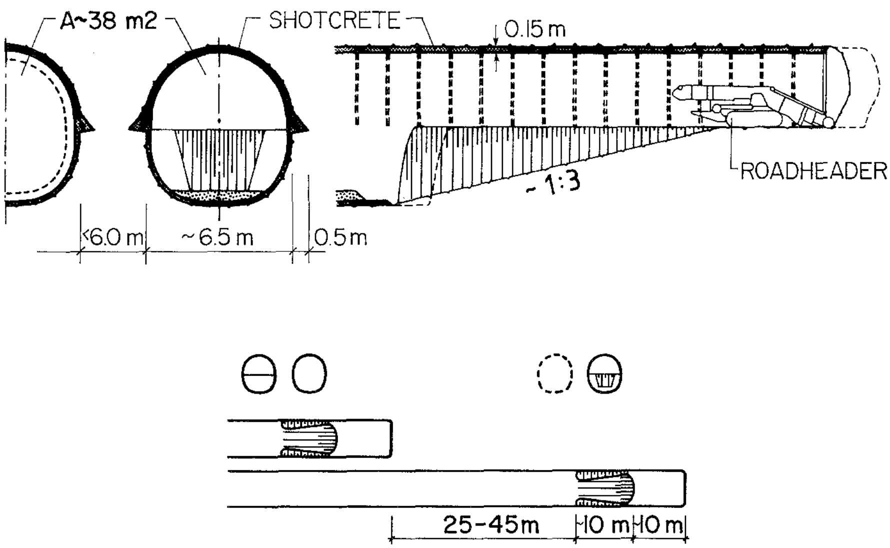


Fig. 16. Method of excavation in the Tertiary with a ramp to the heading

In the variant known as "ramp construction method" (Fig. 16), mucking out is done with a road header that serves alternately via a ramp for the roof and bench sections. Shotcrete base beams at the springings form the foundation in the ramp area and compensate for the retarded ring closure.

To make up for insufficient marl cover, e. g. when breaking through from below, or in erosion trenches, cover grouting or freezing is used. Where possible, this is carried out from the surface or from grouting shafts. In other cases, grouting or liquid-nitrogen freezing is carried out section-wise from the face, as shown in Fig. 17. Since chemical grouting is no longer usable on account of ecological considerations, frozen zones are created instead nowadays. With compressed air, these critical areas can be mastered without freezing (Weber, 1983).

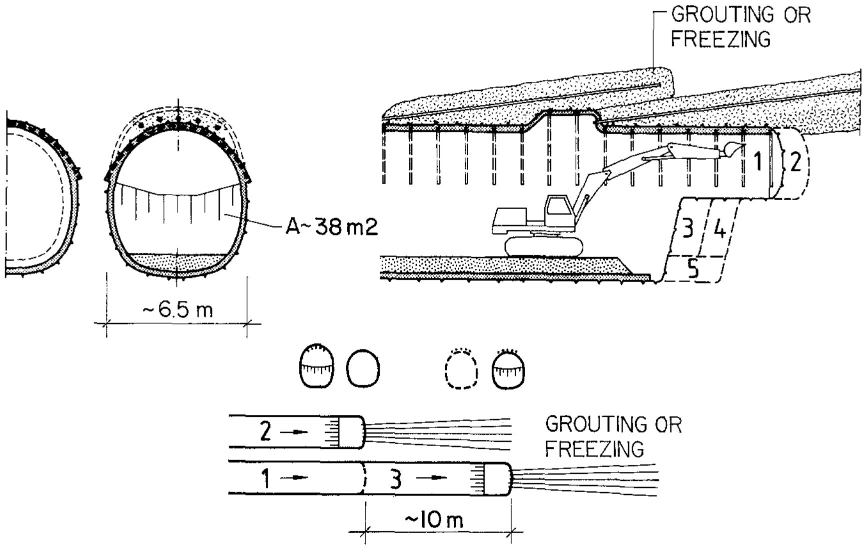


Fig. 17. Tunnelling in the transition zone between Tertiary and Quaternary using grouting of freezing technique

The use of diesel-powered equipment is not permissible when compressed air is used. For this reason, all working equipment, with the exception of air-powered spraying equipment and small apparatus, is electric-powered. Transportation is by rail, with battery-powered locomotives. The principle advantages of compressed air are reduction of settlement by supporting the working face, and an increase in safety when driving in critical areas.

On the other hand, lowering the air pressure upon completion of the driving places heavy demands on the bearing capacity and waterproofing on account of the immediate loading of the shotcrete exterior shell by the groundwater. From an economical and statical point of view, a choice must be made as to whether the outer shell should be designed for the full water pressure or for reduced water pressure combined with partial lowering of the pressure until the inner shell has been built in. The air pressure is lowered in stages (of approx. 0.1 bar per hour) while the shell is kept under continuous surveillance for water leaks and possible damage. It is purposeful to clear up and fill up the carriageway base while the air is still under pressure, in order to be able to maintain observation of the base.

2.3.2 Single-Track Tunnel in Quaternary Soil

With the construction of stretches in the city periphery, where the higher-lying tunnels in the brash level may possibly lie above the groundwater, the share of driving in Quaternary has risen. After the completion of the tunnels currently under construction about ⅓ of all the shotcrete tunnels lie in the Quaternary. The excavation of the roof section and of the bench is staggered in both space and time. Forepoling is carried out only in the roof section. Grouting with bentonite cement is carried out from the floor of the roof section to bond together any loose-gravel layers in the bench zone (Fig. 18). When the grouting is successful, then steel ribs are installed only at 2-metre intervals in the subsequent excavation of the bench. If the grouting is of doubtful success, then rib is built in every metre in the bench, and also forepoling where considered necessary. In special cases (e. g. under buildings or canals), a temporary invert arch in the roof-section is used in addition. Particular problems arise in forepoling work in the roof section whenever conglomerate formations are encountered.

