B. Maidl, M. Thewes, U. Maidl

Handbook of Tunnel Engineering II
Basics and Additional Services for Design and Construction

Ernst & Sohn
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Dedicated to My **G**randchildren

Maximilian

Leon

Antonia

Frederik

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Foreword to the English edition

The “black book of tunnelling” has become a standard work in German-speaking countries since its first German edition in 1984. It can be found on every tunnel site and in every design office – whether contractor or consultant. Students at universities and technical colleges use it as a textbook.

For many years, colleagues from abroad have been asking me for an English edition. Now the time has come to publish the two-volume book in English. An important step was that the publisher of the first German edition, VGE, gave their permission for the publishing of the English edition by Ernst & Sohn, Berlin. Special thanks are due to Dr. *Richter* from Ernst & Sohn for his successful negotiations. However, preparation of the text for the translation showed that the 3rd German edition required updating and extending. In particular, the standards and recommendations have been revised. This will all be included in a 4th German edition, which will be published soon. Changes to the standards and recommendations are given in this edition, with the references stating the latest version.

As with all books, the English edition has also required the collaboration of colleagues. Professor Dr.-Ing. *Markus Thewes*, who has succeeded me as the holder of my former university chair, and my son Dr.-Ing. *Ulrich Maidl*, managing director of the consultant MTC, have joined me in the team of authors. Dipl.-Ing. *Michael Griese* from MTC is the overall coordinator, assisted by Dipl.-Ing. *Stefan Hintz* from MTC. I thank all those involved, also the translator *David Sturge* and the employees of the publisher Ernst & Sohn in Berlin.

Bochum, in September 2013 *Bernhard Maidl*

Writing without violence is impossible.
I constantly put myself under pressure.
Violence is perhaps not the right word.

Daniel **B**oulanger

Foreword to the 3rd German edition

The above quotation introduced the complete revision of the second volume for the 3rd German edition. This became necessary after 15 years because not only the tunnelling technology in the first volume has developed enormously but also the standards and regulations have been revised or harmonised in the European Union. Under these premises, all chapters have been reworked and extended, partly based on my other books like shotcrete, steel fibre shotcrete, shield and TBM tunnelling as well as more recent publications.

The chapter "Dewatering during the Construction Phase" has been extended and is now called "Dewatering, Waterproofing and Drainage" in the second volume; this includes detailed information about hardness stabilisers.

As already in Volume 1, my employees have supported me in every way, although I have also received external help. For example, Dr. *Heimbecher* revised the section about road tunnels in Chapter 1 and Mr *Chromy* contributed to the section about the EU machinery directive in Chapter 8.

I wish to thank them all, and also the collaborators on my former books, which we have referred to for the revision work. Great thanks are also due to the many helpers from the consultancy Maidl + Maidl and for the contributions of several machine manufacturers and the publishers Glückauf.

Bochum, in January 2004 *Bernhard Maidl*

Foreword to the 2nd German edition

The good sales of the “Handbook of Tunnel Engineering” have also accelerated the publication of the second volume. Numerous ongoing and future large tunnel projects lead to a great demand for relevant literature. Reference books about design, tendering and construction are of great importance today, whether for instruction at universities, for practical application in consultancies and on construction sites, but also for the individual engineer interested in gaining further knowledge. So I am personally very satisfied to find the “Handbook of Tunnel Engineering” in use in design offices, on site, and also repeated in the text of university lectures.

On the threshold to the next century, with tunnels becoming ever longer and being constructed under ever more challenging conditions, construction methods are also demanded to comply with ecological, environmental and economic requirements. The necessary development potential covers all construction methods, both in conventional and mechanised tunnelling. The initial requirements, specifically the description of the geology and hydrology with the associated structural verifications and measurements, the environmental requirements, and also the special features of scheduling and cost planning with the associated contractual provisions, are likewise significant factors. This book covers all these subjects.

A complete revision for a new edition would naturally represent the latest state of information, but this is not achievable in the available time. The reader must therefore be asked to consider this when one or other innovation has not yet been included.

I wish to thank the publisher for their processing.

Bochum, in October 1995 *Bernhard Maidl*

Research and **T**echnology demand ever more **I**nterdisciplinary **K**nowledge. **I**t is almost a characteristic of our **T**ime that scientific and technical **P**rogress is increasingly taking place at the **I**nterfaces of traditional **P**rofessions.

Karlheinz **K**aske, 1987

Foreword to the 1st German edition

Four years after the publication of the first volume, work on the second volume is at last complete. This Volume II devotes its first chapters to the fundamentals of design, aspects of engineering geology, structural verifications and instrumentation for monitoring with the intention of providing a clear classification of the known theoretical and practical methods. The composition of the chapter "Structural design verifications, structural analysis of tunnels" proved demanding despite the intensive assistance of Herr Dipl.-Ing. *Jens-Detlev Wolter*. Although tunnels are engineering structures, their structural analysis and calculation cannot be undertaken like structures above ground. Their load-bearing behaviour is decisively influenced by the construction method, also including for the tunnel engineer the time factor as construction progresses. This factor is at least as significant as the effect of the rock mass around the tunnel considered as a "construction material".

The construction method is emphasised in the second volume just as in the first. Construction process technology in tunnelling is a prime example of interdisciplinary research: only an engineer who can master the technical basics of all influential factors can work competently. I could not and did not want to offer a qualitative or even quantitative evaluation of calculation procedures. Experienced structural engineers will form their own opinion. However, construction experience and calculation examples from recent rail, road and underground railway tunnels have been included.

This Volume II also deals with auxiliary works such as dewatering, measurement and control technology and scheduling. The control technology for tunnel boring machines is developing rapidly, particularly for shield machines in different soil types.

Similarly to Volume I, Volume II also includes extensive tables and illustrations in order to represent the idea of a handbook. The examples are taken from numerous newer construction projects.

In the production of this Volume II, I have relied heavily on the assistance of my colleagues at the Institute for Tunnelling and Construction Management. I thank Dipl.-Ing. *Jens-Detlev Wolter*, Dr.-Ing. *Dieter Handke*, Dipl.-Geophys. *Günther Eichweber*, Dr.-Ing. *Harald Brühl*, Professor Dr.-Ing. *Dietrich Stein*, Dipl.-Ing. *Karl-Jürgen Athens* and

Dipl.-Ing. *Jürgen Brenker* for their tireless motivation and months of collaboration, and I particularly wish to thank Dipl.-Ing. *Uwe von Diecken* for the overall leadership of the group. Thanks are also due to Professor Dr.-Ing. *Werner Brilon*, chair of transport I at the Ruhr University, Bochum, for collaboration on Section I.1 in regard to transport technology. I thank my brother Dipl.-Ing. *Reinhold Maidl* for his help and the help of the consultancy, particularly for the assistance with the work load at all times.

I thank the ladies at my chair and the consultancy, Frau *Agatha Eschner-Wellenkamp* and Frau *Hildegard Wördehoff*, and at the drawing office led by Herr *Helmut Schmidt* for their industrious help in the production of the work.

The publisher has assisted me greatly with this volume by looking through the manuscript critically, suggesting improvements and have also presented the work excellently.

Let us hope that Volume II "Basics and Auxiliary Works in Design and Construction" can find its way, as Volume I already has, into the hands of the engineers working for contractors, clients and their supervisors, and not least of the students at the various further education establishments.

Bochum, in May 1988 *Bernhard Maidl*

Table of Contents

*) The titles of some chapters have changed since their initial announcement in Handbook of Tunnel Engineering, Vol I, with no effect to the content itself.

Table of Contents

Volume I: Structures and Methods

1 General Principles for the Design of the Cross-section

1.1 General

The shape and size of the design cross-section derive firstly from the purpose of the tunnel (rail tunnel, road tunnel, sewer, water tunnel or pressure tunnel for a hydropower station) and thus the required clearance gauge. Secondly, the dimensions will also be influenced, as is the alignment, by the geotechnical or structural conditions in the ground to be passed through; whether earth or water pressure could occur or whether no external loading is to be expected. Thirdly, the construction process also has an effect on the design of the cross-section; for a given clearance gauge, the most economic cross-section is that which can be constructed with the least excavation and support technology and with the optimal machinery, taking into account the given basic shape.

1.2 Dependence on intended use

1.2.1 Road tunnels

General. The traffic conditions in a road tunnel should in principle correspond to those in the open air. Road tunnels are, however, special sections of a road and demand stringent requirements for their construction, maintenance and operation. Road tunnels have to meet particular requirements regarding road safety and operational safety. When the needs of traffic management are balanced against economy, it is therefore necessary and justifiable in many cases to limit the speed compared to parts of the road in the open air. The permitted maximum speed is thus normally limited to 80 km/h in road tunnels, which inevitably differentiates the traffic flow in tunnels from roads in the open air.

Tunnel cross-section. Road tunnels with two-way traffic and those with one-way traffic are fundamentally different. Two-way tunnels normally consist of a single tube with one lane in each direction. In one-way tunnels, the traffic in each direction is constructionally separated, for example through the provision of two bores. While in the past each bore was often laid out with two lanes without a hard shoulder, the changing composition of the traffic and ever increasing traffic loading will also demand three lanes without hard shoulder, and in exceptional cases even three lanes with a hard shoulder.

The design of the cross-section of road tunnels has to consider road traffic aspects, operational equipment and the tunnel structure. The design of the cross-section of a cut-and-cover road tunnel is often subject to different constraints from a mined underground tunnel. Some examples of cross-sections of mined road tunnels are shown in Fig. 1-1.

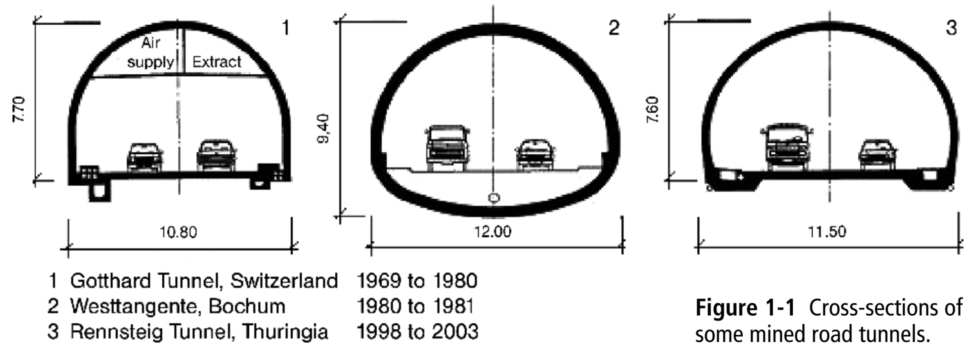


Figure 1-1 Cross-sections of some mined road tunnels.

The starting point of all considerations does, of course, remain the space required for the road intended to run through the tunnel. The required total cross-section can often be twice that of the actual cross-section for traffic, and the cross-sectional area at breakdown bays of autobahn tunnels can be up to 200 m^2 and more. The space required is also influenced by the horizontal and vertical alignments selected for the project.

The design of tunnel cross-sections in Germany is based on the guidelines for the equipment and operation of road tunnels (RABT) [77], also taking into account the guidelines for road design; cross-sections (RAS-Q) [76] and alignment (RAS-L) [75]. These guidelines include requirements for the standard cross-section, the structure or vehicle gauge to be maintained, the transverse and longitudinal gradients in tunnels and the provision of breakdown bays and emergency exits.

Standard cross-section. The standard cross-section of a road tunnel has to provide dimensions to enable the installation of equipment like lighting, ventilation, traffic management and safety technology, normally outside the clearance gauge. Particularly ventilation and signage equipment may demand an enlargement of the tunnel cross-section. In order to limit the multitude of possible cross-sections – also for economic reasons – the standard cross-sections of roads in the open air are assigned to road cross-section types in tunnels. The selection of road tunnel cross-sections is carried out according to [33] (Fig. 1-2).

In tunnels intended for two-way traffic, the standard cross-section type 10,5 T with 7.50 m paved width between the kerbs is normally provided. This cross-section is also used in open-air sections where wider verges are provided due to high heavy goods traffic volumes. In the course of a road with 2 + 1 RQ 15,5 sections (two lanes with an overtaking lane), sections running through tunnels are also constructed to section 10,5 T. The overtaking lane in this case thus has to be terminated in good time before the tunnel. Special solutions like an additional crawler or climbing lanes in the tunnel are an exception. When in exceptional cases tunnel sections on main roads only provide RQ 9,5 section, cross-section 10,0 T should be used [33].

The normal layout in tunnels with multi-lane carriageways in one direction should be a reduced standard road section without hard shoulders (26 t or 33 t), although it is justifiable under certain economic or traffic conditions to provide hard shoulders. Economic aspects in this case could be the construction and operating costs resulting from the length of the tunnel or the costs resulting from congestion and accidents. The hard shoulders are available for vehicles to swerve to the side or stop in an emergency. They often allow continued multi-lane traffic flow after minor accidents or breakdowns and also simplify maintenance work without serious disruption of traffic flow. The width of hard shoulders varies depending on cross-section type (Fig. 1-2). It is

– for cross-section type	29,5 T	2.50 m.
– for cross-section types	26 T and 33 T	2.00 m.
– for cross-section type	26 Tr	1.50 m.

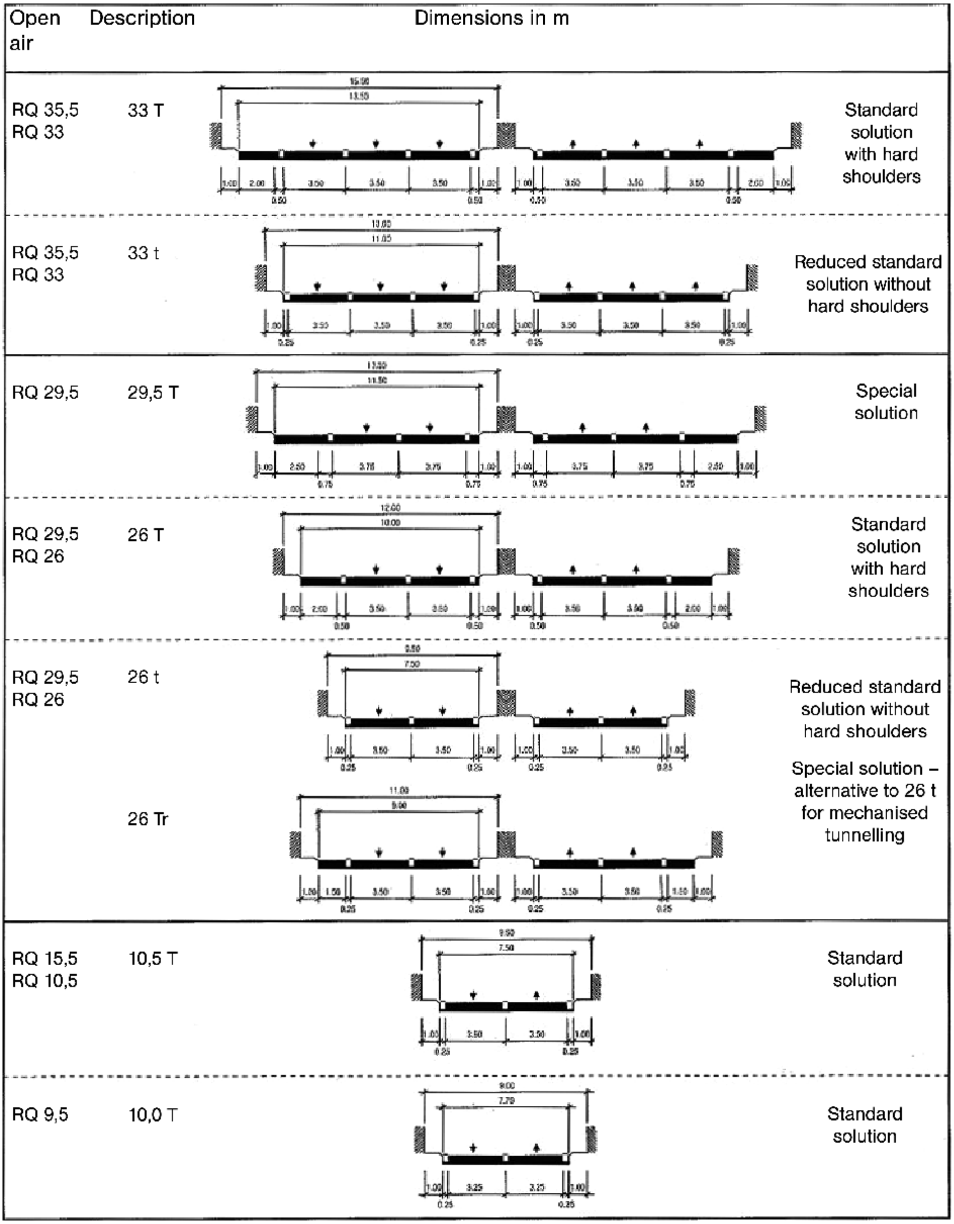


Figure 1-2 Standard cross-sections for road tunnels [33, 77].

For the layout of hard shoulders in tunnels, reference should be made to [33]. Using this decision-making process, it should be checked whether the additional utility resulting from a hard shoulder exceeds its extra cost. Using the diagrams for use with this process, it can be seen that the decision to provide the cross-sections with hard shoulders (26 T or 33 T) can only be justified under very favourable construction conditions or with a high volume of heavy good vehicle traffic combined with steep gradients. This process applies for multi-lane carriageways in one direction in road tunnel up to 2,000 m long.

The reduced form of special cross-section 26 Tr should only be considered for tunnels to be driven with shield machines. In this case, the reduced hard shoulder replaces the otherwise necessary breakdown bays along the entire length [33].

Cross-section type 29,5 T is only worth considering for very unusual cases and in any case only for very short tunnels with an exceptionally low-cost construction method.

Clearance gauge, traffic gauge. The clearance gauge denotes the space for the road cross-section, which has to be kept clear of obstructions. It consists of the traffic gauge and the safety margins at the top and the sides. The necessary cross-sectional area of the clearance gauge ensues from the traffic purpose of the tunnel. It is derived from the applicable standard cross-section in the open air; the permissible restriction of the cross-section inside structures also has to be considered (RAS-Q [76]).

The total width of the clearance gauge is the sum of the widths of the side safety margins, the carriageway, the verges and any additional lanes (for example hard shoulders) (Fig. 1-3).

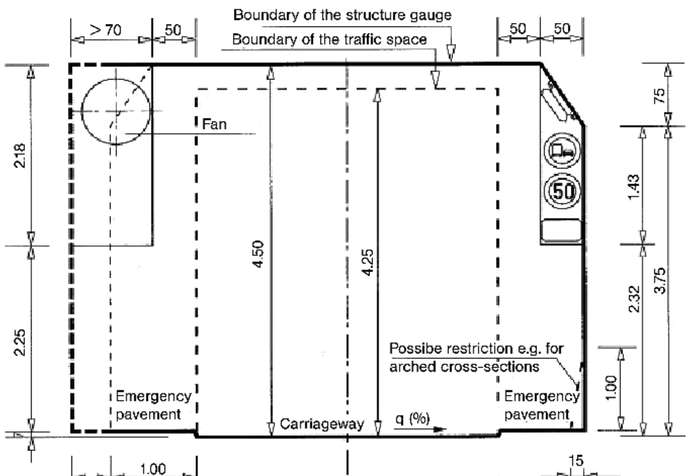


Figure 1-3 Outline of the clearance gauge in road tunnels (standard solution) [77].

The required headroom for road traffic is 4.50 m. For economic reasons, the sides of the outline are normally vertical, demanding a widening of the safety margin when the cross-slope gradient is steep. For circular cross-sections, on the other hand, it can be economic to tilt the clearance gauge with the carriageway. The outline at the sides can then be assumed to be vertical to the carriageway. It is not necessary in such cases to widen the safety margin.

The outline of the clearance gauge includes areas solely reserved for traffic. Emergency pavements are provided on each side of the carriageway, which are 1.00 m wide and have to have clear headroom of 2.25 m. These are separated from the carriageway with kerbs, normally 7 cm high. Part areas are assigned at a height > 2.25 m above the emergency side pavements, in which easily deformable furniture elements particularly traffic signs and notices can be located although these are only permitted to approach within 50 cm of the traffic gauge; jet fans required for ventilation have to be installed in niches or ceiling coves. Easily deformable light fittings are only permitted to approach within 50 cm of the traffic gauge at a height of > 3.75 m. If jet fans are located inside the normal structural dimensions, this results in widenings of the emergency pavements dependent on the diameter of the fans to be installed [77].

It is often practical to locate traffic signs on the end walls of breakdown bays. In exceptional cases, traffic signs can by located down to a minimum of 30 cm from the traffic gauge at a height > 2.25 m above the emergency pavements; but this does not apply where a widening of the emergency pavement has been provided for fans. If traffic signs have to be made with smaller dimensions than stated in the regulations [32], then this has to be agreed with the authority responsible for traffic management.

Light fittings are permitted to approach within 50 cm of the traffic gauge in exceptional cases when it can be ensured that a clear headroom of 4.10 m from the top of the emergency pavement to the underside of the light fitting is maintained at all points. Jet fans with external diameters ≤ 70 cm are permitted in exceptional cases to be located in the safety margin with a minimum distance at the side of ≥ 30 cm to the traffic gauge in the upper corners.

Gradient and cross-slope. According to the RAS-L [75], the gradient in uninhabited areas running through tunnels should be limited to 4 % if possible and a maximum of 2.5 % should be the intention, particularly for longer distances. The chimney effect, which also increases with increasing gradient, normally leads to higher longitudinal flow, which in case of fire can severely impair the rapid and effective removal of smoke by a ventilation system. In order to ensure road safety and due to the chimney effect, gradients steeper than 5 % should be avoided in road tunnels in uninhabited areas.

A minimum cross-slope of 2.5 % is specified for straight stretches in order to drain surface water [76]. Depending on the design speed, the cross-slope may have to be adapted to suit the curve radius [75]. In addition to these conventional requirements, the cross-slope of roads in tunnels has special significance in case of an accident. If a fire breaks out, any leaking flammable liquids have to be drained away as fast as possible, which is ensured by a steep cross-slope and high-capacity drainage. Slot channels with a capacity of 100 l/s should therefore be provided, with firestops spaced at max. 50 m [77].

1.2.2 Constructional measures for road safety in tunnels

Breakdown bays. Breakdown bays should be provided where the provision of hard shoulders is not economically justifiable. They are required in tunnels more than 900 m long, and under special conditions from 600 m (for example ≥ 4,000 HGV · km / bore and day) [77]. The end wall should have an angle of ≤ 1:3 in the travel direction (Fig. 1-4). It can be secured by suitable passive protection according to RPS [78]. Concrete protection walls should have an angle ≤ 1:3. In tunnels with two-way traffic, these requirements apply to both end walls.

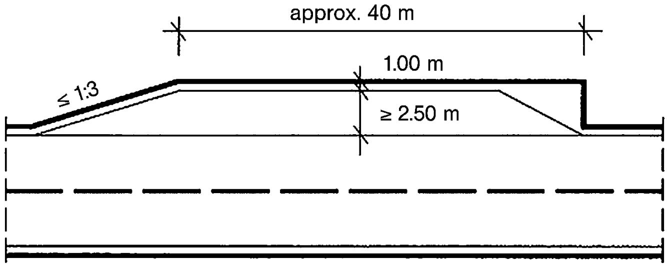


Figure 1-4 Dimensions of a breakdown bay next to a carriageway [77].

The spacing of breakdown bays should be ≤ 600 m in each direction. In tunnels with two-way traffic, the breakdown bays should be arranged opposite each other in order to enable vehicles to run in case of emergencies (turning bays).

Emergency exits, escape and rescue routes. Escape and rescue routes, which are to be signed and have lighting, should be provided in tunnels and the escape route in the traffic gauge should lead to the emergency exit and the rescue route from the emergency exit should lead to the open air directly or through safe areas.

From a tunnel length of ≥ 400 m, emergency exits should be provided at a regular spacing of ≤ 300 m [77]. Emergency exits can lead

- into the open air.
- directly into the other tunnel bore.
- through cross-passages into the other tunnel bore.
- to escape and rescue shafts.
- to escape and rescue tunnels.

Cross-passages in this case denote connecting structures between two parallel tunnel bores. They should be closed from each tunnel bore with doors. In two-bore tunnels, every third emergency access to the other bore can be designed to allow passage for fire service and emergency service vehicles in case this is required by the safety and rescue plan.

In escape and rescue shafts, escaping people are led up stairs to the open air. The stairs have to be 1.50 m wide for two-way access. The design of escape and rescue shafts should give reasonable consideration to the limited physical capabilities of disabled and older people.

Escape tunnels normally run parallel to the tunnel and connect various emergency exits from the tunnel to a common exit into the open air. The gradient should not be more than 10 % and they should have a clear passage of 2.25 m × 2.25 m.

In exceptional cases for tunnels with a high traffic volume, it can be practical to make escape tunnels more than 300 m long accessible for emergency service vehicles. This measure should however be verified as part of an overall safety plan.

The equipment of road tunnels with lighting and ventilation for normal operation and in case of fire, with drainage and also communications equipment, fire detector and extinguishing systems all pose additional requirements for the design of the cross-section. These requirements can lead to various solutions depending on the local conditions and should thus be decided for each project.

1.2.3 Rail tunnels

General. The first rail tunnel in Germany was built near Oberau in the years 1837 to 1839 and had a length of 512 m. The oldest tunnel that is still in operation is the 691 m long Busch Tunnel near Aachen, built from 1841 to 1843. Most of the tunnels that are still in operation were built in the years 1860 to 1880. These had to be maintained at greet cost through the 20th century [118]. The cross-sections of early tunnels were mainly based on the clearance gauge for rolling stock. The clearance gauge encloses the cross-sectional area, into which no part of the train may extend.

For rail tunnels, the horseshoe profile was generally used, in a higher form for single-track tunnels and a flatter form for two-track tunnels. It can also be designed with vertical inner side surfaces. Today an arched profile with or without invert vault is more commonly used for conventionally driven tunnels, and a circular profile for tunnels bored by shield machines. In addition to the cross-sectional areas required for the rolling stock and tracks including signal lamps, contact shoes and any other necessary accessories, rail tunnels require a loading gauge that allows for deviations of the wagons through snaking, for example as a result of broken springs. In addition to the loading gauge determined in this way, space also has to be provided for signals, overhead, cables, lighting, pipes and other equipment required for rail operations and escape routes.

At stations, the tunnel has to be enlarged to house the platforms. It is important for rail operations that the platform is wide and long enough not to obstruct rail traffic, including consideration of traffic peaks. For this reason it is much better to provide sufficient space for platforms in advance than to be forced to undertake rebuilding measures later due to insufficient capacity [93].

Rail tunnel on new high-speed lines (NBS) of German Railways DB AG are designed according to the planned use and the resulting design speed v_E. This is categorised by new regulations (Ril 853) into four categories:

- High-speed traffic with 230 km/h < v_E < 300 km/h.
- Express traffic with 160 km/h < v_E < 230 km/h.
- Passenger and goods traffic with v_E < 160 km/h.
- S-Bahn, urban transit with v_E < 120 km/h.

The gradient on main lines should be limited to 12.5‰ and on urban and side lines 40‰. The permissible gradient should be laid down for each individual case and can, like for example in the Irlahüll Tunnel on the NBS Nuremberg – Ingolstadt at 14.5‰, also lie outside the ideal value stated above. A lower limit should also be maintained – depending on the planned use – of 2‰ (tunnel length l < 1,000 m), or 4‰ (l > 1,000 m). Ideally, the vertical alignments of tunnels should be ramps with the gradient in one direction for fire protection reasons.

The permissible curve radii should be limited to

2,000 m < r_A < 30,000 m

and determined more precisely from the design speed within this range.

The size and shape of the excavated cross-section depend on the loading gauge of the train, the lining thickness and the construction process. Depending on the various planned uses, the guideline Ril 853 specifies different track spacings in tunnels and thus various sizes of cross-sections. An enlargement of the cross-section compared to previous regulations is necessary because high pressures are created when two trains pass each other in a tunnel at high speed. The sudden change of pressure can reduce the travel comfort of the passengers in a small tunnel and more seriously can cause stresses in the windows that endanger operations.

In the following section, the most important parameters demanded in Ril 853 for the cross-sections of rail tunnels are described, depending on the planned use:

1. Tunnels for high-speed traffic at 230 km/h < $v_E \leq$ 300 km/h

In new construction and major refurbishments, the standard track spacing in straights and curves should be exactly 4.50 m, with a specified formation width of 12.1 m and a distance of the track centre to edge of formation of 3.8 m. The radius of the cross-sectional area is specified as 6.85 m for two-track tunnels, resulting in a total area above top of rails (TOR) of A = 92 m^2. The same total area results for the case of a three-centred arch for two-track traffic, for which radii of R_1 = 6.85 and R_2 = 4.00 m should be selected (Fig. 1-5). The permanent way can consist of a ballastless track or tracks laid on ballast. This choice then influences further parameters of cross-section design but not the total area of the cross-section. Details of these minor differences can be found in Ril 853.

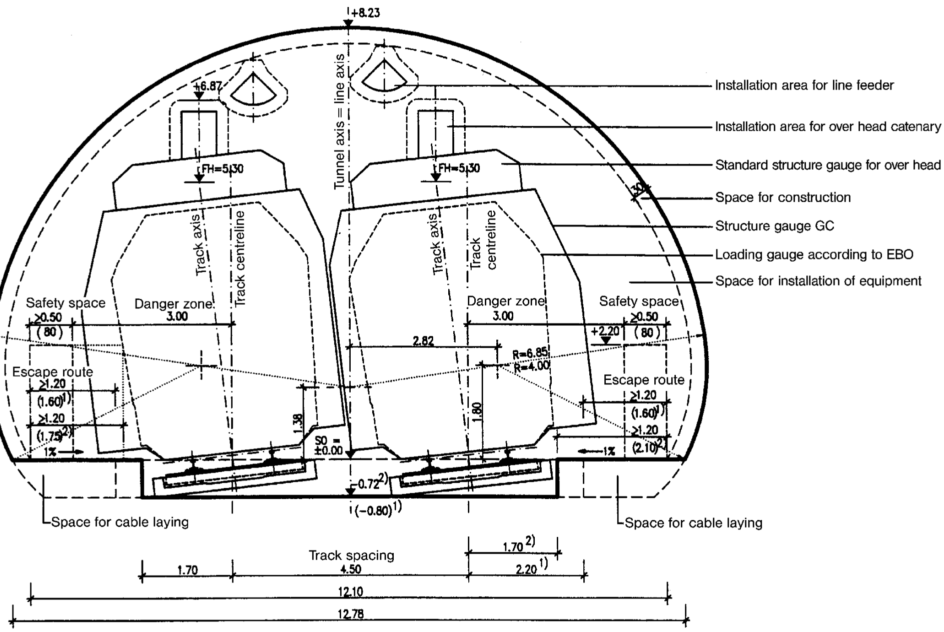


Figure 1-5 Guideline detail for a two-track high-speed tunnel with three-centred arch section according to Ril 853.

In new construction and major refurbishment of single-track tunnels, a safety space has to be maintained on the side of the cable trough, and in multi-track tunnels outside the danger area on each side. This serves for access to the tunnel and for the evacuation of passengers to an exit in case of emergency. The safety space must be at least 2.20 m high and 0.50 m wide. In all new tunnels, there must be one continuous escape and rescue path leading to the open air for each track. The escape and rescue path should lie on the side of the safety space outside the outline of the clearance gauge. The passage width of the escape and rescue path should be at least 1.20 m, and the clear headroom at least 2.20 m.

The illustration (Fig. 1-5) shows an example of these requirements and the other details of the clearance gauge for a two-track tunnel with three-centred arch section on a high-speed line. The corresponding guideline details for a single-track tunnel with circular or three-centred shape of the cross-section can be found in Ril 853.

2. Tunnels for express traffic at 160 km/h < $v_E \leq$ 230 km/h

The cross-section of a rail tunnel for express traffic only differs from that for high-speed travel in the specified dimensions according to the guideline detail. The requirements for safety spaces and escape routes are formulated independently of design speed, so the requirements are identical for all design speeds. Ril 853 specifies a track spacing of only 4.00 m for express traffic in two-track tunnels, so the required formation width at $u = 0$ reduces to 11.60 m. The required spacing of track centreline to edge of formation remains at 3.80 m. The radius of a circular cross-section also reduces to $r = 6.10$ m, from which an altogether smaller cross-sectional area of $A = 79.2\ m^2$ above TOR can be calculated. As with tunnels for high-speed traffic, individual parameters can also vary with the selection of as ballastless or conventional permanent way. This is illustrated below with a guideline detail for a two-track tunnel for express traffic with circular cross-section according to Ril 853 (Fig. 1-6).

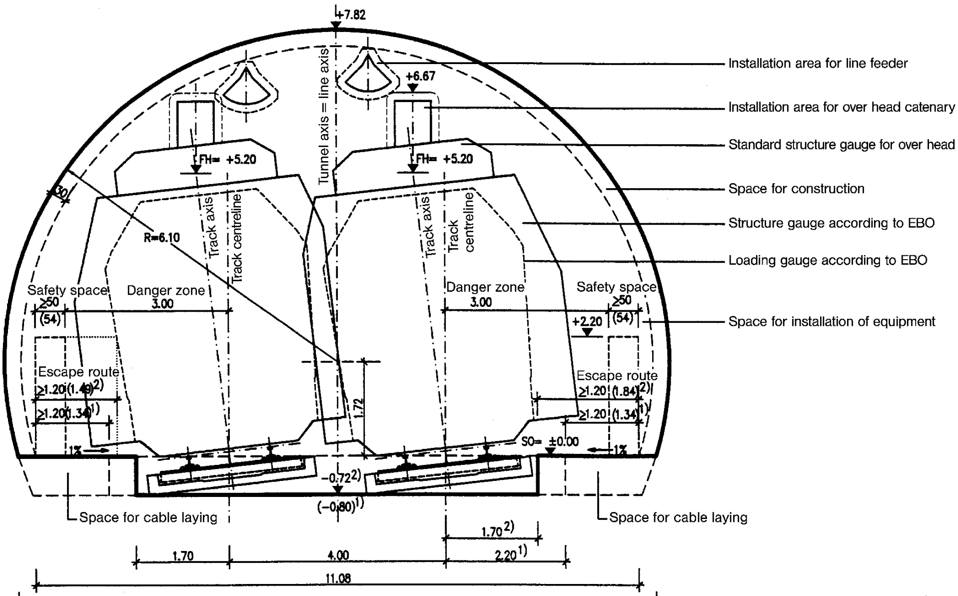


Figure 1-6 Guideline detail for a two-track express tunnel with circular section according to Ril 853.

3. Passenger and goods traffic at $v_E \leq$ 160 km/h

For passenger and goods traffic with a design speed of $v_E < 160$ km/h, the Ril 853 does not provide any guideline details for two-track cross-sections. Because the traffic is mixed, only single-track tunnels should be used in this case according to the guideline for civil protection from the EBA (federal rail authority), so two-way traffic has to run through separate parallel tunnels. Fundamentally, it can be stated that the distance of the track centreline from the edge of formation reduces to 3.30 m in comparison with other layouts and the formation width of open-air sections at $u = 0$ is thus 10.60 m. The dimensions for escape routes and safety spaces still apply for passenger and goods traffic since they are independent of design speed. This is illustrated below with a guideline detail for a single-track tunnel with circular cross-section (Fig. 1-7).

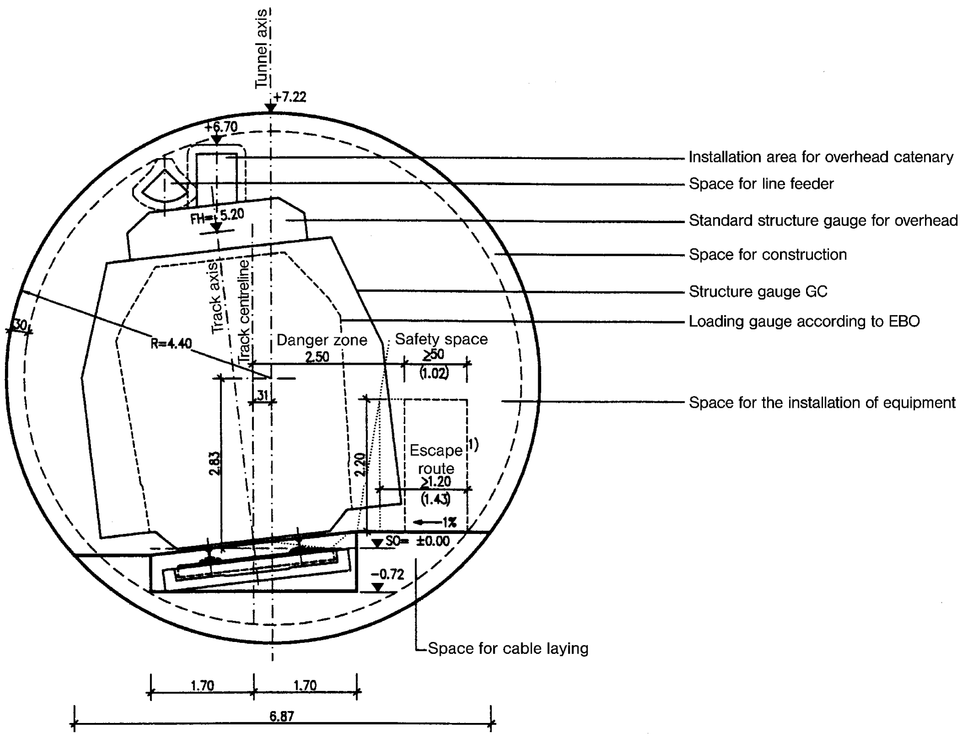


Figure 1-7 Guideline detail for a single-track tunnel with circular cross-section for passenger and goods traffic according to RiL 853.

4. S-Bahn, urban transit traffic at $v_E \leq 160$ km/h

Urban or rapid transit railways (S-Bahn in Germany) are categorised as railways according to the provisions of the general railway law and the railways construction and operation regulations [41] derived from it. In order to take into account developments in tunnelling technology and associated special processes for tunnelling inner-city rapid transit lines, the DB AG guidelines RiL 853, RiL 800.0130 and Ril 997.0101 are applicable, of which the Ril 853 has a chapter dedicated to the special features of urban rail tunnel construction.

In densely built-up urban areas, in hilly terrain or near stations at intermodal hubs, urban rail lines often run underground. S-Bahn lines in the cities of Stuttgart, Munich, Hamburg and Berlin have numerous underground stations. The locomotive-hauled S-Bahn shuttle in the Rhine-Ruhr area also partly runs underground.

S-Bahn tunnels can have either round, vaulted or rectangular cross-sections. With a permissible gradient of 40 ‰ and track radii of $R > 250$ m, smaller cross-sectional dimensions are possible than at speeds of over 120 km/h due to the lower design speed. The specified track spacing is 3.80 m, the distance from track centreline to edge of formation is 3.20 m and the specified formation width at $u = 0$ is 10.20 m.

With a clear width of 9.16 m and a clear height 5.49 m, a two-track rectangular cross-section on a straight line has an area of 50.3 m^2. In curves, this area is slightly greater due to the cant. With a clear width of 9.25 m, the Ril 853 specifies a clear height of 5.59 m and thus a total area of 51.7 m^2. It is also the case here that the selection of permanent way type can change individual parameters of cross-sectional design.

One special detail of S-Bahn tunnels is the layout of the clearance gauge for the overhead. Ril 853.1003 specifies, in contrast to Ril 800.0130, a space to be kept clear for the overhead as shown in Fig. 1-8.

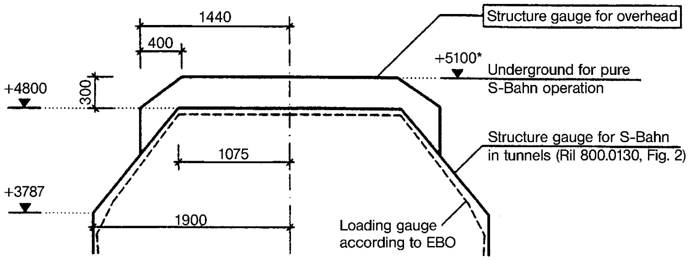


Figure 1-8 Clearance gauge for the overhead in urban rail tunnels.

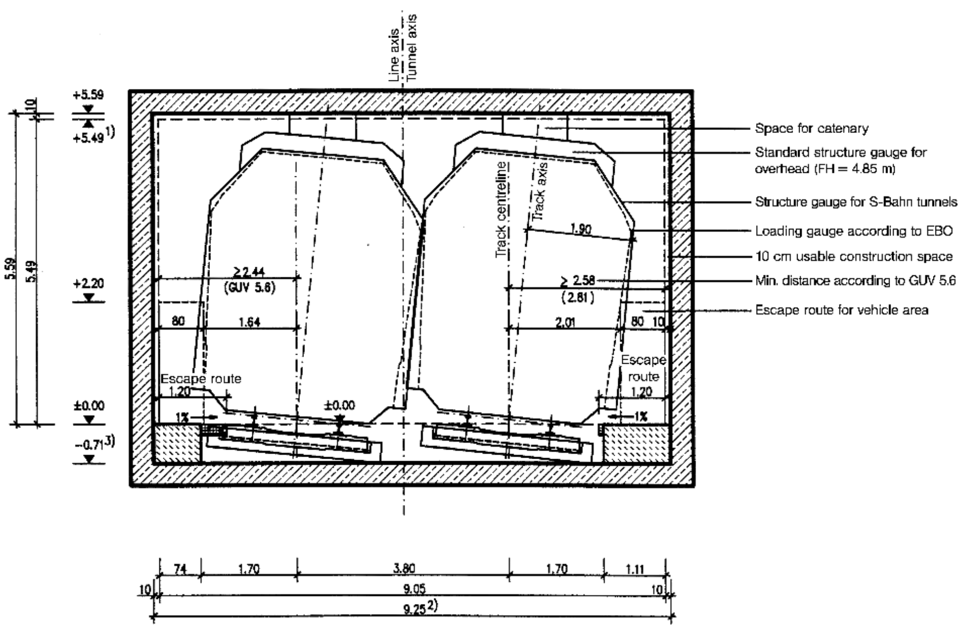


Figure 1-9 Guideline detail for a two-track S-Bahn tunnel in a curve with rectangular cross-section according to RiL 853.

An exception permit is to be obtained from the BMVBW (federal ministry for transport, building and town development) in each individual case for the application of the height of 5 100* mm above TOR according to Fig. 1-8.

In contrast to the details described until now, the safety space in S-Bahn tunnels has to be at least 80 cm wide. For the height and width of the escape route, the dimensions of 2.20 m · 1.20 still apply.

Fig. 1-9 shows as an example the guideline detail from Ril 853 for a 2-track S-Bahn tunnel with rectangular cross-section.

1.2.4 Construction of rail tunnels

The fire and civil protection requirements for the construction and operation of rail tunnels are laid down in the guideline of the federal railway authority (EBA) with the same name from 15 August 2001. The following section describes some important design principles from this guideline. More detailed information can be found in the guideline.

Fire duration and temperature curve. In order to minimise the depth of concrete spalling that could endanger passengers, a curve of temperature against time is to be assumed for design purposes (Table 1-1).

Table 1-1 Temperature curve depending on fire duration according to EBA.

Fire duration [min]	0	5	60	170
Temperature [°C]	0	1200	1200	0

These data can be used to determine the additional stresses, which have to be resisted by constructional measures (for example additional layers of reinforcement).

Safe areas, escape routes, emergency exits. Each track is provided with its own escape route next to it, as has already been mentioned in the description of the various planned uses. Localised narrowing of the escape route is to be avoided or in exceptional cases limited to a length of 2.0 m and a depth of 0.3 m. Handrails should be provided on all escape routes.

For the design and layout of escape shafts and tunnels, the limited physical capabilities of frail people and those with disabled mobility should be considered. Shafts should not exceed a level difference of 60 m and if the level difference is more than 30 m, should be equipped with a lift with dimensions 1.1 · 2.1 m. Stairs should be suitable for people passing and for the transport of stretchers according to DIN 13024.

Escape tunnels must have a cross-section of at least 2.25 m · 2.25 m, and not exceed a maximum gradient of 10 % and a maximum length of 150 m if they do not reach the open air directly but up a shaft. For lengths of more than 300 m long, rescue shafts must be accessible for ambulances.

Rescue areas and access roads. All exits and portals of the tunnel must be accessible by road. For long tunnels, a rescue area is to be provided at each portal and emergency exit. For shorter tunnels, one rescue area is sufficient.

Access roads and rescue areas must

- have secure planning status,
- have secure property status,
- be covered by an access regulation in traffic law.

In this case DIN 14090 should be complied with.

Access roads to rescue areas should be separate from exits. If this is not possible, two-way traffic with 2.50 m width must be ensured, if unavoidable by providing passing places. If rescue areas are connected to dead-end streets, it must be possible to turn.

Extinguishing water supply. Each tunnel portal must have sufficient water supply available for extinguishing fire (at least 96 m^3 at 800 l/min) within a maximum distance of 300 m.

Two-track tunnels are to be provided with a continuous dry fire extinguishing pipeline, which can be supplied from the portals and from the emergency exits.

In single-track tunnels on a two-track line, a continuous dry fire-extinguishing pipeline is to be laid in every running tunnel. In addition to the above provisions, both pipelines must be joined by dry pipes at junction structures.

It must be possible to operate dry extinguishing pipelines in sections and they are to be laid with protection.

1.2.5 Underground railway and underground tram tunnels

General. The cross-sectional dimensions of these tunnels are determined by constraints resulting from vehicle dimensions, dynamic travel properties, alignment elements, layout of safety spaces in the tunnel, location and type of power supply, environmental protection requirements (damping of vibration) and construction.

Guidelines. The following guidelines are applicable for all new design and design revisions for underground railway and underground tramlines:

1. Regulations concerning the construction and operation of tram lines (BOStrab) from 11 December 1987.
2. Guidelines for tunnels in the Regulations concerning the construction and operation of tram lines (Tunnel construction guidelines) from 30 April 1991.
3. Accident prevention regulations (UVV) of the accident insurer for trams, underground railways and railways.

For the design of urban railways, the regulations that have been introduced in the Rhine-Ruhr area [227] are widely regarded as a standard. The Stadtbahngesellschaft Rhine-Ruhr has produced detailed guidelines.

Considering the different types of vehicle at each location, the provisions regarding cross-sections only have the character of a recommendation. The shape of the tunnel cross-section is decisive for the determination of the outline of the loading gauge and the clearance gauge. Tunnels are differentiated into those with rectangular cross-sections and circular or similar cross-sections.

1.2.6 Innovative transport systems

In recent years, the development of guided transport systems for inner city public transport has been advancing in the Federal Republic of Germany.

Bus transport is being further developed, particularly for travel through tunnels, with buses being guided electronically or mechanically in the tunnel. The advantage of this principle is the considerable reduction of the cross-sectional dimensions of the tunnel compared to a manually steered bus (Fig. 1.10), which can greatly cut construction costs. This type of tunnel for buses will mainly be restricted to inner-city areas. Bus tunnels are already being designed and constructed in the cities of Essen and Regensburg. The dimensions of the tunnel cross-section derive from the size of a standard bus. These have a width of 2.50 m plus 0.25 m on each side for the rear-view mirrors and a height of 3.95 m. The mechanical guidance (Fig. 1-11) requires a road trough with a width of 2.95 m.

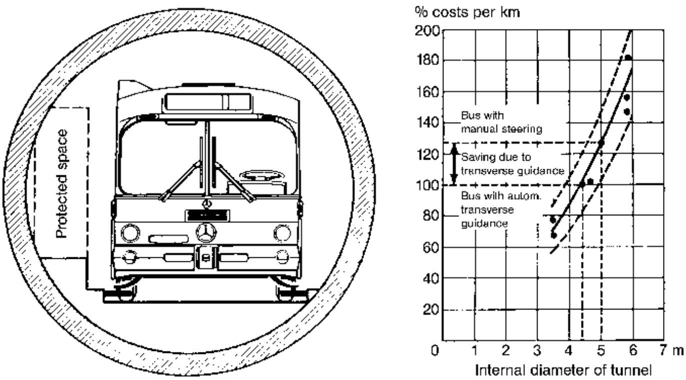


Figure 1-10 Automatically guided bus. Circular tunnel cross-section (left) and possible cost savings (right) [168].

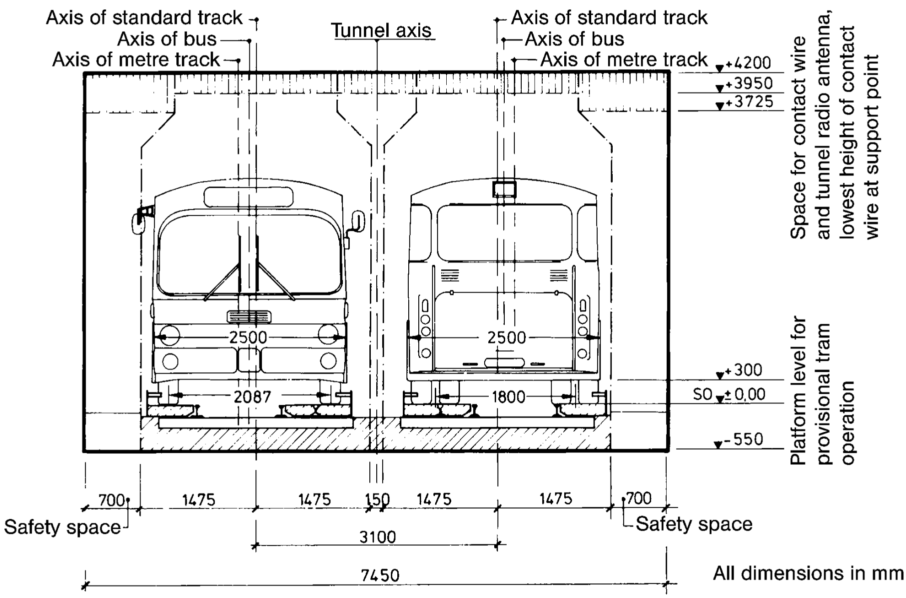


Figure 1-11 Mechanical guidance for busses. Rectangular cross-section for straight stretches of two-way tunnel [15].

1.2.7 Monorail with magnetic levitation, Transrapid, Metrorapid

A new method of transport, which has already been under development for about 30 years, is the "Transrapid" high-speed monorail with magnetic levitation, which represents an alternative between jet and train with a speed of 400 km/h (Fig. 1-12).

After the construction and testing of the Transprapid in Shanghai, an application in Germany is still in the design phase. The planning of the Metrorapid in the Ruhr area from Dortmund to Düsseldorf and in Munich between the airport and the main station has however been abandoned for financial reasons. This would have required a 4 km long tunnel bored by a shield machine.

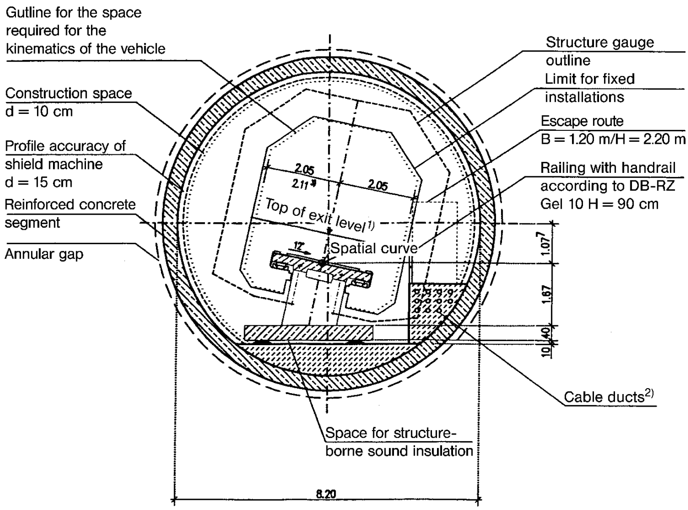


Figure 1-12 Standard cross-section for a shield-driven tunnel for a magnetic monorail.

1.2.8 Other underground works

General. In addition to the road and rail tunnels described above, tunnels are also devoted to the needs of pedestrians, skiers, shipping, drinking water supply and drainage and electricity and gas supply.

Pedestrian tunnels. These are of a similar nature to road tunnels, but the small clearance gauge, small curve diameters and the steeper permissible gradients, which can be up to 10%, and the possibility of joining them into a lift shaft lead to such a decisive simplification of their design and construction that they can be regarded as a different group. Pedestrian tunnels are found almost exclusively in inner cities and only seldom under rural roads.

The best-known pedestrian tunnels are those in Hamburg under the Elbe and in Antwerp under the Schelde. Neither of these has a staircase, but the pedestrians enter and leave the tunnels in lifts, escalators or shafts at the riverbank. The tunnel in Antwerp is a fully independent tunnel only intended for pedestrians, but the tunnel in Hamburg is for mixed traffic since there is a central single-track road with a 1.25 m wide pavement each side, similar to a road bridge [238].

Pedestrian tunnels either have rectangular or circular cross-sections according to whether they are below paving or deeper (including below water).

In order to make the pedestrians feel comfortable, pedestrian tunnels should have generous height and width. The clear headroom should not be less than 2.44 m, better still 2.75 m or more. The width is determined by the number of pedestrians.

Ski tunnels. These are becoming ever more common in many countries [128], [223]. A good example of this new sort of tunnel was opened in Saas Fee, Switzerland in December 1984 for an underground funicular railway to transport skiers. The tunnel, almost 1,600 m long, was driven at rock temperatures of 0 °C, so no water ingress had to be feared. Altogether over 80 % of the tunnel could be driven in excavation class I according to the Swiss SIA standard 198 (see Chapter 2). Fig. 1-13 shows the excavation conditions and the chosen cross-section. The tunnel was bored by a full-face machine with a diameter of 4.20 m.

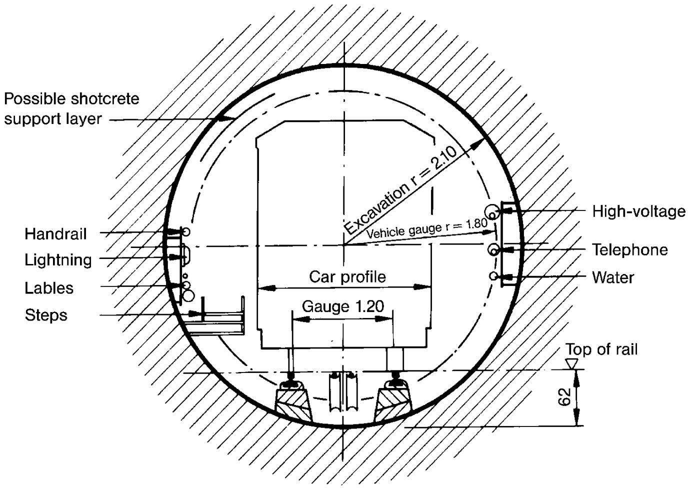


Figure 1-13 Cross-section of the Metro-Alpin Tunnel in Saas Fee [128].

Shipping tunnels. Historically, shipping tunnels were the forerunner of today's transport tunnels, as water transport was formerly more significant. These tunnels are not described in further detail here.

Tunnels to transport water under gravity mostly have a horseshoe section. The cross-sections of pressure tunnels tend to a circular form with increasing water pressure.

Utility tunnels. These are mostly in urban areas and serve to house utility supply pipes. The cross-section is normally circular or rectangular according to the chosen construction process. This sector is currently the subject of much research and development, particularly regarding the repair or replacement of old pipes and small cross-sections. The various construction processes and characteristic cross-sections are described in more detail in Chapter 7 of volume 1.

No further details are given here of the cross-sections of shafts, caverns, chambers or other applications.

1.3 The influence of the ground

General. The shape of the cross-section of tunnels also has to be suitable for the prevailing geological conditions and overburden. The size and direction of external loading mostly depends on the ground pressure. The better the load-bearing capacity of the rock mass being passed through, the less ground pressure has to be assumed in the design of the support, particularly pressure from the side. The higher the lateral pressure is in relationship to the vertical pressure, the more a near-circular cross-section will be suitable. The shape of the cross-section thus depends on the external and internal forces acting on the perimeter of the cavity (Fig. 1-14). In competent rock, which does not tend to be weathered, the excavated profile will stand up without any structural contribution from the support, and a thin layer of shotcrete can resist any effects of weathering. Ways of optimising the tunnel cross-section to an ideal shape are offered by the laws of structural geotechnical engineering.

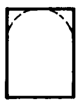
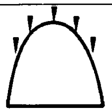
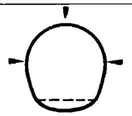
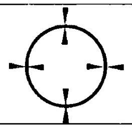
	**Rectangular** Used when the external forces do not lead to any damaging movement of the rock mass into the tunnel.
	**Semi-elliptical, parabolic or semi-circular** Used when vertical forces act.
	**Horseshoe, vaulted** Used when horizontal and vertical forces act.
	**Circular** Used when forces act from all sides and particularly with internal water pressure.

Figure 1-14 Basic cross-section shapes [135].

L. Müller [160] stated the following considerations:

"In competent rock, design is relatively unrestricted: cross-sections with vertical sides or flat vaults, even horizontal crowns are possible in stable rock as long as there are not too many joints, such as in conglomerates, some compact limestones and above all in undisturbed granite. Such tunnels are then mostly not lined or only provided with a weak lining. Fully inappropriate are linings, in this case actually only a facing, which are only installed to present a smooth face; underground cavities have their own aesthetics and should be designed according to the character of the rock; there is no reason to coyly hide it.

In fairly competent but jointed rock, profiles with weak support are used or those where systematic rock bolting provides the actual support. Also in this case we no are longer ashamed to specify an uneven lining that follows the structure of the rock, like for example is created by spraying shotcrete, and we leave the heads of the rock bolts visible even in important structures. This saves support work and concrete, which would only serve to create a geometrically precise outline by filling the hollows formed by overbreak without any actual structural purpose. Why should we not, where the surface structure imposes such an excavation shape, sometimes specify the required clearance gauge as a pointed arch or similar irregular profile, when otherwise rock bolting and concrete would have to be provided to avoid such profiles but without any actual structural purpose?

Rock that is competent but tends to subsequent loosening is often only permanently supported in the top heading, sometimes only with mesh and rock bolts. Formerly, when masonry *lining was used, the lining was often restricted to certain less stable areas and these were supported with masonry bands and corners. This practice died out with the introduction of formed concrete but should be reintroduced today with the availability of shotcrete, which can be applied as required. This would however only become established practice if saving money was regarded as a virtue and the supervisory engineers felt responsible not only for the avoidance of damage but for the economical use of construction materials.*

In competent rock, which is fractured into large to very large blocks by jointing, and if the diameter is less than twice the average joint spacing, support can be designed on the same basis as in competent rock without large joints if the anchoring is installed according to the structure of the rock mass. If the diameter is larger, a type of punching pressure on the support has to be expected, so the lining has to be designed to resist this shear or the joints have to be dowelled.

Brittle rock with medium strength, which requires transverse support to assist the formation of a protective zone, demands well rounded profiles that are approximately circular including the invert and support to all sides, possibly making use of permanent rock bolts behind a thin layer of concrete.

In squeezing and strongly squeezing rock, irrespective whether the ground pressure derives from high primary stress or low rock strength, the cross-section should always be nearly circular, although the invert is normally given a flatter profile (arched profile). The support in squeezing rock is normally made stronger without, however, having to completely abandon the advantages of a relatively slender support layer, as recent knowledge shows that the formation of a supporting ring in the rock mass, normally achieved or reinforced with systematic rock bolting, should always be the intention. The circular structure of the tunnel support must always be closed in squeezing rock, including the invert, and the curvature of the invert arch must be more pronounced the higher the ground pressure is. The closure of the invert is also very important in rock susceptible to softening.

Ground pressure acting on only one side (a), as for example is characteristic of tunnels beneath a slope, particularly slopes susceptible to creep pressure, require an asymmetric lining (Fig. 1.15), in which the abutment on the valley side, that is away from the pressure, is strengthened and this always requires a widening of the foundation.

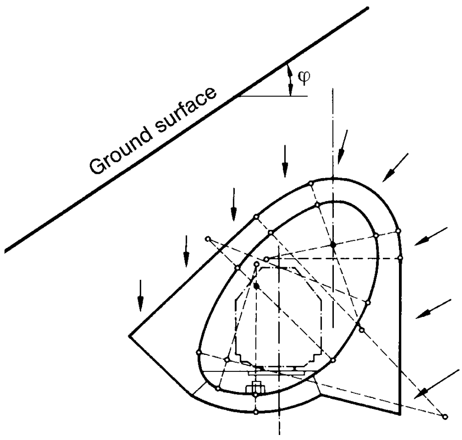


Figure 1-15 Tunnel profile for a tunnel below a slope with strong downhill thrust, designed using a classic line of thrust calculation [160].

Both measures, thickening of the abutment and widening of the foundation, are only sensible where the foundation is founded on firm ground, which is not the case when there is a downhill thrust. If suitable firm ground does not occur naturally, then the foot of the abutment has to be supported artificially with corbels, anchoring or such like.

Sideways yielding of the ground (b) can be due to steeply dipping and open joints or other loosening of the rock structure, also a comparatively high plasticity of the ground associated with a low modulus of subgrade reaction. Under such conditions, negative convergence is often observed during the supporting of the top heading, and even more often during the excavation of the bench, denoting an in-and-out movement of the top heading and bench support.

Both cases (a) and (b) require a strong invert arch; in the case of one-sided pressure, this has the purpose of supporting the pressure from above through the abutment foot on the downhill side; and in the case of ground susceptible to sideways yielding an invert arch is necessary because in this case the load-bearing ring in the surrounding ground lacks sufficient restraint and because such ground tends to plastic and pseudo-plastic invert heaving.

Ground with strong side pressure demands a strong invert arch.

Plastic (and also stiffly plastic) ground also demands an invert arch. Even in stiffly plastic types of ground, the cross-section has to be nearly circular but in plastic ground this is definitely to be preferred.

In ground susceptible to swelling, increased pressure on the invert is resisted by a particularly strong invert arch, or by a very curved invert arch. L. v. Rabcewicz [182], who always followed the principle of waiting for ground pressure to subside before resisting it with high support pressure, suggested two construction methods in ground susceptible to swelling. One has the essential feature that the walls of the cavity are initially supported with a weak support that yields under ground pressure and only when the ground pressure has subsided is a final lining installed against the first support layer to resist the pressure; this is the two-pass lining often used later as part of the New Austrian Tunnelling Method. L. v. Rabcewicz also proposed a second method of leaving a cavity between masonry lining and the rock mass, in which case it is important to make sure that the masonry is sufficiently stable. The stability is provided by masonry ribs, which are in contact with the rock mass and remain between the faces of the intended cavity.

The geological conditions relevant to construction include the primary stress in the rock mass, which denotes the stresses in the rock mass before the excavation of the cavity."

1.4 Dependency on construction process

General. The construction process that is chosen and the machines to be used in the cross-section and along the tunnel (construction and operation method) have an influence on the selection of the overall and partial cross-sections. It has an influence on the size of the final cross-section, also the profile of the excavation and to a certain extent the usable cross-section. The process- and operation-related criteria for the full-face and partial-face excavations are dealt with in Volume 1, Chapter 3.

Cross-section size. This has to be larger than the minimum profile for economic excavation, which is about 5 m^2, except when pipe jacking (see Volume I, Chapter 7) is used. Small cross-sections obstruct personnel and machinery, so the costs rise despite the smaller excavation and support quantities. Another cost factor is the ventilation. The size is no longer limited by the construction process because large cross-sections are normally divided into partial areas. Only when a tunnel boring machine (TBM) is used is the practical size limited by mechanical factors. The largest tunnel boring machines at the moment have reached diameters of 15 m, but still larger machines are planned.

Cross-section shape. This is influenced by the selected construction process and the associated machinery; for example a tunnel boring machine can only bore a circular section. All equipment associated with a TBM has to be adapted to suit the circular shape of the invert. Except for a few exceptions, this also applies to shield machines, although there have been some developments. Conventional tunnelling with its further developments including the shotcrete process also have an influence on the design of the cross-section in that a flat invert is available for mucking and transport and the excavated profile has to be enlarged more or less for the thickness required for the support.

2 Engineering geology aspects for design and classification

2.1 General

Underground cavities can be constructed down to depths of more than 2,000 m below the surface. The range of variation of the ground around the structure, which is structurally or hydraulically affected during and after the formation of the cavity, or which affects the structural stability and serviceability of the structure and the construction processes required to construct it, is thus practically unlimited. This fact requires that the tunnel engineer has a good understanding of geology, geological engineering and geomechanics and demands close cooperation between the different professions. The following section deals with the essential knowledge and factors, which are required for the estimation of the ground conditions as they affect the construction of an underground structure.

2.2 Origin, properties and categorisation of rocks

2.2.1 General basics

Starting comment. Rocks are naturally formed combinations of various minerals (mineral aggregates) or of just one mineral type [50]. They can be solid or loose. Rocks are the basic material that forms the rock mass, although the properties of the rock mass are not identical with the properties of the rock.

Loose ground or soil is a combination of minerals and/or fractured pieces of rock and/or organic components without mineral bonding. It is possible to grade the mineral content according to grain size. Predominant contact between the particles at points is a characteristic. Loose ground consists of several phases: solid-liquid, solid-gas or solid-liquid-gas.

Solid rock is a combination of minerals and/or fractured pieces of rock and/or organic components with mineral bonding, which gives the rock a certain strength. Predominant contact between the particles at surfaces and surface bonding of the components are characteristics.

Each type of rock has been formed by geological processes, which can be assigned to a cycle held in motion by the balancing of energy equilibrium inside the Earth's crust. (Fig. 2-1). In this cycle, the rocks are moved by the effects of endogenous (caused by forces from inside the Earth) and exogenous (caused by forces acting from outside the surface of the Earth) processes into areas, which do not correspond to their original formation conditions. The rocks are changed by the resulting adaptation to new conditions of stability. The processes of formation and alteration of rocks illustrated in Fig. 2-1 are still continuing today and lead to continuous changes at the Earth's surface. Since the duration and intensity of the processes within the cycle can be very varied, different types of rock can be formed, which can be categorised into the following rock types:

Igneous rocks, also called pyrogenic or magmatic rocks, are created by magmatic processes with the cooling and hardening of molten siliceous flows from the inside of the Earth (magma). According to O. Wagenbreth [249], the following types can be distinguished:

- Intrusive igneous rocks solidify within the crust and are mostly acidic. Their structure is fully crystalline and uniformly grained (granite, syenite, diorite, gabbro).
- Hypabyssal igneous rocks penetrate into crevices and solidify as they rise from the inside of the Earth; their structure is partially similar to intrusive and partially similar to extrusive rock (granite-porphyry, syenite-porphyry, diorite-porphyrite, gabbro-porphyrite).
- Extrusive igneous rocks are mostly alkaline and are created when magma exudes onto the surface of the Earth and solidifies relatively quickly. The structure is thus "porphyritic", finely crystalline, sometimes even glassy (older extrusive rocks: quartz porphyry, porphyrite, diabase; younger extrusive rocks: liparite, andesite, basalt).

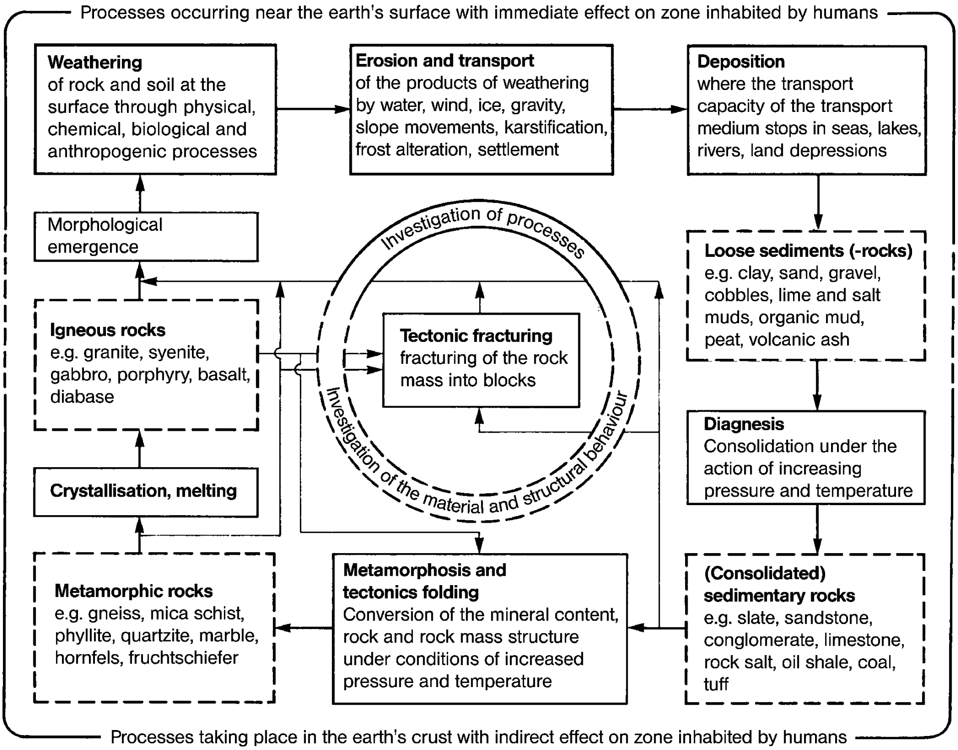
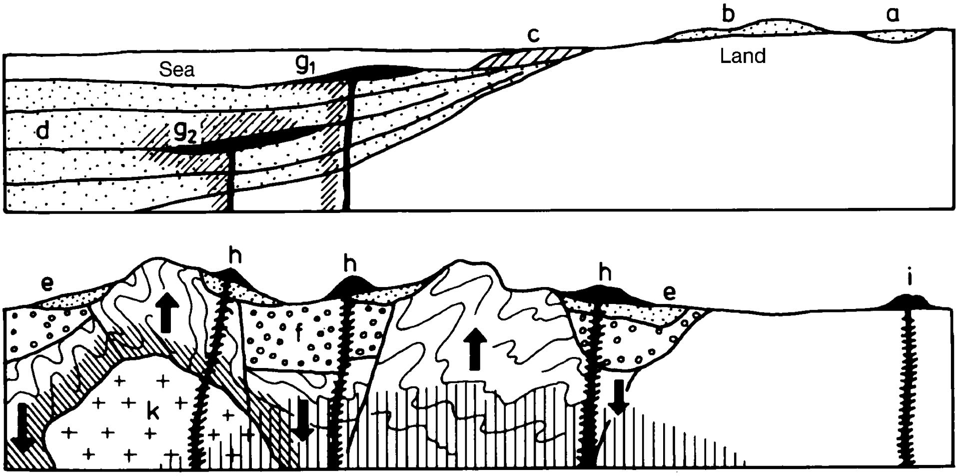


Figure 2-1 Cycle of rock formation [188].

Sedimentary rocks are created by exogenous dynamic effects (weathering, transport, deposition on land or under the sea) on the surface of the earth. According to the degree of consolidation, they can be loose sediments, loose rocks or sedimentary rocks. Some representatives of this type of rock are listed in Fig 2.1.

Metamorphic rocks, of which phyllite, marble, mica schist, quartzite, gneiss and hornfels are well-known and technically significant representatives, are formed from igneous or sedimentary rocks (or also from older metamorphic rocks) by regional metamorphism (increase of pressure and temperature with or without the addition of material), dynamic metamorphism (conversion through movement) or contact metamorphism with and without magma injection (conversion through contact with hot magma). Metamorphites formed from igneous rocks have the prefix ortho-, and those formed from sedimentary rocks have the prefix para-.



Formation of sedimentary rock
a in river valley (cross-section)
b in the desert
c delta deposition in coastal area
d geosynclinal sedimentation in shallow sea
e weathering debris of a mountain range in a trough next to the mountains
f weathering debris of a mountain range in a trough in the mountains (circles denote older, dots younger sediments)

Formation of igneous rock
g undersea outflows g_1 and intrusions g_2 at geosynclines
h volcanism in a mountain range, in troughs in front and within the range
i volcanism as land table mountain
k Intrusion and formation of an igneous intrusion

Formation of metamorphic rock
Vertically hatched: area of regional metamorphosis
Hatching inclined to lower left: Area of contact metamorphosis at extrusive igneous rock
Hatching inclined to lower right: Area of contact metamorphosis at intrusive igneous rock
The arrows show the tendency to block movements

Figure 2-2 Schematic section to show the connections between rock formation and structures in the Earth's crust, from O. Wagenbreth [249].

Rock formation. According to O. Wagenbreth [249], there are rules connecting the formation of rocks and the structure of the Earth's crust (Fig. 2-2). Sediments occur mostly in geosynclines, which are marine depressions, leading to the formation of fold mountains. They can also be found in inland depressions and forearc basins of fold mountains as well as to a limited extent on flat land and coasts without depression. Igneous rocks occur in zones of active mountain formation and also to a limited extent on geologically relatively quiet continental plateaus. Metamorphic rocks are found in the immediate vicinity of igneous rocks, and regional metamorphic rocks in zones of mountain formation.

The essential rock characteristics include the mineral composition and the structural fabric of a rock. The structure of a rock denotes the spatial and geometric configuration of the individual grains in the rock. The following partial complexes can be differentiated:

– The *Structure* denotes the form and size of the mineral components in a rock without consideration of the neighbouring grains and the spatial location [249].
– The *Texture* is the spatial arrangement and distribution of the mineral components in a rock.

Categorisation or even classification of rocks is the definition of properties and features to approximately determine the dependencies on other parameters. This serves to provide consistent assessment criteria, which make it possible to perform an advance estimation of suitability for a particular construction purpose. In general, it can already be stated that there is no generally valid classification of rock types to fulfil these requirements. The reason for this is the different mineral-chemical, structural and textural features of rocks and the wide range of technical aspects (in tunnelling, for example, the various excavation processes to drive the tunnel), under which the civil engineer has to consider rock properties.

The standard DIN 1054 "Subsoil – Verification of the safety of earthworks and foundations" [54] undertakes the classification shown in Table 2-1. This genetic system of classification and naming of subsoil types is generally accepted in soil mechanics, but is inadequate for rock mechanics where a further specific classification is needed. The important aspects of classification that are relevant in underground construction are dealt with in the following sections.

Table 2-1 Classification of subsoil types according to DIN 1054.

Subsoil class	Criteria for differentiation
I. Solid rocks	compressive strength, water solubility, origin
II. Soils (undisturbed ground)	
Non-cohesive	grading distribution, grain shape, density, water content, origin
Cohesive	plasticity, consistency, pore volume, pore ratio, susceptibility to softening, consolidation of macropore soils, origin
Organic	content and degree of composition of the organic materials, origin
Fill (artificial) (fill, artificially altered soils)	composition of the material

2.2.2 Categorisation of rocks

Due to the reasons stated above, categorisation of rocks is very difficult. Neither the attempt to categorise rocks on a genetic basis [20], or the adoption of the principle of strength [195], have led to any practical results. From all these investigations, those of K.J. Klengel [110] should be mentioned, whose categorisation is based on geological engineering and construction principles and who differentiates weather-resistant, weather-susceptible and (easily) water-soluble rocks (Table 2-2).

Table 2-2 Rock classification according to K.J. Klengel [110].

Rocks	Characteristic features	Mineral content	Grain bonding	Examples
Weather-resistant	Resistant grain bonding within a human timescale as the result of the type of grain bonding and the resistance of the mineral substance to the effect of weathering agents	quartz, feldspar, mica, hornblende, augite, calcspar	direct (grain-to-grain bonding), indirect (with weather-susceptible binder)	granite, basalt, porphyry, sandstone (with siliceous and ferrite bonding agent), limestone, gneiss, quartzite, marble
Weather-susceptible	Loss of grain coherence within a short time (days to months) under the effect of weathering agents (surface energy of wetting water, frost and others). Irreversible conversion into cohesive or non-cohesive soils	quartz, feldspar, mica, hornblende, augite in combination with clay minerals or rock glass	Predominantly indirect with clay binders, indirectly loosened	weathered igneous, sedimentary, metamorphic rocks: shale, marlstone, Sonnenbrenner basalt, glassy rocks, slate
Water-soluble (slightly)	Solubility on ingress of 1g per l water, precipitation of dissolved substance with removal of solvent	easily water-soluble minerals (chlorides, sulphates)	direct	rocks containing salt, gypsum, anhydrite

This classification takes into account the effect of weathering in changing the strength and load-bearing capacity of rocks and thus their stability and suitability for resisting loading. Further petrophysical properties and technical characteristics are given for the most significant rocks, for example in [105, 166]. Although a great spread of values can occur in individual cases, these data are quite suitable for preliminary construction assessment.

2.2.3 Categorisation of soils

DIN 1054 divides the undisturbed soils into the following main soil types (see Table 2-1):

Non-cohesive soils. In non-cohesive soils, the individual mineral grains or rock particles form a heap, which only possesses a loose coherence due to the friction of the grains against each other. The properties are influenced by the grain size, the grain size distribution, grain shape and grain angularity. They are resistant to weathering effects. According to DIN 1054, non-cohesive soils include sands, gravels, cobbles and their mixtures when the content by weight of the grain fraction smaller than 0.06 mm is less than 15 %, and also mixed-grained soils with a content by weight of grain sizes < 0.06 mm of 5 to 15 %, unless the fine-grained content determines the plastic behaviour of the soil.

Cohesive soils. In cohesive soils, the individual particles stick to each other through electrostatic surface forces and form a cohesive mass that can be formed. Their properties, above all the strength characteristics, are decisively influenced by the water content, grain size and clay mineral content. Cohesive soils are susceptible to weathering. According to DIN 1054, this soil category includes clays, clayey silts and silts and their mixtures and non-cohesive soil with a content by weight of the grain fraction smaller than 0.06 mm of more than 40 % and also mixed grain soils with 15 to 40 % if the fine-grained content determines the plastic behaviour.

Organic (organogenous) soils. Organic soils according to DIN 1054 include peat or organic sludge and anorganic soils with organic contents by weight of animal or plant origin of more than 3 or 5 % respectively. Depending on the degree of composition, they have a fibrous, matted or earthy structure and high water absorption capacity.

Soils are also classified in DIN 18 300 “German construction contract procedures (VOB) – Part C: General technical specifications in construction contracts (ATV) – Earthworks” [58] with regard to extraction, use and processing into the following classes:

Class 1: Topsoil. Topsoil is the uppermost layer of undisturbed ground, which is particularly rich in micro-organisms and contains humus or clay.

Class 2: Flowing soil types. Soil types, which are of liquid to pasty consistency due to their water content and do not release the water easily such as mud and silt.

Class 3: Easily excavated soil types. Non-cohesive to slightly cohesive sands and gravels and their mixtures up to 60 mm grain size, in which there is no or only very slight cohesion due to loam or clay soil types (< 15 % of grain size < 0.06 mm) and with max. 30 % cobbles of over 63 mm grain size up to 0.01m³ volume.

Class 4: Moderately easily excavated soil types. Mixtures of sand, gravel, silts and clay with more than 15 % grain fraction smaller than 0.06 mm. Further cohesive soil types of low to medium plasticity, which are semi-solid depending to water content and contain max. 30 % cobbles of over 63 mm grain size up to 0.01m³ volume.

Class 5: Difficult to excavate soil types. Soil types in classes 3 and 4, but with more than 30 % cobbles (over 63 mm grain size) up to 0.01m³ volume. Also non-cohesive and cohesive soil types with max. 30 % cobbles (over 63 mm grain size) from 0.01m³ to max. 0.1m³ volume and pronounced plastic clays, which are soft to semi-hard depending on water content.

Class 6: Easily excavated rock and similar soil types. Rock types, which have internal mineral cohesion, but are heavily jointed, brittle, unstable, foliated, soft or weathered and comparably solid or cemented cohesive or non-cohesive soil types, for example due to drying out, freezing, chemical compounds.

Also non-cohesive and cohesive soil types with more than 30 % cobbles of over 0.01 m^3 to max. 0.1 m^3 volume.

Class 7: Difficult to excavate rock. Rock types, which have internal, minerally bonded coherence and high joint strength and which are only slightly jointed or weathered, also densely consolidated, unlettered slate, conglomerate beds, slag heaps of ironworks or similar. Also cobbles with a volume of over 0.1 m^3.

DIN 4022, Part 1 – "Subsoil and groundwater; Naming and description of soil types and rock" [55] is valid for the uniform description of soil types. A soil type is assigned to a soil group according to DIN 18196, "Earthworks and foundations – Soil classification for civil engineering purposes" [57]. This latter standard applies to earthworks and has the aim of combining soil types (loose ground) into groups with approximately similar material composition and similar soil physical properties.

Soil mechanics parameters. In order to be able to describe the condition of a soil layer or soil sample, the following parameters have to be determined in addition to the grading distribution and plasticity:

- ρ Density of the soil (mass of a damp sample related to the total volume of the sample) or weight density γ of the soil, if it has to be regarded as a load or loading for construction purposes.
- ρ_s Grain density (density of the solid grain material).
- n Pore volume (Volume of the pores between the grains of the solid mass related to the total volume of a sample).
- w Water content (ratio of the mass of water in the soil m_W to the mass of a dry sample).
- w_L Liquid limit.
- k_f Permeability coefficient.
- I_p Plasticity index.

In special cases, the air and oil permeability may have to be determined.

If the effective stress in the soil mass is changed by external loading, the shape changes and the state of the soil changes. Since the soil can only resist slight tension forces, only the compression and shear loading in the subsoil are normally of significance. The compaction (squeezing) caused by the application of compression loading is characterised by the stiffness modulus E_s. Shear loading exerts a relative displacement between the individual soil particles, the magnitude of which depends on the shear strength of the soil. The shear strength of a soil is a combination of friction angle φ and cohesion c.

For preliminary design, M. Kany [102] recommends the values given in Table 2-3.

Table 2-3 Average soil parameters and permeability coefficients for preliminary design purposes according to M. Kany [102].

Soils	Weight densities		Friction angle: final strength	Cohesion		Stiffness modulus
	above water γ	below water γ'		Final strength c'	Initial strength c_u	E_s
	kN/m³	kN/m³	degrees ϕ or ϕ'	kN/m²	kN/m²	MN/m²
Non-cohesive soils						
Sand, loose, round	18.0	10.0	30	–	–	20 to 50
Sand, loose, sharp	18.0	10.0	32.5	–	–	40 to 80
Sand, medium-dense, round	19.0	11.0	32.5	–	–	50 to 100
Sand, medium-dense, sharp	19.0	11.0	35	–	–	80 to 150
Gravel without sand	16.0	10.0	37.5	–	–	100 to 200
Coarse gravel, sharp	18.0	11.0	40	–	–	150 to 300
Cohesive soils[a]	(values from experience for undisturbed samples from northern Germany)					
Clay, semi-solid	19.0	9.0	25	25	50 to 100	5 to 10
Clay, hard to knead, stiff	18.0	8.0	20	20	25 to 50	2 to 5
Clay, easy to knead, soft	17. 0	7.0	17.5	10	10 to 25	1 to 2.5
Glacial till, solid	22.0	12.0	30	25	200 to 700	30 to 100
Loam, semi-solid	21.0	11.0	27.5	10	50 to 100	5 to 20
Loam, soft	19.0	9.0	27.5	–	10 to 25	4 to 8
Silt	18.0	8.0	27.5	–	10 to 50	3 to 10
Tidal mud, organic, low clay content, soft	17.0	7.0	20	10	10 to 25	2 to 5
Tidal mud, highly organic, high clay content, soft, marine peat	14.0	4.0	15	15	10 to 20	0.5 to 3
Peat	11.0	1.0	15	5	–	0.4 to 1
Peat under moderate preloading	13.0	3.0	15	10	–	0.8 to 2

	Ground not compacted Stiffness modulus E_S MN/m²	Ground compacted Stiffness modulus E_S MN/m²
Fill[b]		
Gravel	40 to 80	80 to 150
Blast furnace slag	40 to 100	70 to 1500
Sand, pure	18 to 30	30 to 80
Demolition rubble	5 to 30	10 to 50
Mixed mining material	5 to 50	10 to 50
Burnt-out mining waste tip	2 to 30	5 to 40
Slate (tailings)	1 to 10	3 to 30
Ash, washed-out, coarse	2 to 6	5 to 15
Ash, washed-out, fine	2 to 4	4 to 12
Boiler slag	1 to 3	7 to 15
Waste	0.5 to 2	1 to 3

[a] Recommendation (E9) of the working committee "Bank revetment" (leader: Professor Dr.-Ing. E Lackner)
[b] Lecture by Professor Dr.-Ing. Schmidbauer at the building ground course in Essen, May 1963

Table 2-3 continued

Guideline values for permeability coefficient k_f			
Soil	k_f m/s	Soil	k_f m/s
Coarse river gravel	10^{-2}	Silty clay, clay	10^{-7} to 10^{-11}
Coarse gravel, river gravels and coarse sand	10^{-3} to 10^{-4}	Clay, greasy	10^{-9} to 10^{-10}
Sand	10^{-3} to 10^{-4}	Clay, silty	10^{-8} to 10^{-9}
Fine sand	10^{-4} to 10^{-5}	Loess, disturbed	10^{-9} to 10^{-10}
Fine sand and silt	10^{-4} to 10^{-7}	Mud	10^{-9} to 10^{-10}
Silt	10^{-5} to 10^{-8}	Bentonite	0.0033 mm/year

The coefficient of permeability k_f is dependent on soil type, grain composition, size of the individual pores and the pore volume or consolidation density. This value is a decisive for the selection of a suitable process of dewatering including compressed air, for the calculation of groundwater lowering and the assessment of the suitability of the undisturbed ground for grouting. It can be generally stated that soils with a permeability coefficient k_f of greater than 10^{-4} m/s are permeable, less than 10^{-6} m/s are semi-permeable and less than 10^{-8} m/s can be considered (nearly) impermeable [105].

2.3 Engineering geology and rock mechanics investigations

General. Geological engineering and rock mechanics investigations and the design and construction of underground cavities always have to be considered in relationship to each other. Design work should therefore be performed as a close cooperation between the responsible specialists for geology, rock mechanics, design and construction, and this cooperation should start at an early stage of preliminary investigations. This is followed by a description of the tasks and methods of engineering geology and rock mechanics investigations. More detailed information can be found in the relevant publications.

Binding guidelines for such investigations that are applicable for all cases can scarcely be given since ground conditions can vary widely from case to case. The aim of engineering geology and rock mechanics investigations is the assessment of the conditions below ground for construction purposes, entailing the creation of documents for safe and economical construction of the structure (selection of process) including the specification of construction measures for temporary and permanent support and lining of the rock cavity. At the preliminary design stage, suitable methods should be used to determine whether the planned structure is feasible in the intended location including consideration of technical and economic questions. It is important to find out which particular underground conditions and rock properties are of decisive significance. In the further course of processing the project design, the geological and rock mechanical conditions in the project area are determined more precisely and documents are produced for the design and structural design of the structure. Qualified statements should also be made for used by contractors for purposes of estimation, production of a construction schedule and the performance of the works.

Since the entire truth about the ground conditions is only discovered with the excavation of the cavity in the rock, the engineering geology and rock mechanics forecast should be checked as the excavation proceeds. If the encountered ground conditions vary from the forecast, the responsible experts should be informed and the planned measures for support and lining may have to be changes or supplemented.

The extent of all investigations should be suitable for the size and purpose of the structure and for the forecasts required about rock properties (for example strength, elasticity, permeability, in-situ stresses). In this regard it should be mentioned that the engineering geology and rock mechanics investigations are frequently not strictly separated and perhaps cannot even be separated. The assignment of tasks is very dependent on the appointed experts (engineering geologists with knowledge of rock mechanics, rock mechanics specialists with basic training in engineering geology). But the designer responsible for the structural verifications should also be involved in decisions because the assumptions made about loading and calculation have to consider the selection of a model.

2.3.1 Engineering geology investigations

Task. Engineering geology investigations should provide information about

- the rock mass around the planned underground structure,
- the extent of ground pressure or the primary stresses,
- the stability of the cavity,
- the requirement for temporary or permanent support,
- possible water ingress,
- the occurrence of dangerous gases and
- the occurrence of high temperatures.

Investigation of the rock mass also includes the description of the rock and the visible or latently formed interfaces and the determination of the groundwater or formation water conditions:

1. *Rock*
   Rock type
   Mineralogical composition
   Fabric: Structure (for example grain shape, arrangement, bonding and texture (for example layered texture in gneiss)
   State of weathering
   Resistance to air and water (weather resistance, swelling of clay minerals, solubility of salts and limestone (karst), solution pressure of salts)
   Hardness, brittleness, toughness

2. *Interface structure*
   Layout
   Spatial alignment, which is described by the angles of strike and dip (in a sphere diagram or joint rose)
   Spacing of parallel interfaces (frequency)
   Extent (degree of jointing)
   Opening width
   Filling of the individual joints (for example mylonite, calcite, quartz)
   Unevenness and roughness of the interfaces

3. *Formation water conditions*
   Level of the groundwater table
   Rock permeability, flow direction and seepage water quantities to be expected in the rock cavity
   Chemistry of the formation water (for example concrete aggressivity)

Investigation methods. There are various methods of investigating the engineering geological conditions, which can generally also be used for rock mechanical investigations and have therefore been developed in close cooperation between rock mechanics and engineering geology specialists. Direct methods of investigation (for example natural openings, ground movements, boreholes) and indirect investigation methods (mechanical and optical methods) can be differentiated. The most important processes are described in Table 2-4 and Table 2-5. For further descriptions, scope of application, information content and discussion, reference should be made to the work of Kany [102]. Boreholes are especially useful for rock mechanics investigations, so the information that can be gained from the drilling data, samples and the borehole is described in more detail in Table 2-5.

Table 2-4 Engineering geology investigation methods, from M. Kany [102].

Direct investigation methods		Indirect investigation methods	
Process	**Example**	**Process**	**Example**
Natural investigations	Walking the terrain, aerial photography	Evaluation of existing documentation, surface mapping	
Ground movement	Rock opening, test pits, Investigation tunnels and shafts, Test excavation in large or full profile	Mechanical methods	Penetration test
Drilling	Rotary drilling (with core extraction)	Optical methods	Optical sounding, Photographic sounding, TV sounding
		Geophysical methods	Seismology, Geoelectrics

Table 2-5 Properties of rock and rock mass that can be determined by drilling, from DIN 4021.

Row	Column		1	2	3	4	5	6	7	8	9	10	11	12	13	14
	Detectable properties		**Drilling process**			**Drilled samples**								**Drill hole**		**Certain improvement of detection through combination of the following columns**
						Core					**Cuttings**					
			observation of progress[1]	**loss or increase of flushing agent**[2]	**hole falling in**[3]	**complete**[4]	**incomplete**	**number of pieces**	**form of pieces**	**core loss**	**sieve retention**	**sediment**	**alteration of flushing fluid (colour, chemical)**	**television camera probe**	**various drillhole measurements**	
1	Rock	Type (grain size, mineral content)										+				9 & 10
2	Rock	Formation (grain shape, arrangement, bonding, spatial filling)	+													1 & 9 & 10 9 & 10 & 13
3	Rock	Colour, staining										+	+			9 & 10 & 11
4	Rock	Weathering condition			+					+		+	+			1 & 9 & 10 & 11
5	Rock	Decomposition (air/water/frost)	+		+				+	+			+	+	+	9 & 10
6	Rock	Permeability									+	+				
7	Rock	Strength, ductility			+						+					
8	Rock mass	Rock boundaries			+							+	+			1 & 9 & 10 & 11
9	Rock mass	Interfaces – Joint bodies – strike and dip														
10	Rock mass	Interfaces – Joint bodies – frequency (spacing)			+					+						
11	Rock mass	Interfaces – formation and course					+									
12	Rock mass	Interfaces – filling, coating					+				+	+	+		+	9 & 10 & 11
13	Rock mass	Cavities														
14	Rock mass	Permeability			+		+	+		+						
15	Rock mass	Strength, ductility					+	+		+						

Detectable | Not detectable | Incompletely detectable, but detectable with a certain experience | + Not generally detectable, but frequently detectable in certain ground types

[1]) Feed rate, type and condition of drill bit, pressure, flushing pressure and revolutions
[2]) Taking into consideration the type of flushing fluid and additives
[3]) Before performance of any drillhole support such as casing or consolidation
[4]) See Section 4.2 regarding core taking with orientation

2.3.2 Rock mechanics investigations

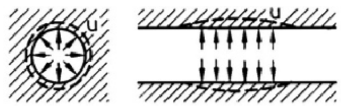
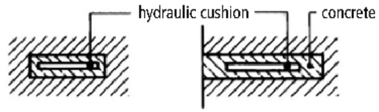
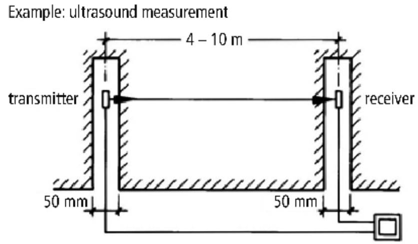
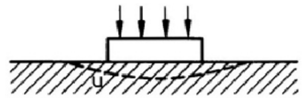
Task. Rock mechanics investigations mainly have the aim of determining the following rock properties:

- Elasticity.
- Strength.
- Primary stress state.
- Permeability.
- Solubility.
- Swelling propensity.
- Determination of the loosened zones to be expected around the perimeter of the cavity after excavation.