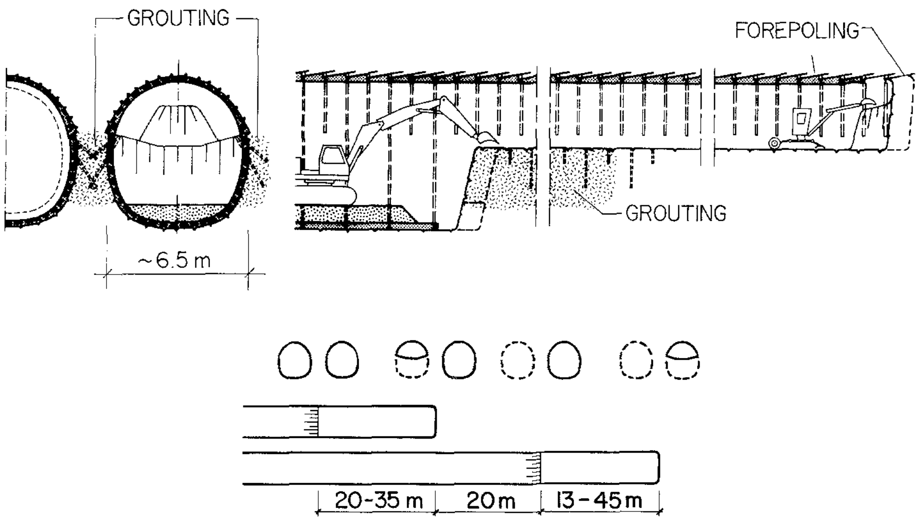


Fig. 18. Head and bench method in the quaternary gravels with free-running grouting from the head

Not only natural but also artificial obstacles have to be reckoned with when tunnels lie so high. Thus on one job, not only old dumpings of brick rubble and pieces of wood, but also the foundation pilings of an underground garage had to be passed through.

Fig. 19 shows the foundation pilings of 60 cm diam. that were exposed by the excavation. They carried a load of about 400 kN. These loads could still be taken up with the outer lining that was reinforced with additional steel ribs. (When the piles were cut through, roof movements of about 5 mm were measured.)

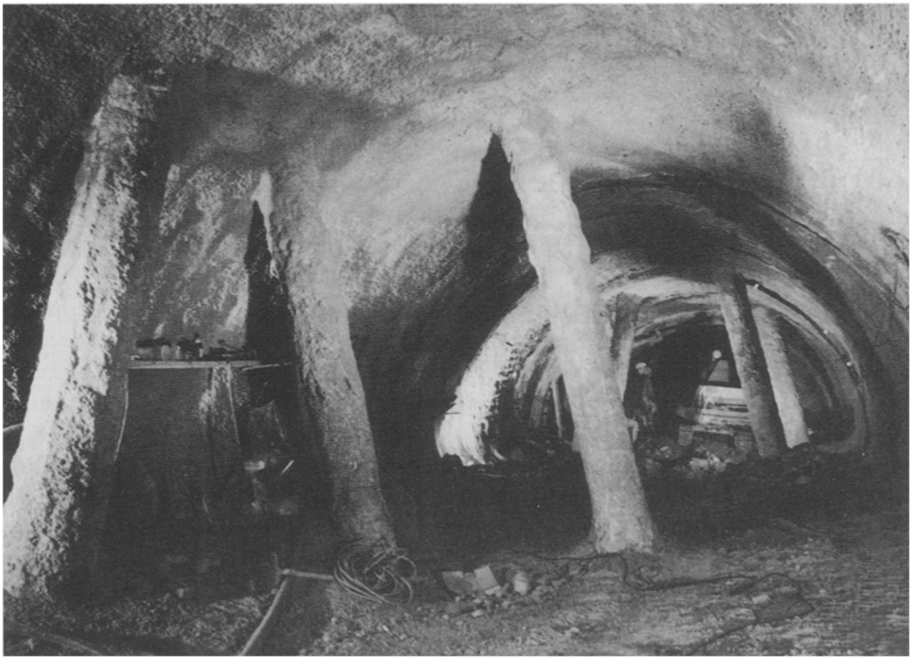


Fig. 19. Tunnel with shotcrete lining traversing the pile group of an underground car park

2.3.3 The Multiple-Track Tunnels and the Platform Tunnels

Excavation of tunnels having a large cross-section is carried out in principle with multiple drifts (Krischke and Weber, 1981). Excavation and support are similar to those in single-track tunnels. There are several reasons for excavating large tunnels with multiple drifts:

- the standup time of the soil places a limit on the size of the excavation cross-section
- in many cases, adequate dewatering can be achieved only for the smaller partial cross-sections and out of them
- the multiple drifts and the purposeful choice of their excavating sequence make it possible to form wide shallow settlement basins step by step.

Up to now, large profiles (A = 75 to 150 m^2) were tunnelled in many cases in "three sections" (Fig. 20). First the two side drifts are driven and fully lined with a shotcrete lining. Then the roof section is driven and the core is excavated, whereby the inner walls of the side tunnels are taken away again. (The choice of the excavating equipment has an influence on the size of the partial drifts. With their cross-sectional area of from 20 to 35 m², they are almost as large as single-track tunnels.)