These parameters form the basis for determination of the free stand-up time of the rock mass, for planning of the construction process, structural calculations, design of the lining and support measures, determination of the deformation or settlement to be expected and the specification of in-situ measurements.

Investigation methods. The most important rock mechanics methods for the determination of rock properties are given in Table 2-6 (page 34).

Table 2-6 Rock mechanics investigation methods.

Process	Principle	Experimentally determined quantity	Quantity to be calculated	Remarks
Deformation properties				
1. Borehole expansion test		u	E, E_V	Investigated area is only slightly disturbed; various depths possible; loaded area small, thus the interface structure is only recorded to a limited extent
2. Pressure cushion test		u	E, E_V, λ	Investigation of the deformation anisotropy is simple by varying the arrangement of the pressure cushions
3. Ultrasound measurements		v_L, v_T	E_{dyn}, ν	Interpretation is difficult and often only unambiguous in combination with other tests
4. Plate pressure test		u	E, E_V	It is only possible to test near the surface. The values cannot normally be directly used for the design of a tunnel structure
5. Sample extraction with deformation measurement				See also Chapter IV

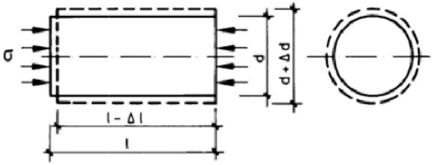
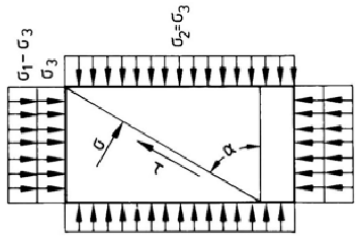
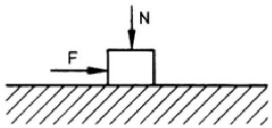
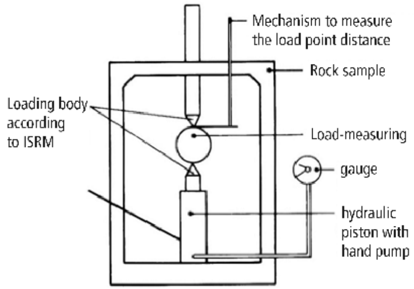
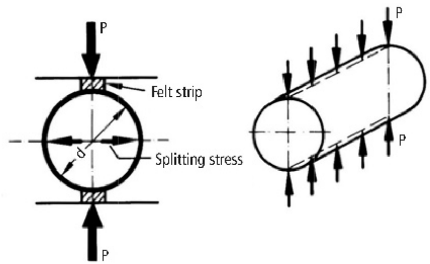
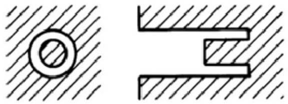
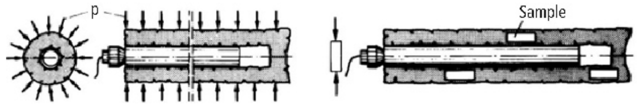
6. Radial pressure test	As 1, but in a tunnel or test heading	*u*	*E*, E_v	Very time-consuming, thus generally only used in larger pressure tunnels with high internal pressures or under difficult geological conditions
7. Uniaxial compression test		Δl, Δd, σ_u	E_v	Laboratory test: records the rock properties Conversion for rock mass properties is necessary (522)
8. Triaxial compression test		Δl, α, σ_{1u}, σ_{3u}	λ, c, φ	see also 7
Strength				
Rock mass strength 9. Direct shear test		F_u	τ_u	see also 6
10. Triaxial shear test	see also 8			

Table 2-6 continued

Rock strength 11. Uniaxial compression test	see also 7	F_u	σ_u	
12. Direct shear test	see also 9	F_u	τ_u	
13. Point load test				Very simple test
14. Splitting test (Brazilian test)				
Primary and secondary stress state				
Stress relief tests in excavation 15. Extensometer		P_u	$\sigma_{z, u}$	See also Chapter IV
16. Convergence measurements				See also Chapter IV

Stress relief tests in excavation 17. Doorstopper		u	$\sigma_1, \sigma_2, \theta_1, \theta_2$	Recording of the biaxial stress state at right angles to the borehole axis
18. Triaxial cell			$\sigma_1, \sigma_2, \sigma_3$ $\theta_1, \theta_2, \theta_3$	Recording of the triaxial stress state
19. Borehole expansion test				See also 1
Compensation measurements 20. pressure cushion test				See also 2
Stress measurement in the support 21. Mechanical, hydraulic and electrical measurement systems				See also Chapter IV
Water permeability of the rock mass (measured in the borehole)				
22. Wished-in-place, percolation and pumping tests (Lugeon test)		p, Q	k_f	Time-consuming. expensive: various evaluation criteria
Water permeability of the rock mass (in a larger zone)				
23. Dye, salting and isotope tests		Q	k_f	

Key to symbols

u Deflection	F Force	E Modulus of elasticity	c Cohesion	Q Water quantity
v_L Propagation velocity, longitudinal	N Normal force	E_v Modulus of deformation	φ Friction angle	k_f Water permeability
v_T Propagation velocity, transverse	θ Principal stress direction	ν Poisson's ratio	τ Shear stress	
σ Normal stress	P Line load	λ Lateral stress coefficient	p Grouting pressure	

2.4 The ground and its classification

2.4.1 Ground

Preliminary note. The part of the earth's crust, in which an underground cavity is constructed, is called the ground or subsoil by miners, tunnellers and rock engineers. The ground was created naturally and consists of soil or rock, or sometimes both in alternation. If the ground consists of rock, it is referred to as rock mass. The rock mass consists of an in-situ bond of similar or dissimilar rock bodies, which can show considerable anisotropy and discontinuity in their physical properties [50]. The properties of the rock mass are however not the same as the sum of the rock properties, and cannot be derived from them using a reduction factor.

Anisotropy occurs when the elements of the rock fabric are arranged with a preferred orientation. In contrast to isotropic bodies, the physical properties such as the modulus of elasticity depend on direction. All transitional forms from isotropy to high degree of anisotropy occur in natural rock (Fig. 2-3). For certain practical problems in construction in rock, however, structurally anisotropic rock bodies can be assumed to be almost quasi-isotropic.

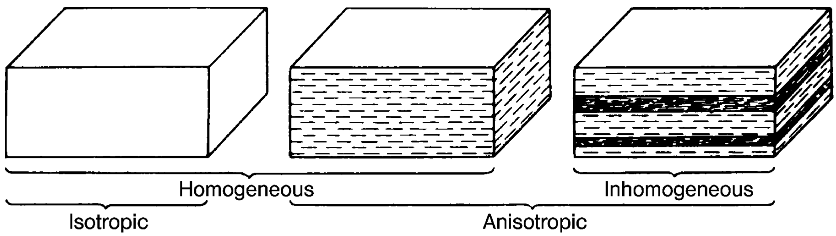


Figure 2-3 Illustration of the terms homogeneous, inhomogeneous, isotropic and anisotropic [166].

Discontinuum denotes a body interfused with discontinuity surfaces. The discontinuity surfaces in a rock body are the interfaces. They interrupt the shape and mechanical continuity of the rock body and have the effect of a sudden or discrete alteration of its physical properties like strength, compressibility or water permeability. The general terms discontinuity or interface include joint, fault, bedding, schistosity and cleavage surfaces. These weak points occur in all magnitudes from sub-microscopic to large fault zones with an extent of many kilometres (Fig. 2-4).

Rock mass properties. For the construction and operation of underground tunnels and caverns, the following properties of the rock mass are particularly significant:

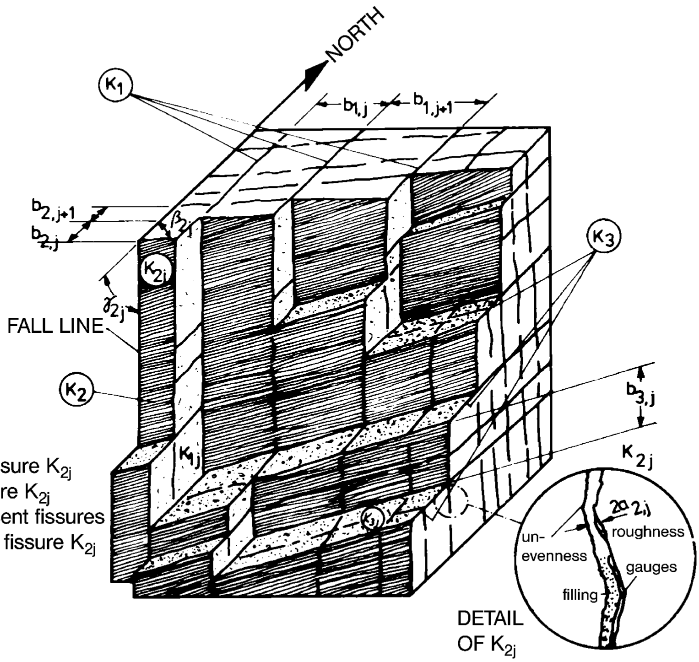
- Strength behaviour.
- Deformation behaviour.
- Rock mass properties significant for technical working methods.
- Properties, which describe the behaviour of the rock mass in relation to formation water.

These properties are a function of the geological and mineralogical composition of the rock mass, the mechanical properties of the rock and the geomechanical and geohydraulic behaviour of the rock composite in the widest sense.

Table 2-7 shows an excerpt of the properties from L. Müller. For the definitions, influences and ways of determining the properties, reference is made to [159].

Table 2-7 Rock mass properties (excerpt) from L. Müller [159].

Strength behaviour	Deformation behaviour	Properties related to technical working methods	Behaviour of the rock mass in relation to formation water
rock mass compressive strength rock mass tensile strength rock mass shear strength strength behaviour under spatial stress states rock mass toughness brittleness	rock mass elasticity transverse deformation rock mass partial moveability flow susceptibility time-dependent flow susceptibility creep susceptibility tendency to elastic after-effects thixotropy	stability rock fall susceptibility squeezing behaviour extraction strength softening susceptibility dimensional stability cuttability frost and weather resistance	water absorption rock mass permeability water flow suscepti-bility



β_{2j} Strike angle of the fissure K_{2j}
γ_{2j} Dip angle of the fissure K_{2j}
b_{1j} Spacing of two adjacent fissures
$2a_{2j}$ Opening width of the fissure K_{2j}
K_1, K_2 Fissure surfaces
K_3 Bedding surfaces

Figure 2-4 Interfaces in rock [49].

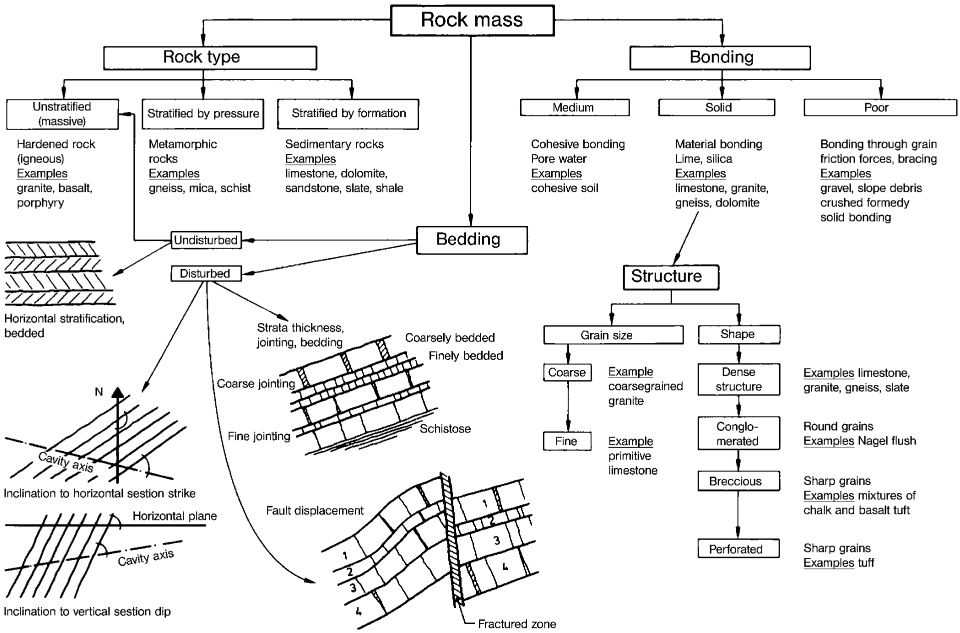


Figure 2-5 Factors of rock structure [34].

The mineralogical composition of a rock mass depends on the creation of the rock, which determines the type of rock and thus also its mechanical properties. The structure of the rock mass is characterised by its bedding, bonding and fabric. The factors of rock structure are illustrated in Fig. 2-5.

Undisturbed bedding is normally encountered in unstratified igneous rocks and in recent and very recent geological strata (sedimentary rocks of the diluvium, soils of the alluvium). Most geological units have, however, been disturbed in their bedding by tectonics (mountain formation processes). Tectonics is the main cause of the formation and inclination of interfaces and of faulting. Strike (angle) denotes the angle of the intersection of a rock surface with the horizontal plane related to geographical North. Dip denotes the angle between the fall line of a rock surface and the horizontal plane (Fig. 2-6).

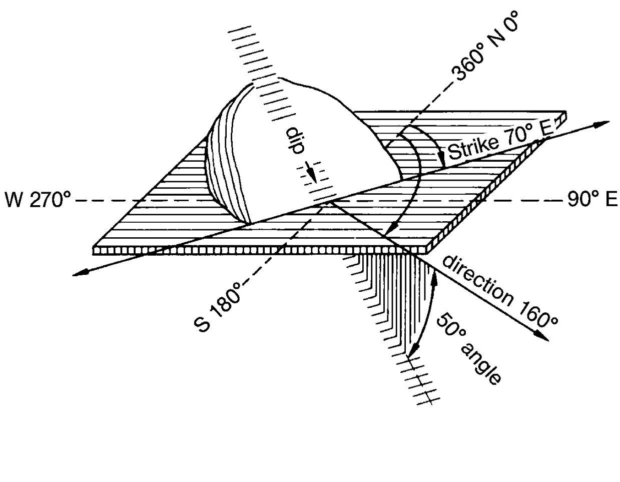


Figure 2-6 Strike and dip of a rock surface (stratum) [50].

A rock mass with coarse jointing is referred to as *coarsely jointed,* one with cracks as *finely jointed*. If the thickness of a bed or stratum is greater than 30 cm, then it is called *coarsely bedded,* thicknesses of 2 to 30 cm are *finely bedded* and thicknesses of less than 2 cm are *schistose or slaty* [34].

A *fault* is a sheared sequence of strata. The shear zone is normally extended and the bedding in the fault is fully destroyed (Fig. 2-5).

The components of the rock can be bonded together by substances (lime, silica), surface tension of the pore water (clay) or friction and bracing forces. This bonding can also be significantly disturbed by tectonic effects, but also particularly by weathering (bursting effect of ice, erosion by water and wind).

2.4.2 Classification of the rock mass

2.4.2.1 General

One of the important tasks in the design, tendering, construction and invoicing of tunnels is to classify the rock mass. The essential objective is to classify the rock mass along the tunnel alignment according to its properties and its behaviour with regard to the excavation and support of the cavity. In addition to the preparation of tenders, this provides the basis for the selection of a construction process, the specification of the required support measures and estimation. During the construction phase, the classification has to be continuously adapted to the actually encountered rock mass to serve as a basis for invoicing.

The behaviour of the rock mass as the tunnel is excavated is determined by the interacting factors of geology, geomechanics and tunnel construction. This complexity justifies the demand that tunnel engineers should possess interdisciplinary knowledge, particularly of geotechnics.

2.4.2.2 Basic system of classification

Until the start of the 1950s, classification of the rock mass for a tunnel was still based on a predominantly geological and purely qualitative description. This was assigned in relation to the possible methods of support, which were decided according to the experience of the specialists involved in the project. Only the combination of rock mechanics and construction technology aspects with the geological description to give a partially quantitative evaluation led to a practically oriented reference to the planned structure. More recent classifications were based on the work of Terzaghi [241] and Stini [234], who were the first to propose a classification according to the behaviour of the rock mass (Table 2-8). Lauffer [121] in 1958 selected the stand-up time and the free span length as criteria for his generally accepted and applied classification process. The system of rock mass properties related to technical working methods developed by Müller, Rabcewicz, Pacher and others [159, 175] together with the introduction of shotcrete in 1957, and later revised many times [compare 130], divides the rock mass classes according to the composition of the rock mass and its behaviour during the creation of a cavity. Seeber attempted, in his proposed classification based on Mohr, to emphasise the differentiation of squeezing rock mass behaviour from brittle behaviour [219].

Thanks to theoretical advances in rock mechanics, quantitative parameters came to be introduced ever more for classification purposes. Particularly worth mentioning are the works of Wickham [256], Barton [14], Bieniawski [22] as well as John and Baudendistel [99]. Since the end of the 1960s, the tendency has moved away from generally applicable rock mass classifications to project-related classification. Further details are dealt with in [142].

Table 2-8 Comparison of basic classification systems.

K. Terzaghi	11 rock mass classes (1 to 11) from "competent rock" to "loosely consolidated sand"
G.E. Wickham	No fixed classes but RSR (Rock Structure Rating) values from 25 to 100, which denote improving rock mass quality with increasing values
J. Stini	5 rock mass classes (1 to 5) from "rock more or less brittle" to "mild rock mass, very squeezing"
H. Lauffer	7 rock mass classes (A to G) from "stable" to "very squeezing"
G. Seeber	3 rock mass classes (1 to 3) from "stable" to "squeezing"
F. Pacher, L. v. Rabcewicz	6 (5) rock mass classes (I to Vb) from "stable" to "loose" (Tauern Tunnel)
W. Berger	7 support classes (0 to 6) from "without structural support" to "special measures"
Z.T. Bieniawski	5 rock mass classes (I to V) from "very good rock" to "very poor rock" with ratings from 100 to 1
N. Barton	No fixed classes but Q (Quality) values from 0.001 to 1000 for "exceptionally poor soil" to "exceptionally good rock"
K.W. John and M. Baudendistel	7 Tunnelling Procedure Classes, (TPC-1 to TPC-7) from "very favourable" to "very unfavourable" with evaluation from 100 to 0 %

2.4.2.3 Q System (Quality System)

The Q System of rock mass classification was developed in 1974 by Barton, Lien and Lunde in Norway [14, 22]. The system is based on an analysis of more than 200 tunnel projects in Scandinavia. Thanks to this analysis, it can be described as a quantitative system. It is intended as a system for engineers to simplify the design of tunnel support.

The Q System is based on the numerical estimation of the following six parameters:

1. Determination of the rock mass quality (RQD = Rock Quality Designation)
2. Number of joint sets as the joint set number (J_n)
3. Roughness of the least favourable joint as the joint roughness number (J_r)
4. Degree of alteration or filling of the weakest joint as the joint alteration number (J_a)
5. Water ingress as the joint water parameter (J_w)
6. Stress conditions (SRF = Stress Reduction Factor)

These six parameters are collected into three quotients, which give a weighted mathematical value for the rock quality Q calculated as follows:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

The value of Q (= rock quality) calculated according to this formula lies in a range from Q = 0.001 to Q = 1000. The value for Q is entered in a diagram with a logarithmic scale and the rock classes can be read off (Fig. 2-7).

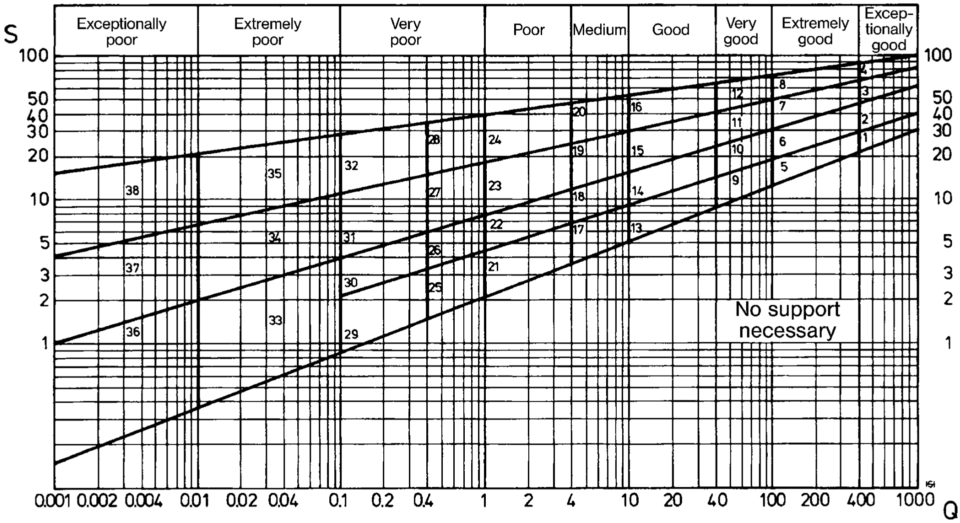


Figure 2-7 Q System: Reference value for the support in comparison to the rock quality Q.

Classification procedure: Table 2-9 to Table 2-14 give the numerical values for each of the factors listed above. They are interpreted as follows:

The first two factors, *RQD* and J_n (joint set number), give an overview of the structure of the rock mass and their quotient as a relative dimension for the joint body size in the drill core. The *RQD Index* proposed as a rock characteristic by D.U. Deere and A.J. Hendron [40] is used. It gives the ratio L_{10}/L in percent, where L_{10} denotes the number of pieces more than 10 cm long contained in core length L.

Table 2-9 *Q* System: Determination of the rock quality – *RQD* [14, 22].

Description	Range of values RQD	Remark
very poor	0 to 25	(i) In case the measured value of *RQD* is less than 10, the value of *RQD* = 10 can be used in the last formula.
poor	25 to 50	(ii) The grading of the values of *RQD* in intervals of 5 is sufficient for further consideration.
fair	50 to 75	
good	75 to 90	
excellent	90 to 100	

Table 2-10 *Q* System: joint set number – J_n.

Description	Range of values J_n	Remark
massive rock, no or few joints	0.5 – 1.0	(i) For junctions of tunnels and cross-passages, $(3.0 \cdot J_n)$ is to be used. (ii) For portal areas, $(2.0 \cdot J_n)$ is to be used.
one joint set	2	
one joint set plus random joints	3	
two joint sets	4	
two joint sets plus random joints	6	
three joint sets	9	
three joint sets plus random joints	12	
four or more joint systems, random joints, heavily jointed "sugar cube" etc.	15	
crushed rock, similar to earth	20	

Table 2-11 *Q* System: joint roughness number – J_r.

Description		Range of values RQD	Remark
(a)	Rock wall contact and		(i) Add 1.0 if the average spacing between the relevant joint sets is more than 3 m. (ii) $J_r = 0.5$ can be used for planar slickenside surfaces as long as the striation is favourably aligned.
(b)	Rock face contact before 10 cm shear		
	discontinuous joint	4.0	
	rough or irregular, undulating	3.0	
	smooth, undulating	2.0	
	slickenside, undulating	1.5	
	rough or irregular, planar	1.5	
	smooth, planar	1.0	
	slickenside, planar	0.5	
(c)	No rock wall contact when sheared		
	the shear zone contains clay minerals sufficiently thick to prevent rock wall contact.	1.0	
	sandy, gravelly or crushed zones sufficiently thick to prevent rock wall contact.	1.0	

Table 2-12 *Q* System: joint alteration number – J_a.

Description		Range of values J_a	ϕ_r (approx.)
(a)	**Rock wall contact**		
A	densely filled, solid, non-softening, impermeable filling	0.75	25 to 35°
B	unaltered joint walls, only surface staining	1.0	
C	slightly altered joint walls, non-softening mineral surface, sandy particles, unbonded disintegrated rock without clay	2.0	25 to 30°
D	silty or sandy clay surface, small clay fraction (non-softening)	3.0	20 to 25°
E	clay mineral surface indicating softened or slight friction	4.0	8 to 16°

Table 2-12 continued

Description		Range of values J_a	ϕ_r (approx.)
(b)	**Rock wall contact before 10 cm shear**		
F	sandy particles, unbonded disintegrated rock without clay	4.0	25 to 30°
G	strongly over-consolidated, non-softening clay mineral filling (continuous, < 5 mm thick)	6.0	16 to 24°
H	medium or weakly over-consolidated, softened clay mineral filling (continuous, < 5 mm thick)	8.0	12 to 16°
J	swelling clay filling, for example montmorillonite (continuous, < mm thick). The value of J_a depends on the content by percent of clay-type minerals susceptible to swelling and the ingress of water.	8.0 to 12.0	6 to 12°
(c)	**No rock face contact in the shear jointing**		
K	zones or drifts with disintegrated or crushed rock and clays (compare also G, H and J for the clay description)	6.0, 8.0 or 8.0 to 12.0	6 to 24°
L	zones or bands of silty or sandy clays, small clay fractions (non-softening)	5.0	
M	thick, continuous zones or bands of clay (compare also G, H and J for the clay description)	10.0, 13.0 or 13.0 to 20.0	6 – 24°

Note: (i) The values for ϕ_r should be regarded as approximate reference values for the mineral properties of the altered products, in case such are present.

Table 2-13 *Q* System: Joint water parameter – J_w.

Description		J_w	Approximate water pressure [kg/cm²]	Note
A	Dry excavation or slight water ingress up to 5 l/min	1.0	< 1	(i) The values for cases C-F are coarse estimates. Increase J_w in case dewatering measurements are available. (ii) Special problems caused by ice formation have not been considered.
B	Medium water ingress or pressurised water loss of individual joints	0.66	1.0 to 2.5	
C	Heavy water ingress or high pressure in stable rock mass with unfilled joints	0.5	2.5 to 10.0	
D	Heavy water ingress or high pressure, considerable water loss from individual joints	0.33	2.5 to 10.0	
E	Exceptionally heavy water ingress or water pressure after blasting, which declines with time	0.2 to 0.1	> 10.0	
F	Exceptionally heavy water ingress or persistent water pressure with no noticeable reduction	0.1 to 0.05	> 10.0	

The quotient of the third, J_r (joint roughness number), and the fourth factor, J_a (joint alteration number), can be considered an indicator of the shear strength at the interfaces between the individual blocks of rock.

The fifth parameter, J_w (reduction factor for the influence of joint water), is a measure of the water pressure, while the sixth factor, *SRF* (stress reduction factor), permits various interpretations:

– loosening pressure in the case of shear surfaces and rock containing clay,
– ground pressure in a stable rock mass and
– squeezing and swelling pressures in plastic, instable rock mass.

This sixth parameter is regarded as the overall pressure parameter. The quotient of the fifth and sixth parameters describes the active stress.

Barton regards the parameters J_n, J_r and J_a as more significant than joint orientation. This is implicitly included in the parameters J_r and J_a, since these relate to the most unfavourable joint.

Using the Q value, it is indirectly possible to come to a conclusion about the necessary support measures with reference to the actual size of the cross-section and the intended use of the tunnel. The ratio for the support measures is a function of the constraints described above. It is determined by dividing the opened length, the diameter or the height of the excavated cross-section by a defined factor determined by the intended use of the structure, the Excavation Support Ratio ESR.

$$S = \frac{A_L, D \text{ or } H \ [m]}{ESR} \quad \text{with}$$

S equivalent support measures number
A_L excavation length [m]
D tunnel diameter [m]
H cross-section height [m]
ESR Excavation Support Ratio

Table 2-14 *Q* System: Stress Reduction Factor – *SRF*.

Description		Range of values	Note
(a)	Weak zones at cross-passages, which could cause a loosening of the rock mass on excavation.		Reduce the given values for the *SRF* by 25 to 50 % in case the relevant shear zones only influence but do not cross the excavation.
A	Multiple occurrence of weak zones with clay or chemically stabilised rock, very loosened surrounding rock (any overburden)	10.0	
B	Isolated weak zones with clay or chemically stabilised rock (overburden ≤ 50 m)	5.0	
C	Isolated weak zones with clay or chemically stabilised rock (overburden > 50 m)	2.5	
D	Multiple shear zones in stable rock mass (clay-free), loosened surrounding rock (any overburden)	7.5	
E	Isolated shear zones in stable rock mass (clay-free) (overburden ≤ 50 m)	5.0	
F	Isolated shear zones in stable rock mass (clay-free) (overburden > 50 m)	2.5	
G	Loosened, open joints, heavily jointed or "sugar cube-like", etc. (any overburden)	5.0	

Description		$\frac{\sigma_c}{\sigma_1}$	$\frac{\sigma_t}{\sigma_1}$	Range of values	Note
(b)	Stable rock mass, rock mass stress problems				If heavily anisotropic stress fields are measured, the following applies: if $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8 \cdot \sigma_c$ and $0.8 \cdot \sigma_t$; if $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6 \cdot \sigma_c$ and $0.6 \cdot \sigma_t$; with: σ_1 and σ_3 principal stress and secondary stress, σ_c = uniaxial compression strength and σ_t = tension strength
H	Little pressure, near surface	> 200	> 13	2.5	
J	Medium pressure	200 to 10	13 to 0.66	1.0	
K	High stress, very dense structure	10 to 5	0.66 to 0.33	0.5 to 2.0	
L	Slight rock spalling (massive rock)	5 to 2.5	0.33 to 0.16	5 to 10	
M	Heavy rock spalling (massive rock)	< 2.5	< 0.16	10 to 20	

Description		Range of values	Note
(c)	Squeezing rock, plastic yielding of unstable rock under the influence of high ground pressures		Very few cases are known, in which the depth of the top heading below ground level is more significant than the diameter. For such cases, the value of *SRF* should be increased from 2.5 to 5 (see also H)
N	Slightly squeezing ground pressure	5 to 10	
O	Heavily squeezing ground pressure	10 to 20	
(d)	Swelling ground, chemically caused swelling due to reaction with water		
P	Slightly swelling ground pressure	5 to 10	
R	Heavily swelling ground pressure	10 to 15	

Table 2-15 Values for the *ESR* depending on the intended use.

Excavation category		ESR
A	temporary excavations	3 to 5
B	shafts with a round cross-section	2.5
	shafts with a rectangular cross-section	2.0
C	permanent excavations, unpressurised water tunnels for hydropower systems, pilot tunnels, headings and directional tunnels for large excavations	1.6
D	storage caverns, water treatment works, smaller road and rail tunnels, surge tanks and access adits	1.3
E	power stations, large motorway and rail tunnels, civil protection bunkers, portal areas, connections between tunnels	1.0
F	underground atomic power stations, railway stations, factories	0.8

The *ESR* relates to the use of the excavated cavity and the required degree of safety, see also Table 2-15.

The relationship between the Q index and the reference parameter for the support determines the appropriate support measures. For temporary support, either the Q value is increased to 5 or the ESR to 1.5 *ESR*, if the temporary support is intended for a period of less than one year. The *ESR* can be reduced to 0.5 for long transport tunnels. It should be noted that the required anchor length is determined as follows, with *B* being the excavated width:

$$L = \frac{2 + 0.15 \cdot B}{ESR}$$

The maximum unsupported round length (unsupported) = 2 $ESR \sqrt[5]{Q^2}$

The relationship between the Q value and the permanent support pressure P can be determined from the following formula:

$$P = \frac{2.0}{J_\mathrm{r}} \frac{1}{\sqrt[3]{Q}}$$

In case the number of joint sets is less than three, the formula can be expressed as follows:

$$P = \frac{2}{3}\sqrt{J_\mathrm{n}}\,\frac{1}{J_\mathrm{r}}\frac{1}{\sqrt[3]{Q}}$$

2.4.2.4 RMR System (Rock Mass Rating System)

The *RMR* system [23, 230, 22] is mainly used in countries with American influence. It was essentially developed by Z. T. Bieniawski in the years 1972 and 1973. The system can also be referred to as the “Rock Mass Rating System” or also as “Geomechanics Classification”. Over the last twenty years, the system has been confirmed in over 350 practical applications and has been continuously improved with supplements. The RMR system, like the Q system, is a quantitative classification system, which is calibrated from completed projects and is thus continuously improved.

Classification procedure. The following six parameters are used for the rock mass classification in the *RMR* system:

1. Uniaxial compression strength of the rock material.
2. Determination of the rock mass quality (*RQD*).
3. Discontinuity spacing.
4. Condition of the interfaces.
5. Water ingress.
6. Discontinuity orientation.

In order to used the geomechanical classification, the rock mass is split into sections, in which the condition of the rock has nearly the same properties. Although the rock mass as a natural material is not homogeneous, individual sections can be delineated according to the already mentioned aspects and used for the investigation (homogeneous sections). The characteristic properties of section are entered in a data sheet and evaluated with the aid of Table 2-16 and Table 2-17. It is important that Table 2-16 can be used independently of the orientation of any faults and the results are then corrected using Table 2-17 for their orientation and for the structure to be constructed.

Table 2-16 Classification parameters and their evaluation.

	Parameter	Range of values							
1	Strength of the intact rock (Mpa)	Point load strength index	> 10	4 – 10	2 – 4	1 – 2	-		
		Uniaxial compression strength	> 250	100 – 250	50 –100	25 – 50	5 – 25	1 – 5	< 1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD [%]		90 – 100	75 – 90	50 – 75	25 – 50	< 25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2	0.6 – 2	0.2 – 0.6	0.06 – 0.2	< 0.06		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		very rough surface, not continuous, no separation, unweathered wall rock	slightly rough surface, separations < 1 mm, weathered walls	slightly rough surface, separations < 1 mm, highly weathered walls	slickenside surface or slip < 5 mm or separations 1 – 5 mm continuous	soft slips > 5 mm or separations > 5 mm continuous		
	Rating		30	25	20	10	0		

Table 2-16 continued

	Parameter	Range of values					
5	Ground-water	Inflow per 10 m tunnel length [l/min]	none or	< 10 or	10 – 25 or	25 – 125 or	> 125 or
		Joint water pressure to principal stress	0 or	< 0.1 or	0.1 – 0.2 or	0.2 – 0.5 or	0.5 or
		General condition	completely dry	damp	wet	drips	streaming
	Rating		15	10	7	4	0

Table 2-17 Evaluation correction for the strike direction of the fault.

Strike and dip direction of the fault		**Specially favourable**	**Favourable**	**Acceptable**	**Unfavo-urable**	**Very unfa-vourable**
Evaluations	Tunnels and mines	0	up to 2	up to 5	up to 10	up to 12
	Foundations	0	up to 2	up to 7	up to 15	up to 25
	Slopes	0	up to 5	up to 25	up to 50	up to 60

If the results of the two tables are added, this gives a characteristic value, which enables assignment to a rock mass class with the aid of Table 2-18. The higher this value is, the better is the prevailing rock. The added range of values lies between 0 and 100, bad to very good.

In Table 2-19, the practical evaluation of the individual rock mass classes is explained using examples from engineering practice. Since the rock mass consists of the most varied sections, those with the most unfavourable faults for the future structure are decisive. Future construction measures have to be planned for this section, although the situation with regard to rock strength and other parameters may be good. In case two sections with different parameters dominate the entire cross-section, the evaluation numbers are weighted according to their area of occurrence and averaged to one characteristic value.

Table 2-18 Determination of rock mass classes from the overall evaluation.

Evaluation	**100 – 81**	**80 – 61**	**60 – 41**	**40 – 21**	**< 20**
Rock mass class	I	II	III	IV	V
Description	very good rock	good rock	acceptable rock	bad rock	very bad rock

Table 2-19 Evaluation of rock mass classes.

Rock mass class	I	II	III	IV	V
Average free stand-up time	self-supporting over 15 m for 20 years	self-supporting over 10 m for 1 year	self-supporting over 5 m for 1 week	self-supporting over 2.5 m for 10 hours	self-supporting over 1 m for 30 minutes
Cohesion of the rock mass [kPa]	> 400	300 to 400	200 to 300	100 to 200	< 100
Friction angle of the rock mass [°]	> 45	35 to 45	25 to 35	15 to 25	< 15

Strengths and limits. The RMR system is very simple to use; the classification parameters can be gained from analysis of drill cores or from geomechanical records. This procedure is applicable and adaptable to many situations in mining, for the stability of foundations and slopes and in tunnelling. The geomechanical classification is very well suitable for use in expert systems. On the other hand, the results of the RMR classification method tend to be conservative, which mostly leads to an over-dimensioning of support measures. This can be compensated by continuous monitoring during the construction period, with the evaluation system being adapted to local conditions.

2.4.2.5 Relationship between Q and RMR systems

Working from over 100 cases studies, it proved possible to establish an originally unintended, empirical relationship between the *RMR* and *Q* systems [23, 230, 22]. For tunnels, this can be given as:

$$RMR \approx 9 \cdot \mathrm{In}Q + 44 \qquad \text{or} \qquad Q \approx e^{\frac{(RMR-44)}{9}}$$

Barton sees the relationship as given by the following formula:

$$RMR \approx 15 \cdot \log Q + 50 \qquad \text{or} \qquad Q \approx 10^{\frac{(RMR-50)}{50}}$$

The relationship between *Q* and *RMR* is also very well visible in Fig 2.8.

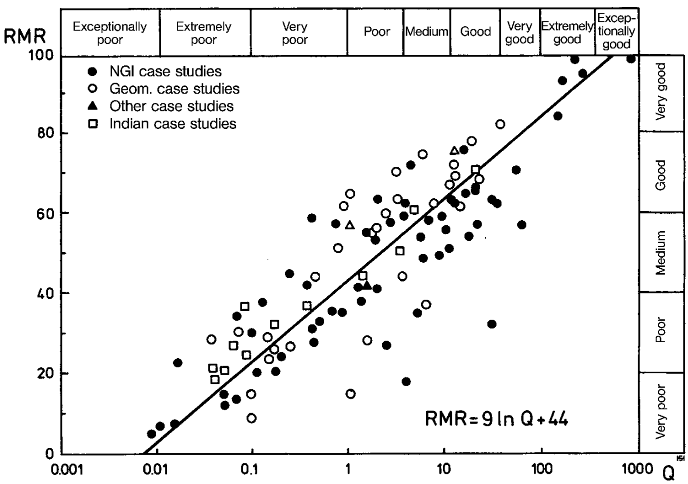


Figure 2-8 Relationship between RMR and Q systems.

2.4.3 Standards, guidelines and recommendations

The classification becoming established in German-speaking countries today is based on a classification of the rock mass as a construction material and its behaviour when the tunnel is excavated and supported. The main emphasis is placed on systematic grading of the works that determine performance of tunnelling and support. The influence on time is also considered in order to quantify obstructions. In newer guidelines, rock mass and support are generally collected into tunnelling classes, which are based on all works with an influence on the advance rate. Detailed, quantified classification is reserved for the project-related tunnelling classification.

In Austria [173], classification is carried out according to the excavation and support works that determine the advance rate, like type of excavation and round length differentiated according to top heading, bench and invert; and type, extent and installation location of support, depending on the rock mass behaviour to be expected as the tunnel is driven.

In Switzerland, classification is also carried out according to the excavation and support works that determine the advance rate. The behaviour of the rock mass as the tunnel is driven is considered, as in Germany, without additional direct assignment of rock mass types through the type of excavation depending on rock mass and type and extent of support [218].

The classification systems in Austria and in Switzerland can be directly transferred into a project-related classification through the numbered grading of the round lengths and the support in the normal case.

Special features in the standards, guidelines and recommendations related to mechanised tunnelling are dealt with in Section 2.5.6.

2.4.3.1 Classification in Germany

DIN 18312 VOB Part C "General technical contract conditions for construction works – Tunnelling", issue 12/2002 [59]. Tunnelling classification for tunnel construction projects is regulated by the DIN 18312 VOB Part C 2002-12 in Germany. The contract classification according to this standard makes the assumption that the cross-sectional size and shape of the cavity and the construction process and excavation and support measures are specified and the grading of soil and rock into the individual classes is only undertaken in compliance with these specified data.

In the very generalised tunnelling classification, classes 1 to 7A are differentiated for conventional tunnelling according to the extent of support work and the resulting obstruction or delay to the advance (Table 2-20), and for tunnel boring machine drives the classes TBM 1 to 5 (Table 2-29) and for shield machine drives the classes SM 1 to 3 (Table 2-30). The special features of mechanised tunnel drives are explained in Section 2.5.6.1.

Table 2-20 General tunnelling classes (VKL) [59].

VKL	Type of excavation
1	Excavation without support
2	Excavation with support, which can be installed together with the construction process so that excavation and loading are not obstructed
3	Excavation with support following at a short distance behind the face (for vertical shafts: shaft floor or crown), for the installation of which excavation and loading have to be interrupted
4	Excavation with immediately following support
4A	Excavation according to tunnelling class 4 with division of the excavated section for reasons of stability
5	Excavation with immediately following support including support to the face
5A	Excavation according to tunnelling class 5 with division of the excavated section for reasons of stability
6	Excavation with immediately following support and pre-support
6A	Excavation according to tunnelling class 6 with division of the excavated section for reasons of stability
7	Excavation with immediately following support including support to the face and pre-excavation support measures
7A	Excavation according to tunnelling class 7 with division of the excavated cross-section for reasons of stability

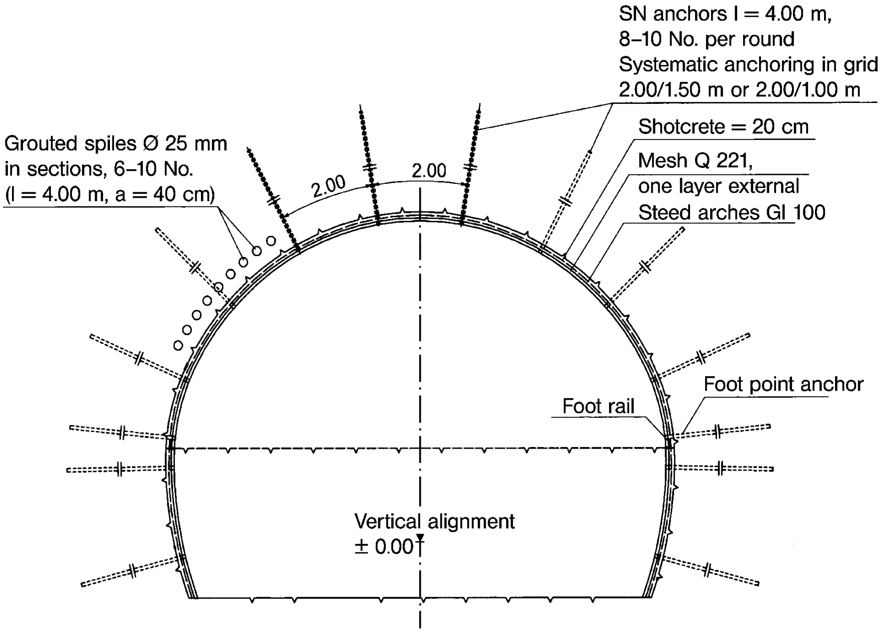
Guideline 853 "Design, construction and maintenance of rail tunnels" of the Deutsche Bahn AG, Issue 06/02. The design, construction and maintenance of rail tunnels for German Railways have to comply with guideline 853 [193], which specifies that a tunnelling classification according to DIN 18312 is to be undertaken for the production of tender documents. Previous experience with similar ground conditions should also be taken into account for the classification. The support measures necessary for each tunnelling class are to be stated. Alternative tunnelling classes are also to be tendered for each tunnel section.

The tunnelling classes and tunnelling methods to be used in each case are determined during the construction phase, proposed by the contractor in agreement with the client. The tunnelling classes are determined according to the condition of the "freshly" excavated rock mass. In order to decide any differences of opinion between the contract parties concerning the determination of tunnelling classes that cannot be agreed, an accredited independent expert should be agreed between contractor and client as an arbitrator and appointed at the conclusion of the contract [193].

Additional technical contract conditions and guidelines for engineered structures, Part 5: Tunnelling, Section 1: Confined mode, (ZTV-ING, T.5-A.1), Issue 01/2003 [263]. This guideline also includes the classification of excavation and support in tunnelling classes according to DIN 18312. Sub-categories are normally required for each specific project. Essential points for further sub-division could include:

– the selection of individual support measures and the time of their installation,
– the effect of water ingress and
– the round length.

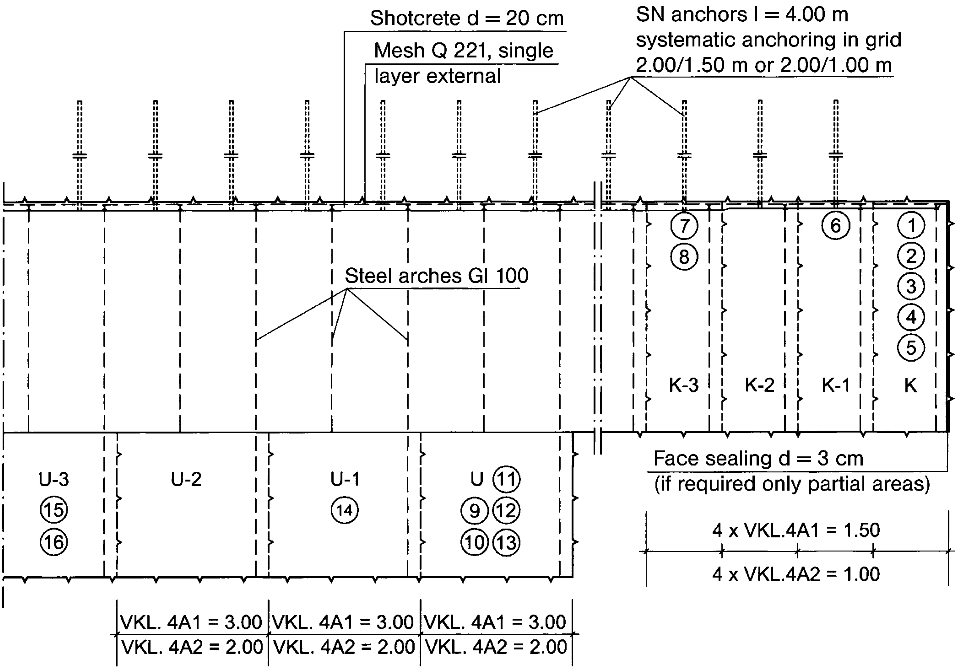
The specification (excavation and support) required for each excavation class is described and shown in drawings according to the RAB-Ing (guideline for the production of designs for engineering structures, issue 1995) [183]. The sequence and extent of support in the individual working stages are also laid down. An adequate range of options should be provided for excavation and support. Figs. 2.9 and 2.10 show the example given in the ZTV-Ing, T.5-A.1.



	Tunnelling class VKL 4A1/4A2	Top heading
Excavation	Process	blasting
	Round lengths	4A1 1.50 m 4A2 1.00 m
	Remarks	–
Support	Presupport	grouted spiles Ø 25 mm in sections 6–10 No.
	Shotcrete	d = 20 cm
	Reinforcement	one layer Q 221, external
	Anchors	SN anchors l = 4.00 m 8–12 No.
	Arches	Steel arches Gl 100
	Top heading invert	–
	Face	d = 3 cm (if required only for partial areas)
	Remarks	–

	Tunnelling class VKL 4A1/4A2	Bench / shoulder
Excavation	Process	blasting
	Round lengths	4A1 1.50 m 4A2 1.00 m
	Remarks	–
Support	Presupport	–
	Shotcrete	d = 20 cm
	Reinforcement	one layer Q 221, external
	Anchors	SN anchors l = 4.00 m 2–4 No.
	Arches	Steel arches Gl 100 (each 2nd arch extended)
	Top heading invert	–
	Face	–
	Remarks	–

Figure 2-9 Illustration of the excavation classes and the required support in cross-section (example according to ZTV-Ing, T.5-A.1 [263]).



	Working steps, top heading
1	Advance field K
2	Shotcrete sealing d = 3 cm field K
3	Single layer of mesh Q 221 field K
4	Erect arches field K
5	Spray to inside of arches field K
6	Set anchors field K-1
7	Tighten anchors field K-3
8	Remaining shotcrete field K-3

	Working steps, bench
9	Advance field U
10	Shotcrete sealing d = 3 cm field U
11	Single layer of mesh Q 221 field U
12	Erect arches field U (extend every 2nd arch)
13	Spray to inside of arches field U
14	Set anchors field U-1
15	Tighten anchors field U-3
16	Remaining shotcrete field U-3

Figure 2-10 Illustration of the excavation classes and the required support in longitudinal section (example according to ZTV-Ing, T.5-A.1 [263]).

During the construction phase, the excavation class to be used is agreed between client and contractor in each case. The contractor proposes the excavation classes. Repeated changing of the excavation classes has to be expected, although there is no special payment for the changeover itself [263].

Recommendations of the working group “Tunnelling” of the Deutsche Society for Geotechnics (ETB) [52]. This also recommends the use of the general classification scheme according to DIN 18312, which should be used as a basis for the project-related classification of a tunnel in each case. The purpose of this is:

– before the construction phase, to implement the design work into tunnelling classes to enable performance-related price formation for tunnelling in various ground conditions,
– during the construction phase, to regulate on site the selection of the excavation class and support measures that are appropriate for the ground conditions and
– after the completion of construction, to provide a basis for payment and comparability for later tunnel projects.

The excavation and support and in some cases support to the face are the works, which determine the advance rate. For this reason, this recommendation also selects the description tunnelling class, which corresponds to the DIN 18312 (draft 4/1995).

A division into sub-classes can achieve a refinement of the general tunnelling classification, for example by consideration of additional obstructions. The following special conditions are given for universal cases and also for TBM and shield machine tunnels:

– Exceptional geological-hydrogeological conditions.
– Change of tunnelling class (which is not considered through the tunnelling class itself, but through an appropriate item in the bill of quantities).
– Effects on the groundwater or measures to limit them.
– Difficulties caused by groundwater or formation water (generally considered through obstruction items in the bill of quantities and not in the tunnelling classes).
– Data given by the bidder about advance rates for the relevant tunnelling classes.
– Tunnelling beneath buildings/facilities that are susceptible to settlement.

Definite special situations related to a specific project are only included for universal tunnelling (blasting, roadheaders, excavator, manual) as follows:

– Quantities for the type and extent of support in bands (shotcrete thicknesses, number and length of rock bolts, type of arches and spacing, type of reinforcement: one or two layers. A certain overlap between the bands in adjacent tunnelling classes is useful).
– Details of round lengths and the free unsupported span.
– Details of the installation location and sequence of support for all stages of excavation.
– If appropriate, limitation of the advance rate to achieve the load-bearing capacity of the support (shotcrete/bolt grouting).
– Restrictions due to building or facilities when the overburden is shallow.
– If appropriate, restrictions taking blasting vibration into consideration.
– Consideration of extensive advance measures to consolidate or waterproof the rock mass (grouting, freezing) and if appropriate special classes.
– Consideration of the special features of shafts and caverns.

The special features in the ETB for tunnel boring machines (TBM) and shield machines are dealt with in Section 2.5.6.1.

The classification should be performed as part of the design for the client. It is based on:

- the ground conditions recorded by geotechnical investigations and the evaluation of their effect on tunnelling.
- The shape and size of the cavity.
- The tunnelling method, which is determined by the type of excavation and support.

After the classification into project-related tunnelling classes, a forecast of the proportions of the various tunnelling classes along the tunnel is produced, with clearly different ground conditions being delineated. On site, the tunnelling classes are agreed between the contractor and the client based on the approved construction design in the form of support specifications before each excavation process. If the opinions differ, then the client specifies the tunnelling class. The tunnelling classes that are finally executed are to be documented in an appropriate drawing and compared to the forecast tunnelling class [43].

2.4.3.2 Classification in Switzerland ("Klassierung" according to the SIA standard)

Excavation classes according to the Swiss engineers' and architects' association SIA 198 [218]. The standard SIA 198 – Underground Structures – was issued in 1993 and is part of the tender documents and construction contract in Switzerland. It applies to the tendering and construction of underground works like mined tunnels, caverns and shafts, and includes drill and blast, excavation by roadheader or TBM and tunnels in loose ground. Section 5.2 of the standard concerning the payment of excavation works for drill and blast in rock and is based on the idea that the costs of the construction of an underground cavity are directly related to the type and extent of the necessary support works, but also with the point in time that such measures have to become effective. The SIA 198 therefore chooses the support measures as criterion for the excavation classes. Classification is therefore based, on one hand, on the type and extent of support measures, specifically rock blots, shotcrete, steel arches with or without lagging, and on the other hand on the location of implementing these measures and thus indirectly their time of installation. The following areas are differentiated:

- face area L1,
- advance area L2 and
- rearward area L3.

The linking of type, extent and installation location of the support measures gives the relevant excavation class. Support measures outside the designated working area have no effect on the classification of the excavation. Table 2-21 defines the five excavation classes used for drill and blast in rock for any cross-sectional form and size. Quantitatively, the measures are not laid down rigidly in the standard in order that the various local conditions can be taken into account. Guideline values are, however, given for the values to be taken according to profile size (Table 2-22).

The main idea behind the firm definition of excavation classes should in every case by the degree of obstruction to the tunnelling works from the support works, categorised as follows:

I. The excavation support causes a negligible obstruction of the excavation cycle.
II. The excavation support causes a slight obstruction of the excavation cycle.
III. The excavation support causes a considerable obstruction of the excavation cycle.

IV. The excavation support causes an interruption of the excavation cycle (immediate support after every stage of excavation).
V. The excavation support is installed continuously with the excavation and requires immediate support to the face or a pre-support measure.

The excavation classes generally apply both for drill and blast in rock and for roadheader excavation in rock (Section 5.3 of the standard) and for the use of tunnel boring machines in rock (Section 5.4 of the standard). However, different evaluations are given for these.

The amount of work involved in drill and blast excavation in rock depends on the type of excavation and the excavation class. This is taken into account by a grading of the excavation bill items in accordance with Table 2-23, where each field corresponds to a unit price.

Table 2-24 shows with an example the excavation classes for a tunnel with an excavation width b = 12m, correlation of the support types and their installation locations.

Table 2-21 Determination of the excavation classes due to the excavation support for drill and blast in rock (SIA-198, Table 8) [218].

	Class I	**Class II**	**Class III**	**Class IV**	**Class V**
Excavation classes for tunnels					
Face area L1		≤ n anchors around perimeter	> n anchors around perimeter ≤ anchors in face shotcrete with or without mesh around ≤ 1/3 of perimeter	bringing up temporary support to face after every round steel sets with or without anchors and mesh around 1/3 of perimeter > n anchors in face face support with shotcrete over ¼ face area	continuous steel installation during excavation with Marciavanti continuous shotcrete during excavation with or without anchors and mesh face support over > 1/4 face area face support with shotcrete over > 1/4 face area advance support (e. g. spiles)
Driving area L2	Mesh as rockfall support, fixed with anchors and rock bolts	≤ 3 n anchors around perimeter shotcrete with or without mesh around 1/3 perimeter	> 3 n anchors around perimeter shotcrete with or without mesh around > 1/3 perimeter steel sets in series (min. 3 arches with or without lagging	not applicable	not applicable

Table 2-21 continued

	Class I	Class II	Class III	Class IV	Class V
Rear area L3	-	> 3 n anchors around perimeter shotcrete with or without mesh around > 1/3 of perimeter	not applicable	not applicable	not applicable
Excavation classes for raised shafts					
Working area H	Mesh as rockfall support, fixed with anchors and rock bolts	≤ p anchors around perimeter shotcrete with or without mesh, but not after every round	anchors in face > p anchors around perimeter shotcrete with or without mesh after each round	Steel sets with or without lagging but not advanced as Marciavanti	-
Excavation classes for sunk vertical shafts					
Working area S	Mesh as rockfall support, fixed with anchors and rock bolts	≤ p anchors around perimeter shotcrete with or without mesh, but not after every round	> q anchors around perimeter shotcrete with or without mesh, but not after every round	Steel sets with or without lagging but not advanced as Marciavanti	-
Excavation classes for caverns					
Planned excavation surface	≤ 0.4 anchors per m^2	> 0.4 anchors per m^2 around perimeter shotcrete with or without mesh, but not immediately after every round	Shotcrete with or without mesh immediately after each round	Steel sets with or without lagging but not advanced as Marciavanti	-

Table 2-22 Guideline values for L1, L2, L3, n, H, p, S and q (SIA-198, Table 7) [218].

Max. excavation width [m]	3	6	10	15
Tunnel (clause 5 23 2)				
L1 Face area [m]	2	3	5	5
L2 Advance area [m]	15	20	25	35
L3 Rearward area [m]	150	200	250	300
n Number of rock bolts per running m tunnel	2	4	5	9
Excavated shafts (clause 5 23 3)				
H Working area	2	3	5	5
p Number of rock bolts per running m shaft	6	12	18	27
Sunk vertical shafts (clause 5 23 4)				
S Working area [m]	6	6	6	6
q Number of rock bolts per running m shaft	3	6	9	13

Table 2-23 Matrix of the excavation types and excavation classes for drill and blast (SIA-198, Table 6) [218].

Excavation types	**Excavation classes**				
	I	**II**	**III**	**IV**	**V**
A Full-face excavation	A I	A II	A III	A IV	A V
B Top heading	B I	B II	B III	B IV	B V
C Divided top heading			C III	C IV	C V
D Side headings			D III	D IV	D V

Table 2-24 Excavation classes (AK) for drill and blast (simplified).

	AK I	**AK II**	**AK III**	**AK IV**	**AK V**
L1...		< 9 rock bolts	< 9 rock bolts mesh & shotcrete < 1/3 U	steel arches shotcrete > 1/3U	HEB rolled steel sets + Marciavanti
L2...	mesh & rock bolts against rock fall	≤ 27 rock bolts mesh & shotcrete < 1/3 U	> 27 rock bolts mesh & shotcrete > 1/3 U		
L3...		> 27 rock bolts mesh & shotcrete > 1/3 U			

L1: Face area........................ 5 m
L2: Advance area: .... 35 m
L3: Backup area .... 300 m
n: number of rock bolts per tunnel n = 9
u: circumference

$\}$ for excavated span b = 12 m

The application of SIA 198 "Underground works" for mechanised tunnelling is described in Section 2.5.6.2.

2.4.3.3 Classification in Austria

The classification of the construction of underground works in Austria was regulated until 2001 in the Austrian standard ÖNorm B 2203 "Underground works" from 1994 [170], which is currently being completely revised. The new standard is divided into the standards ÖNorm B2203-Part 1 (Cyclical advance), which was already published in 2001 [172], and ÖNorm B2203-Part 2 (Continuous tunnelling with tunnel boring machines), which was published in 2005 [172a].

The procedure according to the old standard is described in [142, 148]. Since the new standard has been valid since 2001, the procedure for cyclical (sequential) tunnelling in the new standard is dealt with here. In addition, the Austrian Society for Geomechanics has issued a guideline for the geomechanical design of underground construction works with cyclical advance [173], which provides further assistance than mentioned here.

In order to determine the tunnelling classes and thus the classification of the rock mass, the guideline [173] includes the flow diagram shown in Fig. 2-11 for the geomechanical design procedure.

The extensive description of the geological situation provided in the previous standard with the description of rock mass behaviour and the requirements for excavation and support measures is no longer found in the current ÖNorm B2203-Part 1 [172]. Instead, reference is made to the guideline [173] for the characterisation of the rock mass. The *Determination of rock mass behaviour types (GVT)* is performed using the basic list of rock behaviour types shown in Table 2-25, which is supplemented by the following information:

The following minimum information is required for every determination of rock mass behaviour types:

- Sketch of the expected rock mass structure and failure mechanisms.
- Rock mass type(s).
- Orientation of the decisive interfaces relative to the tunnel.
- Description of the loading on the rock mass.
- Formation water: quantity, effect on rock mass behaviour.
- Rock mass behaviour (behaviour on excavation of sides and face, type of overloading, failure mechanisms, long-term behaviour).
- Deformation of the perimeter of the cavity, estimate of size and direction.

The delineation of the individual rock mass behaviour types to each other within a category can be undertaken, for example, under the following aspects:

- Rock mass type.
- Rock mass structure.
- Formation water.
- Kinematics, failure type.
- Deformation measurements.

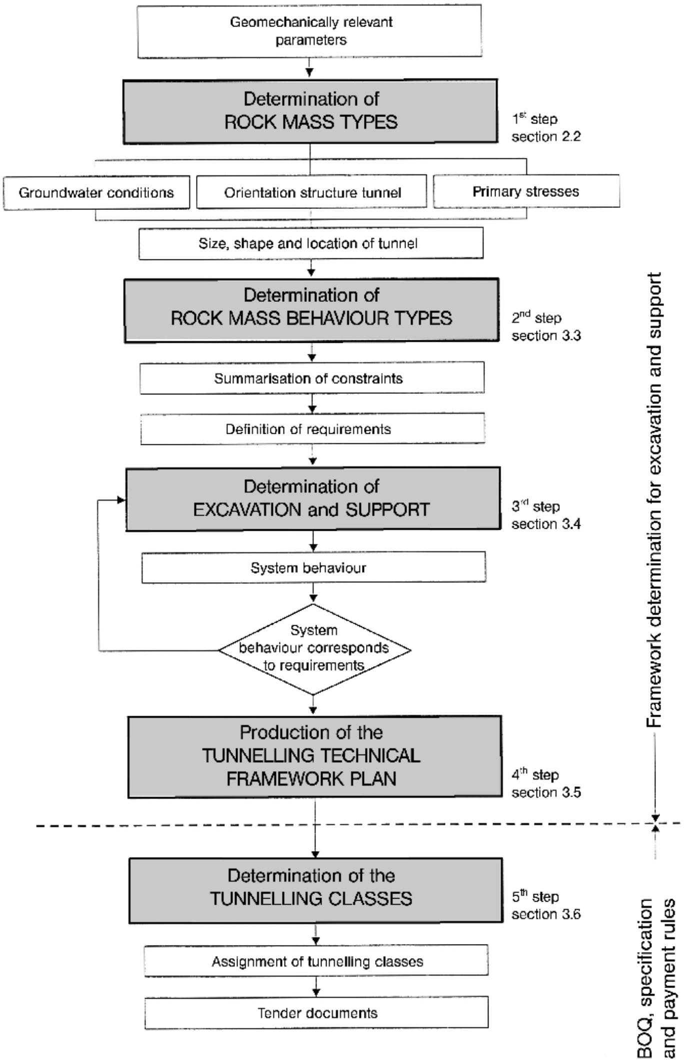


Figure 2-11 Flow diagram of the geomechanical design procedure according to [173].

Table 2-25 Basic categories of rock mass behaviour types [173].

Rock mass behaviour types	Description of the rock mass behaviour (without support measures)
1 Stable rock mass	Stable rock mass with potential falling out or sliding out of small-volume jointed bodies under gravity
2 Rock falls dependent on jointing	Deep-reaching rock falls due to jointing or gravity, isolated local exceeding of the shear strength of interfaces
3 Overloading around the cavity	Stress-dependent softening or plastification of the rock mass around the cavity in combination with rock falls dependent on jointing
4 Deep-reaching overloading	Stress-dependent deep-reaching softening or plastification of the rock mass with great deformation

Table 2-25 continued

Rock mass behaviour types		Description of the rock mass behaviour (without support measures)
5	Rockburst	Sudden ravelling due to high stresses in combination with brittle rock
6	Strata buckling	Buckling of slender strata, frequently in combination with shear failure
7	Shear failure at a low stress level	Potential of large-volume rock falls and progressive shear failure under low stress
8	Loose ground	Running out of mostly non-cohesive, dry to damp soil
9	Flowing ground	Flowing out of soil with high water content
10	Swelling ground	Time-dependent expansion of soil through physical-chemical reaction of the soil and water in combination with stress relief
11	Ground with quickly changing deformation properties	Strong variability of stresses and deformations due to block matrix structure (for example fault zones, tectonic melange)

The forecast required support and lining measures for the forecast ground behaviour types are laid down within defined homogenous zones.

In order to determine the tunnelling classes according to ÖNorm B2203-Part 1, two ordinal numbers are determined. The *1st ordinal* represents the round length. For the invert excavation, this is the range of opened lengths.

As the *2nd ordinal,* the support factor *sf* is given. In order to calculate the support factor, the sum of the evaluated support measures per metre tunnel is divided by the relevant rating area of top heading and bench. These rating areas must be given and defined in the tender documents of each project.

The support factor [12] is calculated as follows:

$$sf = \frac{\sum(sq \cdot rf)}{ar}$$

sf support factor
sq number of support measures per running metre of tunnel (supporting quantity)
rf rating factor according to ÖNorm (see Table 2-26)
ar rating area according to ÖNorm (see Table 2-12)

In the determination of the 2nd ordinal, the valid ranges according to Table 2-27 are to be complied with. Fig. 2-13 shows an example of the determination of the 2nd ordinal.

The intersections of the 1st and 2nd ordinals produce matrices, in which the unit prices per cubic metre of excavation and the guaranteed advance rate per working day guaranteed by the contractor are to be entered. The adjacent horizontal fields of the defined matrix fields, which are also filled in, give a horizontal and vertical range of payment for the advance of the tunnel within specified limits of tunnelling classification. Should zones be unexpectedly encountered, which have not been included in the matrix, then the new fields of the matrix are to be derived linearly from the existing fields. this provides the client a certain cost security and reduces the potential for discussion about payment.

An example of a tunnelling matrix is shown in Fig 2.14 and 2.15.

Table 2-26 Rating of the support measures and additional measures for cyclical tunnelling [172].

Support and supplementary measures		Rating factor per unit of quantity	Unit	Remarks
Anchors	Swellex or similar	0.8	m	
	SN grouted anchors	1.1	m	
	self-drilled anchors	1.7	m	
	grouting tube anchor	2.0	m	
	prestressed grouted anchor	2.5	m	
Face anchors	number of anchors in round	8.0	number	1)
	setting of anchor plate without prestress	1.7	number	2)
	setting of anchor plate with prestress	5.0	number	3)
Spiles	rammed spiles	0.5	m	
	ungrouted spiles	0.6	m	
	grouted spiles	0.9	m	
	self-drilling spiles	1.3	m	
	grouting tube spiles	1.6	m	
Grouting over 10 kg per anchor, spile, foot pile		0.1	kg	
Mesh reinforcement	external with arches	1.0	m^2	3)
	internal with arches	1.5	m^2	3)
	external without arches	2.0	m^2	3)
	top heading invert	0.8	m^2	3)
	additional reinforcement, face reinforcement	2.0	m^2	3), 4)
Arches and load distributors		2.0	m	
Shotcrete	top heading and bench	20.0	m^3	5)
	top heading invert, top heading feet	12.0	m^3	5)
	face	14.0	m^3	5)
	filling of angles and extra consumption	14.0	m^3	5), 6)
Deformation slots	without yielding elements	3.5	m	7)
	with yielding elements	5.0	m	7)
Forepoling boards		5.5	m^2	
Foot piles	foot piles Ø ≤ 38 mm	4.5	m	
	foot piles Ø > 38 mm	5.0	m	
Partial areas		22.0	number	8)
Excavation of widened top heading feet (elephants' feet)		50.0	m	9)
Excavation of top heading invert arch in bench advance		50.0	m	10)

1) Number of anchors in each round. The rating factor takes into account setting, shortening and difficulties of loosening
2) Number of anchor plates set in the relevant face
3) Theoretical quantities without consideration of longitudinal and side overlapping
4) Exposed area covered by reinforcement – starter reinforcement top heading/bench and bench/invert is not taken into account
5) Theoretical quantities without consideration of overprofile or rebound
6) Filling of planned angles (with forepoling boards etc.) or filling of known overbreak outside the boundary area A
7) Running metres of slot length
8) Partial excavations are only rated as partial area if they are maintained immediately after the opening of a first support
9) For both top heading feet, per running metre of tunnel
10) Length of the top heading invert arch for the relevant round of the bench, independent of any necessary partial excavations

Table 2-27 Valid ranges for the 2nd ordinal [172].

Round length of top heading up to	Maximum scope of validity for the 2nd ordinal (support measures) in top heading	Round length of bench up to	Maximum scope of validity for the 2nd ordinal (support measures) in bench
not given	± 0.35	not given	± 0.45
4.0.m	± 0.35		
3.0 m	± 0.45	3.0 m	± 0.70
2.2 m	± 0.60		
1.7 m	± 0.80	2.0 m	± 1.20
1.3 m	± 1.00		
1.0 m	± 1.30	1.0 m	± 2.10
0.8 m	± 1.60		
0.6 m	± 2.10		

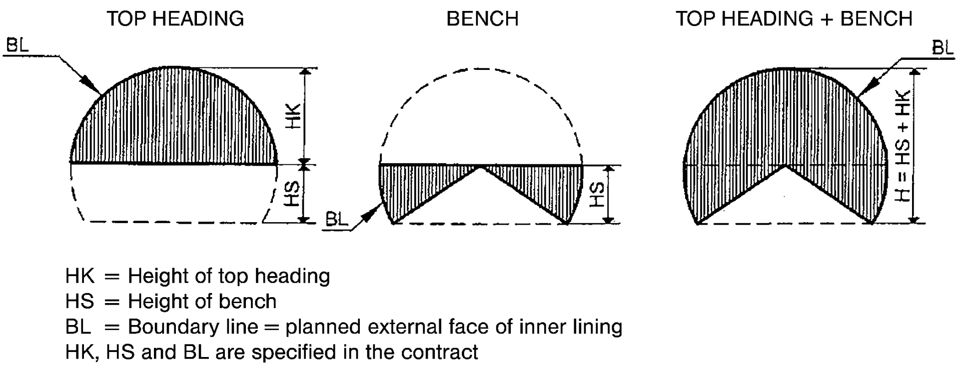


Figure 2-12 Diagram of rating areas [172].

B1. Example for the determination of the support measures number

Cross-section values:	Height of top heading	HK = 5.00 m
	Height of bench	HS = 3.75 m
	Rating area top heading	47.7 m²
	Rating area of bench	22.4 m²

Top heading	**Round length 1.3 m → 1st ordinal number = 6**							
Support measures	per m tunnel	unit	Thickness/ length	unit	Quantity/m	unit	Rating factor (Tab. 3)	rating
Shotcrete – top heading	16.60	m²	0.25	m	4.15	m³	20.00	83.00
Shotcrete – face	28.00	m²	0.10	m	2.80	m³	14.00	39.20
Shotcrete – top heading invert	11.00	m²	0.20	m	2.20	m³	12.00	26.40
Excavation top heading foot	1.00	m	1.00	-	1.00	m	50.00	50.00
Mesh – external	16.60	m	1.00	m	16.60	m²	1.0	16.61
Mesh – internal	16.60	m	1.00	m	16.60	m²	1.5	24.90
Mesh – top heading invert	11.00	m	1.00	m	11.00	m²	0.8	8.80
Mesh – additional	2.10	m	1.00	m	2.10	m²	2.0	4.20
Arches	12.77	m	1.00	-	12.77	m	2.0	25.54
Spiles – grouted	18.00	No.	3.00	m	54.00	m	0.9	48.60
Anchors – SN grouted	4.61	No.	4.00	m	18.44	m	1.1	20.28
Anchors – SN grouted	1.54	No.	6.00	m	9.24	m	1.1	10.16
							Total	**357.69**

Support measures number = 2nd ordinal = sum of rating / rating factor for top heading = 7.50

Area of validity for tunnelling class top heading: 6 / 7.50 according to Table 4: from 6.50 to 8.50

Bench	**Round length 2.6 m → 1st ordinal number = 3**							
Support measures	per m tunnel	unit	Thickness/ length	unit	Quantity/m	unit	Rating factor (Tab. 3)	rating
Shotcrete – bench	7.60	m²	0.25	m	1.90	m³	20.00	38.00
Excavation top heading invert	1.00	m	1.00	-	1.00	m	50.00	50.00
Mesh – external	7.60	m	1.00	m	7.60	m²	1.0	7.6
Mesh – internal	7.60	m	1.00	m	7.60	m²	1.5	11.4
Arches	2.93	m	1.00	-	2.93	m	2.0	5.9
Anchors – SN grouted	3.08	No.	4.00	m	12.32	m	1.1	13.6
							Total	**126.41**

Support measures number = 2nd ordinal = sum of rating / rating factor for bench = 5.64

Area of validity for tunnelling class bench: 3 / 5.64 according to Table 4: from 4.94 to 6.34

Figure 2-13 Example of the determination of the 2nd ordinal (support factor) [172].

FIRST ORDNINAL	ROUND LENGTH UP TO: TOP HEADING OR TOP HEADING + BENCH	ROUND LENGTH UP TO: BENCH	SECOND ORDINAL – SUPPORT MEASURES NUMBER: 1	2	3	4	5	6	7	8	9
1	not given	Is to be specified for each project									
2	4.0 m										
3	3.0 m										
4	2.2 m				4/2.4	4/3.6					
5	1.7 m						5/4.5		5/6.1		
6	1.3 m							6/5.5		6/7.5	
7	1.0 m										
8	0.8 m										
9	0.6 m										

Figure 2-14 Tunnelling classes matrix for the advance of top heading, bench, or top heading with bench [172].

FIRST ORDNINAL	OPENING LENGTH UP TO	SECOND ORDINAL – SUPPORT TYPE: OPEN INVERT (1)	INVERT SLAB (2)	INVERT ARCH WITH LENGTH DIVISION (3)	INVERT ARCH WITHOUT LENGTH DIVISION (4)
1	not given	1/1			
2	36.0 m		2/2		
3	24.0 m		3/2	3/3	
4	12.0 m				4/4
5	6.6 m				5/4
6	4.4 m				
7	2.2 m				

Figure 2-15 Tunnelling classes matrix for the advance of invert [172].

2.4.4 Example of a project-related classification according to DIN 18312 for the shotcrete process

The project-related classification according to DIN 18312 [62] is now illustrated through the example of the drill and blast driving of the Oerlinghausen road tunnel (construction period 9/1994 to 3/1996) on the route of the L 751.

2.4.4.1 Procedure at the Oerlinghausen Tunnel

Working from the geotechnical investigations and the intended construction process, a project-related tunnelling classification with support types according to DIN 18312 was developed for the excavation and support works. Rock mass behaviour, excavation and support were each defined through one tunnelling class (Fig. 2-16).

The excavation of the entire cross-section was generally divided into top heading, bench and invert, not only for reasons of stability but also for construction process reasons (Fig. 2-17 and 2.18).

In the areas of the top heading, bench and invert, the same tunnelling class applies within one cross-section. Within a tunnelling class, round lengths and support are laid down differently for the areas of the top heading, bench and invert. The geological and rock mechanics documentation and the interpretation of the geotechnical measurements are used in order to determine the tunnelling classes as the tunnel advances.

The individual support measures are laid down according to the behaviour of the rock mass for tunnelling at the location.

The support measures are agreed according to local conditions within the range given in the plans (Fig 2.17 and 2.18). It is not possible to derive any change of the tunnelling class from this, however, as long as the classification criteria for the excavation remain constant. The round lengths given in the tunnelling classes are maximum values.

Table 2-28 Shares of the forecast tunnelling classes for the Oerlinghausen Tunnel.

Section	Length	Expected tunnelling class					
		4.1	4.2	4.3	4.4	6	7
	[m]	[m]	[m]	[m]	[m]	[m]	[m]
1	32	–	–	–	–	16	16
2	224	–	90	74	45	15	–
3	85	55	30	–	–	–	–
4	51	–	15	20	13	3	–
5	50	–	–	–	–	25	25
Sum	442	55	135	94	58	59	41
Sum %	100	12	31	21	13	13	10

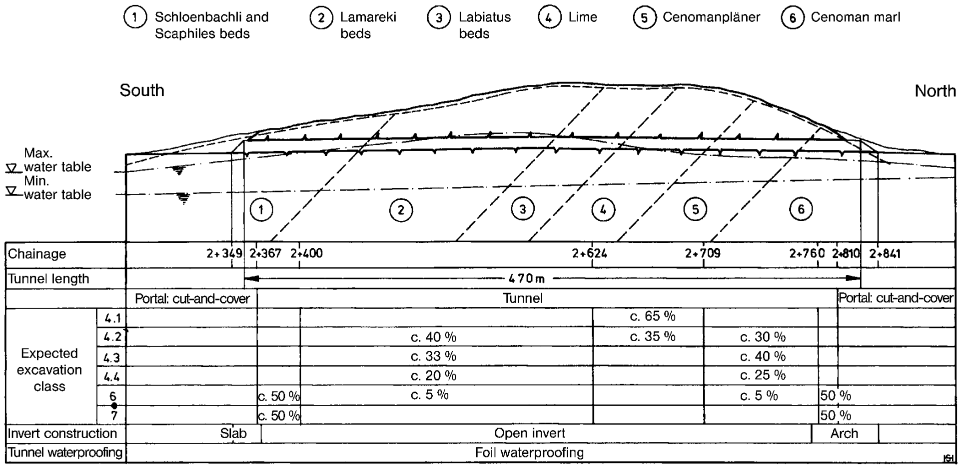
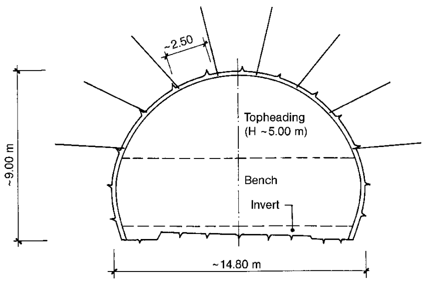
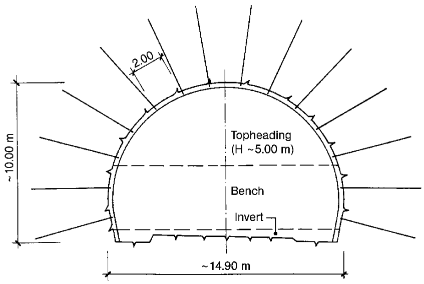


Figure 2-16 Distribution of the tunnelling classes along the length of the Oerlinghausen Tunnel (1995).



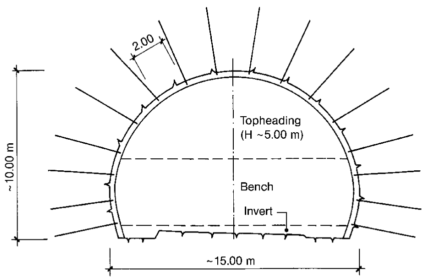
Tunnelling class 4.1

Excavation + support in the areas				
	Top heading	**Bench**	**Invert**	**Quantity / m**
Excavation	~53.4 m³/m	~59.0 m³/m	~5.7 m³/m	~118.1 m³/m
Shotcrete	d = 10 cm (~1.86 m³/m)	d = 10 cm (~0.97 m³/m)	-	~2.81 m³/m
Reinforcement	Q 188; 1 layer	Q 188; 1 layer	-	~28.1 m³/m
Anchors	8 – 10 No. / L = 4.0 m grid 2.5 x a	-	-	average ~3.2 No./m
Arches	-	-	-	-
Spiles	-	-	-	-
Lagging boards	-	-	-	-
Excavation depth	2.5 – 3.5 m	3.5 – 6.0 m	-	-



Tunnelling class 4.2

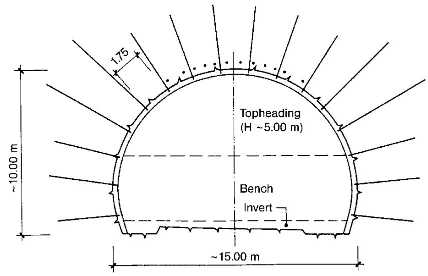
Excavation + support in the areas				
	Top heading	**Bench**	**Invert**	**Quantity / m**
Excavation	~54.3 m³/m	~59.5 m³/m	~5.7 m³/m	~119.5 m³/m
Shotcrete	d = 15 cm (~2.79 m³/m)	d = 15 cm (~1.46 m³/m)	-	~4.22 m³/m
Reinforcement	Q 188; 1 layer	Q 188; 1 layer	-	~28.1 m³/m
Anchors	10 - 12 No. / L = 4.0 m grid 2.0 x a	4 - 6 No. / L = 4.0 m grid 2.0 x a	-	average ~7 No./m
Arches	GI 110 A = 2.0 – 3.0 m	GI 110 A = 4.0 – 6.0 m	-	average ~9.4 running m/m
Spiles	-	-	-	-
Lagging boards	-	-	-	-
Excavation depth	2.0 – 3.0 m	3.0 – 6.0 m	-	-



Tunnelling class 4.3

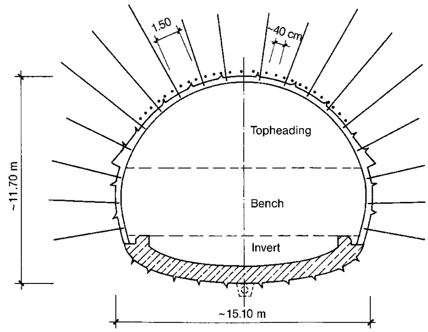
Excavation + support in the areas				
	Top heading	**Bench**	**Invert**	**Quantity / m**
Excavation	~55.3 m³/m	~59.9 m³/m	~5.7 m³/m	~120.9 m³/m
Shotcrete	d = 20 cm (~3.27 m³/m)	d = 20 cm (~1.93 m³/m)	-	~5.62 m³/m
Reinforcement	Q 257; 1 layer	Q 257; 1 layer	-	~28.1 m³/m
Anchors	10 - 12 No. / L = 4.0 m grid 2.0 x a	4 - 6 No. / L = 4.0 m grid 2.0 x a	-	average ~9 No./m
Arches	GI 110 A = 1.5 – 2.5 m	GI 110 A = 3.0 – 5.0 m	-	average ~11.9 running m/m
Spiles	-	-	-	-
Lagging boards	-	-	-	-
Excavation depth	1.5 – 2.5 m	2.5 – 5.0 m	-	-

Figure 2-17 Summary of the tunnelling classes for the Oerlinghausen Tunnel (VK 4.1/4.2/4.3).



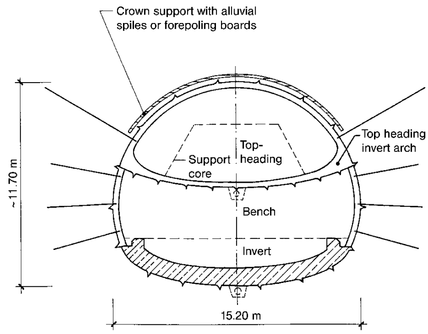
Tunnelling class 4.4

Excavation + support in the areas				
	Top heading	**Bench**	**Invert**	**Quantity / m**
Excavation	~55.7 m³/m	~60.1 m³/m	~5.7 m³/m	~121.5 m³/m
Shotcrete	d = 20 – 25 cm (~4.16 m³/m)	d = 20 - 25 cm (~2.17 m³/m average)	-	average ~6.33 m³/m
Reinforcement	Q 257; 2 layers	Q 257; 2 layers	-	average ~56.2 m³/m
Anchors	10 - 12 No. L = 4.0 & 6.0 m grid 1.75 x a	4 - 6 No. L = 4.0 m grid 1.75 x a	-	average ~11.5 No./m
Arches	GI 110 A = 1.0 – 1.5 m	GI 110 A = 2.0 – 3.0 m	-	average ~18.7 running m/m
Spiles	poss. Ø 25, localised	-	-	-
Lagging boards	-	-	-	-
Excavation depth	1.0 – 1.5 m	2.0 – 3.0 m	-	-



Tunnelling class 6

Excavation + support in the areas				
	Top heading	**Bench**	**Invert**	**Quantity / m**
Excavation	~56.8 m³/m	~60.4 m³/m	~5.7 m³/m, poss. ~26.5 m³/m	~143.7 m³/m
Shotcrete	d = 25 cm (~4.69 m³/m)	d = 25 cm (~2.42 m³/m)	poss. d = 25 cm (~3.2 m³/m)	~10.32 m³/m
Reinforcement	Q 257; 2 layers	Q 257; 2 layers	Q 257; 1 layer	~84.7 m³/m
Anchors	10 - 12 No. L = 4.0 & 6.0 m grid 1.5 x a	4 - 6 No. L = 4.0 m grid 1.5 x a	-	average ~18.7 No./m
Arches	GI 110 A = 0.8 – 1.2 m	GI 110 A = 0.8 – 1.2 m	-	average ~28.1 running m/m
Spiles	number as requ. Ø 25, L = 2.5 – 3.0 m	-	-	30 No./m
Lagging boards	-	-	-	-
Excavation depth	0.8 – 1.2 m	1.6 – 2.4 m	max. 20 m	-



Tunnelling class 7

Excavation + support in the areas					
	Top heading	**Bench**	**Invert**	**Top heading invert arch**	**Quantity / m**
Excavation	~67.5 m³/m	~50.4 m³/m	~26.5 m³/m	-	~144.4 m³/m
Shotcrete	d = 30 cm (~10.2 m³/m)	d = 30 cm (~2.5 m³/m)	poss. 25 cm (~4.0 m³/m)	poss. 20 cm (~3.0 m³/m)	~12.7 m³/m
Reinforcement	Q 377; 2 layers	Q 377; 2 layers	poss. Q 377; 2 layers	poss. Q 257; 2 layers	~86.2 m³/m
Anchors	2 No. L = 6.0 m	6 No. / L = 4.00 m	-	-	~10.7 No./m
Arches	GI 110 A = 0.6 – 0.9 m	GI 110 A = 0.6 – 0.9 m	-	-	average ~37.5 running m/m
Spiles	poss. Ø 25	-	-	-	-
Lagging boards	2.0 – 2.5 m long, driven in advance	-	-	-	~45.6 m²/m
Excavation depth	0.6 – 0.9 m	0.6 – 0.9 m	max. 10.0 m	-	-

Figure 2-18 Summary of the tunnelling classes for the Oerlinghausen Tunnel (VK 4.4/6/7).

2.4.4.2 Description of the tunnelling classes for the Oerlinghausen Tunnel

Altogether six tunnelling classes are assigned according to the geological survey and the geological report.

The support measures shown in the tunnelling classes can be slightly altered if required without this changing the tunnelling class. Alternative items are available for this. The essential criteria for the classification of the rock mass conditions are:

– the stand-up time of the cavity in the rock mass depending on the effective span from excavation until the support becomes effective,
– the possibility or necessity of excavating smaller partial drifts,
– type and support resistance of the support and
– the point in time for the installation of the individual support measures.

The tunnelling classes resulting from the different rock mass conditions are described as follows (Fig. 2-17 and Fig. 2-18):

Tunnelling class (VKL) 4.1 (Fig. 2-17). VKL 4.1 applies for sections in rock liable to rock fall.

The freshly excavated cavity remains free standing until the next round without any occurrence of loosening. The discontinuities only have a slight effect and only cause slight ravelling and spalling. Any formation water has no influence on the stability of the rock mass.

The round length in the top heading is 2.50 m to maximum 3.50 m and in the bench 3.50 m to maximum 6.0 m.

The temporary support consists of:

– a 10 cm thick layer of shotcrete,
– one layer or mesh Q 188 externally and
– systematic bolting of the top heading.

The entire support is to be installed before the next round.

The following may also be necessary according to a decision on site:

– anchoring of dangerous sequences of strata,
– shotcrete sealing directly after the round.

Tunnelling class (VKL) 4.2 (Fig. 2-17). VKL 4.2 applies for sections in rock liable to heavy rock fall.

The rock mass is clearly fissured. The discontinuities have a mechanical effect. The occurrence of loosening in the unsupported state and of water ingress from joints, which could impair the stability of smaller zones of the rock mass, have to be expected.

The round length in the top heading is 2.00 m to maximum 3.00 m and in the bench 3.00 m to maximum 6.00 m.

The temporary support consists of:

– steel arches with distances according to the round length,
– a shotcrete layer altogether 15 cm thick,
– one layer of Q 188 mesh reinforcement externally and
– systematic anchoring in the top heading and invert.

The entire support is to be installed around the entire perimeter before the next round.

The following may also be necessary according to a decision on site:

– anchoring of dangerous sequences of strata and
– shotcrete sealing directly after the round.

Tunnelling class (VKL) 4.3 (Fig. 2-17). VKL 4.3 applies for sections in slightly unstable rock.

The interfaces are clearly formed and partially completely separated, which leads to rockfalls, mainly in the crown. The interfaces are partially open or filled with claystone interburden. Particularly with water ingress, individual jointed bodies can slide out or rockfalls can occur with ravelling at the interfaces.

The round length in the top heading is 1.50 m to maximum 2.50 m and in the bench 2.50 m to maximum 5.00 m.

The temporary support consists of:

– steel arches with distances according to the round length,
– a shotcrete layer altogether 20 cm thick,
– one layer of Q 257 mesh reinforcement externally and
– systematic anchoring in the top heading and invert.

The entire support is to be installed around the entire perimeter before the next round.

The following may also be necessary according to a decision on site:

– Anchoring of dangerous sequences of strata and
– shotcrete sealing directly after the round.

Tunnelling class (VKL) 4.4 (Fig. 2-18). VKL 4.4 applies in sections in unstable to slightly squeezing rock mass.

The strength of the rock mass is exceeded locally. The rock mass loosens with heavy rockfalls.

The short stand-up time of the rock mass requires sealing (from the muck pile) of the tunnel sides immediately after blasting.

The round length in the top heading is 1.00 m to maximum 1.50 m and in the bench 2.00 m to maximum 3.00 m.

The temporary support consists of:

– sealing of the tunnel sides with 3 to 5 cm shotcrete,
– steel arches with distances according to the round length,
– a shotcrete layer altogether 20 to 25 cm thick,
– two layers of mesh reinforcement Q 257 and
– systematic anchoring in the top heading and invert.

In case of heavy water ingress, see special measures.

Sealing of the tunnel sides with 3 to 5 cm shotcrete is required immediately after blasting.

The following may also be necessary according to a decision on site:

– Widening of the top heading arch foundation (elephant foot) d = 60 to 80 cm,
– face support and
– pre-support with spiles dia. 25 mm.

Tunnelling class (VKL) 6 (Fig. 2-18). VKL 6 applies for sections in very unstable or squeezing rock and for fault zones with pronounced karst features.

The severe loosening can lead to large rockfalls, which makes pre-excavation support necessary. Fractures in the rock mass and the rock pressure demand an early ring closure and lead to a considerable limitation of the timing of the advances of top heading, bench and invert.

The round length in the top heading is 0.80 m to maximum 1.20 m and in the bench 1.60 m to maximum 2.40 m.

The temporary support consists of:

– pre-excavation installation of steel spiles in the top heading,
– steel arches with distances according to the round length,
– a shotcrete layer altogether 25 cm thick,
– two layers of mesh Q 257
– systematic anchoring in the top heading and bench and
– the installation of a load-distribution rail and an elephant foot.

In case of heavy water ingress, see special measures.

Sealing of the tunnel sides with 3 to 5 cm shotcrete is required immediately after blasting.

The following may also be necessary according to a decision on site:

– support, division or buttressing of the top heading face,
– support of the invert with shotcrete 20 cm thick and a layer of mesh Q 257 and
– pre-injection support in the form of a grouted canopy.

Tunnelling class (VKL) 7 (Fig. 2-18). VKL 7 applies to sections in strongly squeezing rock mass, in loose material and in fault zones with pronounced karst features.

The effects of pressure are apparent on the cross-section and at the face.

in addition to pre-excavation support, it is necessary to buttress the face or excavate in partial drifts with immediate support.