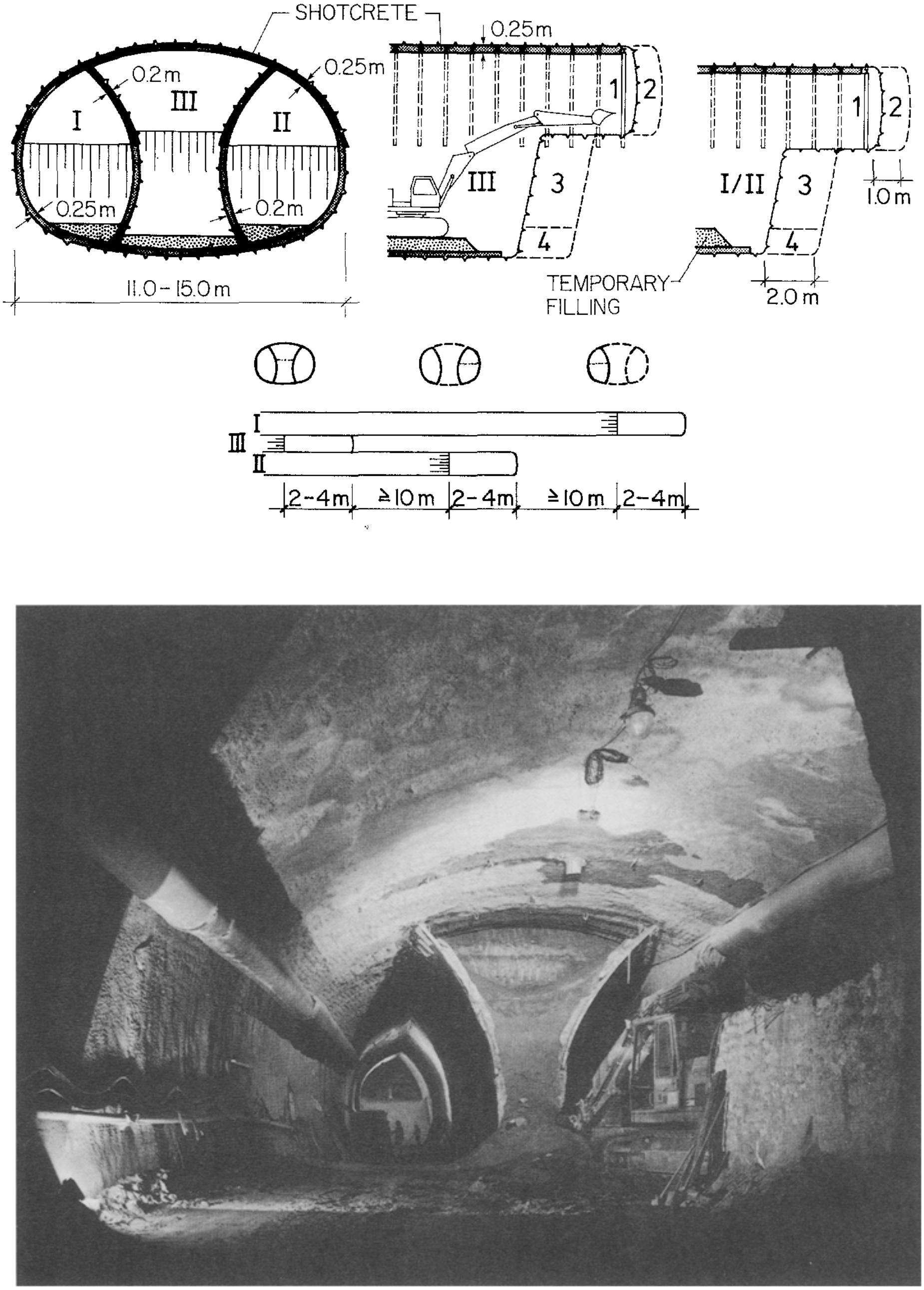


Fig. 20. Side drift method for large tunnels ($A = 75$—150 m²) in the Tertiary

In the case of tunnels of up to 90 m² cross-section and with good soil conditions, a two step excavation has also performed well (Fig. 21). This has only one preceding side tunnel and one that follows after at an interval of at least 10 m.

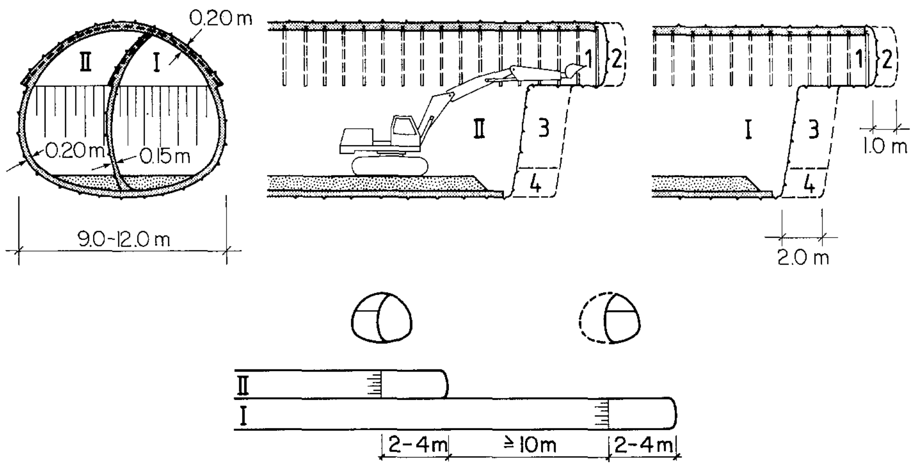


Fig. 21. Construction method for 2-track tunnels and stations with smaller cross-section ($A=50$—$90\ m^2$) in the Tertiary

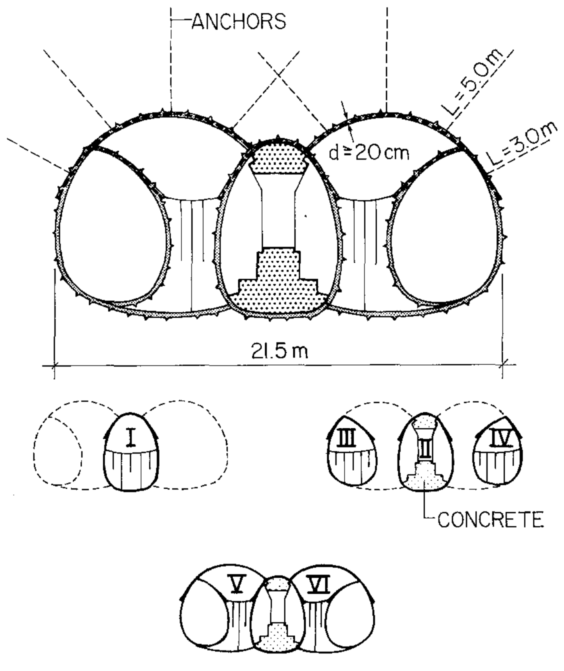


Fig. 22. Sequence of excavations for a station with a cross-sectional area of $A=176\ m^2$ using shotcrete support

An example of the construction of a two-cell cross-section of 176 m^2 excavation surface, with partial use of final construction, is the station tunnel with a platform in the middle (Fig. 22). Here a large middle tunnel of 44 m^2 (I) was constructed first. The later support construction (floor beams, supports, roof-ridge beams) was built into this. At the same time, two side tunnels (III, IV) were driven, each having an excavated cross-section of 32 m^2. Between the side tunnels and the central supports, the two roof sections (V, VI) were excavated, and the remaining excavation of the cores, each of which was 34 m^2.

2.4 Construction of the Inner Lining

After the emplacement of the outer lining, all tunnels acquire a reinforced-concrete shell of waterproof concrete (Krischke, 1982). With