The round length in the top heading is 0.60 m to maximum 0.90 m and in the bench 1.20 m to maximum 2.40 m.

The temporary support consists of:

– pre-excavation installation of forepoling sheets or steel spiles,
– a sealing of the tunnel sides with 3 to 5 cm shotcrete,
– steel arches with distances according to the round length,
– a shotcrete layer altogether 30 cm thick,
– the installation of an invert floor of shotcrete,
– two layers of mesh reinforcement Q 377 and
– systematic bolting of the sides.

In case of heavy water ingress, see special measures.

During excavation, it is necessary to support the drifts and the face immediately. The support in the top heading and bench must be complete within three arch spacings at least.

The following may also be necessary according to a decision on site:

– Additional division of the top heading section,
– support to the invert with reinforced shotcrete immediately after invert excavation and
– pre-injection support in the form of a grouted canopy.

Special measures. The appearance of karst features has to be expected at any time as the excavation advances. This will normally mean karst fissures with an average width of about 10 cm, although voids up to many metres wide have to be expected. Voids, which intersect the tunnel alignment, have to be permanently filled with suitable material (for example grout).

In zones of rock subject to heavy weathering, it may also be necessary to grout with cement suspension to consolidate the structure of the rock mass.

When a tunnel is driven into a karst void, large quantities of water have to be expected. This applies particularly to areas where the tunnel section is at the level of the karst water table or the formation water table.

In this case it is necessary to dewater the ground around the tunnel by drilling drainage holes or installing hoses. These works can be carried out in advance of the tunnel, but also subsequently (perhaps shortly before the installation of the waterproofing).

Soil filling flowing out of karst voids also has to be expected. For this reason, it can be necessary to carry out additional support works at any time, such as the installation of lagging sheets, spiles or a grouted canopy.

The localised use of the special measures described here does not in itself justify a change of class. The extra costs should be included in the estimation of the prices for special measures, including operating costs.

2.5 Special features for tunnelling machines

2.5.1 General

When tunnelling machines are used, additional information about the ground conditions is required in order to plan the construction process using the machine, determine costs and perform a risk analysis. This includes the boring process with wear and sticking problems, machine stroke and support.

For example knowledge of the stratification, particularly transitions from rock to loose ground and pronounced faults, is more important for TBM drives than with drill and blast or the use of roadheaders. The following information is taken from the book “Hardrock Tunnel Boring Machines” [148].

2.5.2 Influences on the boring process

In Fig. 2-19 shows the factors influencing the penetration of the boring process.

The factors that have a great effect on the boring process are the strength properties of the rock, the abrasiveness and the size and bonding of jointed bodies or blocks. The latter factor determines whether a TBM with disc cutters mounted in detachable brackets, as is common in the smaller diameters, can be used at all. Blocks breaking out can do great damage to the cutterhead of a TBM. Modifications to the cutterhead during a TBM drive are expensive and disc changing is time-consuming. Such cutterhead modifications had to be carried out, for example, to the Jarva Mk 15 gripper TBM for the pressure tunnel of the Amsteg hydropower station, or the shielded TBM at the Colorado-Arizona irrigation system, after entire disc mounting brackets had been torn off both machines.

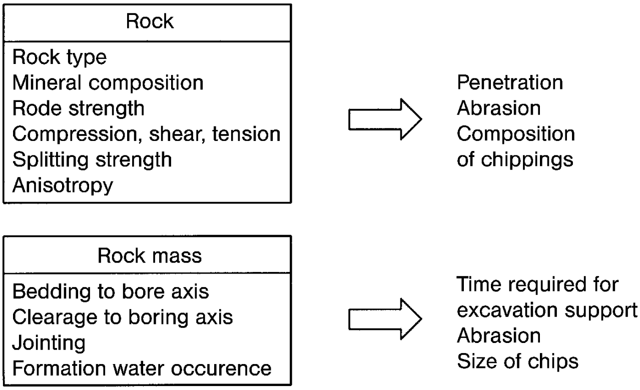


Figure 2-19 Factors influencing tunnelling penetration.

The abrasiveness of a rock can be described quite well by the CAI test. In a few hard rocks like very crumbly sandstones, abrasiveness can reach an exceptional severity, and the methods usually used to determine abrasiveness do not deliver realistic values in such rocks. Under such conditions, the LCPC abrasiveness test according to [148, Section 3.3.5] can be of assistance. If this sort of crumbly sandstone with high quartz content, such as often occurs as riverbed sandstones in the lower freshwater Molasse, also contains water, then the abrasive mixture of sand and water acts as a grinding paste.

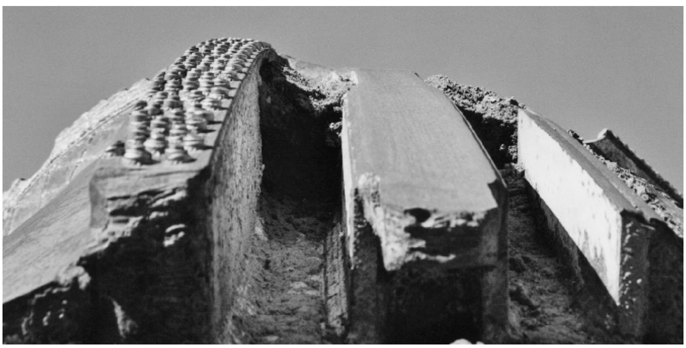


Figure 2-20 Section of the rim of the cutterhead Murgenthal Tunnel TBM.

In the Murgenthal Tunnel, this type of abrasive sandstone caused alarming wear to the rim of the cutterhead (Fig. 2-20).

The rock strengths are mostly determined from cores, a process that produces good results. Cores taken from rock mass under high primary stress, at great depths or due to restrained residual stresses are still relaxing days after their extraction. This causes microcracks in the rock structure, which reduces the strength of the samples. This sort of false strength value can lead to incorrect conclusions for the selection of TBM and disc cutters. In order to maintain the estimated performance during the construction phase, the contractor is then forced to drive the TBM with higher cutter loadings, presuming his machine permits this at all.

This type of microcrack formation has been demonstrated by microscopic examination of thin sections, particularly in dense granite or granitic gneisses:

- at the Amsteg hydropower project in the Central Aare granite,
- the Piora investigation tunnel in the Leventina granite-gneiss,
- the Paute project in Ecuador in granodiorites,
- the Lötschberg Tunnel in Gastern granite.

In the central Aare granite at the Amsteg hydropower station, the reduction of wave velocity in the drill core through microcrack formation averaged about 15 % within four days of extraction. It is evident that the compression, shear and tension strengths are reduced by such microcracks. Within the range of tolerance, it may be assumed that the strength reduction is proportional to the wave velocity reduction. In some zones of the Aare granite at the Amsteg power station, there is a clear although not very pronounced orientation of the mica minerals. This slight foliation should lead to compressive strengths that are much higher at right angles to the orientation plane of the mica minerals than in the plane of foliation. The anisotropic factor is then always greater than one. Measurements on drill cores have however also shown a contradictory result, which can only be explained through the formation of microcracks. Thin section investigations have confirmed these assumptions; instead of the logical anisotropy values of about 1.2, values of 0.91 to 0.75 were determined.

When the face is bored, rock has to be excavated that has not yet been able to relax. The strengths that have to be overcome in such cases are considerably higher than the material properties determined using conventional methods. If preliminary investigations show such variable anisotropy values, additional investigations should be carried out to clarify the uncertainty.

As long as the tension strength remains in a normal relation to the compression strength, a statement about the compression strength is adequate for rough dimensioning. If the rock mass is of extreme toughness – which is categorised as hard to bore – detailed investigations, even including thin section observations, are essential. If these investigations are omitted, the necessary penetration will not be achieved and the tunnel drive can then only be completed after changing the disc cutters to button cutters, with the associated very high cost.

2.5.3 Influences on the machine bracing

The bracing (or gripping) forces have a magnitude of about double the thrust forces (see [148, Chap. 4]). The large-diameter disc cutters used today lead to correspondingly high thrust forces and thus to remarkable stresses, which have to be transferred into the rock mass through the gripper unit (Fig. 2-21).

The values recorded for various TBMs in use on construction sites show loadings of 2 to 10 MPa on the side of the cavity. Related to the primary stress in the rock mass, this is a significant extra loading.

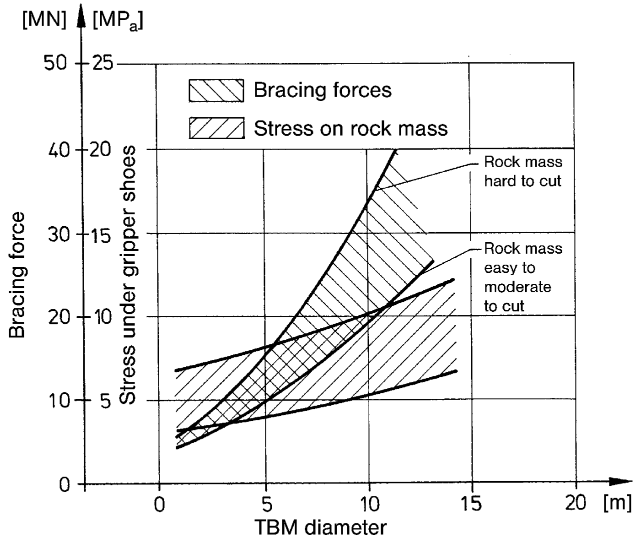


Figure 2-21 Bracing forces of an open TBM and the stresses they apply to the rock mass depending on cuttability and TBM diameter.

With appropriate geological-geotechnical investigations and sensitivity analyses, it is possible to perform the selection of a TBM better and take measures as precautionary decisions in order not to be surprised in case of occurrence.

There are three essentially different problems with the bracing of a TBM:

- The rock strength is too low for the bracing of a gripper TBM. Soft rocks, which often occur as poorly cemented sediments or at geological fault zones, do not provide an adequate reaction for the gripper shoes. For example, in the access tunnel for the junction structure of the Rosenberg Tunnel in St. Gallen in the soft Molasse beds, the bracing area had to be enlarged by inserting oak sleepers (Fig. 2-22). At the Westtangente in Bochum, a Mini-Fullfacer could not brace at all due to a fault zone at the start of the drive and abutments had to be concreted (Fig. 2-23) [232]. Bessolow and Makarow have reported very impressively about the problems with a TBM on the Baikal-Amur-Magistrale in Siberia [21].
- The large bracing force in a very hard rock mass that is categorised as difficult to cut leads to stress transfers, which can precipitate or considerably worsen rock bursts (see also [148, Chap. 4]). In the Amsteg pressure tunnel, spalling similar to rock burst occurred in massive granite between the roof of the stator and the front bracing unit – the gap between stator and the bracing unit at the start of work was about 1 m (Fig. 2-24). The spalling indicates stress transfers due to the bracing unit. Myrvang [162] reports rock burst while driving a power station tunnel through the geologically very old granites of Kobbelv in Norway. These occurred surprisingly and severely between the roof of the stator and the bracing unit of the Robbins machine. In some areas, this also had the effect that the bracing of the machine was only possible with lagging below the bracing units, since the bursts extended too deep into the side walls of the tunnel.
- The alternating loading of the tunnel sides by the bracing unit advancing in steps can cause the displacement of jointed bodies in a blocky rock mass. This can result in collapses in the machine area.

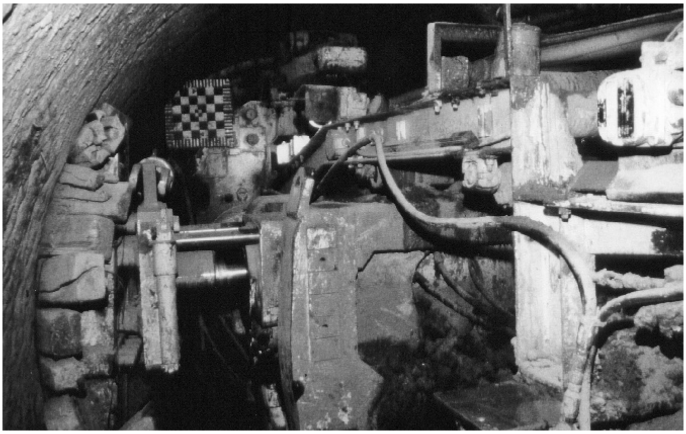


Figure 2-22 Rosenberg access tunnel with enlargement of the bearing area of the bracing units.

Figure 2-23 Abutment for bracing in a fault zone for the Mini-Fullfacer, Westtangente Bochum.

The geotechnical investigation and geomechanical analysis lead to the assignment of excavation classes with their associated support systems. In the higher excavation classes, the temporary excavation support is installed as a temporary measure or as a system in front of or next to the bracing units of a TBM. Temporary support and bracing unit need to be matched to each other or else the temporary support can be destroyed by the high pressures [194].

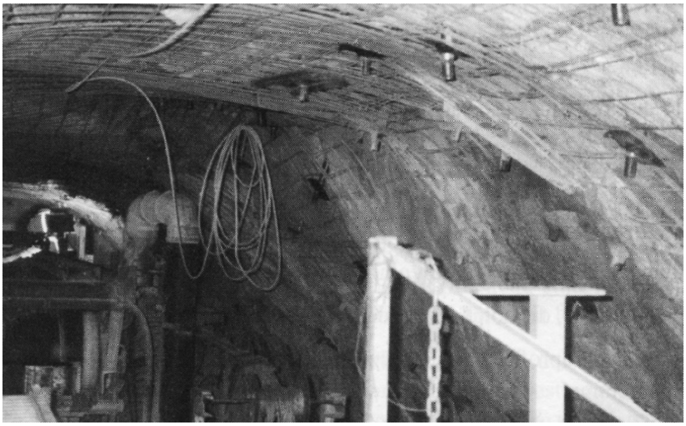


Figure 2-24 Tunnel wall of the Amsteg pressure tunnel with spalling described as rock burst.

2.5.4 Influences on the temporary support

The rock mass is already deformed in front of the face to a certain extent as a tunnel advances. Depending on the relevant geomechanical properties, more or less deformation of the tunnel sides occurs shortly after the opening of the cavity. This deformation is however to be kept within bounds by the temporary support. The well-known method of the ground reaction curve illustrates the problem ideally [124].

The design goal for temporary support must therefore be to select support measures, which permit the intended range of deformation but also permit systematic installation with correspondingly fast advance rates (see [148, Chap. 1]).

While pure loosening pressure permits stiff support and indeed often demands stiff support in order to limit settlement at the surface, genuine rock pressure demands a system, which enables deformation in the installed state.

Proven and possible measures, which permit such required deformation, are:

- Steel sets with bell profiles, which slip at the joints under a certain loading.
- Deformation slots in the shotcrete support. These will however scarcely be effective on a TBM drive since they can only be installed 40 to 50 m from the face and the deformation at this location will only increase slightly or has already reached the permissible magnitude.
- Deformable support dowels in the segment joints or also hydraulic cylinders in the invert segment.
- Compressible filling of the annular gap through a segment lining and compressible grains in the stowing material, for example cement-coated Styrofoam balls.

Investigation of the necessary deformation behaviour is therefore necessary for the appropriate selection of temporary support (see [148, Chap. 15]).

2.5.5 Classification for excavation and support

2.5.5.1 General and objective for mechanised tunnelling

The classification systems known previously were almost exclusively produced for drill and blast tunnelling. Classification provides the basis for payment and schedule. For a TBM drive, cuttability is also a decisive factor, because the time to bore a stroke increases in less cuttable rock and more time is thus available for temporary support.

In addition to rock mass classification as a grading of stability, mostly through the required extent of support measures, cuttability should be integrated in a suitable manner in order to satisfy the basic objective. If the determination of these matters is omitted from the contract, diverging opinions between the contract parties are inevitable.

2.5.5.2 Classification systems and investigation of suitability for tunnel boring machines

Classification according to rock mass properties. Terzaghi, Stini and later Lauffer, Packer and Rabcewicz all developed rock quality classes based on the rock mass properties or also the behaviour of the rock mass. All grade the rock mass between “stable” and “squeezing” with various numbers of intermediate classes. In particular, Bieniawski created the *RMR* System (Rock Mass Rating System) and Barton the *Q* System (Quality System) from theoretical knowledge of rock mechanics [142]. Both systems are based on quantitative rock mass parameters without consideration of the cuttability for TBM drives.

Suitability of the RMR System (Rock Mass Rating System). The procedure for the *RMR* System and its strengths and weaknesses are dealt with under 2.4.2.4.

In the absence of such a system for TBM drives, the RMR System can also produce usable results for TBM drives, provided the conditions are also considered critically.

This rock mass classification was developed from and for of conventionally driven tunnels. For TBM drives, the tearing of the rock outside the intended contour due to blasting is not applicable. The positive influence of the use of mechanised tunnelling is described according to Alber et al. by the following formula [5]:

$$RMR_{\mathrm{TBM}} = 0{,}84\ RMR_{\mathrm{D+B}} + 21 \qquad (20 < RMR_{\mathrm{D+B}} < 80)$$

RMR_{TBM} represents the *RMR* value for TBM drives and $RMR_{\mathrm{D+B}}$, that for drill and blast. In a rock mass of poor to very poor quality, the difference between conventional and mechanised tunnel drives is less significant.

Suitability of the Q System (Quality System). The *Q* System described in Section 2.4.2.3 was further developed for TBM drives by Grimstad and Barton [13, 83]. The intention was to create a Q_{TBM} Index based on the *Q* Index.

This rock mass classification for TBM as a combination of a forecast for temporary support with a forecast for the cuttability and material wear may be usable in some cases. In general, the cuttability and disc cutter state cannot be aligned with the rock classification. Barton attempted to estimate the penetration rate *PR* and the advance rate *AR* of a TBM drive using Q, Q_{TBM} and additional values. In addition, the Cutter Life Index (*CLI*) as a wear parameter with a scale of values limited to a maximum (Fig. 2-26) is not very convincing when the lifetime of cutter rings can vary from below 200 km to more than 20,000 km rolling distance.

The penetration rate *PR* and the advance rate *AR* are calculated from:

$$PR = 5 \cdot Q_{\mathrm{TBM}}^{-1/5} \qquad [\mathrm{m/h}]$$

$$AR = 5 \cdot Q_{\mathrm{TBM}}^{-1/5}\, T^{\mathrm{m}} \qquad [\mathrm{m/h}]$$

with:

$$Q_{\mathrm{TBM}} = Q_0 \cdot \frac{\sigma}{F^{10}/20^9} \cdot \frac{20}{CLI} \cdot \frac{q}{20} \cdot \frac{\sigma_\Theta}{5}$$

$$\sigma = \sigma_{\mathrm{cm}} = 5 \cdot \gamma \cdot Q_{\mathrm{c}}^{1/3} \text{ mit } Q_{\mathrm{c}} = Q_0 \cdot \frac{\sigma_{\mathrm{c}}}{100} \text{ für } \beta > 60°$$

$$\sigma = \sigma_{\mathrm{tm}} = 5 \cdot \gamma \cdot Q_{\mathrm{t}}^{1/3} \text{ mit } Q_{\mathrm{t}} = Q_0 \cdot \frac{I_{50}}{100} \text{ für } \beta < 30°$$

The value σ is intended to take into account the rock strength, considering the uniaxial compression strength σ_c and the tension strength σ_t, here expressed as the strength index I_{50} and the angle β between bedding and tunnel axis. Furthermore, the value *m* is calculated to consider the wear patterns on the TBM (Fig. 2-25):

$$m = m_1 \cdot \left(\frac{D}{5}\right)^{0.20} \cdot \left(\frac{20}{CLI}\right)^{0.15} \cdot \left(\frac{q}{20}\right)^{0.10} \cdot \left(\frac{n}{2}\right)^{0.05}$$

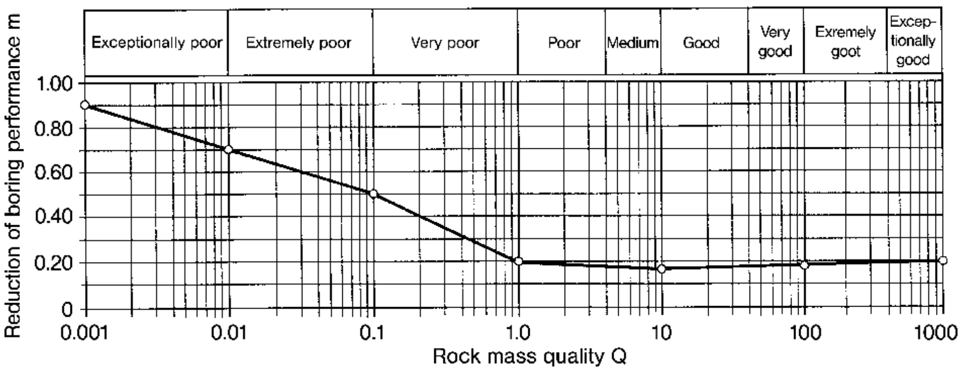


Figure 2-25 Reduction factor m_1 depending on the *Q* value [13].

The time required to drive a tunnel section or a specified geological zone with the length *L* using a TBM can be estimated as *T*:

$$T = \left(\frac{L}{PR}\right)^{\frac{1}{1+\mathrm{m}}}$$

with:

m reduction of boring progress due to wear [-]
T time required to drive a tunnel section
F thrust force per disc cutter [t]
m_1 base value for the reduction of boring progress due to wear [-]
D tunnel diameter [m]
CLI Cutter Life Index, a specific wear parameter of disc cutters determined from laboratory tests [-]
q concentration of quarzite in the rock [%]
n rock porosity [%]
γ rock density [kg/dm³]

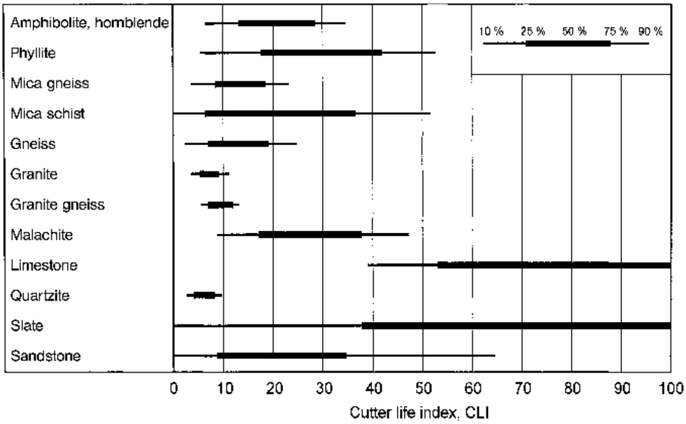


Figure 2-26 Wear index *CLI* depending on the rock type [13].

Classification according to cuttability and abrasiveness. A simple excavation classification is not adequate for TBM tunnelling. Parameters that determine penetration are therefore necessary for the overall grading of the rock mass to be bored.

Such a classification can be performed directly through the description of the geological stratum or through the direct determination of penetration using test strokes, as with SIA 198.

The two classifications of the rock mass that can determine the performance of a TBM were recognised in 1974 by Rutschmann [194]. In order to determine the cuttability, he considered four factors decisive:

– Stress-strain relationship: E modulus at 50 % compression strength $E_{t\,50}$.
– Uniaxial cylinder compression strength β_D.
– Brazilian tension strength β_Z.
– Hardness, for example the saw hardness *SH*.

The rock mass classification according to Rutschmann essentially describes the possibility of bracing a TBM. Both classifications correspond to tests of a qualitative description.

Rauscher developed a classification system based on models of the cuttability of a rock mass. This cuttability is expressed as an analytic relationship between the independent variable F_V (thrust force) and the dependent variable N_B (power of the cutterhead drive) and v_n (net boring speed). Using this relationship, he produced a nomogram that enables the classification of different rock mass types (Fig. 2-27). In the nomogram, the different cutting coefficients f_{SP} are entered as a group of hyperbolae and the performance ratio κ (quotient of N_B and the translational power $N_V = F_v \cdot v_n$) as a function of v_n and p (penetration) on one side and as a variable of F_v on the other side. In this way, κ can be used the decisive characteristic value for classification.

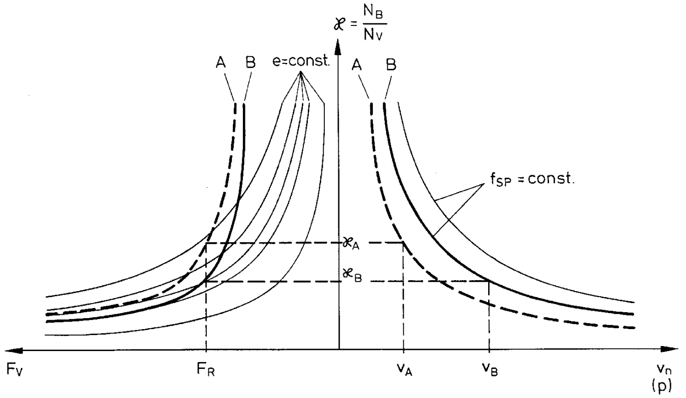


Figure 2-27 Nomogram for the evaluation of different rock mass types using rock mass-specific characteristic curves, according to Rauscher [185].

At this point, reference should also be made to the work of Beckmann, who investigated influential factors and their effect on the advance rates of TBMs based on experience and data from more than 40 km of tunnel driving [17]. Beckmann is able to prove that the rock mass has an influence on the advance rate of a TBM both according to its cuttability and its stability. The cuttability determined in in-situ tests is graded into six classes, as is the stability. Both parameters can now be used as additional criteria for payment, with cuttability and stability classes being combined in a price matrix, which is compared with a corresponding performance matrix by the contractor.

The cuttability according to geological conditions (Fig. 2-28) was produced by Schmid in 1988 for the Swiss Master Builders Association.

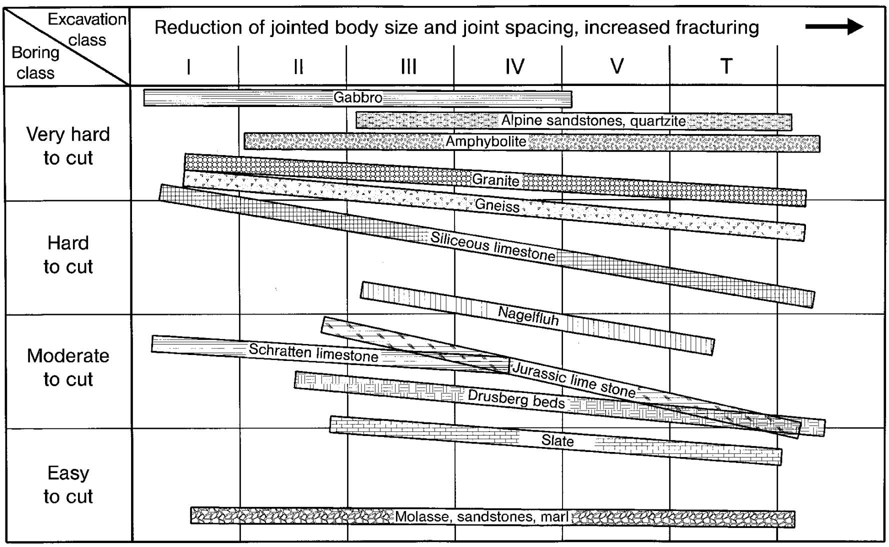


Figure 2-28 Cuttability and geological conditions [206].

The abrasiveness of rock has a direct effect on the cost of tunnelling. A limited and indirect influence results from the increased proportion of cutter changing time and any interruptions for the changing of wearing parts like lips or buckets. A wear class in addition to cuttability could therefore be suitable for the payment of wear, but scarcely for rock mass classification as a basis for advance performance.

Classification according to the type, extent and installation location of the necessary support measures and bracing feasibility. An experienced tunneller is normally capable, with intense discussion with the geologists, of undertaking a classification according to the type, extent and installation location of support measures. He compares the information gained with the facts from similar completed structures or with knowledge of any differences from projects that used similar construction methods. Whether this achieves better results than a classification according to rock mass properties is questionable. A better procedure would be to check the results of a classification according to rock mass properties against experience.

The determination of the necessary extent of temporary support including the compulsory addition of a safety margin for the miners is undoubtedly only possible at the face after excavation.

The national standards in Germany, Switzerland and Austria all recognise this as they are based on the extent of temporary support.

All are however based on the classification for drill and blast tunnelling without consideration of the fact that certain support measures cannot be used at any location in a TBM tunnel. For example shotcrete applied in the machine area is still included as a classification factor.

Without exception, these classifications provide no support measures in the first class, corresponding to a stable rock mass. The bodies responsible for insurance, SUVA, TBG, AUVA, however, demand head protection from a profile height more than 3 m. If according to the definition the installed support forms the basis for classification, it is immaterial for what reason it is necessary. It thus determines the class in every case.

2.5.6 Standards, guidelines and recommendations

2.5.6.1 Classification in Germany

DIN 18312 VOB Part C “General technical contract conditions for construction works – Tunnelling”, issue 12/2002 [59]. The basics of this standard are described in Section 2.4.3.1. Regarding mechanised tunnelling, the standard provides sub-classes TBM 1 to TBM 5 (Table 2-29) for tunnel boring machines and classes SM 1 to SM 3 (Table 2-30) for shield machines. For TBM drives, the tunnelling classes are differentiated according to:

– required support,
– obstruction of mechanical excavation,
– requires support installation in the machine area and
– special measures.

Table 2-29 Tunnelling classes for tunnel boring machines (TBM) [59].

Tunnelling class	Type of excavation
TBM 1	Excavation without support
TBM 2	Excavation with support, the installation of which does not obstruct excavation
TBM 3	Excavation with support immediately behind the machine or already in the machine area, the installation of which does not obstruct excavation
TBM 4	Excavation with support in machine area immediately behind the cutterhead, with excavation having to be interrupted to install it
TBM 5	Excavation with measures of a special nature, with excavation having to be interrupted to carry them out

While no support is necessary in the most favourable tunnelling class TBM 1, increasing instability of the rock mass and rising tunnelling class lead to the obstruction of boring by support measures, or boring even has to be interrupted. In tunnelling class TBM 5, special measures are necessary, for example for bracing the machine, to remove fallen debris in the machine area, to probe the ground and/or to improve the ground.

The classes reflect the installation location, with machine area and backup or rearward areas being differentiated. This classification is only general, as the standard does not provide any sub-classes or detailed explanations. Only in Section 3 “Construction” is there a note regarding excavation and support that all measures to be undertaken are a matter for the employer unless special contractual agreements have been specified about their type and extent.

For shield machines, the classes SM 1 to SM 3 are defined (Table 2-30). There is no delineation of the two systems, tunnel boring machine and shield machine.

Table 2-30 Tunnelling classes for shield machines [59].

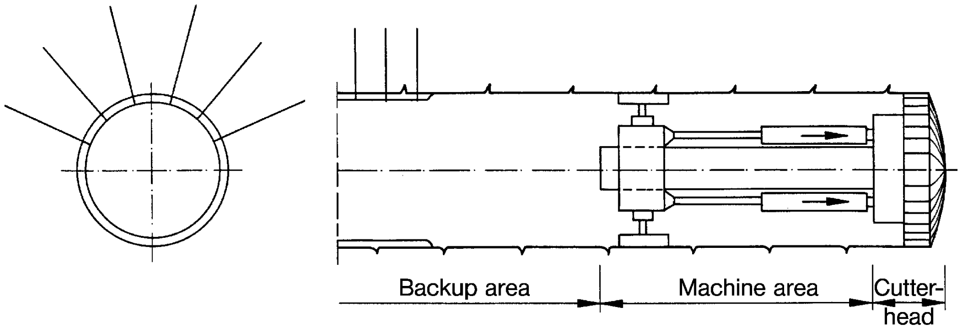
Tunnelling class	Excavation type
SM 1	Excavation without support to the face
SM 2	Excavation with partial support to the face
SM 3	Excavation with full support to the face

Recommendations of the “Tunnelling” working group of the German Society for Geotechnics (DGGT) (ETB) [52] The “Tunnelling” working group recommends the general classification scheme from DIN 18312, which is intended to serve as the basis for a project-specific classification for each tunnel. In addition to the classes from DIN 18312, TBM 1 to TBM 5 and SM 1 to SM 3, the additional classes SM-V1 to SM-V5 are also defined for shield machines with full-face excavation (Table 2-31).

Table 2-31 Tunnelling classes for tunnel boring machines and shield machines [43].

Machine type	Tunnelling class	Note
Tunnel boring machines (TBM)		For the determination of tunnelling classes for tunnel boring machines, the type and extent of the support measures and the installation location (in section and along the tunnel) and installation sequence together with the associated obstruction of full-face excavation are decisive.
	TBM 1	Excavation requires no support.
	TBM 2	Excavation requires support, the installation of which does not obstruct excavation.
	TBM 3	Excavation requires support immediately behind the machine or already in the machine area, and this obstructs excavation.
	TBM 4	Excavation requires support in the machine area immediately behind the cutterhead and its installation requires an interruption of excavation.
	TBM 5	Excavation requires special measures and excavation has to be interrupted to carry these out (for example measures to brace the machine, to remove fallen debris in the machine area, to probe the ground and/or to improve the ground from the machine).
Shield machines with full-face excavation (SM-V)		For the determination of tunnelling classes for shield machines, the type of face support and whether this obstructs excavation are decisive. Temporary or permanent support is installed as a closed ring inside the protection of the shield. Changing the process technology is not generally possible.
	SM-V1	Excavation without support to the face, and excavation is not obstructed.
	SM-V2	Excavation with mechanical partly or fully supported face, and excavation is not obstructed.
	SM-V3	Excavation with face supported by compressed air, and excavation is not obstructed.
	SM-V4	Excavation with slurry support to the full face, and excavation is not obstructed.
	SM-V5	Excavation with earth slurry support to the full face, and excavation is not obstructed.
		Excavation, which requires special measures with an obstructive effect on excavation, is to be taken into account by a project-specific classification with further sub-division of the classes (for example measures for bracing with SM-V1 $\Rightarrow$ SM-V1.1 or probing ground conditions from the machine in SM-V1 $\Rightarrow$ SM-V1.2).

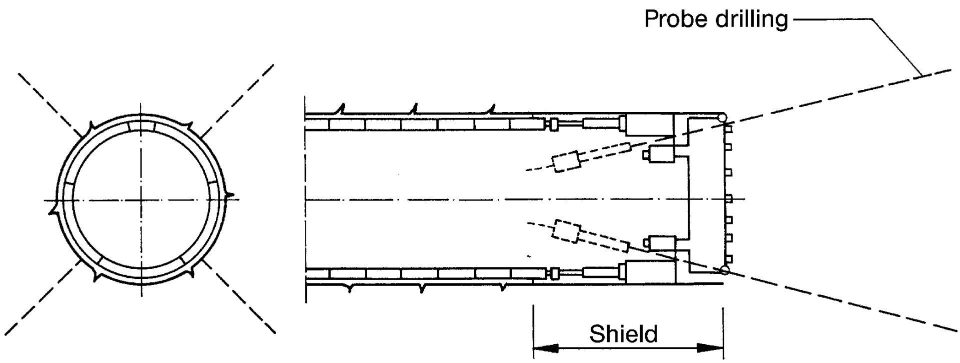
Fig. 2-29 shows an example of a classification according to the DGGT recommendations. This classification also complies with DIN 18312. In the example, the support is installed without obstruction of excavation.



Excavation		Tunnel boring machine
Support, installation without obstruction of excavation	Shotcrete	B 25, d = 10 cm
	Reinforcement	Q 257, 1 layer
	Anchors	SN anchor 6–7 N°, l = 4 m, e = 2 m
	Support arches	- - - -

Figure 2-29 Example of tunnelling class TBM 2 [43].

Fig. 2-30 illustrates an example of a classification for shield machines according to the DGGT recommendations. The example shows a tunnelling situation with special measures (drilling) to probe the ground from the machine. The support consists of a closed ring of reinforced concrete segments installed in the protection of the shield skin. The division into sub-classes is intended to refine the classification, for example to taken into account increasing difficulties. This covers defined, project-specific specialities.



Excavation	Full-face shield machines, with special measures (drilling) to probe the ground from the machine
Support	Reinforced concrete segments B 35, d = 40 cm as closed ring in the protection of the shield

Figure 2-30 Example of tunnelling class SM-V1.2 [43].

The general special features described by the ETB, which have already been described in Section 2.4.3.1, are also applicable here according to the ETB.

The following special features for TBM tunnels are listed:

- Recording of the quantities of the type and extent of support measures in ranges (shotcrete thicknesses, rock bolt number and length, type of arch, spacing, type of reinforcement, single or two layers; a certain overlapping of ranges of adjacent tunnelling classes is suitable according to the DGGT).
- Installation location and installation sequence of support in the machine area, backup or rearward area.
- Cuttability of the rock (mineral compositions – particularly quartz content), mineral formation, mineral grain size, compression strength, hardness, abrasiveness, jointing structure.
- Grading by given net boring speeds.
- Consideration of the special features of shafts.

The following special features for shield machines are listed:

- Grading distribution, fines content, mineral composition, bedding density, consistency, water content, groundwater table.
- Special features, which could obstruct the advance rate, like mineral-chemical cementation, adhesiveness, soil plasticity, influence of water content in changing the soil, boulders, intercalated hard layers, high/very variable permeability, separation in the machine area (not due to machine).
- Measures to maintain the stability of the face, interventions in front of the cutting wheel.
- Removal of obstructions.
- Type of annular gap grouting with regard to obstruction of advance.
- Restrictions due to building or facilities with shallow overburden.
- Consideration of measures ahead of the face like probing the ground from the machine and/or ground improvement from the machine (grouting or freezing).

2.5.6.2 Classification in Switzerland

SIA 198 “Underground Structures”, issue 1993, reissue 2004 [218]. The basics of this standard are described in Section 2.4.3.2 (page 57).

Section 5.4 of the standard is about payment for excavation works with a TBM in hard rock and is based on the idea that costs are directly related to the type and extent of necessary support works, but also the point in time when the measures have to become effective.

In Switzerland, tunnelling classes are defined by SIA 198 (1993) based on the costs of tunnelling. Excavation types A (full face) and E (excavation in phases with a pilot tunnel or pilot shaft and subsequent enlargement; both with TBM) are differentiated. The subsequently explained provisions apply to both of these excavation types. If a pilot tunnel bored by a TBM is combined with enlargement using another method of excavation, the relevant provisions for bored tunnels apply to the pilot tunnel and those for drill and blast or roadheader excavation as appropriate apply for the enlargement.

The tunnelling classes result from a combination of excavation and tunnelling classes. The excavation classes are based, as in Germany or Austria, on the required support measures and the installation location along the TBM. Five excavation classes are normally differentiated, with no support measures being intended for the rock mass in excavation class 1 and heavy excavation support being necessary in the machine area in excavation class 5. But if the excavation support consists of an immediate and continuously installed ring of segments, the excavation classes no longer apply; excavation class T is provided for this case.

The consideration of the installation location is more complex for TBM drives than for drill and blast. The working areas for boring with a TBM are described as follows in Switzerland:

– L1 machine area.
– L2 backup area.
– L3 rearward area up to 200 m behind the backup.

Within the areas L1, L2 and L3, the working zones L1*, L2* and L3* are defined, in which the temporary support is installed according to the conditions on the project and the constraints of the machine type (Fig. 2-31).

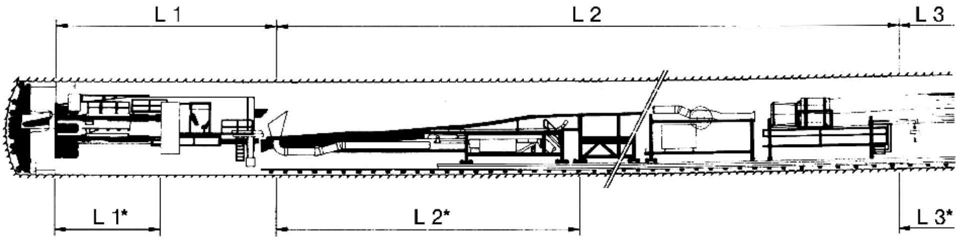


Figure 2-31 Working areas and working zones for a TBM [218].

It is the task of the contractor to include with the tender details of the lengths of the working areas L1, L2 and L3 and the working zones L1*, L2* and L3* on a plan of the intended tunnel boring machine, and also to state which support works are possible for the tendered works; he has to take into account the conditions regarding installation location of the temporary support specified in the tender documents.

The combination of type, extent and installation location of the support measures gives the relevant excavation class. Support measures outside the designated working area have no influence on the classification of excavation.

The main principle for the firm definition of the excavation classes should be in every case the degree of obstruction due to support works (Table 2-32). Exact assignment is according to Table 2-34.

Table 2-32 Excavation classes according to Swiss standard SIA [218].

Excavation class	Obstruction of excavation
AK I.	The excavation support causes an insignificant obstruction of the excavation cycle.
AK II.	The excavation support causes a slight obstruction of the excavation cycle.
AK III.	The excavation support causes a considerable obstruction of the excavation cycle.
AK IV.	The excavation support causes an interruption of the excavation cycle (immediate support after every stage of excavation).
AK V.	The excavation support is installed continuously with the excavation and requires immediate buttressing of the face or pre-excavation support.
AK T.	The excavation support consists of a closed segment ring, which is installed immediately and continuously.

The excavation classes basically apply for drill and blast in solid rock (SIA 198, Section 5.2) and for roadheader drives in solid rock (SIA 198, Section 5.3) and for TBM drives in solid rock (SIA 198, Section 5.4), but different evaluations are offered by the SIA 198.

Table 2-33 Matrix of excavation and boring classes, diagram from SIA 198.

Excavation class	Boring classes		
	X	Y	Z
I	I X	I Y	I Z
II	II X	II Y	II Z
III	III X	III Y	III Z
IV	IV X	IV Y	IV Z
V	V X	V Y	V Z
T	T X	T Y	T Z

For the application of the excavation classes, tunnels and inclined shafts are dealt with in the same way. For vertical shafts, which are enlarged from bottom to top with a cutterhead, there is no division into excavation classes.

The boring classes correspond to the cost of excavating solid rock with a TBM and take into account both penetration and tool wear as decisive factors. Boring classes are specified by the employer from essential rock and rock mass characteristics. In this way, the length of tunnel to be driven can be summarised as sections with the same class with regard to the cuttability of comparable geological formations or rock types. The number of boring classes is thus determined by the prevailing conditions and is specified in the tender documents for each project. The decisive rock mass characteristics are to be given as precisely as possible with their range of variation.

The boring classes X, Y, Z are to be understood as symbols for differently cuttable rocks along the tunnel; X could stand for a gneiss defined in the longitudinal geological section, Y for a granite. On another project, X could represent Schratten limestones, Y marl of the Drusberg beds, Z a siliceous limestone.

Alternatively, the standard SIA 198 also permits the grading of boring classes from penetration rates driven in-situ. The determination of the net boring speed is performed contradictorily, daily on a stretch of tunnel that is representative for the drive, using a test procedure specified in the contract (test stroke length, thrust force, condition of the cutting tools and their state of wear). The procedure is however only regarded as suitable for special cases, on the one hand as too many factors determine the penetration and on the other as it is questionable whether the test stretch is representative.

The classification of the tunnel drive into individual classes is done by linking the relevant excavation and boring classes. The combination of excavation and boring classes is performed in a matrix; an example is shown in Table 2-33. Each entry in the matrix corresponds to a stage of boring and the support work obstructing a stage of boring and is assigned a unit price per metre of advance for the payment of this tunnelling class.

With this matrix, it can be that the advance achieved is determined solely by the boring class. In rock mass that is hard to bore, it is possible to install the excavation support of classes II, III or even IV in the time it takes to bore a stroke, unless the support is shotcrete.

Table 2-34 Determination of the excavation classes due to the excavation support for TBM tunnelling in solid rock[1)].

	Class I	Class II	Class III	Class IV	Class V	Class T
Excavation classes for tunnels						
Machine area L1		≤ n anchors around tunnel perimeter	> n anchors around tunnel perimeter with mesh and/or lagging sheets	Anchor with mesh and partial arch Shotcrete as sealing around > 1/4 of the profile perimeter Pressure-distributing elements at the bracing shoes	Closed steel sets with or without lagging Full-surface steel elements	Closed segment installation
Backup area L2	Mesh as rockfall support, fixed with anchors or bolts Invert segments in case provided in design for all rock classes	**Systematic arrangement** > n anchors with partial arch around > 1/4 of the profile perimeter Mesh, shotcrete around < 1/2 of the profile perimeter **Non-systematic arrangement** > n anchors	**Systematic arrangement** Mesh, shotcrete around the entire profile perimeter except invert Partial arches with anchors around ≤ 3/4 of the profile perimeter **Non-systematic arrangement** Mesh, shotcrete around ≤ 1/2 of the profile perimeter > n anchors with partial arches around < 1/4 of the profile perimeter	**Systematic arrangement** Invert support with shotcrete, if no invert segments are provided Closed steel sets, possible supported on invert segments **Non-systematic arrangement** Mesh, shotcrete around the entire profile perimeter except invert Partial arches with anchors around ≤ 3/4 of the profile perimeter Closed steel sets with or without lagging		
Rearward area L3 up to 200 m behind backup	Excavation support in L3 is insignificant for the assignment of excavation classes. In case such support is intended, waiting time items are to be provided.					

values for n (number of anchors per running m of tunnel)	max. excavation diameter:	4.00 m	6.00 m	9.00 m	12.00 m
	n	2	3	4	6

[1)] If there is more than one support measure in a field of the table, each measure alone is sufficient for classification.

2.5.6.3 Classification in Austria

ÖNorm B 2203 "Underground works – contract conditions" has recently been revised [172, 172b].

"Part 2 – continuous tunnelling" is intended to regulate the special features of TBM drives. According to [12], however, Part 2 is mainly based on the former standard ÖNorm B2203-Part 1, version 2001, (see 2.4.3.3) with adaptations for the special features of TBM tunnelling. The descriptions of the geological situations from the old ÖNorm B2203 from 1994 were not taken over analogously to the new Part 1. To overcome this problem, a separate guideline for the geotechnical design of TBM drives has been developed, according to [12].

For the procedure in ÖNorm B2203, reference is made here to the explanations in [148].

2.5.7 New classification proposal

The classification systems that have been discussed so far are based to a considerable degree on support methods, which are not always suitable for mechanised tunnelling or ignore the problems of cuttability and feasibility of bracing, although these are significant for the advance rate.

These deficiencies often lead to serious differences between the contract parties, perhaps because construction times cannot be maintained or due to claims based on changed conditions.

The objective of classification must be to create a means of understanding, which permits the fairest possible treatment for both contract parties without however believing that every case of changed conditions can be classified in a system.

With the exception of extreme cases, only the excavation support and the penetration of the TBM to be used are decisive for the advance rate. An excavation classification should therefore include both these main components. The wear has an effect above all on the excavation price.

For a TBM drive, the classification should consider the following components if possible:

– Support systems as an excavation class with the actual possibilities.
– Cuttability with facts of a geological and/or geotechnical nature, which enable a bounding of the penetration rate.
– Wear with the basics of abrasiveness as it affects the excavation process, for example Cerchar [148, Chapter 3]; proposal: if a Cerchar test is not possible, details of the content of abrasive minerals.

Proposal for excavation classes: The excavation classification from Schmid [148] is based on support systems, which can be installed quickly. This enables interruption of boring to be minimised with increased overall cost-effectiveness. Table 2-35 shows the proposed excavation classification with guideline values for support systems. The profile types are to be defined for each case.

According to current experience at the Lötschberg Tunnel and the Gotthard Tunnel, an additional class is required, support with shotcrete in the L1 area, i. e. immediately behind the cutterhead.

Proposal for boring classes: Tunnel sections with similar cuttability are to be graded into boring classes. This can be undertaken according to geological sections (Fig. 2-28) or also in exceptional cases by specifying penetrations in the construction contract, to be verified during the construction phase using test strokes with a defined condition of the disc cutters and the pressing force.

The procedure with test strokes does however have grave deficiencies. If the contractor uses a very powerful TBM for the conditions, he is penalised because the payment is less for the improved performance. If he uses a TBM with scarcely adequate power, then the employer is correspondingly disadvantaged.

Assignment according to geological conditions (Section 2.5.5.2) with specification of the geotechnical properties compression strength, tension strength for the abrasiveness (Cerchar) leads to a good contractual basis.

The contractor tenders a price matrix according to Table 2-33.

Table 2-35 Excavation classes according to the proposal of Schmid [148].

Gripper TBM with systematic support					**Shield TBM**
Support system	**very light** (head protection in large diameters)	**light**	**medium**	**heavy**	**segment**
With invert segment					
in machine area	Mesh as rockfall safety measure	Light arch 1 to 1.5 m spacing with light lagging e. g. mesh as unit	Medium arch 0.8 to 1.2 m spacing and lagging as unit	Heavy arch 0.75 to 1.0 m spacing with heavy lagging as unit	
in backup area	shotcrete 30 – 50 m behind cutterhead				
Without invert segment					
in machine area	analogous, except continuous arches in invert				
in backup area	shotcrete also in the invert				
Profile types	support elements defined as profile types for the tender documents				
	Profile type 1	Profile type 2	Profile type 3	Profile type 4	T
Excavation class	I	II	II	IV	T

According to Gehring, corrections of penetration during construction can be controlled very well using the ratio of the forecast and measured compression strengths. The correction factor leads to a penetration corresponding to the curve in Fig. 2-32.

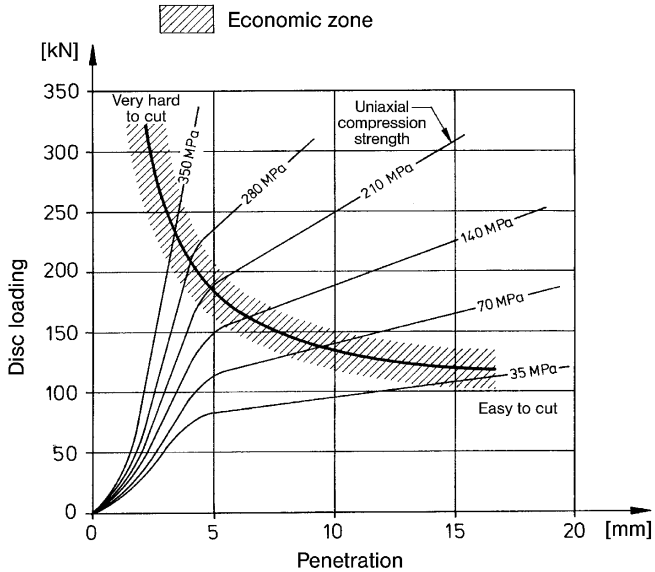


Figure 2-32 Robbins curves from 1970, overlain with disc pressures and penetrations as a function of rock strength.

$$K_p = \frac{1}{\left(\dfrac{\sigma_{ucs.measured}}{\sigma_{ucs.prognosis}}\right)^{\lambda}}$$

K_p = correction factor for penetration
$\sigma_{ucs.measured}$ = measured uniaxial compression strength
$\sigma_{ucs.prognosis}$ = prognosis of uniaxial compression strength
λ = exponent 1.0 to 1.2 (frequently 1.1)

For the boring process, tension strength would indeed be a decisive factor, but including it in evaluations always fails. The main reason for this is that with increasing compression strength, the tension strength also increases but not to the same degree. The basic value of compression strength thus also includes the tension strength.

The ratio of compression strength to tension strength seems to be more significant. This often varies between values of 12 and 15 with an extreme range of 8 to 22. For the construction contract, it is therefore important to give the forecast span of a realistic magnitude.

Wear. The increase or also reduction of wear, due to the great variety of cutter types in shape and steel quality, can only be determined sensibly by directly forming the quotient of actual value to forecast value. The extra cost or cost saving is the result of multiplying the wear costs in the estimation by the quotient.

3 Structural design verifications, structural analysis of tunnels

3.1 General

The terms “load determination”, “structural analysis”, “calculation”, “dimensioning”, and “safety coefficient” cannot be transferred from structural engineering to tunnelling without comment. The usual practice until now of using the existing standards and guidelines for reinforced concrete, steel construction and others, which were normally intended for structures above ground, for tunnelling should also be reconsidered. The exploitation of interpretations of or even deviations from standards when local conditions are not relevant for use in tunnels is not a way to perpetuate existing experience for innovative new developments. For many years, structural verifications have been worked out from scratch, working with new engineers responsible for checking the calculations. By the end of the project, a more or less good and perhaps also economic compromise will have been found. But compromises are often far from the optimal solution. Collected experience is only transferred sparingly, because the next project will be based again from the existing standards and guidelines and start once more from the beginning. The extent of the efforts made by the specialist world of tunnelling to fit tunnel engineering into the existing environment of verification calculations is demonstrated by the chapter heading “Attempts at structural calculation” in the book “Rock construction” volume 3, by L. Müller [160].

In order to justify the special situation of tunnelling, the following differences compared to conventional structural engineering can be stated:

Rock mass. The rock mass is simultaneously a loading and load-bearing element. The loads are not normally acting on a load-bearing member from the outside. The designer only has information at isolated locations from the geological survey. The available information is, however detailed, not adequate to perform a conclusive structural verification. Measurements made as the tunnel progresses demonstrate the real behaviour of the unsupported and supported tunnel as a full-scale model, presuming they are appropriately performed and evaluated. Unexpected changes of ground conditions can however still lead to collapses, even when great caution is exercised.

Construction process. The size and shape of the cross-section influence the ground pressure and the loading. The excavation process influences the degree of loosening of the rock mass. The time taken for support to become effective influences the distribution of the development of ground pressure.

Support materials. Support materials can support at points (steel profiles) or as a bonded composite (shotcrete) and they influence the loosening of the rock mass and its contribution to structural action. The strength development of the construction materials with a hydration process influences the development of ground pressure and deformation. The quality requirements for the construction materials, like for example concrete strength, are often negatively influenced by local conditions.

Collaboration of the contract parties. Employers, consultants, contractors, appointed specialists and supervisors must have not only professional qualifications but also experience and the will to achieve technical and economic success. (The number of parties involved should be as few as possible). Measurements and calculations serve as criteria for structural stability and for further proceeding. The trade skills of the men on site have a decisive influence on the loosening of the rock mass, avoidance of rock falls and the quality of support materials.

These special features may seem negative but are balanced by positive features:

– The reserves of load-bearing capacity in the rock mass as a composite with the support materials are high, as long as the rock mass is dealt with expertly.
– The possibility of evaluating measurements made on site and comparing them to design calculations provides evaluation criteria for structural stability.
– If the construction process is adapted in good time with additional strengthening, this helps to avoid large deformation or even collapses.

The variety and range of variability of the special features mentioned in comparison to normal structural engineering make tunnel construction one of the most interesting but also challenging engineering disciplines. In no other field do theory and practice have such an effect on each other and have to be overcome with knowledge of structural analysis, materials technology, geology, machine technology and particularly construction process technology. Each of the parties involved in the project, whether site manager, consultant, appointed specialist or supervisory engineer needs basic knowledge of these varied disciplines. The site manager cannot interpret measurements and introduce immediate measures to adapt the process without knowledge of the structural design and geology; and the consultant cannot produce a meaningful design without knowledge of construction process and procedures on site.

Requirements. The verification of structural stability has to include analysis and calculation of the play of forces between rock mass and support at various stages, taking into account the construction process and excavation methods. Verifications of structural stability are normally performed as part of dimensioning the elements. The special features already mentioned do not permit this to be done unambiguously, so construction processes and constructional solutions have to be investigated and critical phases identified. In addition, building regulations, at least in Germany, require verifications of structural stability to be performed, and the procedure to be adopted also has to be justified with calculations to professional colleagues, certainly after the occurrence of a collapse or failure.

In order to avoid repeating material from the extensive literature, which reaches back into the 19th century, the most important terms and models connected with the verification of structural stability will now be summarised and arranged according to their application. Examples from tunnels built in recent years are intended to document current practice.

3.2 Ground pressure theories

The construction of a tunnel disturbs the existing equilibrium in the rock mass. The type and magnitude of the action of this technical intrusion are the result of a complex interaction of rock mass and tunnel construction. The rock mass is provided with all its properties. The construction of the tunnel structure is involved in the interaction, which causes the loading of action and reaction of ground mass and tunnel, particularly through the selection of a construction process and the associated or decisive decision about its depth, cross-sectional dimensions and construction. The construction of the tunnel is particularly drawn into the interaction between the loading of action and reaction of rock mass and tunnel by the selection of a construction process and the associated or decisive decisions about the vertical alignment, cross-sectional dimensions and construction.

3.2.1 Historical development

Since the construction of the important rail tunnels in the previous two centuries, numerous engineers and geologists have published their analyses of the loading on tunnels and rock mass.

Qualitative statements. A. Heim [89], writing in 1878 and 1905, considered the loading, which he mostly discovered in the form of damage to tunnels with open inverts, to result from a ground pressure acting equally all around the cavity, similar to hydrostatic pressure: the hydrostatic pressure corresponds to the weight of the rock mass overburden. The prompt installation of a closed tunnel tube prevents damage by activating the internal friction of the rock mass. The lining tube should therefore by fully mortared to the rock mass.

W. H. Trompeter [246], writing in 1899, assumes from his observations in coalmines that the excavation of coal seams forms spatial protective envelopes, which relieve the cavity when the depth is sufficient, particularly in the longitudinal direction.

E. Wiesmann [258] repeated in 1912 the view that a protective envelope is formed across the tunnel, since the loading on deep tunnels from overburden pressure would otherwise be so high that such tunnels would "have to be left alone".

R. Maillart [152] in 1923 extended the theory of Heim with the view that the horizontal pressure and the vertical pressure can be of different magnitudes and the lining of the tunnel increases the strength of the surrounding rock mass "because the strength of a body is not solely a function of the largest pressure acting in a certain direction, but is also the minimum pressure acting in one of the other dedicated main axes".

Quantitative statements. After the more qualitative considerations (A. Heim, W.H. Trompeter and others) about ground pressure, which were summarised by E. v. Willmann [259] and E. Wiesmann [258], the ground pressure was then regarded as an external load parameter and introduced into structural calculations. To start with, the ground pressure was considered as a fill covering acting on the tunnel structure with the full surcharge weight. Later, this ground pressure was reduced, W. Ritter, K. Culmann [259]. For the magnitude of ground pressure for calculation purposes, observations of crown settlement from O. Kommerell [113] and the influences of tunnel dimensions and rock mass structure from K. Terzaghi [241] were taken into consideration.

In order to be able to find quantitative statements to explain the interaction of ground pressure and tunnel construction, mathematical and mechanical models were used. The structural analysis model of the vault of the tunnel covered by fill was replaced by the model idea of the elastic continuum (H. Schmid, 1922 [202], R. Fenner, 1938 [73]) and the bedded continuous beam (B. H. M. Hewett and S. Johannesson, 1922 [93], A. Bull [30]). The ground pressure was thus introduced into a model as a load and deformation quantity for calculation purposes. For the model of a deep elastic beam, H. Kastner in 1962 [103] developed the plastic zones and a lining resistance in order to illustrate the effect of the ground pressure on the tunnel structure.

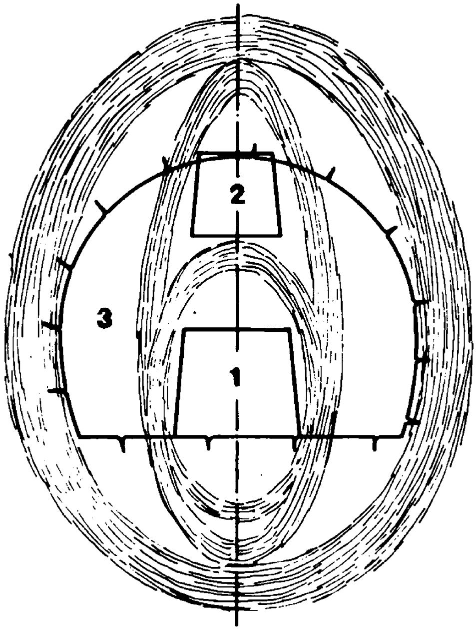


Figure 3-1 Diagram of the development of a protective envelope at various stages of excavation of a tunnel cross-section according to L. v. Rabcewicz [182].

The ground pressure theories were produced in line with the relevant state of the technology of the classic construction methods for deep tunnels under mountains and still serve as thought models, for example as protective envelopes or load bells, as aids to interpretation for similar or also new methods of construction and operation, in order to understand and overcome the interactions of ground pressure and tunnel construction methods. L. v. Rabcewicz [182] in 1944 returned to the formation of a protective envelope across the tunnel axis for deep tunnels beneath mountains (Fig. 3-1). Later, he used this idea for the description of the New Austrian Tunnelling Method (see also Volume 1, Chapter 4).

Types of ground pressure can be differentiated according to their causes [182]:

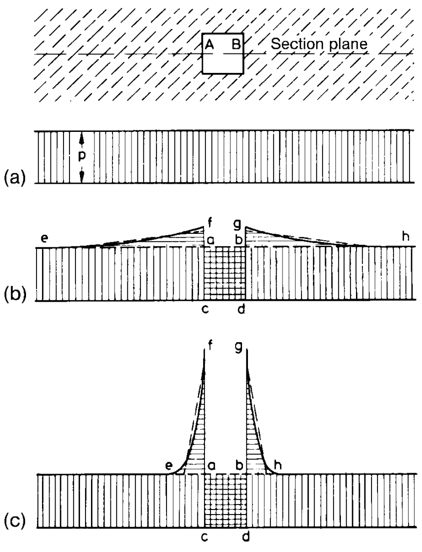
1. Surcharge pressure from the weight of the overburden, from the topography of the tunnel location.
2. Tectonic pressure from the movement of the Earth's crust.
3. Loosening pressure from the loosening of the rock mass and from disturbance by technical intervention.
4. Redistributed pressure from the disturbance of the equilibrium through technical intervention, through volume increase of the rock mass as swelling pressure.

The various terms for ground pressure are given various meanings in the literature. So L. Müller, 1978, takes up the recommendations of the "International working group for geomechanics" for the unified use of ground pressure terms and differentiates ground pressure according to the "primary stress distribution" and the "secondary stress distribution" [160]. The ground pressure is no longer separable from the type of technical intervention into the rock mass and is used in various ways for the explanation and interpretation of observations from tunnel drives. In addition to this came the development of calculation models using numerical processes (Finite Element Method), which input the ground pressure as "primary pressure".

With the introduction of numerical calculation techniques, the ground pressure is used as a model quantity in the calculation, introduced into the model as a parameter for the magnitude of loading and deformation instead of the ground pressure theory. The sequence of excavating the tunnel is simulated with various types of ground pressure for the relevant model of rock mass and tunnel. The associated ground pressure types can be analysed through observations and through measurements.

3.2.2 Primary and secondary stress states in the rock mass

When a tunnel is driven, the existing pressure conditions in the rock mass are disturbed. The primary stress state becomes a secondary stress state in vertical and transverse directions around the cavity (Fig. 3-2).



(a) In an undisturbed rock mass
(b) Immediately after the tunnel advance in yielding rock mass
(c) Immediately after the tunnel advance in brittle rock mass

Figure 3-2 Ground pressure distribution according to E. v. Willmann [259].

3.2.2.1 Primary stress state

The primary stress state is defined as a spatial stress state, which is very much influenced by the type and development history of the rock mass. Strata, fissures, folding, tectonics and the slope location of the rock mass form the primary stress state. It is difficult to measure and thus can almost only be understood theoretically under the assumption of a homogeneous, isotropic and undisturbed rock mass as a ground pressure, which does or can correspond to the overburden pressure when, for example, no additional tectonic pressures or other effects are suspected. The primary stress state is influenced by:

– Depth,
– Overburden weight,
– Rock mass type (loose ground, rock),
– Characteristics of the rock mass (tectonics, bedding conditions, water conditions).

Fig. 3-2 (a) shows the primary stress state according to Willmann [259] in a planar model.

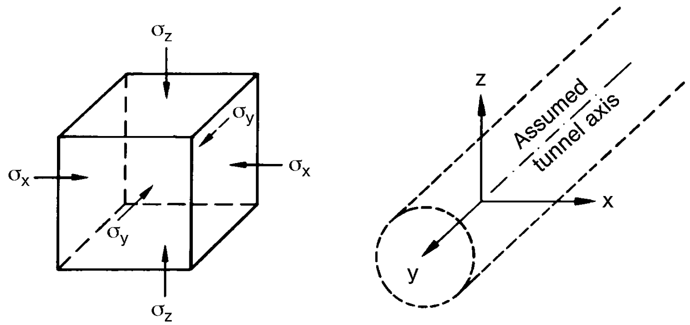


Figure 3-3 Principal stress direction of the spatial stress state with assumed tunnel axis.

The directions of the principal stress of the triaxial stress state are mostly defined in relation to the tunnel axis (Fig. 3-3).

3.2.2.2 Secondary stress state

Fig. 3-2 (b) and (c) show the secondary stress state according to Willmann [259] in a planar model. In order to forecast the effects of driving the tunnel on the formation of a secondary spatial stress state depending on the time-dependent complex interaction of rock mass, tunnel structure and construction process, model concepts with the aid of idealisations are used to order and simplify the numerous influential parameters for the load-bearing behaviour, the loading and the material behaviour of rock mass and tunnel. One aim of this is to make the influences and the complexity calculable through experience and conceptual models.

3.3 General steps of model formation

The generally formulated phases of operational research methods are also applicable for the problem being discussed here (Fig. 3-4):

1. The formulation of the problem based on reality.
2. The design of the mathematical model for the system to be investigated.
3. The derivation of a solution from this model.
4. The testing of the transferability of the model to reality.
5. The precautionary monitoring and adaptation of the solution.
6. The practical implementation of the solution.

The special problem only at first glance seems to be soluble on one hand with trial-and-error approaches based on plenty of practical experience, or on the other hand through a purely formal and theoretical approach,

- since the material values for the rock mass as a construction material cannot be unambiguously determined and change continuously.
- since the very complicated numerical calculation procedures do not approach the problem with sufficient flexibility, that is they scarcely represent the practical behaviour of the rock mass, do not consider the time factor, are much too expensive and can scarcely be influenced from outside by the parties directly involved on the site.
- since the working sequence with its time-dependencies does not input directly into the calculation.
- since measurements continuously deliver new information about load-deformation behaviour and make corrections necessary.
- since a historical development of experience is available, which is recognised even by theoretically oriented experts.

The result is that the calculation does not provide a basis for the specification of a construction process and the closely associated dimensioning, but serves as a verification of engineering solutions. The calculation also delivers essential additional information for the evaluation of risk.

Further evaluation makes an objective method of consideration seem more questionable.

A stepwise inclusion of the engineer with his experience in the numerical calculation process could represent a possibility of solving the overall problem. Information data for further stepwise decisions is delivered by the computer. No more information can be gained from the calculation procedure, so all important decisions are left to the engineer. Safety considerations and concepts could also be used [65], [66].

The general scheme for the solution of complex problems using computers with interactive intervention by an engineer in tunnelling is presented in Fig. 3-5, which shows the stages of the dialog between engineer, computed results, measurements and experience.

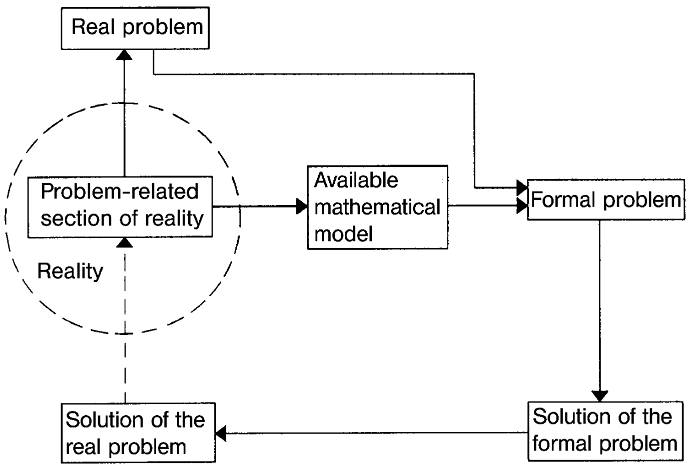


Figure 3-4 Scheme of a procedure with mathematical methods for modelling in tunnelling [139].

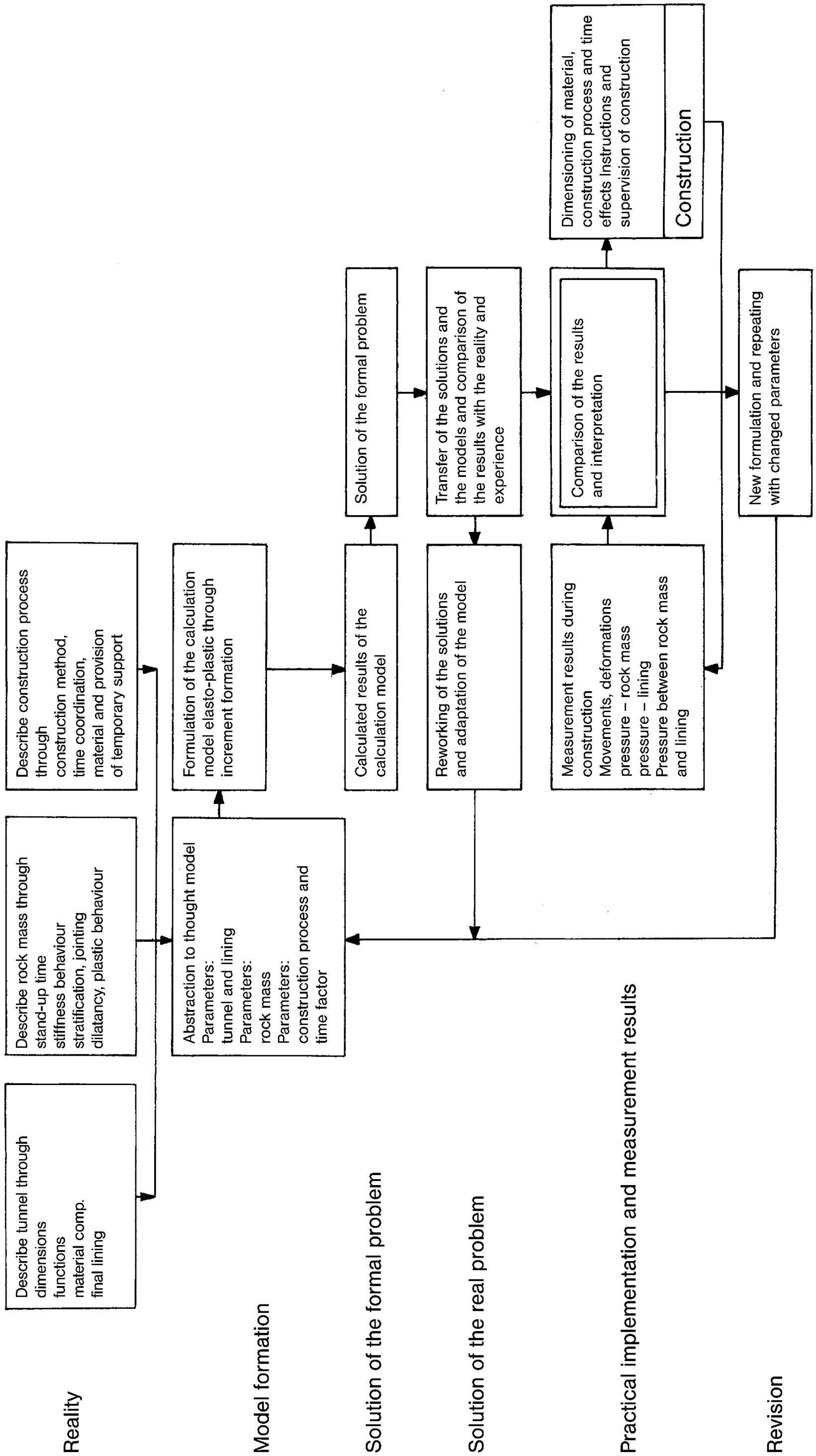


Figure 3-5 Methods of evaluating construction states in tunnelling [139].

3.4 Analytical processes and their modelling

There is a basic difference between tunnels in loose ground and those in solid rock regarding loading assumptions and modelling. For tunnels in loose ground, shallow and deep tunnels are also differentiated, with a tunnel being considered deep when the overburden is at least twice the diameter, or the equivalent diameter of non-circular sections (Fig. 3-6).

The difference between these three groups of tunnels is significant above all in analytical calculations. With the increased use of numerical calculation methods – above all the Finite Element Method (FEM) – this categorisation has become ever less important. Analytic processes are now briefly discussed.

Two-dimensional models, which are the most commonly used at the moment, are presented for the three groups. Recently, however, three-dimensional models have been used increasingly as these are becoming ever more practical for daily use with the development of computers and programmes.

3.4.1 Modelling of shallow tunnels in loose ground

Shallow tunnels in loose ground are normally calculated using a model of a pipe covered by loose material. It is assumed that the rock mass or soil cannot form a load-bearing ring and that the tunnel support has to resist the entire load. Tunnels of this type with overburdens of up to one diameter are often driven in an open trench, since the soil cannot be assumed to have sufficient stand-up time until the support is sufficiently load-bearing.

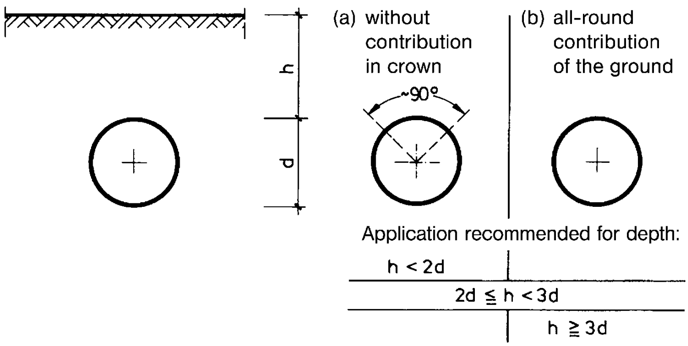


Figure 3-6 Approaches for the contribution of the ground to load-bearing.

3.4.2 Modelling deep tunnels in loose ground

For deep tunnels, the assumption is made in accordance with Terzaghi's silo theory [238, 242] that the actual ground pressure is nearly independent of the actual overburden and is determined by the ground properties. The model assumes that a vault forms at a certain height due to the internal friction of the material and this resists loading from above. The formation of this vault depends on a deformation of the ground or the soil, which results from the relaxation of the sides of the tunnel as the section is excavated. A similar effect can be observed in a silo, when the material "hangs" at the side walls of the silo due to friction between the material in the silo and the wall surface and the material does not flow out although the silo is full.

The vault formed in this way transfers its support reactions into the area of ground beside the tunnel, which is thus compressed in addition to the increased compaction due to the overburden, so a certain subgrade reaction can be expected.

Therefore these tunnels are idealised as circular rings in the elastic continuum or partial continuum, with the crown area of the ground in this case (about two to three times diameter) being assumed as full loading (for greater overburdens, there is a basic reduction). The remaining areas are idealised as a composite load-bearing system of tunnel support and ground (Fig. 3-7).

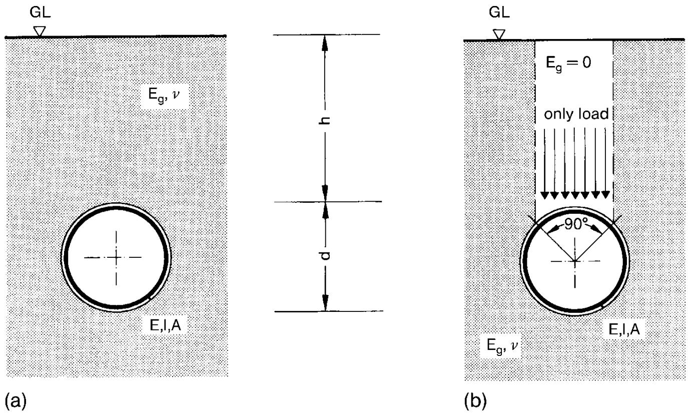


Figure 3-7 Circular ring in the elastic continuum (a) and in the partial continuum (b) [4].

3.4.3 Modelling tunnels in solid rock

In solid rock, a very intensive composite action of tunnel support and rock mass is assumed. A large proportion of this load-bearing effect is assigned to the load-bearing capacity of the rock mass. In some extreme cases, the load-bearing capacity of the rock mass is so high that no support is required. This case is idealised by an elastic slab with a hole [130]. The overloading of the rock strength that occurs in the elastic calculation can however not be explained by this model. For this reason, Kastner developed the model further with the introduction of plastic zones around the tunnel cross-section [103].

The analytic processes still have their significance today, as it is possible to achieve usable approximations for a given problem with relatively little work. As soon as the solutions contain non-linear constitutive laws or complicated geometries, analytic processes reach their limits.

A further disadvantage of analytic processes is that they generally include no information about failure mechanisms. The determination of the failure load or the determination of the failure state is, however, one of the most important tasks in geomechanics, since the failure state is mostly used as a basis for the definition of safety margins [215].

These disadvantages, above all the cross-sections of tunnels that often deviate from circular, were the initial reasons for the development of numerical methods in tunnelling.

3.4.4 Bedded beam models

The interaction between ground and tunnel construction is simulated in this model by the so-called bedding or stiffness modulus procedure. The tunnel is represented by a non-linear bedded beam and is wholly or partially surrounded by bedding. A subgrade reaction is activated as soon as deflection into the ground occurs, which simulates the load-bearing effect of the ground. A certain proportion of the load is resisted by the subgrade reaction, controlled by the modulus of subgrade reaction. The tunnel construction can be represented in analytic calculation methods as an elastically bedded non-linear beam in the elastic continuum or if the geometry is more complex, as a bedded continuous beam ring in a numerical calculation procedure. Bedded beam models can also be used to model the geometry of cross-sections with partial areas that are not embedded. The numerical model of a bedded beam is frequently used in continuous beam programs specially modified for tunnel design.

The modulus of subgrade reaction is not a soil constant but depends on the modulus of elasticity of the ground E_g, the radius of the tunnel R and the factor C. Usual values for C lie between 0.5 und 3.0 [4, 160]:

$$k_s = \frac{C \cdot E_g}{R}$$

In addition to the full-round continuously bedded system, partially bedded systems are also used. In this case, a 90° unbedded crown segment is typical, since the support in most cases deforms inward in this zone. For the calculation, care should generally be taken that displacements into the tunnel do not activate any tension forces in the bedding, since in reality the ground will only provide a subgrade reaction in compression. The value for the bedding around the perimeter does not have to be constant but can change linearly or in steps, particularly for systems with unbedded crown. Radial or tangential bedding can be differentiated according to the direction of the subgrade reaction. While the assumption of radial bedding is always realistic, the applicability of tangential bedding in contrast must always be checked. This applies particularly to calculations modelling the inner lining of two-layer constructions separated by a foil.

Depending on the depth of the tunnel, the vertical overburden pressure can be assumed to act fully, or for deep tunnels partially reduced, as an external load. The horizontal load component is determined from the appropriate coefficient of lateral ground pressure. When a bedded beam model is used, different variants are available depending on the beam program used. The optimal representation of the tunnel geometry is as an arch consisting of curved members. It is also however possible to approximate as a polygonal continuous beam. In case no continuously bedded beams are available, the bedding can also be represented by individual radial and tangential springs or hinge-ended columns at the node points (Fig. 3-8).

Even at a time when complex calculation programmes are available, the non-linear bedded beam is still a useful model for the design of shallow tunnels in loose ground. Because of its relatively simple manageability, the processing time is short compared to other numerical processes. One disadvantage is the limited extent of the results, which are restricted to the internal moments, forces and deflections in the members and the subgrade reactions.

No further statements can be made, in particular concerning surface settlement. This can only be extrapolated from the deflection of the beam model and thus estimated.

3.5 Numerical methods

The working method of the analytical process is based on the exact solution of the differential equations that describe the relevant problem.

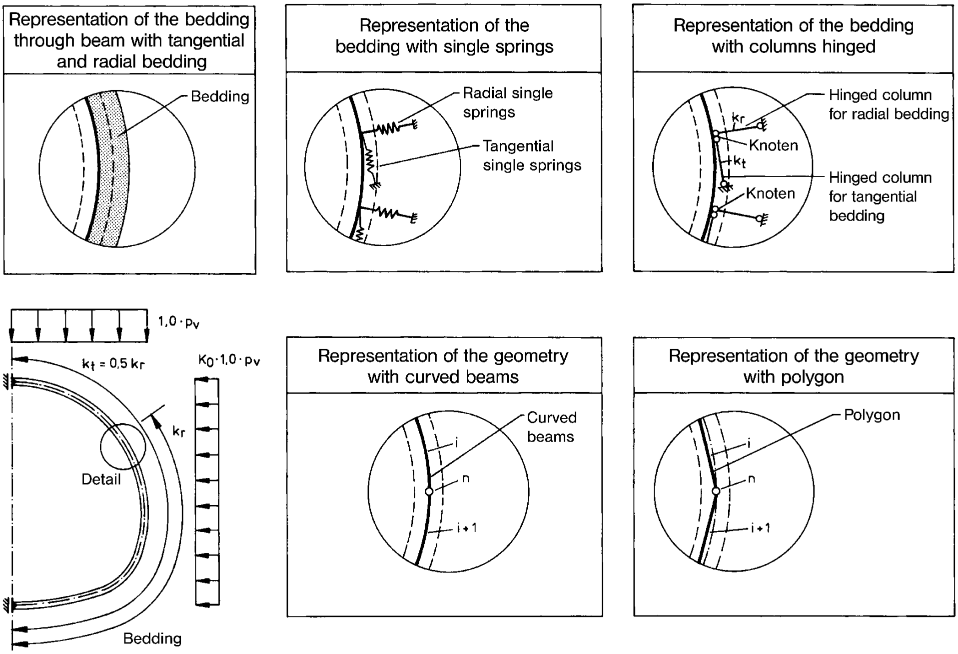


Figure 3-8 Possible discretisations for bedded non-linear beam calculations.

In the numerical methods, on the other hand, a sufficiently large calculation area is discretised in a suitable form, which means it is split into elements of a finite size – finite elements. This achieves the situation that the unknown functions (for example the deflection of the support), which could only be precisely determined analytically with an infinitely large number of parameters at every point in the area, are replaced by a finite number of unknowns. The linear and non-linear equation system resulting from the discretisation delivers an approximation solution of the differential equation, the precision of which depends on the discretisation [215].

Four numerical processes are used in geotechnical engineering:

– Finite Difference Method (FDM).
– Finite Element Method (FEM).
– Boundary Element Method (BEM).
– A combination of finite element and boundary element methods.

3.5.1 Finite Difference Method (FDM)

In the finite difference method, the (partial) derivatives in the differential equations at the points of the mesh, which result from the discretisation, are replaced by algebraic operations. This method can however only be used with difficulty when the boundaries of the calculation area are non-linear. The method therefore has no significant application in tunnelling.

3.5.2 Finite Element Method (FEM)

In contrast to the finite difference method, the function being sought is approximated by the approach of an assumed curve inside an element. This means the relationship between the nodes is no longer determined by the discretisation and any geometry is possible. For this reason, FEM has become well established in tunnelling.

A further advantage is that both plastic and time-dependent behaviour of ground and support can be simulated. Further details of application and the extensive literature can be found in Sections 3.6 and 3.7.

3.5.3 Boundary Element Method (BEM)

In the Boundary Element Method, not the entire space of the tunnel but just the boundaries of the area are discretised (Fig. 3-9). Such boundaries could be, for example, not only the tunnel perimeter but also the boundaries of geological regions or faults. The amount of discretisation and calculation is thus relatively low. The edge elements are based on fundamental analytic solutions, for which the relevant constraints have to be laid down. The results are indeed only directly applicable to the boundaries, but subsequent calculation can determine stresses and distortions for any point, since the curves are functionally known. The decisive disadvantage of the BEM is that only elastic material behaviour can be modelled [216].

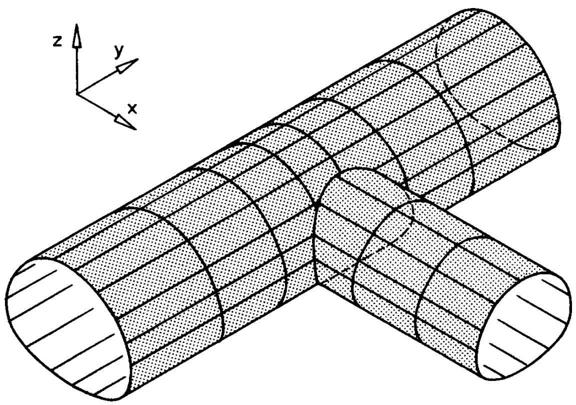


Figure 3-9 Discretisation example for the Boundary Element Method (REM).

3.5.4 Combination of finite element and boundary element methods

For tunnelling, a combination of finite element and boundary element methods is suitable because plastification of the rock mass can occur in the immediate vicinity of the tunnel. Outside this plastic zone, however, the rock mass remains elastic. The combination of the FEM with the BEM has the effect that the advantages of both these methods can be exploited and the weaknesses can be compensated. The inner, plastic zone is modelled using FEM and the outer, elastic layer using BEM. The advantage is that the discretisation and calculation time is relatively low despite the good representation of the real conditions (Fig. 3-10).

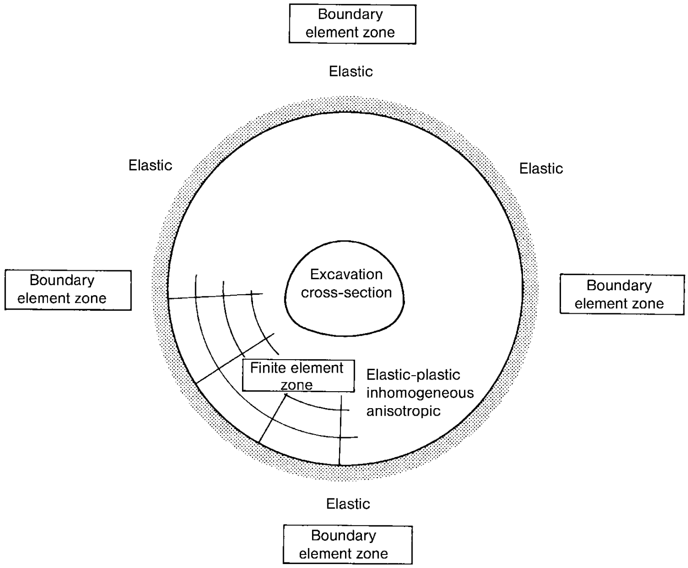


Figure 3-10 Discretisation example for the Boundary Element Method (REM).

Fig. 3-11 shows a typical application. The inner, geometrically complex zone is discretised with the finite element groups A, B and C. The rock mass into infinity is modelled by the boundary element group D.

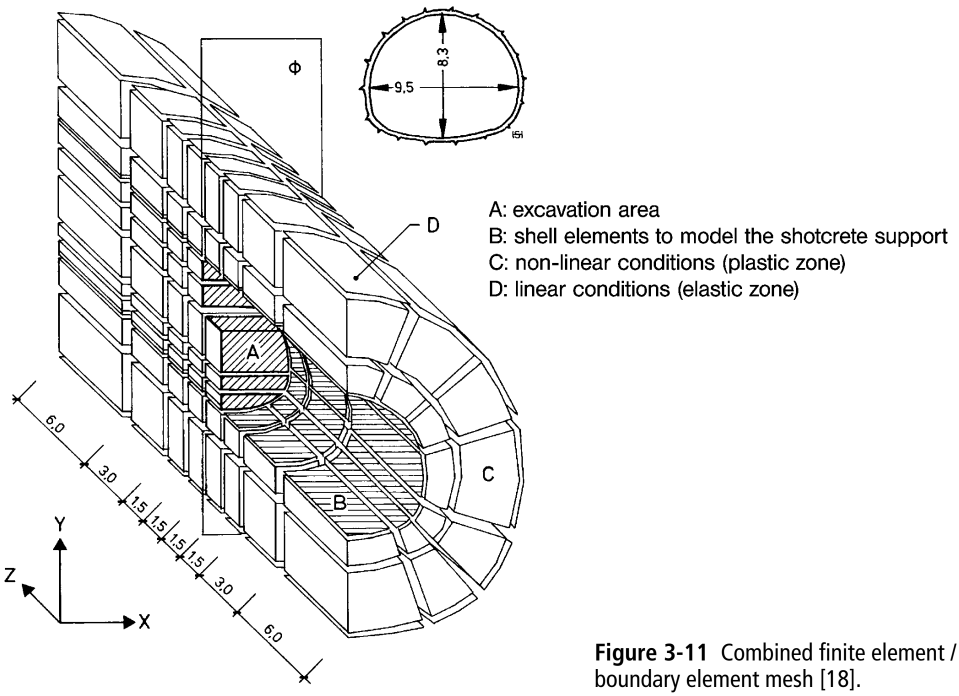


Figure 3-11 Combined finite element / boundary element mesh [18].

3.6 The application of the finite element method in tunnelling

Due to its flexible discretisation potential, the finite element method has become very significant in tunnelling. The first successful tests were for very shallow tunnels, a field in which the interaction of shotcrete and soil is still relatively insignificant. The next step was the consideration of the stress relaxation ahead of the drive for the simulation of three-dimensional load-bearing. Since it is not possible to make any statement about the magnitude of the stress relaxation in 2D models, even when the geomechanical parameters from the modulus of elasticity to the failure criteria are fully known, this was dealt with from the start by incorporating measurements of displacement. This process is still normal practice today.

As a result of the mathematical modelling of deep tunnels, the simulation of the interaction between support and ground and the simulation of the advance of the tunnel have now become particularly significant [51]. The basic procedure is not new and is described in the references [16, 64, 66, 139, 160].

There follows a description of three FEM applications based on [186].

3.6.1 "Step-by-Step" technique

The basic idea of this method is to simulate the construction process as the tunnel advances in many steps, which approximate to the procedure in practice.

The procedure for the step-by-step simulation of a tunnel drive is illustrated in Fig. 3-12 through the example of a top heading advance with an idealised cross-section. Starting from the primary state, in which the calculated section is only loaded by the self-weight of the rock mass (γ_F), the advance of the top heading is simulated. For the 1st construction state, it is assumed that the top heading is excavated in one step along a greater length and the shotcrete support is installed simultaneously. The support is not installed right up to the temporary face; there remains an unsupported length of tunnel, whose length equals the round length. The face is driven forward by this distance in the following working steps. The entire length of the top heading excavated and supported in the first step is selected according to the length stated above, after the three-dimensional influence of driving the advance has subsided to about 1.5 to 2 d.

The simulation is achieved by assigning the excavated element the stiffness 0, meaning that this element can no longer transfer stress.

3.6.2 Iteration process

This method makes the assumption that the alteration of the loading on the ground and the shotcrete lining repeats with each advance of the tunnel with the same construction process. The tunnel has an axis running parallel to the ground surface and uniform ground with time-dependent stress-strain behaviour in the direction of advance. The state of displacement and stress around the face, which no longer changes with further advance and is achieved by stepwise simulation of the tunnel advance with the step-by-step method, is calculated using an iteration procedure.

The basis of the iteration procedure is a calculation section, which moves in the direction of advance. The external dimensions of the calculation section are selected so that the stresses and displacements at the vertical boundaries do not significantly change due to the displacement of the face by one round length. The required length of the calculation section along the tunnel is shorter compared to the length necessary with the step-by-step technique, which is selected according to the number of successive construction states in the advance direction among other factors. This leads to a smaller number of element slices for the same discretisation in the plane, and thus considerably reduces the number of elements required in comparison to the step-by-step technique.