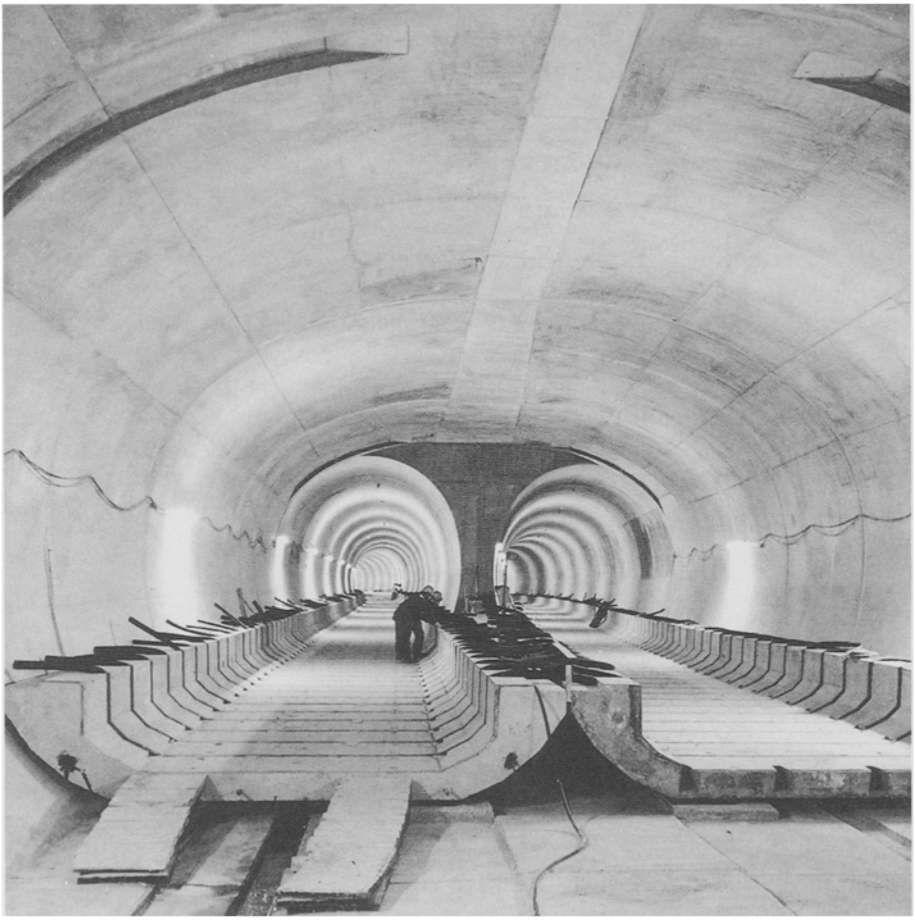


Fig. 23. Two-track tunnel with final lining showing the elastically supported rail troughs against noise propagation

large tunnel cross-sections, the inner shell is introduced in two parts: the invert arch, and then the vault. In the case of single-track tunnels, concreting is carried out in a single pour with fullround shuttering.

The inner lining is constructed in blocks of 10 m and is concreted directly onto the outer lining. In spite of the direct connection with the rough outer lining, corresponding concreting technology must be used to attain sufficient waterproofing. Nevertheless, isolated wet spots have to be reckoned with. These have to be made watertight later on by means of grouting.

In the case of multi-track cross-sections, the inner lining is constructed about 50 cm thick (the reinforcement content is about 65 kg per m^3 of concrete); in the single-track tunnels, the thickness is about 35 cm (the reinforcement runs to 40 kg steel per m^3 concrete, in the case of higher situated tunnels about 60 kg steel per m^3). Fig. 23 shows a completed double-track tunnel, and as continuation, two single-track tunnels each with an inner lining, and with elastically supported prefabricated track channels. The latter effect a decoupling of the track superstructure of the tunnel construction and protect buildings above or nearby from the vibrations of the ensuing operation of the subway. These have also become a constructive prerequisite for the acceptance of the construction of a subway line by the population.

3. Evaluation of Practical Experience with the Shotcrete Method

In this section, the practical experience gained from the planning and execution of subway lines using the shotcrete method will be given a closer look. This experience is founded on 12 years of continuous use, in the course of which 34 km of single-track and 4 km of multiple-track tunnel and station tunnels were constructed. The great extent of the work, which was carried out under difficult but largely comparable geological and hydrogeological conditions, justifies statistical support of the following statements:

3.1 Tender and Award, Alternative Proposals

On the basis of the authorisation for the construction of the subway, the Committee for the Subway (U-Bahn-Referat) draws up the proposed tender and the list of prices for the individual jobs. These tender documents are highly detailed in regard to the structures, the construction methods and the sequence of construction operations including consideration of traffic needs. Time-consuming and costly replanning after the award can thus be avoided in most cases, and alternative proposals can be judged more accurately and speedily.

Tendering must be open and public. Participation of unqualified bidders is relatively rare. Exclusion of such bidders in favorable price situations has not led to any difficulties up to now.

Alternative proposals and partial alternative proposals are admissible in principle, so long as the bidder also submits a main bid on the basis of the official design. Nevertheless, they should contain a technical improvement and be elaborated to the extent that they can be unequivocally evaluated both technically and economically. Bogus alternative proposals that just boil down to a quantity reduction in the official proposal are dismissed out of hand. The positive attitude of the awarding authority toward alternative proposals promotes the readiness of the contractor for innovation. Limitation of risk must be added, acceptable to both parties, as well as a corresponding contractual regulation to make the risks involved plain and clear. The first time a new construction method is used, a well-understood qualified cooperation between the contractor and the awarding authority is necessary. This applies especially to questions of safety. In the case of tunnel construction within heavily built-over urban areas, the estimation of risks and forward-looking engineering thinking on the side of contractor and owner are indispensible and take precedence over economic considerations. In this way, out of an innovative idea via technical examination and practical testing, with possible modifications, a new technical standard can be achieved.

Fig. 24 shows the economical result of such a development strikingly with the comparison of the tunnel measurements of 1968 and 1973. In 1973, an alternative proposal for a platform tunnel with a 25-cm-thick shotcrete shell was accepted for the first time. The award was made with the stipulation that the construction method first be tested in a 60-m-long test stretch that was available. By the use of the shotcrete construction method, the ratio of the excavated to the required cross-section could be reduced from 2.3 to 1.3.

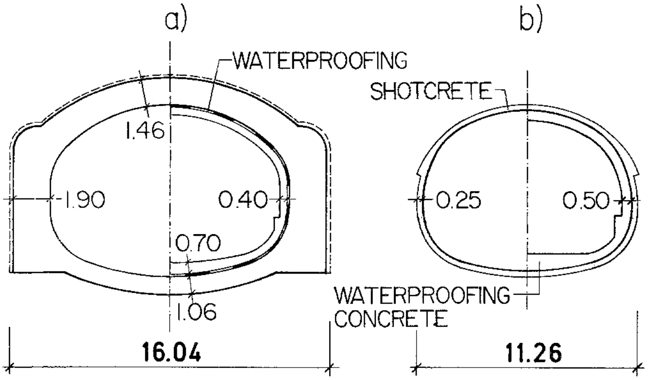


Fig. 24. Conventional and advanced design for two-track tunnels
a) Construction in 1968 involving 150 m² excavated surface
b) Construction in 1979 involving only 75 m² excavated surface

3.2 Checking the Quality of the Shotcrete Shell

When tunnels are excavated with the use of shotcrete, the development of the early strength of the shotcrete is a decisive factor, because the excavation cycle and the attainable excavation speeds are significantly influenced by it.

At the same time, the shotcrete has to attain a sufficient 28-day strength, as it often constitutes the only support of the excavated space for a period of a year and more, until the inner lining is built in. Up to now, the dry-spraying process has been employed exclusively. It turned out in the course of examining various concrete recipes for the manufacture of shotcrete that the requirements of high early strength and high 28-day strength are, as a rule, difficult to combine. The concrete recipes that demonstrate early strength development often bring little hardening thereafter (Fig. 25). On the other hand, in the case of mixtures with a slow increase of strength in the first hours, there is usually very good subsequent hardening. The uni-axial 28-day compressive strength of Mixture *B* (350 kg/m³ PZ 45 F, 6 % Fluresit accelerator for rapid hardening) is 25 % less than that of Mixture *A* (370 kg/m³ Portland blast-furnace cement 35 L, 4 % MC—Sprayaid accelerator for slow hardening). It is remarkable in this regard that the strength development is influenced not only by the kind of cement and the strength class, the kind of accelerator and the amount of additive. With the same kind and amount of accelerator, for example, cements of the same type and hardness class, but from different factories, often yield very different strength results.

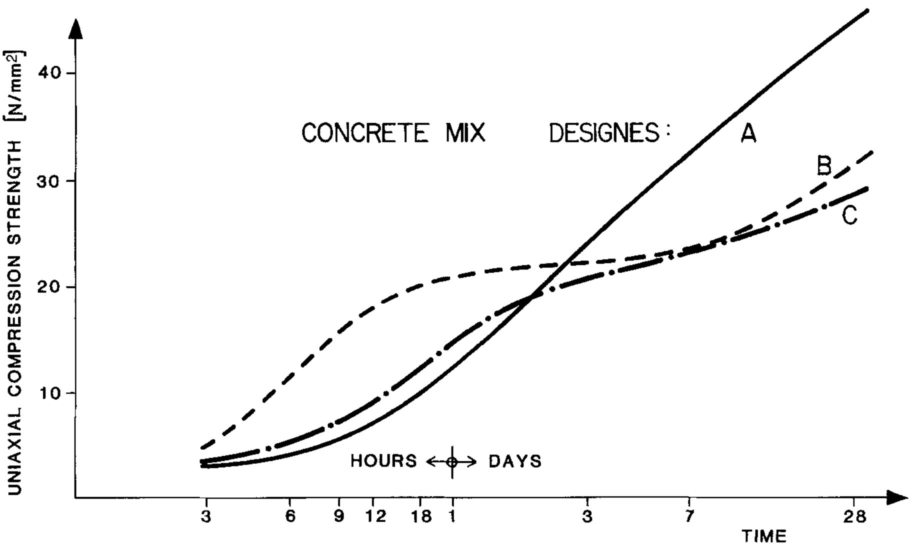


Fig. 25. Shotcrete lining. Mean values of the uniaxial compression strength for different mixtures
Mixture *A*: HOZ 35 L, 370 kg/m³, Injection Aid 4 % of cement content (kg weight)
Mixture *B*: PZ 45 F, 350 kg/m³, Fluresit 6 % of cement content (kg weight)
Mixture *C*: PZ 35 F, 380 kg/m³, Guttacrete 3 % of cement content (kg weight)

For the shotcrete recipes *B* (v. above) and *C* (with 380 kg/m^3 PC 35 F, 3 % Guttacrete accelerator), that were employed on five jobs on the Munich Subway, extensive test results are available, so that statistical evaluation of the strengths becomes possible. The statistics shown in Fig. 26 are based on an average of about 450 individual values for every time step. The development of strength in the first hours was determined here with the Kaindl-Meyco test apparatus. For the values over 3 days, the tests were carried out on bored cores. The confidence interval is shown within which 90 % of all test results lie in the statistical mean. It may be seen here that the choice of a mixture with faster strength development is paid for with a certain increase in the mean variation, primarily in the early strength.

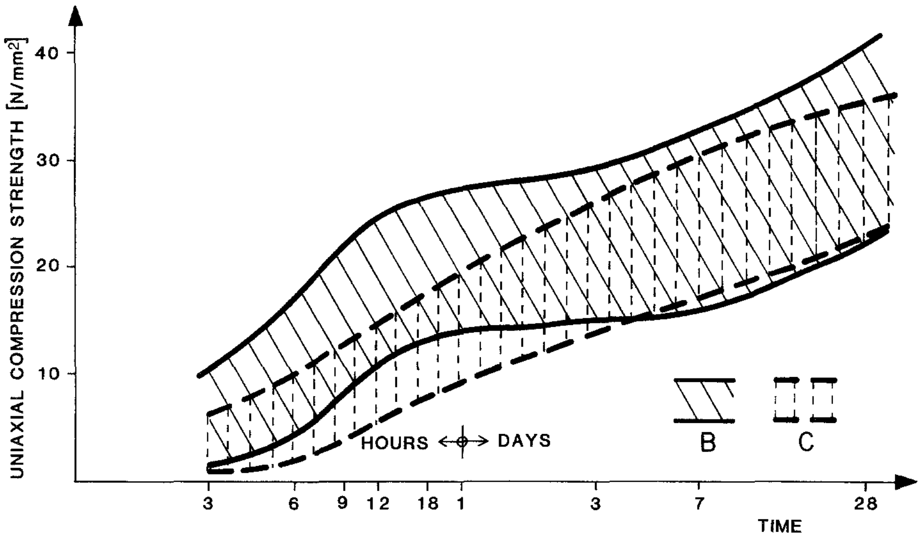


Fig. 26. Shotcrete lining. 90 % confidence limits for the mixtures *A* and *B*

3.3 Safety Considerations

No one will dispute that tunnel excavation in heavily built-over urban areas, as is ordinarily the case in the construction of a subway, is work that is fraught with dangers and that inexorably entails problems of safety. Several years ago, following several accidents, all efforts were made (with success) to raise the safety standard as much as possible. The starting point for this was careful accident analysis to determine causes of accidents, to develop preventive measures and to avoid repeat cases.

In addition to human error, a series of special external circumstances can be regarded as dangerous. Such situations are shown schematically in Fig. 27.

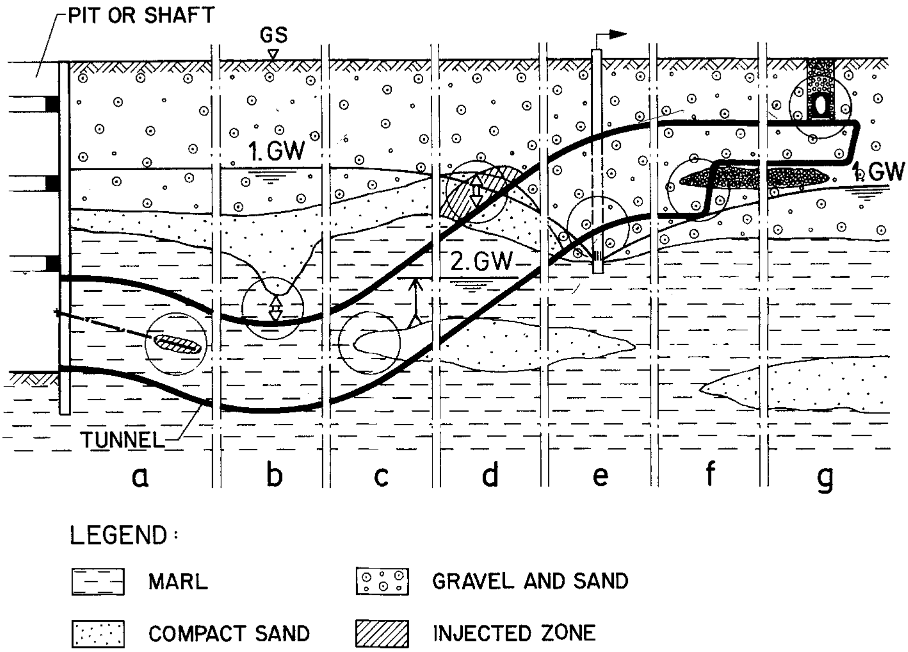


Fig. 27. Critical hazards when tunnelling with shotcrete support in Munich. *a* to *g* indicate sources of risk