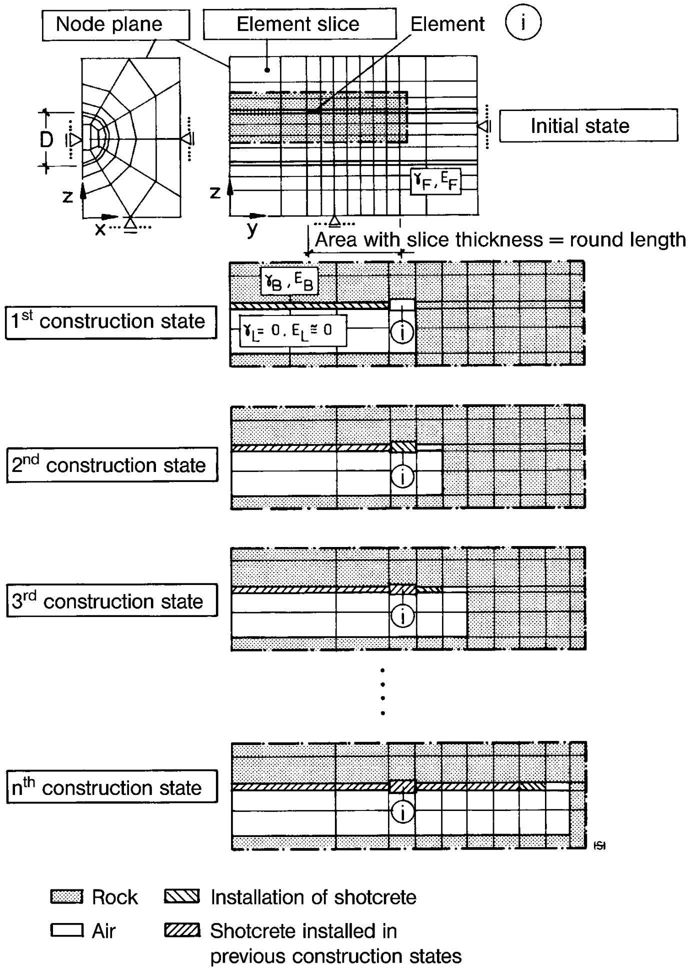


Figure 3-12 Simulation of a tunnel advance using the step-by-step technique [186].

The basic approach of this calculation procedure is illustrated in Fig. 3-13 using the example of the same idealised top heading advance already used in the section above. For the following explanation, the simplifying assumption is made that the slice thickness, as shown in Fig. 3-13, is equal to the round length and remains constant in the calculation section.

Starting from the primary state, and a following initial state, which corresponds to the 1st construction state as already described, the first calculation step is to simulate the advance of the top heading in an iterative calculation. In the course of the iterative calculation, which is illustrated in Fig. 3-13 through the 1st to mth iteration steps, the calculation section is moved forward by one round length in the direction of advance compared to the relevant previous state. In the process, the concrete element i-1, which lies behind the face in the first round supported with shotcrete, is installed into the already deformed system of the previous calculation step. The displacements that are used are equal to those of element i in the unsupported section of the top heading. In the course of the iterative calculation, the actual displacements and stresses, which occur in practice in the rock mass and the shotcrete layer with each advance step, are converged towards. In this way it is possible to determine the magnitude of the displacements due to the tunnel advance and the associated loading on the shotcrete layer.

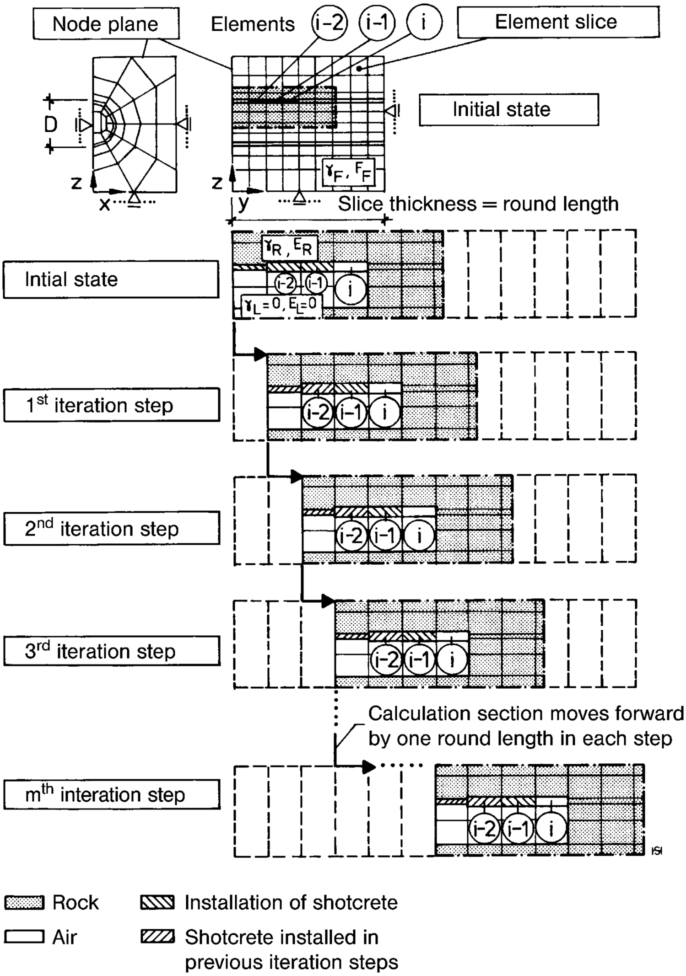


Figure 3-13 Simulation of a tunnel advance using the iteration process [186].

3.6.3 Simulation of uncoupled partial excavations

If a tunnel is driven with a number of partial excavations, for example a top heading advance (first partial drift) with subsequent excavation of the bench and invert (second partial excavation), then the stress and displacement state of the ground changes with the driving of the top heading. The advance of the bench and invert that follows the top heading occurs in a state that has changed from the original primary state, in which the ground was untouched. The main characteristic of this state is the cavity that is now present in the ground, which is wholly or partially supported with shotcrete. The loads, which were transferred through the area of the cavity before the advance of the top heading, are diverted around the shotcrete layer. This loads the shotcrete layer due to the displacements resulting from the advance of the top heading after the layer was applied. As a result of the subsequent excavation of bench and invert, the state of displacement and stress present in the rock mass and support is changed again.

For the calculation of the changed state of displacement and stress due to a further partial excavation, the stepwise simulation of the tunnel advance using the step-by-step technique is basically suitable. The partial excavation in this case is decoupled from the previous partial excavation. When two or more partial excavations are calculated, the location of the calculation section in the direction of advance should be selected so that each following partial advance lies outside the area affected by the face of the previous advance. In the discretisation of the calculation section, the excavation cross-section of each partial advance is to be modelled in the planes perpendicular to the advance direction. Different round lengths are normally considered for the element slices in the direction of advance. This leads to a size of FE meshes and thus to equation systems, which can often no longer be calculated at an economically justifiable expense. Should nonetheless many partial advances be calculated using the step-by-step technique, then this will only be possible with relatively coarsely discretised calculation sections with the various round lengths are considered in a simplified form.

The method for the simulation of a number of successive partial advances is based on the idea that the stresses and displacements determined for the preceding advance are taken as a basis for the initial state of the following partial advance. The displacement and stress state of a zone, which is no longer affected by the events at the face, are decisive. For the calculation of the individual partial advances, the step-by-step technique and the iteration process are equally suitable. This, however, only applies when the tunnel profile is constant and the ground is homogeneous in the area to be investigated and the overburden depth and the primary stress state in the ground do not change along the tunnel. Another precondition is that calculation sections with the same dimensions and discretisation in the plane perpendicular to the advance direction are selected for the investigation of the individual partial advances. In the discretisation, the partial excavation cross-section of each partial advance is to be considered. The discretisation is to be selected for each partial advance independent of the associated round length. For the dimensions of the calculation section, the remaining criteria stated for the step-by-step technique apply. Fig. 3-14 shows the discretisation for an already described calculation.

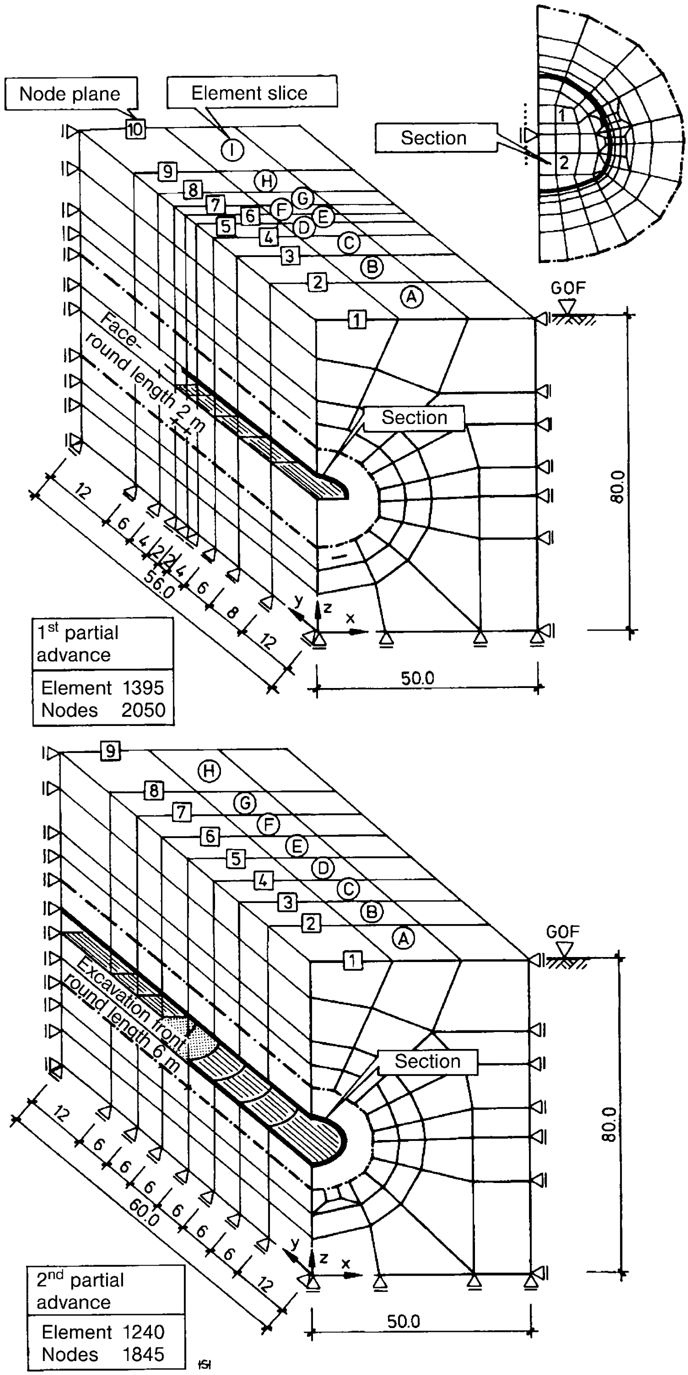


Figure 3-14 Calculation section: element mesh [186].

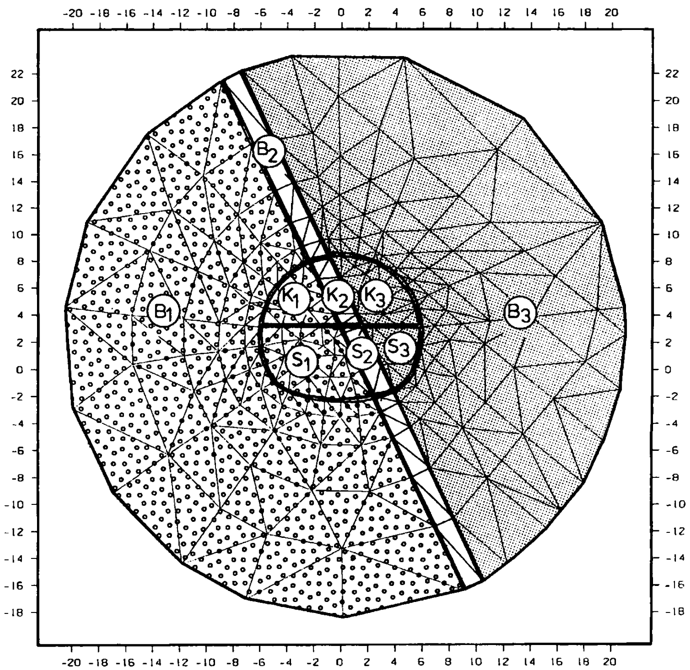
3.7 Special applications of the FEM in tunnelling

Two special applications are now described to make clear the usefulness of the FEM in tunnelling.

3.7.1 Modelling of deformation slots

Unfavourable geology, for a example heavily squeezing rock mass, can lead to problems with high deformation in deep tunnels in rock. For this reason, movement slots can be installed in the shotcrete layer parallel to the tunnel axis, which largely protect the layer from destruction by large displacements. This was practiced for the first time in the Tauern Tunnel. In the Inntal Tunnel, the process was practiced regularly and backed up with structural verifications.

The appropriate measures and their numerical modelling are reported in [236]. Fig. 3-15 shows the discretisation of the rock mass with unfavourable geology for tunnelling; Fig. 3-16 shows the modelling of the slots in the shotcrete layer.



$K_{1,2,3}$ Top heading with different rocks
$S_{1,2,3}$ Bench and invert with different rocks
$B_{1,2,3}$ Different rocks

Figure 3-15 Numerical model with definition of geological zones [236].

Until the slot is finally closed, no or only very slight normal force is transferred from shotcrete zone A to zone B. Slight transfer is caused by shotcrete residues in the slots or over the tunnel vault. Contact elements offer the capability to specify both a gap spacing *a* and an elastic stiffness in the gap. This means that force is only transferred through this element when the gap *a* has been exceeded. The working curve based on this assumption can be seen in Fig. 3-16.

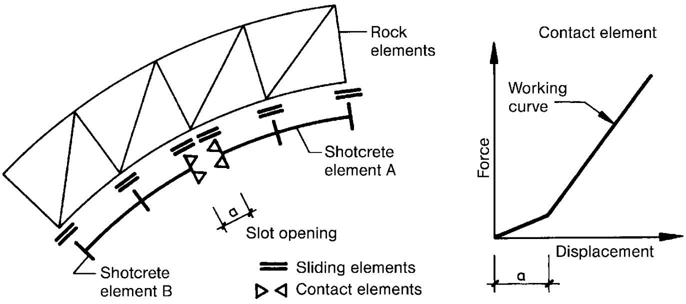


Figure 3-16 Numerical model of the movement slot [236].

A further important property is the sliding process between the shotcrete layer fixed to the rock mass by rock bolts and the rock mass. The sliding elements described here, which are also based on the theory of decoupled finite elements, can also describe this process very well by using a Coulomb failure criterion.

For the modelling of the shotcrete shell, a beam element is used, which delivers normal forces, moments and shear forces without subsequent integration of stresses.

3.7.2 Determination of the loosening of the rock mass from blasting

Another possible application of FEM in tunnelling is described in [237]. This is a calculation model intended to provide the first method of considering the changed load-bearing of the rock mass due to drilling and blasting.

The particular feature of this application is the fact that a dynamic loading has to be simulated, which propagates in waves in the continuum. For example the wavelength and the reflection of the waves at the edges have to be considered in the discretisation. Since, for example, reflection at the edge of the discretisation can influence the results of the calculation, elements were included at the edge to simulate viscous damping. Fig. 3-17 shows the modelling of a wedge cut, and Fig. 3-18 shows the entire FE mesh with the configured load function of the blasting effect and the stress curves as result, each plotted against time.

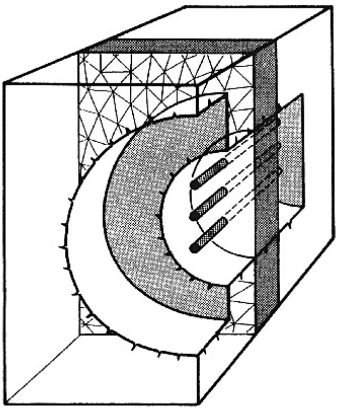


Figure 3-17 Wedge cut with FE model [237].

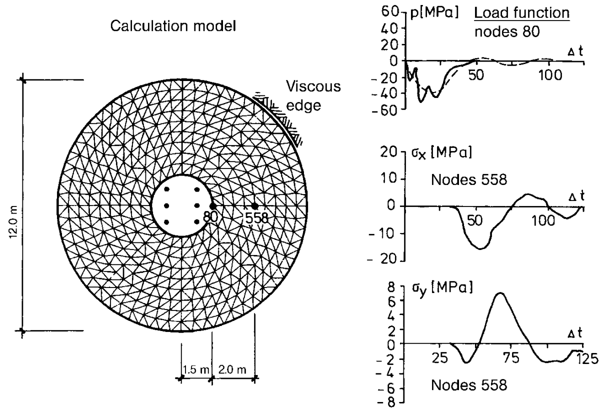


Figure 3-18 Stress curves with FE mesh [237].

3.8 Structural design

3.8.1 General principles

Despite many reservations about the idealisations and the model character of calculation models in tunnelling, structural safety analyses have to be performed. No tunnel is constructed today by just relying on the experience of a practical person. The design work has to include structural calculations based on geological investigations and forecasts, mostly combined with parameter studies, and these are useful and required for the design of the tunnel construction and also for construction planning. The chosen construction process and the specified construction are checked with measurements during the construction period and adapted if required. The possible dangers of recalculating and calculating to a predetermined result can however often be difficult to avoid. Absolute safety can neither be ensured in the original design nor in recalculation from measured data.

When selecting a calculation model for a tunnel structure, the person responsible for the verification of structural safety is faced with the complex problem of including all these considerations in a realistic model:

- The construction states for the selected construction process with its special excavation and support phases,
- The interaction of ground and support,
- The spatial and time-dependent alteration of the support with the different load-bearing functions, particularly at the face and
- The estimation of the loading on the ground and support in the excavation and support phases.

The recommendations of the DGEG [48] and the project-specific guidelines of the Deutsche Bahn AG [193] are included in tender documents and suggest the use of a calculation model, which normally takes account of the following factors in the determination of the forces and moments in a section:

- Primary stress state and bedding structure of the ground.
- Hydrological conditions in the construction state and completed state.
- The partial relaxation of the ground that occurs before the support measures become effective, which has the result that the temporary support is only loaded by part of the full ground pressure.
- Actions that occur later (for example in the completed state: traffic, rising water, frost, excavation, building).
- Additional loosening of the ground from the effects of deformation, especially in the crown and in some cases with time-dependent behaviour of the ground. These effects can normally only be estimated.

The design is performed according to the applicable regulations for reinforced concrete, steelwork etc. For the special features of tunnelling, the guideline 853 from DB Netz AG (version 01/06/2002) is quoted here:

Structural design and safety

Loading cases

(5) The following three loading cases are differentiated in underground tunnelling:

- *Loading case 1: Main loads and additional loads (without temporary loads during the construction period according to Clause 14) in the least favourable combination in each case,*
- *Loading case 2: Main loads and additional loads (including temporary loads during the construction period according to Clause 14) in the least favourable combination in each case,*
- *Loading case 3: Main loads and additional loads (without temporary loads during the construction period according to Clause 14) and special loads in the least favourable combination in each case.*

In the determination of the least favourable combination of loads, the range or variation of the individual loads is taken without safety factors. It should also be considered that individual loads do not act in certain cases or can reduce to almost zero. Particularly in the case of two-layer lining, the case should be investigated for the inner layer that no actions from the rock mass according to Clause 9 are present, unless this can be safely excluded.

Design of the lining

(18) The final lining is to be designed for loading cases 1, 2 and 3 (see Clause 5).

The temporary support is normally only to be designed for loading case 2, unless it is considered as load-bearing and thus part of the final lining.

The outer layer may only be considered for design purposes as part of the final lining if its durability is guaranteed.

Shield constructions are to be dealt with as a temporary support.

When pre-grouting is carried out before tunnelling below a railway line, the strength under vibration is to be verified according to ATV worksheet 161 (DVGW bulletin GW 312).

The design of the preliminary support and the final lining should be performed under consideration of the interpretation of the already available measurements.

Designing of the lining for loading case 2

(19) For short-term construction states lasting up to a few days in loading case 2, reductions of the safety factors of loading case 1 are permissible. The safety factors for concrete and reinforced concrete may be

- *reduced by 10 %, when the safety of rail traffic and other public safety are affected, and*
- *reduced by 20 %, when the safety of rail traffic and other public safety are not affected.*

For the determination of the permissible stresses in steel for the general verification of stresses, the minimum value of the yield limit according to the applicable quality standard may be reduced by the safety factor.

- *$\gamma = 1.5$; when the safety of the operational facilities of the DB network and other public safety are affected and*

$\gamma = 1.33$; when the safety of the operational facilities of the DB network and other public safety are not affected.

For the investigation of the structural stability of frost bodies intended to serve as assisting vaults for the temporary support of the ground during construction, the safety factors shall be specified in each individual case. The value shall be at least $\gamma = 2.0$.

Design of the lining for loading case 3

(20) The design of the final lining for loading case 3 is, unless differentiated safety concepts are used, to be performed using the following safety factors:

- *Concrete and reinforced concrete: $\gamma = 1.0$ and*
- *Steelwork: corresponding loading case HA of the DS 804.*

For grouted anchors, the permissible forces are to be considered according to the assumption of passive earth pressure according to DIN 4125.

Special note

(21) For the final lining, special verification should also be carried out with the following considerations:

- *Ground characteristics taking into account knowledge from former geotechnical measurements,*
- *Increased ground pressures with 1.5 times the calculation values of the regular loading cases, not exceeding the full overburden pressure,*
- *Self-weight of the lining and all normally installed building elements,*
- *Self weight of the permanent way and*
- *Point loads from overhead equipment.*

The outer layer may not be considered in this verification.

The moments and forces on the section determined using these considerations shall be used to check that the limit loading on the section according to DIN 1045 (safety factor $\gamma = 1.0$) is not exceeded. Any verification of water impermeability that may be required for the serviceability state is omitted for this. For this special verification, the displacements are also to be determined and disclosed.

3.8.2 Design method for steel fibre concrete tunnel linings

General. This is a new method of structural design, which takes into consideration the properties of the material steel fibre reinforced concrete [Volume I, Chapter 2].

Possible applications of steel fibre reinforced concrete in tunnelling. When sprayed as steel fibre shotcrete, it can be used as the tunnel advances as temporary support to the exposed ground. As an in-situ concrete inner lining, it can serve as a permanent construction to provide structural stability and serviceability for the entire lifetime of the tunnel. In shield tunnelling, single-layer, waterproof linings of steel fibre concrete are already being used and offer further potential for development.

From the point of view of construction process technology, the advantage of using steel fibre concrete as an alternative to conventionally reinforced concrete is the saving of the time required to fix the reinforcement. This can have a positive effect on the sequence of construction operations, which demands particular attention in tunnelling because the site is linear. The use of steel fibre concrete offers the opportunity to considerably shorten the time required.

Load-bearing properties of steel fibre concrete. Steel fibre concrete is a construction material, which is able to resist tension only to a limited extent. Its tensile strength is determined by the tension strength of the concrete matrix. In contrast to unreinforced concrete, the formation of macro-cracks under tension and bending tension loading do not lead to sudden failure. The steel fibres work to prevent the formation and widening of cracks; the influence on crack formation and crack propagation behaviour of concrete starts with the formation of micro-cracks and results in a finer distribution of the cracks in the structure with smaller crack widths.

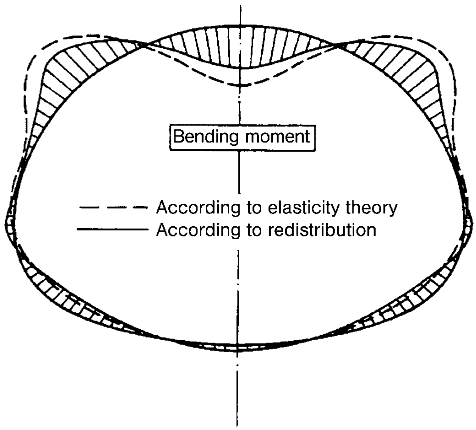


Figure 3-19 Bending moments and subgrade reactions with reduced bending stiffness.

The use of steel fibre concrete in tunnelling is limited from the structural point of view. The conditions, with high compression force acting on a tunnel lining combined with relatively low bending moments, are indeed basically suitable for steel fibre concrete, but the load-bearing capacity of steel fibre concrete reduces considerably with increasing eccentricity of the normal compression force. If the loading on a tunnel lining leads to reinforcement being required for structural reasons, it is normally difficult to verify structural stability for steel fibre concrete as an alternative.

Load-bearing behaviour of tunnel linings. Static systems for the structural analysis of tunnel linings under consideration of the ground-structure interaction have a high degree of static indeterminability. While a tunnel lining that is relatively stiff in bending compared to the ground bears most of the acting load itself, a lining that is weak in bending suffers more deformation, which activates a stronger subgrade reaction of the ground and thus reduces the share of load-bearing on the lining. In a lining that has been deformed by "weakening", a resulting loading form is established, for which the geometry of the structure forms approximately the line of thrust, meaning that bending moments are reduced and normal forces increase [28].

A construction material that is optimal for tunnel linings is thus characterised by the property of accepting high deformation without failing and resisting high normal forces. Qualitatively, this applies to steel fibre concrete. When the forces and moments in the section are determined through the use of the elasticity theory, however, these properties are not considered in the calculation. Only when crack formation is considered with the resulting loss of stiffness in the parts of the tunnel lining subjected to bending can the effect of redistribution of forces into less highly loaded parts of the structure and increased activation of the subgrade reaction be included in the calculation.

Fig. 3-19 shows an example of the qualitative effect of an assumption of reduced bending stiffness on the resulting moments in a two-track rail tunnel cross-section. The rotations resulting from the introduction of plastic hinges are damped by the increased activation of subgrade reaction by the ground around the tunnel. The result is a redistribution of the bending moments with a simultaneous increase of the subgrade reaction.

The weakening of the system in areas of high bending action is associated with the assumption of deformation, which has to be limited to a permissible degree. Steel fibre concrete is indeed a markedly ductile construction material in comparison to unreinforced concrete, but its plastic deformability cannot be assumed as limitless. Eurocode 2 [67] requires that processes that deviate from the elasticity theory to determine the forces and moments in the section of reinforced concrete structure also include a verification of the plastic rotations required to redistribute the bending moments. No standardised procedures for the verification of the plastic rotation capability of steel fibre concrete are yet available (Fig. 3-19).

Simplified verification procedure for shotcrete tunnel outer linings. In tunnelling practice, there is a verification procedure for tunnel outer linings of shotcrete, which approximately considers the mechanism of moment redistribution described above, and is based on the investigations of Schikora/Ostermeier [200]. The basis of this procedure is the calculation formula from DIN 1045 [53] for unreinforced concrete under eccentric compression loading (1).

$$N \leq \beta_R \cdot d / 2{,}1 \cdot (1 - 2 \cdot e / d)$$

N normal compression force
d thickness of the cross-section
β_R calculation value of compression strength according to DIN 1045
e eccentricity of the normal compression force

According to the modified verification procedure of Schikora/Ostermeier [200], bending moments are not necessary for the equilibrium of forces due to the static indeterminacy of a bedded tunnel lining. They can be interpreted as constraint loading and then do not require the application of the full factor of safety. Returning to an investigation for an underground railway cross-section in loose ground based on the FE method, taking into account the possible plastification of a shotcrete tunnel lining, the following calculation is recommended:

$$N \leq \beta_R \cdot d / 2{,}1 \cdot [1 - 2 \cdot (e_a + e) / (d \cdot 2{,}1)]$$

e_a additional eccentricity to take into account imperfections

The application of this simplified calculation formula for the verification of a tunnel outer lining assumes, like the application of the more precise non-linear procedure, the plastic rotation capability of the shotcrete or steel fibre shotcrete. Strictly speaking, this requires an appropriate verification.

Design basis for steel fibre concrete under eccentric compression loading. The bulletin "Steel fibre concrete" of the German concrete and construction technology association from 2001 [46] is valid as a design basis in Germany. This has superseded the formerly valid bulletin "Design basis for steel fibre concrete in tunnelling" (Fig. 3-20).

In accordance with the spacing concept, the design concept of the German bulletin is based on a stress-strain curve. The individual crack is not considered discretely, but the crack widths are interpreted as equivalent strains of the cracked zone. The following section will describe how the influence of an eccentric compression loading can be considered in the determination of the characteristic values of the stress-strain curve.

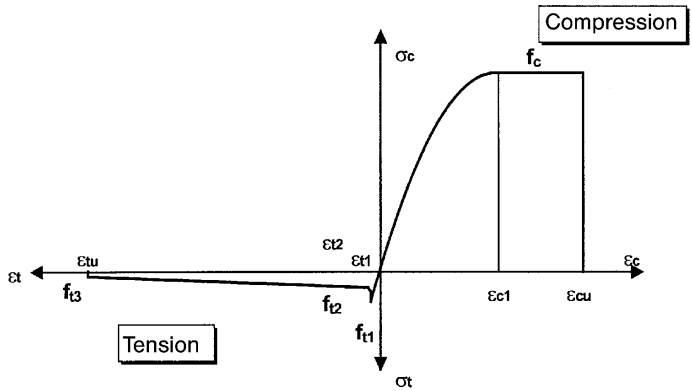


Figure 3-20 Stress-strain curve for the design of steel fibre concrete.

The M/N test rig. The Moment-Normal force test rig of the Ruhr University, Bochum (M/N test rig) (Fig. 3-21) has been developed in order to perform experimental investigations of the load-bearing, deformation and cracking behaviour of steel fibre concrete under the loading combination of bending moment and normal compression force that is typical for tunnel linings.

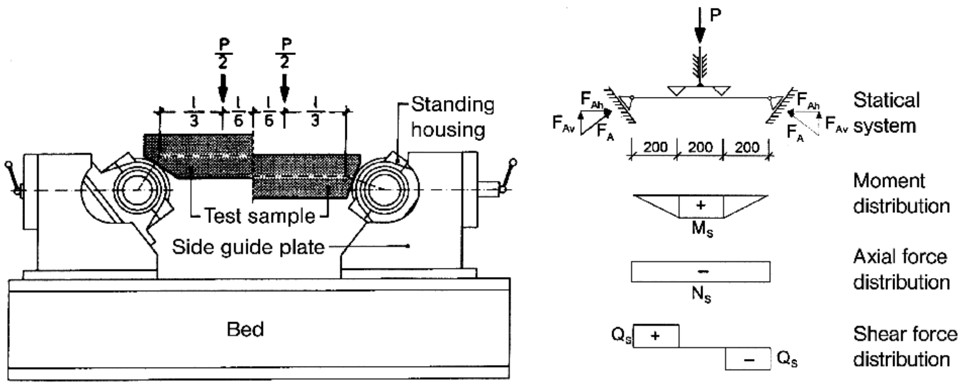


Figure 3-21 Diagram and structural system of the M/N test rig.

The construction of the M/N test rig enables the performance of bending tension tests on beams based on the four-point bending test. In contrast to the standardised test rig, the reaction forces in the M/N test rig are not introduced vertically at the underside of the beam but orthogonally to the sides of the beam ends, which have the appropriate angles. This means that the test beam is also subjected to compression loading. Different eccentricities can be configured by varying the angle.

Using the M/N test rig, numerous investigations of the load-bearing, deformation and cracking behaviour of steel fibre concrete under the loading conditions of a tunnel lining have been carried out at Ruhr University, Bochum, of which some results are described here.

Cracking behaviour of steel fibre concrete. While under pure bending ($e/d = \infty$) it is normal for just one crack to form, multiple crack formation can be observed under eccentric compression loading with a normal fibre content. Fig. 3-22 shows the relationships determined using the M/N test rig between the crack width w and the strain of the tension side of the beam for various steel fibre types and contents. As can be seen clearly, the crack widths of the individual cracks in case of multiple crack formation under eccentric compression loading are less than the crack widths in the case of simple crack formation under pure bending (Fig. 3-22).

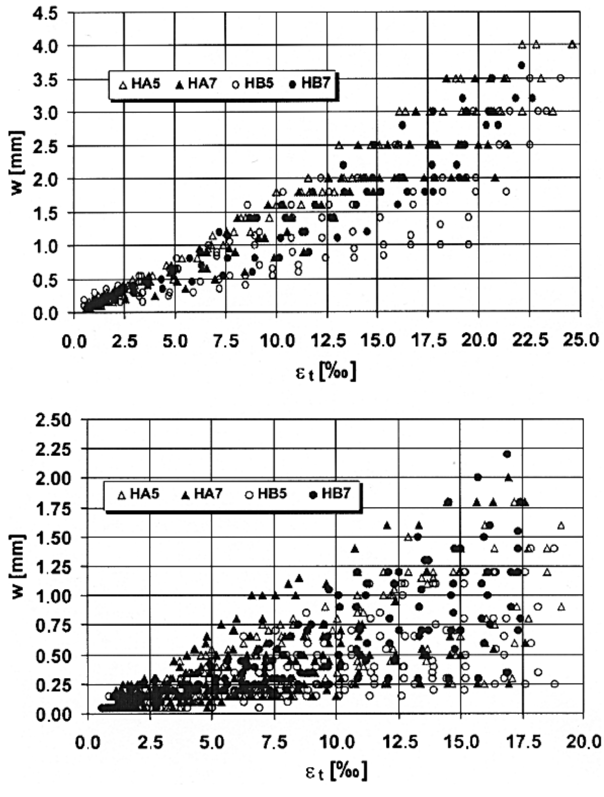


Figure 3-22 Crack width w depending on the strain at the tension side of the beam, for $e/d = \infty$ (top) and $e/d = 0.5$ (bottom).

The introduction of a limit strain into the stress-strain curve for the design of steel fibre concrete is intended to limit the crack width at the limit state of load-bearing capacity. Since, as shown, the equivalent strain of the cracked zone is distributed over numerous cracks, it is justified to permit higher limit strains under eccentric compression loading. Similar dependencies can be derived for the strain parameter ε_{t2}. Fig. 3-23 shows the practical implementation of these qualitative relationships. The given values in this case correspond to the lowest strains, at which a crack width of 2.0 mm was reached in the tests that were performed.

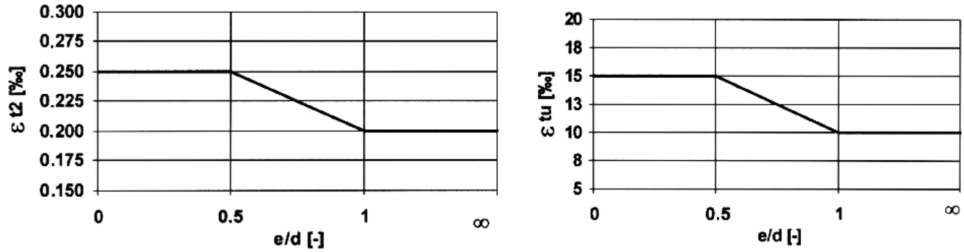


Figure 3-23 Consideration of multiple crack formation in the stress-strain curve.

Failure under compression loading. The failure of concrete under uniaxial compression loading can be traced back to the formation of transverse tension cracks and the resulting destruction of the internal structure. Steel fibres act against this crack formation from the start of the formation of transverse tension cracks. While this has relatively little influence on the compression loading that can be resisted, the compression strain on reaching the maximum load and the compression strain at failure can be increased compared to normal concrete.

This can be used in the design of steel fibre concrete by setting higher strain values for the parameters ε_{c1} and ε_{cu} than given in DIN 1045 into the stress-strain curve according to Fig. 3-23. In the literature, values of up to 2.6‰ for ε_{c1} and 4.85‰ for ε_{cu} can be found (see [169]).

Rotation capacity of steel fibre shotcrete as the precondition for the application of a non-linear procedure. The influence of the eccentricity of the normal force on crack formation is reflected in the rotation capacity of steel fibre concrete. Large rotation angles are reached under small eccentricities. Fig. 3-24 shows a summary of the results of the tests carried out. As was already the case with the equivalent strain, the reaching of a crack width of 2.0 mm also applies here as a limit criterion.

If the eccentricity is reduced further, as soon as the cross-section is in a completely compressed state, the rotation capacity reduces again (Θ_{pl} = 0 for axial compression loading). Since eccentricities of *e/d* < 0.5 are however not practical with the M/N test rig, the unloading part of the relationship was calculated (Fig. 3-24).

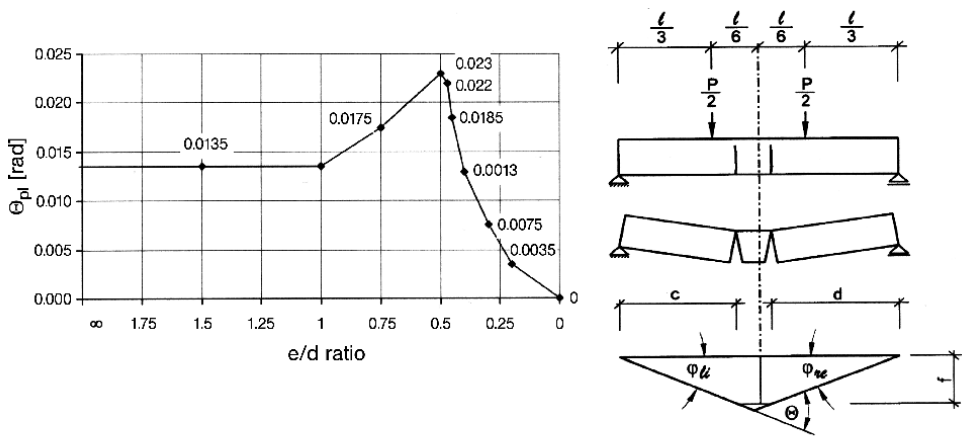


Figure 3-24 Plastic rotation angle depending on the relative eccentricity *e/d*.

3.8.3 Conventionally reinforced shotcrete versus steel fibre shotcrete

Differences in the production process

The production process of a shotcrete layer reinforced with two layers of mesh consists of the following working steps (ignoring further possible support elements like spiles, pipe screen, top heading foot piles):

1. Sealing of the exposed sides (first shotcrete layer).
2. Installation of support arches.
3. Fixing of the first layer of mesh.
4. Spraying of the arch and the second layer of shotcrete.
5. Drilling and setting of the rock bolts.
6. Installation of the second layer of mesh.
7. Completion of the shotcrete layer.

In contrast, the process to produce a steel fibre shotcrete layer consists of the following working steps:

1. Sealing of the exposed sides (first shotcrete layer).
2. Installation of support arches.
3. Spraying of the arch and the second layer of steel fibre shotcrete.
4. Drilling and setting of the rock bolts.
5. Completion of the steel fibre shotcrete layer.

The installation of the layers of mesh is a time-consuming manual activity, which can scarcely be automated. It should also be considered that at least the first layer of mesh is normally fixed in the first field next to the face, which is the immediate danger zone.

When steel fibre shotcrete is used, mesh does not have to be fixed, leading to an effective reduction of the time taken to install support and thus the achievement of altogether faster advance rates. If a shotcrete manipulator is used, only the setting of the arches demands that the miners work in the immediate danger area at the face.

Quality differences. The reinforcing mesh represents an obstruction of the process of applying and compacting the shotcrete. Aggregate grains bounce from the steel and increase the production costs as rebound. Voids can form in the shotcrete structure behind the reinforcing bars, called the spraying shadow, and vibration of the mesh leads to a danger of loosening and impairs the bonding between layers of shotcrete. Altogether, the resulting inhomogeneous shotcrete application leads to inconsistent strengths. Another negative effect is the water-permeability, which is particularly significant for single-layer applications [147].

When steel fibre shotcrete is used, these diminishing effects on quality and strength no longer apply. The danger of spraying shadows only applies to spraying around the arches. If the nozzle is optimally guided, however, this can be minimised or even completely prevented.

Finally, occurrence of loosening of the layer due to vibration from blasting should also be mentioned. Mesh reinforcement leads to stronger connection between loosened and broken areas of the shotcrete layer. This means that damaged areas in the vault may not be noticed and are then sprayed over with the application of the next layer.

When the layer consists of steel fibre shotcrete, damaged areas are more easily recognised or come loose independently after blasting and can be repaired with the application of the next layer.

As a conclusion, it can be maintained that tunnel outer linings of steel fibre shotcrete have a more homogeneous structure, less defects in the form of spraying shadows, a better bond between the individual layers of shotcrete and show altogether better load-bearing capacity and permeability than mesh-reinforced shotcrete.

Comparison of the load-bearing capacity in bending. In order to compare the load-bearing capacity in bending of mesh-reinforced and steel fibre shotcrete, a parameter study was carried out for a 25 cm thick shotcrete cross-section of concrete grade B25 to develop moment-curvature curves (M-κ curves) for various levels of the normal compression force N. In order to describe the relevant level of normal force, the term load factor is used below to describe the relationship between the compression force under consideration and the permissible compression force under axial loading.

For the mesh-reinforced shotcrete, two layers of reinforcement (each Q188) and the parabola-rectangle diagram from DIN 1045 [53] were assumed for the production of the M-κ curves.

For steel fibre shotcrete A, a ratio of strength parameters of $f_c/f_{t1}/f_{t2}/f_{t3}$ = 1/0.1/0.03/0.015 was assumed. In deviation from DIN 1045, the compression strain parameters were set at $\varepsilon_{c1} = -2.2$ ‰ and $\varepsilon_{cu} = -4.5$ ‰. In addition, a 5 % higher compression strength was assumed for steel fibre shotcrete B in order to quantitatively consider the qualitative influential factors listed in Section 4.2 (Fig. 3-25).

Fig. 3-25 shows an example of the M-κ curves determined for load factors of 25 % and 50 % normal compression force. It can be clearly recognised that much higher curvatures of the cross-section are reached for steel fibre shotcrete at a load factor of 25 % than with mesh-reinforced shotcrete. In the latter case, the bending moment that can be resisted is about 7 % higher. Due to the small height of the compression zone when the load factor of the normal force is low, scarcely any difference can be determined between steel fibre shotcretes A and B.

At a load factor of the normal force of 50 %, the bending moment that can be resisted by the mesh-reinforced shotcrete is still about 2 % higher than that of steel fibre shotcrete A. Compared to steel fibre shotcrete B, however, the mesh-reinforced shotcrete shows about 2 % less bending moment. Analogously to the load factor of 25 %, the steel fibre shotcretes reach greater curvatures.

Fig. 3-26 shows as a summary of the parameter study the relationship of the bending moment capacity between the mesh-reinforced shotcrete and the steel fibre shotcrete B depending on the load factor of the permissible compression force. While at lower normal force load factors, the mesh-reinforced cross-section can resist higher bending moments, the steel fibre shotcrete has the advantage at higher load factors (Fig. 3-26).

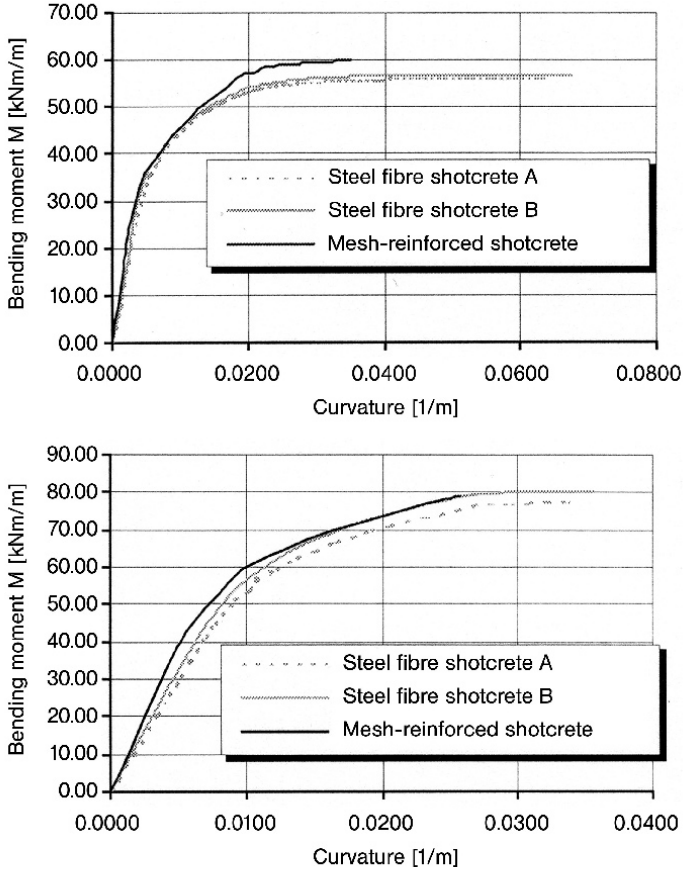


Figure 3-25 *M*-κ curves for a load factor of the normal compression force.

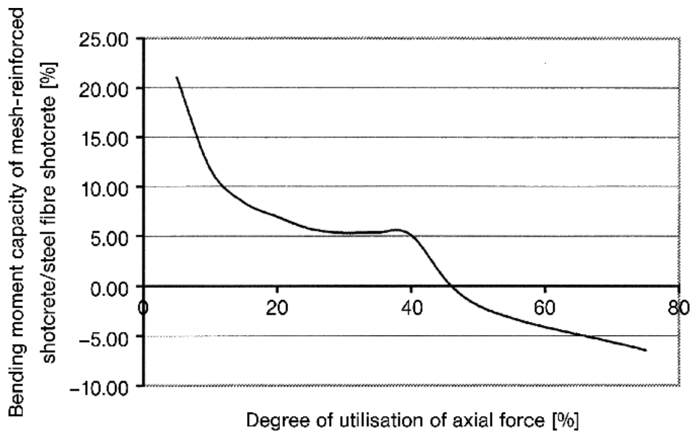


Figure 3-26 Comparison of the moment capacities depending on the load factor.

The results of the calculation show that greater curvatures of the cross-section were determined for the steel fibre shotcrete than for mesh-reinforced shotcrete in every case. Related to the overall structure, this means that higher angles of rotation can be reached in cracked zones and thus higher moment redistribution can be achieved. In summary, a tunnel lining of steel fibre shotcrete is thus capable of resisting more load.

Summary. Due to its properties of following bending deformation without fracture and simultaneously resisting high normal forces, steel fibre shotcrete is generally suitable as a material for tunnel linings. If the redistribution of bending moments due to crack formation and the increased activation of subgrade reaction are to be useful for calculations, the required plastic rotation should be verified. The experimental investigations carried out at the Ruhr-University, Bochum with eccentrically loaded steel fibre concrete beams show how steel fibre concrete can be designed and the verification of rotation capability can be performed for tunnel linings.

A qualitative comparison of mesh-reinforced shotcrete and outer tunnel linings of steel fibre shotcrete shows that the latter has a more homogeneous internal structure, less defects in the form of spraying shadows, a better bond between the individual layers of shotcrete and the altogether higher load-bearing capacity and water permeability can be achieved.