The points b) through d) refer to driving in Tertiary, those of f) and g) to that in Quaternary, while points a) and e) are relevant in all soil layers.

a) Disturbance of the natural soil in approach areas (open cut, shaft), e. g. due to soil anchors.
b) The presence of unforeseeable erosion trenches in the marl horizon, which inadmissibly reduce the marl cover and thereby establish a connection to loose rock containing water.
c) Tertiary sand lenses that are unknown and therefore not depressurised (2nd GW).
d) Tunnel driving in the transition zone of the Tertiary marl and sands in the water-rich Quaternary brash. Inadequate frozen bodies or grout layers that are too weak also pose particular risks here.
e) Dewatering wells in the immediate vicinity of the tunnel. They can cause strong water flow and washing in of material.
f) The presence of brash lenses or benches with uniform kernel diameter (cobble), which thus shows neither true nor apparent cohesion.
g) The presence of channels, drainage pipes and artificial fills in the ridge zone of the tunnel poses a further danger of cave-in.

The analysis of the causes of accidents showed that in the case of 95 % of the total amount of collapsed material groundwater was the principal hazard (Kovari et al., 1982). In the case of 60 % of these material quantities, it was the presence of foreign objects in the tunnel area that was the most dangerous accompanying factor. The accidents were furthered by human failing across the board.

Such dangers must be countered by suitable technical measures and stricter checking. But beyond these purely technical measures it is also imperative again and again to sharpen the awareness of possible dangers on the part of all those responsible at the site, in order to counteract an attitude of indifference that, as experience shows, develops with the continuous performance of hazardous activities.

For this purpose there was developed a concept of the forward-looking way of regarding dangers, similar in principle to a checklist, which prescribes regularly mandatory checks to be entered into the record at the site. The core of this concept consists of:

— a safety plan for the entire job, with specific hazards depicted, and as a particularly important element,
— a tunnelling prognosis, to be drawn up in writing by the contractor every week for the tunnelling work of the coming week, and to be presented to the foreman.

Those responsibles are thereby forced to take note of the problems of tunnelling steps of the immediate future and to come to grips with them ahead of time.

3.4 Measuring Programs, Results of Measurements

Measuring is an important aid in inner-city tunnel construction. Careful planning and execution of measurements in the field and the interpretation of the results of measurements increase safety and promote economy.

The typical measuring program for a job is divided into observations made

— of the surface of the terrain
— of nearby structures (buildings, neighbouring tunnels, etc.)
— in the tunnel
— in the soil around the tunnel

The following equipment is mainly used as instruments:

— leveller (transverse and longitudinal basins of the settlement at the surface, as well as crest sinking and floor rise in the tunnel)
— settlement clinometer (measurement of tilt of buildings)
— convergence device distometer with Invar wire (horizontal and vertical convergence in the tunnel)
— theodolite (convergences with optical measurement of angles)
— sliding-micrometers (distribution of strain in the soil in the vicinity of the hollow spaces along drill-holes)
— "TRIVEC" borehole probe (distribution of all three displacement vectors along horizontal boreholes in the soil)

The systematics of the measurements in the field and the instruments employed, as they were used in Munich, have been dealt with at length elsewhere (Kovari et al., 1982).

The measurement of settlements at the surface during the driving of two single-track tunnels in the Tertiary sediments has demonstrated first of all that the driving sequence has a significant influence on the total amount of settlement (Weber, 1979, Fig. 28).

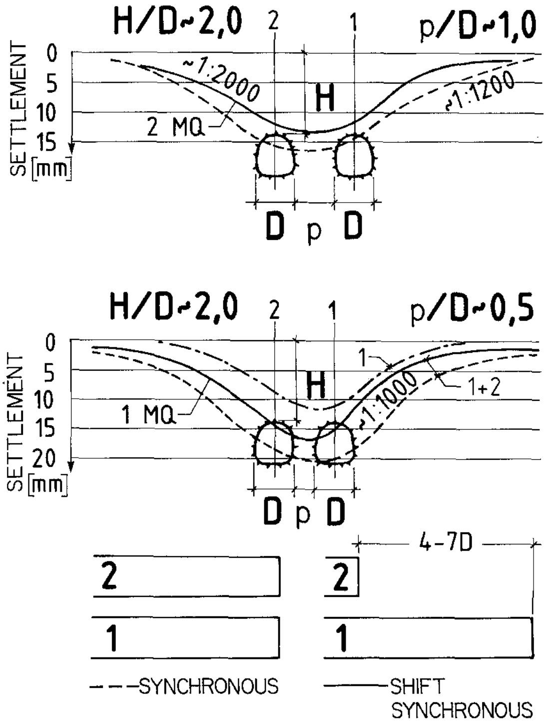


Fig. 28. Surface settlements for different spacings p of the tubes ($D \cong 6.5$ m)

If two tunnels located side by side are excavated not simultaneously but staggered by an amount which in practice could be from 25 to 50 m, then the total settlement is reduced by 20 % to 25 %. At the same time, the gradients of the settlement basin are gentler.

However, the width of the pier *(p)* between the two tunnels also has a significant influence on the amount of settlement. The thinner this pier is, the greater the total settlement becomes. With the help of sliding-micrometer measurements in the pier between two tunnels, this influence can be observed precisely (Fig. 29). For example, with a pier 5.5 m wide and a cover of 14 m, it was shown that about ⅔ of the surface settlement that ensues when the second tunnel is excavated past the one previously excavated can be attributed to compression in the pier zone.

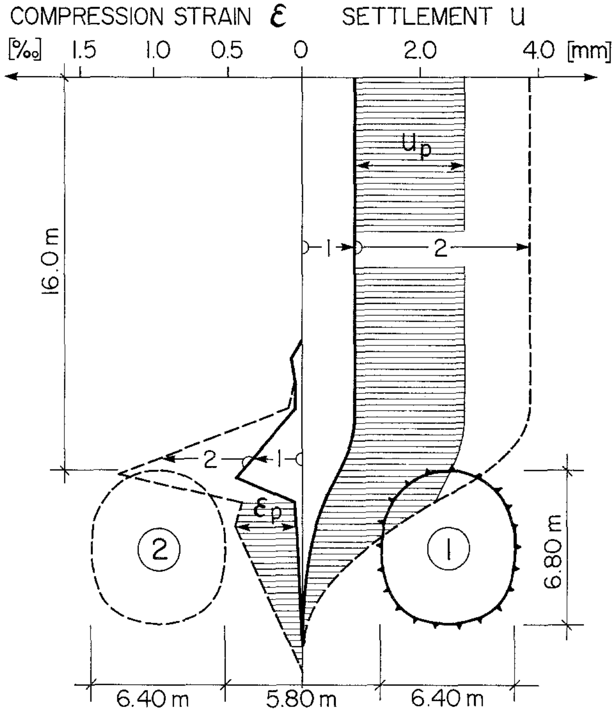


Fig. 29. Additional surface settlement u_p due to pillar stressing caused by excavation 2

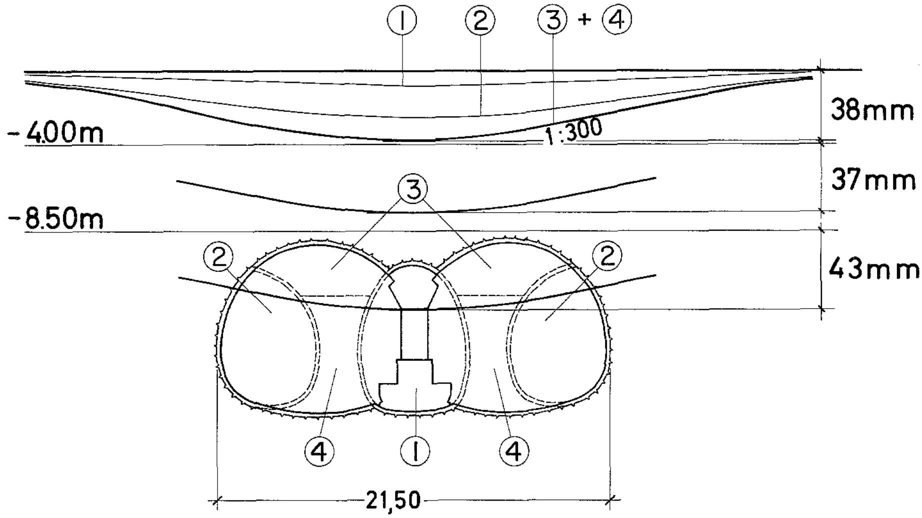


Fig. 30. Surface settlements due to different stages of excavation (tunnel in Tertiary)

In the case of multi-track tunnels, it is also attempted by means of a suitable choice of excavation sequences to reduce the settlements resulting from the continuous development of shallow settlement basins. Where required, the inner lining was taken into account included for support. How the settlement basin is composed of components from the different drifts can be followed in Fig. 31 for a large triple-track cross-section 150 m² in area with two tunnels in the stretch, and in Fig. 30 for a large station tunnel 176 m² in cross-section.

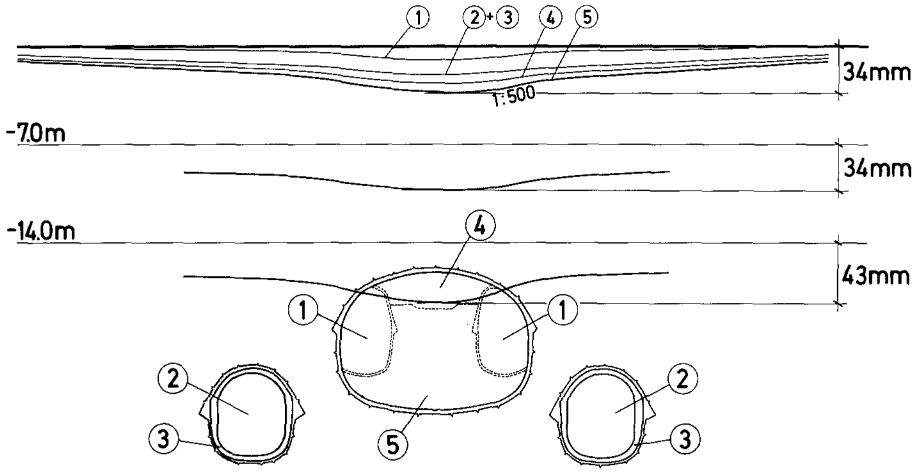


Fig. 31. Surface settlements due to different stages of excavation (tunnel in Tertiary)

A significant further development of the shotcrete method is its combination with compressed air as a means of dewatering. This construction method was successfully tried for the first time in Munich in 1978. Since then, five jobs have been executed with this method (Weber, 1983). Before the excavation of the station at Odeonsplatz underneath buildings extremely sensitive to settlement (Lessmann et al., 1986), the suitability of the planned tunnelling method was investigated in a test stretch with surface levelling and sliding-micrometer measurements in 3 mutually uninfluenced measured cross-sections, with and without compressed air (Amstad and Kovari, 1984). In Fig. 32 can be seen that the gradient of the settlement basin decreases by almost a half where compressed air is used, as do likewise the settlement values.

The sliding-micrometer measurements in Fig. 33 provide the explanation for the above:

— The cause of the surface settlements is the compression that occurs in the soil around the tunnel.
— This compression is significantly less when driving is done under compressed air than at atmospheric pressure.
— The soil settlements derived from adding up the compression strains confirm the settlement values obtained at the surface by levelling.

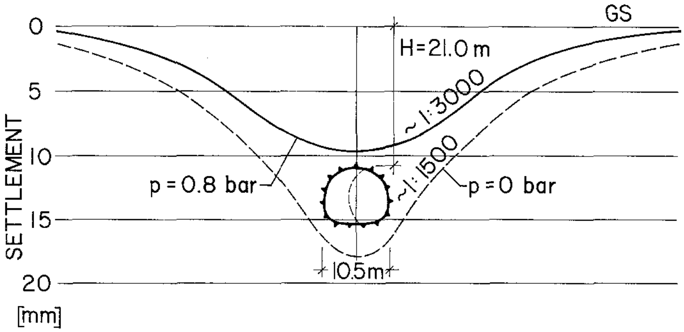


Fig. 32. Settlement troughs under atmospheric and compressed air conditions in the tunnel ($A = 80$ m^2)

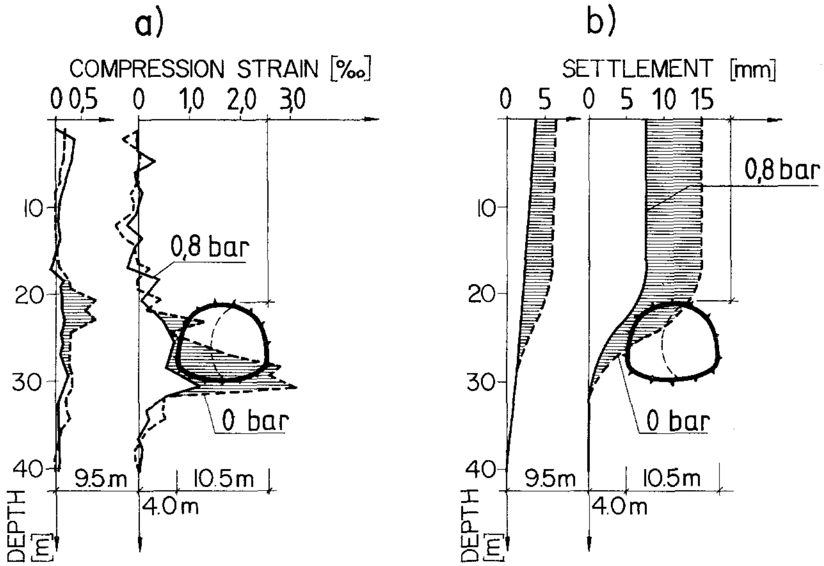


Fig. 33. Ground deformations under atmospheric and compressed air conditions in the tunnel
a) Strain distribution along two boreholes; b) Vertical displacement along two boreholes

3.5 Tunnelling Advance with and without Compressed Air

What effect does using compressed air with the shotcrete method have on the tunnelling progress? In order to answer this question, two jobs were selected in which single-track tunnels were excavated at atmospheric pressure as well as with compressed air. Table 2 shows that peak progress was attained at atmospheric pressure in the first job, and under compressed air in the second.

Table 2. *Tunnelling Progress Through Tertiary, 1-Track Tunnel*

Job	Tunnel length m		Mean weekly progress*, m		Maximum weekly progress*, m	
	ATM	CA	ATM	CA	ATM	CA
1	200	930	11.1	15.0	25.0	20.0
2	100	740	11.6	19.1	15.0	24.0

ATM: Atmospheric pressure; CA: Compressed air.
* 5-day work week.

It is clear, however, that in both jobs, the average progress under compressed air was considerably above that at atmospheric pressure. Apparently decisive for this result were, among other things:

— the absence of dewatering measures at the tunnel face
— the obligation to rigidly organise the construction operation on account of the bottleneck at the air lock for materials

The recorded weekly progress of 5 jobs at atmospheric pressure and 4 jobs with compressed-air tunnelling has opened up the possibility of a statistical evaluation. The evaluation shown is based on single-track tunnels in Tertiary with a total tunnel length of 1470 m without and 2230 m with compressed air.

The Gaussian distribution curves (Fig. 34) show clearly that under compressed air not only the mean values attained for weekly tunnelling progress are higher, but also that the mean variation of the achievement values is less. This overall more uniform speed of tunnelling under compressed air thereby also considerably reduces the deadline and cost risk.

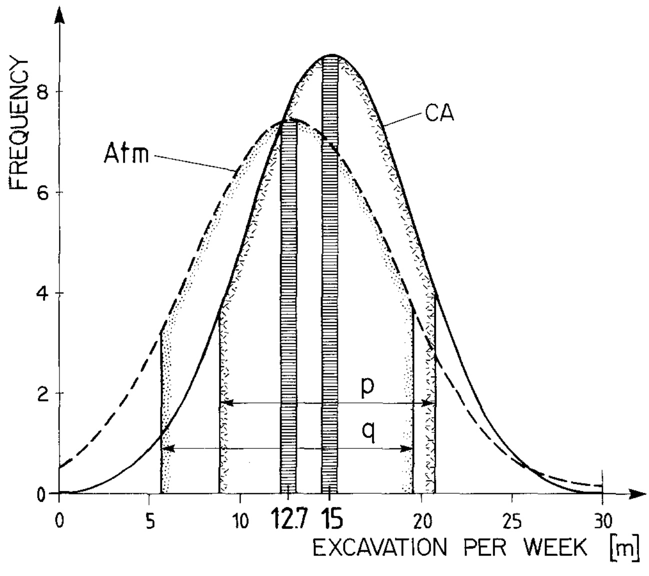


Fig. 34. Advance rates of 1-track tunnels with and without using compressed air
p and *q* are 80 % confidence limits

3.6 Cost Considerations

With the shotcrete method under compressed air, the question of economy naturally arises as well. On 5 jobs, the bids for tunnelling under compressed air lay between 0.5 % and 6.5 % below those for working at atmospheric pressure. The comparison made in Table 3 of the cost shares of atmospheric vs. compressed-air tunnelling for 3 jobs (bidding 1982/83) shows that the comparatively high cost of dewatering with grouting or freezing at atmospheric pressure exceeds the increased expense of tunnelling and equipping the site with compressed air, so that compressed-air tunnelling turns out to be more favourable in cost overall.

Table 3

		Jobs 1982/83					
		1		2		3	
		ATM	AP	ATM	AP	ATM	AP
Length of stretch	m	1350		945		940	
Equipping site	%	13	19	16	25	16	22
Dewatering + grouting/freezing	%	32	7	33	9	27	16
Driving + shoring	%	37	53	34	48	40	45
Inner shell	%	18	21	17	18	17	17
Cost of 1 m of stretch*	DM	41	37	46	42	45	46

* The stretch consists of 2 single-track tunnels.

Table 4 shows the development of costs in Quaternary driving above groundwater under atmospheric conditions (bidding 1984/85), such as is carried out in the zone of the extension lines. It will be seen that the absence of groundwater can lead to considerable cost reductions in spite of many difficulties in the gravel.

Table 4

		Jobs 1984/85			
		4	5	6	7
Length of stretch	m	495	510	705	650
Equipping site	%	11	13	17	15
Driving and propping	%	63	62	59	60
Inner shell	%	26	25	24	25
Cost of 1 m of stretch	DM	24	25	26	24

4. Final Remarks

On the basis of the experience described, it can be determined that the shotcrete method, particularly in combination with compressed air, has proven itself in Munich's subsoil. In the field of hydrogeological problems, an increase in safety has been attained, and the reductions in settlement make it possible to tunnel under structures that are sensitive to settlement without damaging them. Moreover, shotcrete tunnelling with compressed air is less injurious to the environment because it is no longer necessary to drill and operate wells.

As a side effect, shotcrete tunnelling has contributed to the lowering of the costs of mining constructions which at the outset were double those of the open-cut method. Thus subway tunnel construction using shotcrete support will play an important part in further expansion of the Munich subway network.

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