The quantitative comparison of calculated M-κ curves makes clear that the moment capacity depends on the magnitude of the normal force. Steel fibre shotcrete can show a higher moment capacity then mesh-reinforced shotcrete at higher load factors of normal force. It has also been demonstrated that greater curvatures of the cross-section can be achieved with steel fibre shotcrete. Related to the overall structure, this means that higher angles of rotation can be reached in cracked zones and thus higher moment redistribution can be achieved.

4 Measurement for monitoring, probing and recording evidence

4.1 General

In modern tunnelling, measurement is an important part of the verification of structural safety. Above all, measurements to monitor the stability parameters (deformation and stress) and comparison of the measured data with the results of the design calculations lead to technically and economically more successful tunnel construction. The programme of measurements must be laid out so that the assumptions made for the structural analysis and its results can be checked.

Fig. 4-1 shows a flow chart of the tasks of geotechnical measurement technology in the course of a project and the interaction between design, construction and measurement on a tunnel project (see also Fig. 3-5).

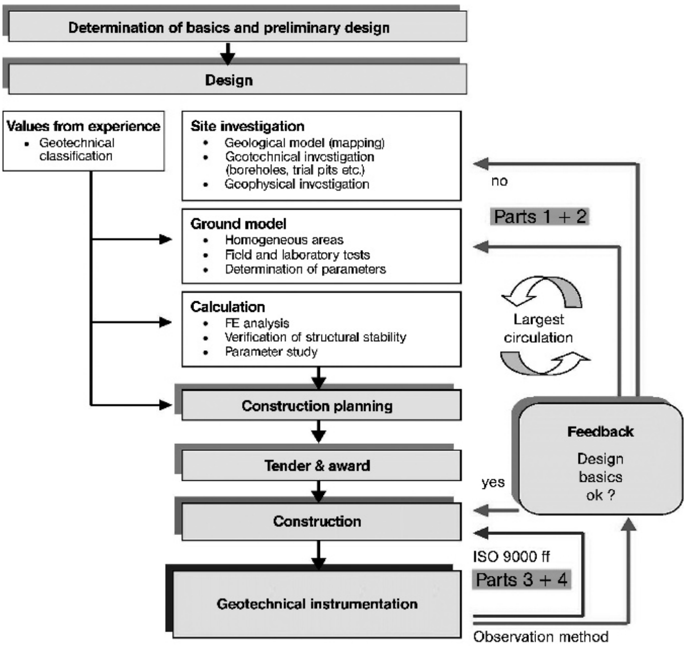


Figure 4-1 Application of geotechnical measurement technology for tunnel projects [25].

Measurements are generally made in and around the tunnel before, during and after the construction period in order to check the stability of a rock cavity and adjacent structures, for the final decision of dimensions and to check the effectiveness of the chosen support ands lining measures. By comparing the results of measurements with those of the design calculations, conclusions can be reached about the suitability of the calculation procedure used, and comparative calculations can also be performed to correct the rock mass parameters used as a basis for calculation. Calculations based on the improved rock mechanical parameters can then be used to check whether the support is sufficiently dimensioned or additional measures may be necessary. Measurements of stress and displacement are generally essential in the construction of rock cavities because they offer the only currently available method of checking the results of the investigation of structural stability performed at the design stage and thus making a reliable statement about structural safety.

The planning of a measurement programme has to consider the preliminary investigations in geological engineering and rock mechanics, the design calculations and the construction process. The type of instruments, construction schedule and any resulting obstruction of construction also have to be considered. It is generally true that the measurement programme has to be planned for each project and there can be no standardisation.

The extent of the measurement programme and the layout of the monitoring cross-sections are determined by the requirement that the measured data should deliver geotechnical parameters that can be regarded as representative and can be extrapolated for larger areas. In fault zones with an unfavourable relation to the tunnel and when impairment of the structural safety of existing buildings is a risk, additional targeted measurements are undertaken. Advance probing should also be carried out to enable better estimation of the geology along the tunnel alignment.

Instrumentation should be installed as early as possible to enable the observation and evaluation of deformation tendencies caused by tunnelling from the start. If this is not the case, the geological conditions can be interpreted incorrectly and the overall behaviour of the rock mass would not be recorded, particularly in case of very high initial deformation.

The measurement frequency should be appropriate for the rate of deformation or loading. Monitoring should continue after the completion of excavation until the deformation rate has finally tended to zero. The measured data is then interpreted in combination with the design calculations and supplementary investigations.

In tunnels driven by shield machines, the monitoring requirements are different due to the immediate stiffness of a segment lining and the high degree of mechanisation. These special requirements are described in more detail in Section 4.2.4 of this chapter.

4.2 Measurement programme

4.2.1 General

Monitoring of the displacement and force parameters of the rock mass and the support carried out during the driving of a tunnel have the following essential objectives:

- To obtain information about the actual behaviour of the rock mass under the specific excavation and support methods being used.
- Early detection of unforeseen deformation of the rock mass in combination with the support.

– To obtain measurement data including changes with place and time to evaluate the structural safety.
– To check the assumptions of design calculations.
– To assist decision-making about the use of individual support measures in relation to their effectiveness and cost-effectiveness.
– To make decisions about the design of the inner lining.
– Observation of long-term behaviour.

4.2.2 Measurements of construction states

During the construction state, measurements can be undertaken at standard monitoring sections (Fig. 4-2), principal monitoring sections (Fig. 4-3) and on the surface (Fig. 4-4).

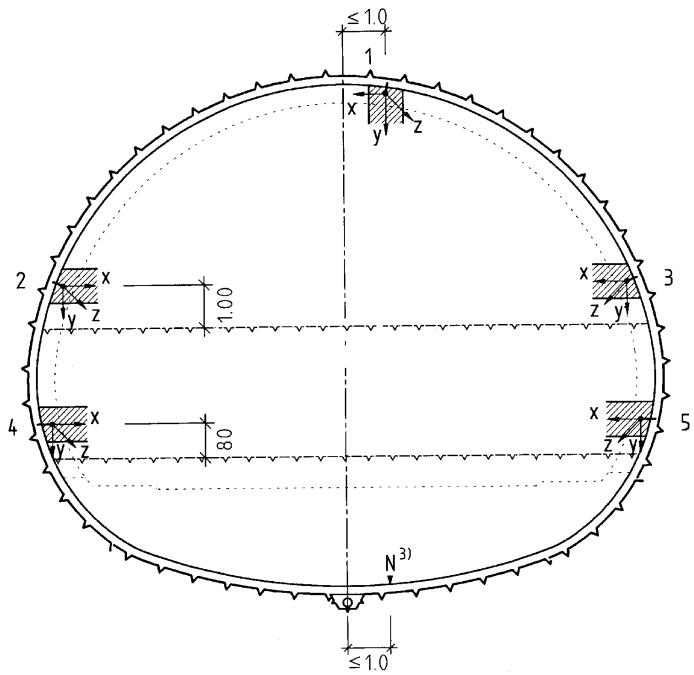


Figure 4-2 Standard monitoring section with measurements of deflection, with extensometers 1 to 5 and in special cases levelling at N.

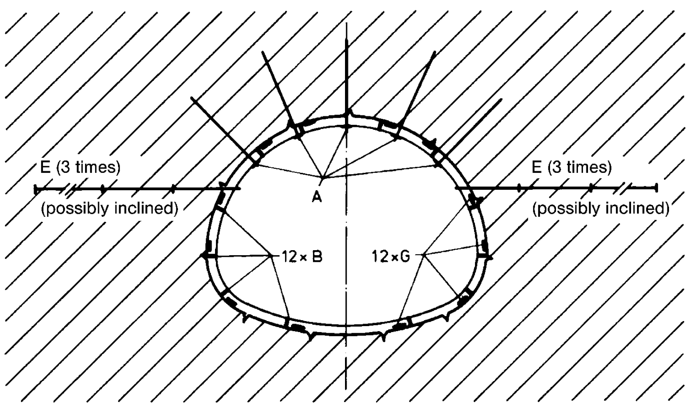


Figure 4-3 Principal monitoring section for the outer support (A anchor extensometer plate, B concrete pressure cells, G pressure cells, E extensometers).

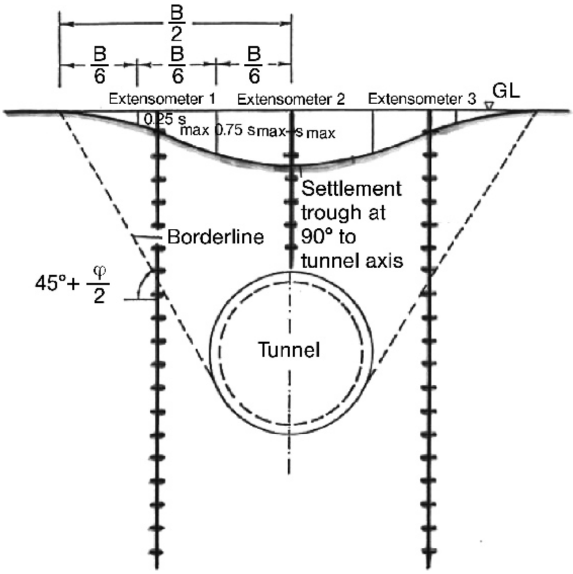


Figure 4-4 Example of surface measurement with mobile extensometers [25].

K convergence measurement
N levelling of the crown N_F, top heading foot points N_K, bench foot points N_{St} and the invert N_S
E extensometer measurement
G pressure cell to measure loading from the rock mass on the shotcrete layer
D pressure cell to measure the pressure between inner and outer support layers
B concrete pressure cell to determine the ring forces in the shotcrete layer
A anchor extensometer plate

The extent of measurements in the specification is determined as an estimate and then laid down according to the conditions encountered during the construction period. This applies not only to extent but also type and distribution. When changes are indicated, these should be discussed promptly with the parties involved and introduced into the construction schedule.

4.2.2.1 Standard monitoring section

Measurements are generally made daily (Fig. 4-2) as the tunnel is excavated and serve to evaluate the structural stability of the temporary support. Simple monitoring sections serve to monitor the local deformation of the tunnel lining. They can be used to determine displacement rates and the magnitude of the final displacement. The results should be extrapolated to determine whether a confirmation can be expected of the relevant structural verification, the scope of which was based on the last principal monitoring section and the geological conditions encountered in the meantime. At each monitoring section, the absolute position of the survey points is determined in all three directions. The displacement is determined from the difference between measurements on different days. In zones of rock susceptible to swelling, levelling points are also provided in the invert. Anchor measurements may also be necessary.

Spacing. The following spacings of the simple monitoring sections can be seen as a rule of thumb and altered to suit specific local conditions:

Stable rock $a \approx 30$ m

Unstable rock $a < 20$ m

Squeezing rock $a < 10$ m

In special cases, for example when unsymmetrical loading can be expected or the displacements are too high, additional instruments or extra sections may be necessary (Fig. 4-2).

Location and extent of displacement measurements. The spatial position of selected points on the outer support layer is determined absolutely in three dimensions by optical surveying. These measurements can be used to derive settlements, convergences, divergences and displacements along the tunnel.

Survey points are installed at a distance of less than 1 m from the face before the next round and surveyed (zero position). Subsequently, the points are normally surveyed at least once a day for the first few days. Near fault zones or in the event of heavy deformation, the interval is shortened. When a number of successive measurements show decreasing rates of deformation, the measurement interval can be lengthened in agreement with the site supervision.

The vertical movements of the points at the feet of the top heading cannot be directly surveyed since the points have to be about 0.8 to 1.0 m above the invert in order to be recorded by instruments. If large displacements occur at the feet of the top heading, it is therefore often better to measure the force at the foot of the top heading with load cells.

Extent of anchor measurements. The forces in anchors are recorded at regular spacings of 100 to 200 m. These measurements should not be confused with the separate requirement to check the load-bearing capacity of anchors.

4.2.2.2 Principal monitoring sections

The main purpose of principal monitoring sections (Fig. 4-3) is to check the input data for the design calculations (rock mass parameters and assumed loadings) and the suitability of the calculation model, and if appropriate quantitatively determine any necessary corrections. It should be possible from this data to derive the loading on the temporary support and the resulting displacements and stresses in the surrounding rock mass, globally and in their actual magnitude. Principal monitoring sections should thus be provided at the start of the scope of applicability of each structural verification. It is possible to check the measurements by investigating the compatibility of measured loadings, displacements and stresses.

Measurements. The following individual measurements are normally carried out:

- Horizontal and diagonal convergences.
- Settlement of the crown; for appropriate cases of bench excavation, also the settlement of the foot points.
- Displacement of the surrounding rock mass.
- Radial pressures on the outer support layer.
- Stress in the outer support layer and the steel arches.
- Loading of the anchors.

In addition to the standard monitoring sections, at least two principal monitoring sections should be provided in every tunnel as shown in Fig. 4-3. It may also be appropriate to provide surface monitoring sections at the same location if it can be expected that deformation could extend to the surface. As soon as the working sequence has become established after driving the first 50 to 100 m of tunnel, the first principal monitoring section should be set up as near as possible to the round. The second principal monitoring section then serves to check and interpret data from the first. In longer tunnels, or when the geological conditions are very variable, for example in fault zones, the provision of a number of principal monitoring sections is to be recommended. The data from principal monitoring sections also provides the basis for dimensioning and deciding when to install the inner lining.

Invert heave monitoring section. In areas with clay minerals susceptible to swelling, invert heave is also measured. Sliding micrometers are only provided in special cases. If a number of these measurements are also performed in the inner lining, they are generally assigned to the same monitoring station.

Monitoring section to evaluate the time to install the inner lining. In sections of tunnel where the inner lining is installed although the residual deflection rate is still above 2 mm/month, internal monitoring sections are installed at a spacing of at least 500 m at the same locations as the external monitoring sections. Measurements at the internal measurement section are undertaken at least once per month.

4.2.2.3 Surface measurements

Surface measurements are necessary for shallow tunnels, particularly when there are buildings in the area affected by the tunnel. This normally entails precision levelling to observe the behaviour of settlement with time, supplemented when necessary with extensometer or sliding micrometer measurements to determine the movement vectors. All measuring equipment should be set a sufficient time before the arrival of the tunnelling works. Measurements are generally continued until a month after the completion of the invert slab or invert arch. Such measurements are specified according to the local conditions.

4.2.2.4 Basic rules for implementation and evaluation

The installation of measuring instruments and data loggers should be undertaken with great care in order to avoid damage. Projecting sensors should be provided with protection against blasting and vehicles. Data should be recorded at all measurement locations as soon as they are installed and for as long as they are accessible. The reading intervals should be adapted to suit the geological conditions, for example starting daily, then every few days and finally one measurement per month, with shorter intervals if required.

Operational details. There are various procedures for purchase, installation and logging:

Instruments are installed by the contractor or by an appointed specialist. The contractor also provides auxiliary equipment like ladders, lifting equipment, lighting and transport. All measurements are undertaken by the contractor. The measurement programme specified in the contract serves as a guideline for the instrumentation and its arrangement and installation as well as the reading of the data. The extent of measurements to be undertaken is specified by the employer or his appointed experts with the agreement of the structural engineer and the construction company.

Measurement procedure. Convergence should be measured with an accuracy of at least ± 0.2 mm. Levelling in the tunnel should initially be performed frequently in order to detect spreads. The accuracy should be at least ± 2.5 mm.

The individual lengths of the extensometer rods are agreed with the employer. These should be selected particularly to record length changes near the cavity.

Pressure cells should be of types, which can be shown to have already been used successfully, and their accuracy should be ± 5 % of the expected maximum reading. Concrete stress sensors should be directly readable.

Anchor forces should be measured with an accuracy of ± 5 % of the expected maximum reading. The instruments should be directly readable.

Measuring instruments and targets to be installed in the tunnel should be placed as soon as possible after excavation. Targets should be installed and protected so that they are not damaged or made unusable by construction operations. The zero measurement of the target should be surveyed immediately after its installation and should be repeated at least once.

Evaluation of measured data and check calculations. Measurement data should be evaluated within 24 hours of reading, presented in an updated graphic together with the tunnelling data and made available to the site supervision. Evidently unusual results, on the other hand, should be reported to the employer immediately. The measurement data should be made available at regular meetings in a clear form. The meaning of the data regarding excavation process, support measured and structural verifications is to be interpreted. In special cases, written reports can be appropriate.

Check calculations should be performed for the principal monitoring sections to record the local rock mass conditions and the actual construction process as realistically as possible (also without safety factors). The interpretation also includes calculations with parameter variation for the most important factors. Check calculations should be tested in advance and prepared so that calculation can start straight after the receipt of the measured data. A detailed presentation should be handed in at the latest 14 days after the arrival of the data.

The contractor should propose the consequences for the tunnel drive and support measures of the data from the individual monitoring sections. After the completion of the tunnel drive, all data should be summarised in an extensive documentation and delivered to the employer.

4.2.3 Measurement of the final state

The measurements of the final state that are now described are intended for cases where the continuation of monitoring seems sensible after the completion of the inner lining. They serve for long-term monitoring and the evaluation of structural stability.

4.2.3.1 Measurement programme

The measurements shown in Fig. 4-5 can be performed at a monitoring section. In addition, the pressure cells G for the temporary support, as shown in Fig. 4-3, that are installed in the same section for the temporary support should be kept in operation if possible despite concreting of the inner lining in order to gain further valuable data. At least two monitoring sections as shown in Fig. 4-5 should be provided in every tunnel. The locations of principal monitoring sections used during the construction phase are generally still suitable as monitoring stations. Measurements in the completed state should generally be read at ever increasing intervals until it is clear that deformation has fully subsided. It can be assumed that instruments are read at least once a month during the construction period. The same requirements for the measuring instruments apply as for the construction phase.

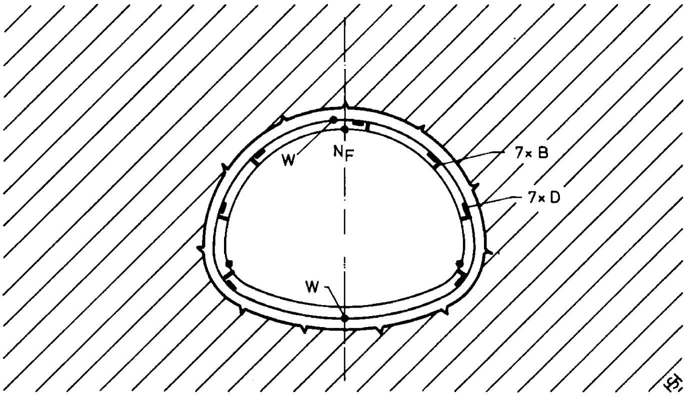


Figure 4-5 Principal monitoring section for the inner lining (W = piezometer, D = pressure cell). Additionally: continuation of reading in the pressure cells in the outer support layer at the same monitoring section (see Fig. 4-3).

4.2.3.2 Evaluation

The measured data is entered in a continuously updated graphical display, partly also as a continuation of records from the construction state. Clearly unusual events should be reported to the employer and his consultants immediately. The measured data is to be presented in a clear form and interpreted at regular intervals. Readings should also be compared to the original structural calculations. This comparison should be extensively interpreted in writing.

Measured data, the associated comparative calculations and interpretations should be summarised in a detailed report after the completion of measurements and delivered to the employer.

4.2.4 Special features of shield drives

The measurement programme for a shield-driven tunnel is intended on the one hand to record the effects on the environment and surface building, and on the other to determine the deformation of the segment lining and the loading on the lining. In addition, the monitoring of tunnelling by recording and interpretation of machine data is particularly important in shield tunnelling [43].

4.2.4.1 Instrumentation

Surface settlement. In order to monitor settlement, transverse chains of five to six survey points are arranged as at a spacing of 25 m above the route of the tunnel to determine the surface settlements (Fig. 4-6, top right).

The required measurement intervals are related to daily advance rates. Settlements should be measured once per 10 m of advance but at least once a day. Measurements are normally recorded for every point in an area 25 m in front of and 25 m behind the passage of the shield machine. Subsequently, the measurement interval is reduced until the settlement has stopped altogether.

Newly introduced automatic measurement of settlement has proved successful. All the points of the monitoring section are surveyed automatically at continuous intervals. Combined with direct data transmission to a server and presentation of the results in the Internet, this can make the results directly accessible for all contract parties (Fig. 4-6, bottom).

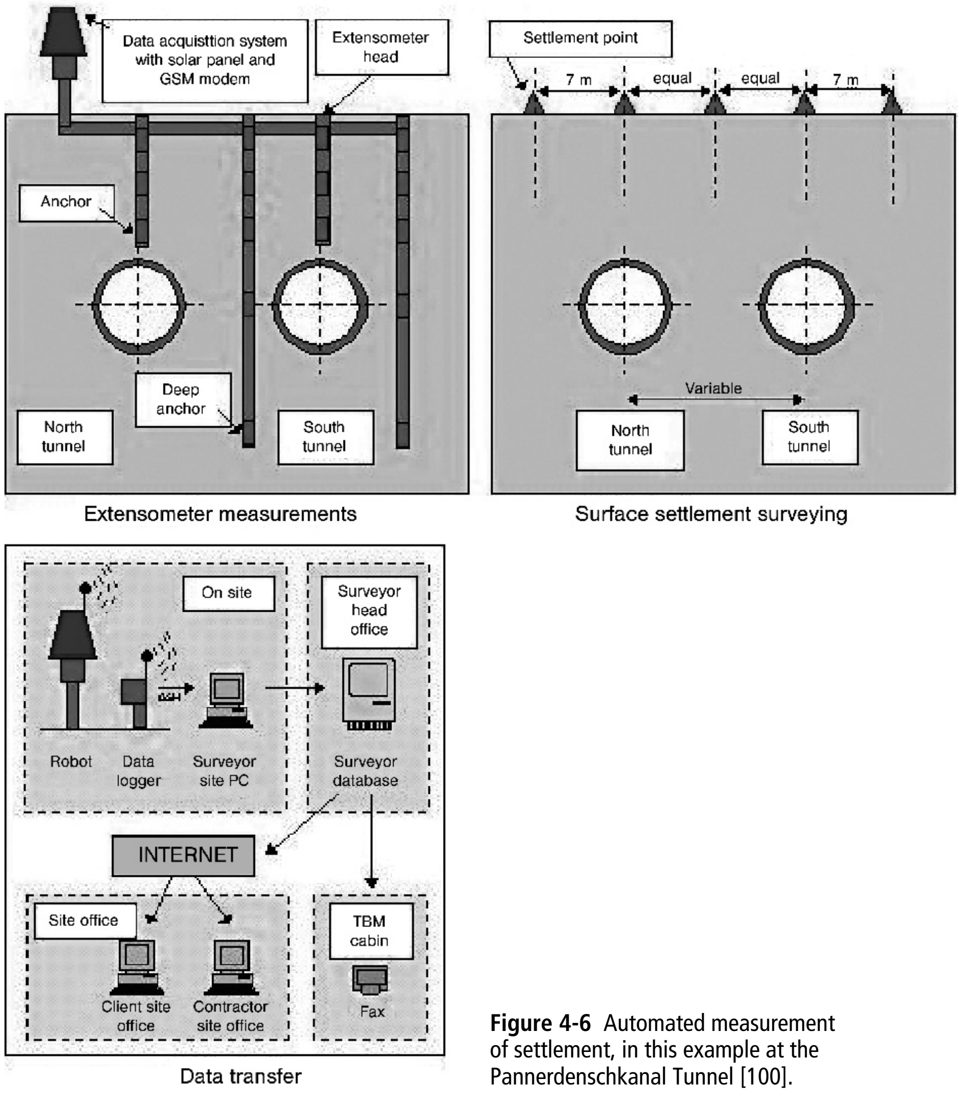


Figure 4-6 Automated measurement of settlement, in this example at the Pannerdenschkanal Tunnel [100].

Monitoring sections. Special monitoring sections are laid out in advance or when required at passages that are especially sensitive to settlement. The number of points in longitudinal and transverse directions is increased, with a spacing of only 5 m along the alignment. Extensometers can also be installed in the ground, sometimes supplemented by inclinometers. These measurement points are installed in advance of the passage of the arrival of the shield machine around the location of the tunnel (Fig. 4-6, top left).

Based on the results of these measurements in combination with the data recorded at the machine, the effect of support pressure, shield progress and annular gap filling on the surrounding ground can be analysed.

In addition to the described geotechnical measurement programme, further measurements can also be made on the tunnel lining as the drive advances.

Convergence measurement. Convergences of tunnel tubes are measured at specified spacings. Starting with the first reading directly after the completion of ring building and still within the protection of the shield skin, further measurements are made as the lining leaves the shield tail, the annular gap is grouted, the annular gap grout hardens and the backup passes by. Particularly in case of defects, the causes of structural damage to the ring in the critical construction phase can be determined from the convergences.

The measurements can be made with inclinometers, theodolite or also distometers. Regarding the measured data, inclinometer measurements have the advantage that they record deformations continuously.

Crown point measurements. In order to provide further information about the deformation behaviour of the latest segment ring, crown points are measured. This records the vertical movement of the tunnel crown in the freshly grouted annular gap. Based on this data in combination with other investigations, conclusions can be drawn about the drainage and hydration behaviour of the annular gap grout and any necessary adaptation of the grout mix can be made.

The behaviour of the ring as it leaves the shield has a particular potential for damage to the segments due to the lack of solid bedding and the flexibility of the fresh grout.

Measurement of joint offset and joint openings. In order to analyse the behaviour of the segments in more detail, the offsets and openings in the ring joints as well as the radial joints can be measured in addition to the convergence and crown point measurements. This makes it possible to record the course of ring deformation and the appearance of structural deformation and analyse the causes.

Pressure cells. In order to check the resultant grouting pressure in the annular gap, pressure cells can be installed in the segments at the monitoring section to record the curve of pressure of annular gap grouting from the time of injection to its hardening. This can detect pressure losses in the grouting pipes and also the drainage and hydration behaviour of the annular gap grout.

4.2.4.2 Recording and evaluation of machine data

Numerous technical innovations [134, 141, 253] have developed shield tunnelling in loose ground to a high-performance construction process in recent years. The scope of application and performance has been significantly extended. Tunnelling machines with 10 m diameter reach peak advance rates of over 35 m per working day and more than 200 m per week, corresponding in monetary terms to an overall turnover of 3 to 4 million Euros per week. This has fundamentally increased the demands on process control and monitoring. Comparison of current procedures in tunnelling with standardised methods of industrial production shows a need to catch up.

The necessary adaptations of the machine to changing geological and hydrogeological conditions have been based until now on the expert qualifications and experience and even the intuition of the people on site. Holistic scientific evaluation of the recorded tunnelling, measurement and settlement data to gain information for the control of the machine has not yet been consistently used. Important decisions therefore still have to be made without sufficient understanding of all relevant information that is specific to the situation.

The interpretation of the machine data is mostly performed manually by experts and is mainly used to document the shield drive and investigate the cause of interruptions. Due to the great amount of work, this is normally restricted to the evaluation of average values, for example of one ring as the machine advances.

This manual interpretation of machine data is carried out with a time delay as post-processing of the previous advances. This makes the results of the analyses very dependent on the people involved and their experience. The technical qualifications of the experts on site do, however, enable detection of risks in advance to avoid dangers and unplanned stoppages in mechanised tunnelling and thus improve the success of a project. Fig. 4-7 shows an example of the currently usual display of machine data, which is used for interpretation by the experts on site.

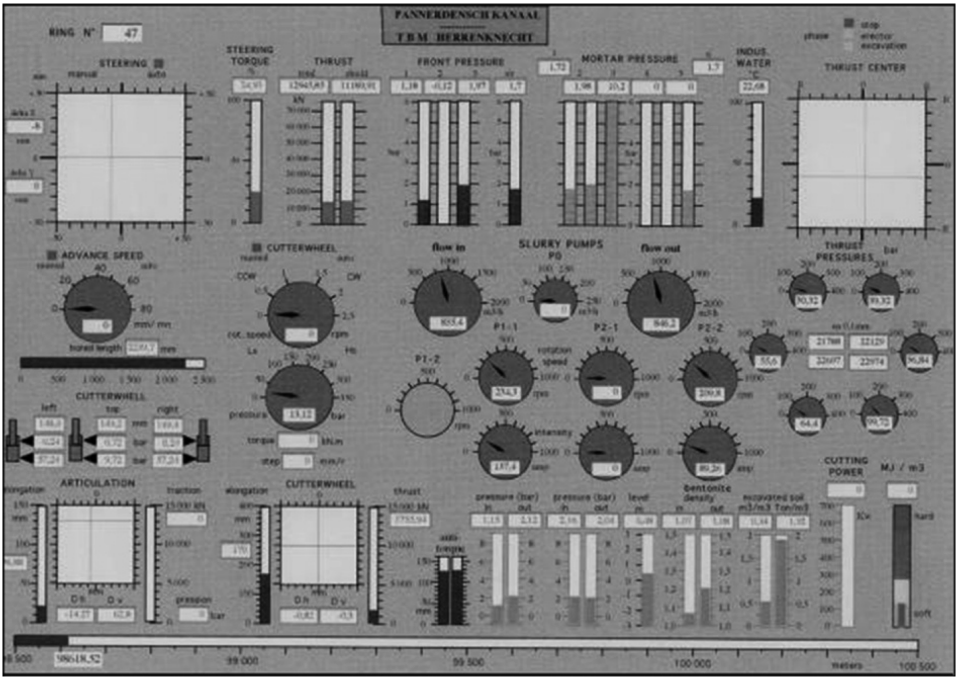


Figure 4-7 Online display of machine data [100].

Systematic data logging. Since almost all functions of a tunnelling machine are controlled electrically or electro-hydraulically, the relevant variables can be measured and processed digitally. Complete logging of the machine data is the state of the technology for modern shield tunnelling machines. 200 to 400 various pieces of machine data are recorded at intervals of 1 to 10 seconds, the momentary values. This produces 1.7 to 35 million data records per day, which can be automatically collected into average and final values for each advance cycle.

Separately from this, it is also usual to log extensive geodetic and geotechnical data as the tunnel advances. Although this is still often performed manually, resulting in very long and variable intervals, measurement robots have already been introduced on some more recent projects (for example the tunnel under the Pannerdensch Canal, Netherlands) to record defined measurement points automatically and transmit the data digitally.

In addition to the machine data and the geodetic data, data is also recorded at other parts of the process (separation plant, segment production works etc), which is normally only available in analog form as shift or daily protocols. In order to consider these, they first have to be processed and are only of limited use for the control of the drive due to the time delay. Table 4-1 shows a typical example of the recorded data.

Table 4-1 Overview of the measurement data recorded during a shield drive.

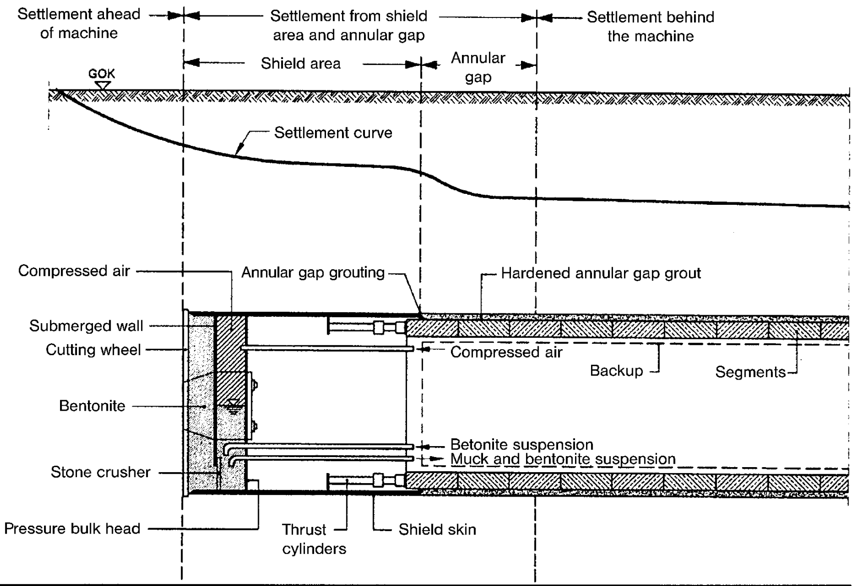
Machine data				
Progress	**Face support**	**Mucking**	**Grouting**	**Operational state**
– advance rate – penetration – overcut, conicity – jack forces – revolutions, torque and direction of the cutting wheel – consumption of motors	– air bubble pressure and bentonite level (Hydroshield) – distribution of the support pressure in the excavation chamber and at the cutting wheel – density in the excavation chamber – pressure and flow in the pipes of the conditioning agent (EPB shield)	– pressure, revolutions and torque of the screw conveyor (EPB shield) – pressure, flow and density in the slurry supply pipes (Hydroshield)	– grouting pressure and quantity of grout	– temperature and pressure in the hydraulic circuits – temperature and pressure in the cooling circuits

Surveying data		**Additional analog data**
Surveying in tunnel	**Geodetic data**	
– actual position of the machine – theoretical position of the machine – rolling – machine inclination / tilt	– settlement measurements on the surface and on buildings – extensometer, inclinometer readings – pressure cell and piezometer readings – internal monitoring sections and deformation measurements	– properties of the support medium (density, filtrate water discharge and liquid limit), (Hydroshield) – properties of the conditioning agent (EPB shield) – composition and properties of the annular gap grout – damage to the lining – installation position of the rings – shift protocols

The purpose of systematic data logging must be to gather all data on a project centrally, organise it systematically and make it available to all project parties for further processing (data and interface management). The formation of information and knowledge islands should be avoided and it should be made possible to link readings from the various areas to each other and analyse them. Evaluation can only be based on central recording, bundling and processing of the data from different sources in a structured database management system.

The central recording and bundling of data for the analysis of the course of settlement can serve as an example.

Fig. 4-8 shows the data required to analyse settlement, divided into the catagories face, shield skin, annular gap and subsequent settlement. In addition to the geodetic data from the surface terrain, such an analysis requires a range of machine data and surveying data from the tunnel. Further to the data mentioned in the illustration, the overburden at a location and any stoppages also have an effect on all three categories of settlement. Efficient evaluation as the tunnel advances is thus only possible when all the required information from the various parts of the tunnel site are available centrally and can be accessed.



Ahead of tunnel	Shield area and annular gap	Subsequent
• Deformation and instability, overbreak at the face • Alteration of the primary stress state in longitudinal direction	**Shield area** • Overbreak and shield taper • Crushing and vibration caused by steering and driving curves • Shear stresses along the shield skin • Deformations of the shield skin • Subsidence of the machine **Annular gap** • Properties of the filing material • Deficit in the fill quantity • Deficit in the grouting pressure • Deformation of the segment tube due to loading from the thrust jacks and the backup	• Settlement due to the increased compressibility of the soil reshaped by the tunnel advance • Deformations of the segment tube due to long-term effects, e.g. creep

Figure 4-8 Diagram of the interaction between the course of settlement and the advance of the machine.

Systematic data evaluation. Due to the complex time-dependent interaction between machine, support medium, annular gap grouting, segment lining, ground and the inconsistent recording and saving of the data, there is still no standard procedure for the evaluation and analysis of the tunnelling data. The enormous amount of data therefore makes it very likely that essential parameters and relationships will fail to be recognised and much information will be lost.

The aim of systematic data evaluation must be to analyse the arriving data as the tunnel advances in order to increase the quality and quantity of the information gained from the data and make it available to all parties in real time. The specialists on site should be supported by computer-assisted automatic evaluations, with the displayed results being available to all project parties. This can include statements about machine condition concerning wear and damage and the effects on the surroundings with guidelines for the adjustment of the machine to suit the prevailing geology and hydrology. This enables the proportion of avoidable settlement to be minimised and the performance and availability of the machine to be optimised.

Working from the already described structured recording and archiving of data in a database system, there are various processes for computer-based evaluation, which are now briefly described. For a detailed examination of this subject, reference is made to [151].

Classic statistics. For the purpose of statistical evaluation of tunnelling data, the known processes of multi-dimensional regression and correlation analysis should be mentioned first. Correlation analysis serves to obtain a quantitative magnitude for a relationship, in other words to determine the degree of relationship. Regression analysis is a process to detect relationships between various variables. The mathematical derivation is not given here and reference should be made to the relevant specialist literature (for example [88]) for further details. In the following, only the possible applications of data analysis for a shield drive will be presented.

Unusual deviations or fluctuations, which could easily be missed in the quantity of data, can be detected immediately from trend analyses and forecasts of characteristic initial values, like the relationship of specific energy consumption to penetration rate [97] or to the settlements produced [165], and can be used to trigger automatic alarms (example: collapse warning [145a]). Presuming that the system behaviour follows reproducible rules, information about the effect of adjusting individual parameters can also be used to optimise the process in the next time step. For example, the relationships between advance and settlements could be analysed in sections and the settings on the shield machine adjusted so that the configured ideal values are not exceeded [151].

Fuzzy logic makes it possible to take into account both engineering know-how (expert knowledge) and also intuition in data analysis and process control. The excellent compatibility and integration capabilities in standardised control and SPS or PLC (Programmable Logic Controller) systems have proved particularly useful for shield tunnelling applications.

In contrast to conventional methods, fuzzy logic is not based on the processing of sharply or discretely defined values, but on the theory of unsharp or fuzzy quantities, a recognised discipline in Mathematics. The basic principle is the description of a condition or a situation through linguistic variables instead of numbers and formulae. Rules are not described by elaborate mathematical models but by simple rule-based conclusions related to human thinking. The already described difficulties in the recording of non-linear interaction relationships between ground and machine, the numerous parameters and the time constants that are difficult to define can thus be bypassed in many areas.

The derivation of the mathematical and deeply theoretical description of fuzzy logic is not given here but reference can be made to the relevant specialist literature [6], [7], [245], [262]. The following section is intended to give examples of the possible applications and demonstrate the advantages for data and process analysis.

Example: Monitoring of the jacking forces in an EPB machine. The lifetime of a disc cutter depends not only on the ground parameters but also decisively on the machine and operating condition parameters. Numerous investigations define the exceeding of the permissible axial load (pressing force) and transverse shock loads in uneven tracks in fractured rock as particularly critical events. Both these loading components are often not noticed and can lead to high wear costs and long-term interruptions of the advance.

The objective of a fuzzy-based analysis of the process is to avoid the overloading of discs and ensure maximum boring progress.

Fig. 4-9 shows a process control system consisting of a database management system (DBMS), the fuzzy controller and the logical links to any proportional-integrator controllers (PI controllers) or programmable logic controller (PLC) units.

The decisive ideal or set values are defined as the cutting wheel revolution speed and the advance rate. The advance rate can generally be characterised as linearly dependent on the oil volume flow in the thrust jacks. For the output variable in the present example, the relative change of advance rate expressed as the oil volume change dependent on the instantaneous value is used instead of the absolute value of advance rate.

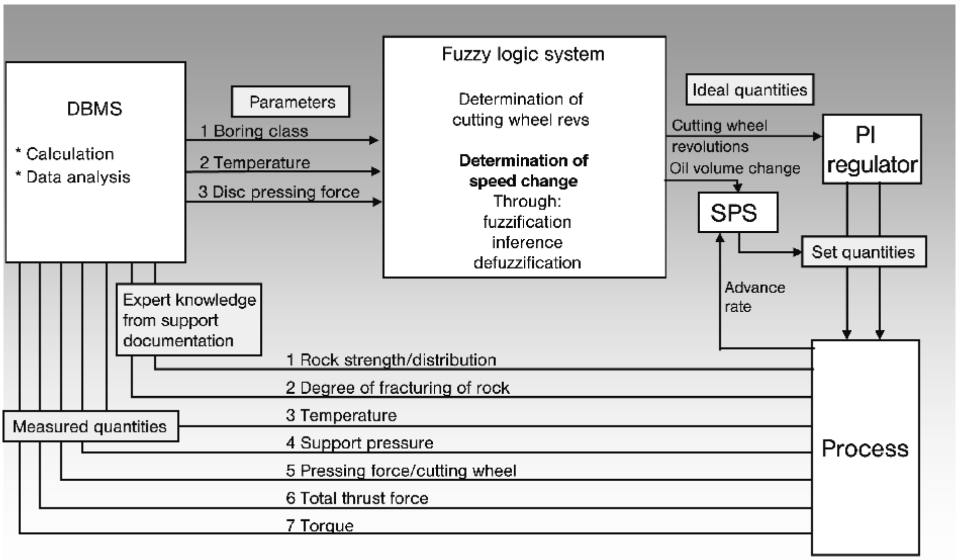


Figure 4-9 Fuzzy controller for knowledge-based determination of the ideal settings for cutting wheel revolution speed and oil volume change for operation in mixed face conditions.

In order to evaluate the process, important information about the face and ground conditions is required in addition to the measured values of support pressure, total thrust force, pressing force on the cutting wheel and temperature. It should be noted for example that in mixed face conditions (50 % rock; 50 % loose ground), only half of the discs are loaded, although these may be loaded with twice the force under some conditions. Since reliable advance probing systems are still in development at the moment, expert knowledge from the contract documents in the form of standardised check lists is entered manually into the DBMS for the description of the decisive ground parameters.

Seven relevant data information streams reach the DBMS. In the DBMS, the input data is filtered, subjected to preliminary analysis and processed to the characteristic parameters of boring class, temperature and cutter press force (averaged). The determination of a boring class for the specific process requires an independent subordinate fuzzy logic system to analyse the ground parameter data. Proven methods are already available in the literature [261], which can be adapted for the specific conditions of the process and the project. In order to determine the average cutter press force, the action and reaction relationships have to be evaluated by the DBMS based on known deterministic relationships.

The fuzzy logic system finally performs a problem-oriented data analysis to determine the ideal values for cutting wheel revolutions and oil volume flow difference. The characteristic procedure always includes the processes of fuzzification, inference and defuzzification.

Fuzzification can be understood as transforming the given sharp values (in this case the parameters temperature, cutter press force, boring class) and the ideal values (in this case cutting wheel revolutions and oil volume change) into unsharp values. The degree of belonging of the individual values to the relevant unsharp quantity is described through linguistic variables. These form a so-called fuzzy set for each parameter of a data set.

Inference. Extensive expert knowledge was already necessary to create the fuzzy set, which now to be processed by inference into processing rules. According to the principle of rule-based reasoning, one step is taken after another according to the principle IF ... AND ... THEN, similar to the human use of symbols in thinking. The evaluation of the IF ... AND part is described as aggregation and the evaluation of the THEN part is described as composition.

This rule matrix provides the core of the following data analysis. All subsequently incoming data is evaluated solely on the basis of these rules, so that a high degree of care and expert knowledge is demanded here.

A simple possible rule would be, for example:

Aggregation: IF cutter press force = low AND boring class = good rock

Composition: THEN oil volume flow = increase.

Precondition: temperature = low

The aggregation represents the combination of the individual conditions into a rule. Many hundreds of fuzzy operators are now available, although however the Minimum/Maximum operator [6] is mostly used in practical applications. The Minimum operator in this case corresponds to a simple AND, the Maximum operator a simple OR.

The compensation, the selection of the decisive rules, is mostly implemented by the Maximum operator, as either one rule or the other should be active for each value. Further processes to evaluate individual rules with the aid of a degree of plausibility are also available, although this will not be described in detail.

Defuzzification. At the end of the fuzzy inference, the result for the oil volume flow is initially given as a linguistic variable and has to be converted back into a technical quantity. This step is described as defuzzification. The relationship between the technical quantity and the linguistic interpretation is explicitly described by the membership function in the creation of the fuzzy set. Various methods of defuzzification are also described in the literature (see for example [6, 101, 262]), and the centre of gravity method is usual in practice, using the weighted average of all membership functions of the sharp output value for the calculation.

Artificial Neural Networks (ANN) are mostly used when the complexity of the problem under consideration is too large or the knowledge is insufficiently structured [262]. In contrast to conventional deterministic methods, closed mathematical modelling of the system is omitted in this case, which means that no statements are necessary about the significance of the individual variables or their interaction.

Neural networks are based on the structure of the human brain and consist of neurons arranged in different layers (input layer, hidden layer, output layer). Each neuron is linked with those in the following and preceding layers. The processing of the data, which is described as "learning", occurs in the so-called "hidden layers". Similarly to the human brain, the weights of the links between the individual neurons are modified so that the input and output data are best reproduced. The best-known learning process for ANN is "back-propagation" [101], of which there are now various modified forms (for example fastback, batch back-propagation [8, 211]), although the advantages and disadvantages of these cannot be described in detail here. For further details, reference is made to the specialist literature [8, 211, 262].

ANNs are already in use in various fields, like for example handwriting recognition, the forecasting of share values, meteorological forecasts, the noise of motors [211] and also in conventional tunnelling for the forecasting of settlement [220, 222].

The main weak point of an ANN is its so-called "black box" procedure, in which the user cannot comprehend how the ANN determines the solution for each problem. The user can only test the ANN by inputting further data and comparing the determined results with actual results, but cannot analyse the sequence of actions. ANNs cannot thus be manually optimised nor is it possible to enter basic knowledge or fundamental relationships manually.

Since ANNs require a large number of good quality, i.e. complete data sets with as few errors as possible, most problems with their practical application are connected with the quality of the available data. The amount of example data required can only be estimated with difficulty in advance of an analysis. The disadvantage of the computer time required for training an ANN is only significant for very complex networks. It can however be that a large number of trials with various network configurations (network type, number of hidden layers, number of neurons per layer) and training runs (number, training type, weighting of errors etc.) are required until a network suitable for a specific task has been found.

ANNs are particularly suitable for the analysis of data in tunnelling due to their capability of completely independently learning, from example data sets, non-linear relationships of any complexity, such as could no longer be comprehended by an expert. The problem with conventional solutions, that methods for extensive problems can only be found with difficulty, does not occur because no mathematical formulation has to be undertaken. Suspected rules and relationships in data fields can thus be easily tested or determined. Tunnelling situations can be simulated with networks that have been trained once and the effects on the surroundings can be determined. Furthermore, comparison of forecast with newly recorded data can be used to control the tunnel drive as with statistical analysis and can give warnings about changes in the ground or the condition of the machine.

Neuro-fuzzy. Neuro-fuzzy denotes the combination of artificial neural networks with fuzzy technology. The combination of the two methods offers several advantages, which are now briefly described.

– The influence of individual input parameters and the relationship between them can be determined using an ANN so that the input parameters required for a fuzzy system can be reduced to fewer but characteristic parameters.
– The result of the learning process is a normally controllably fuzzy system with a method of working, which in contrast to the ANN is understandable. This avoids the "black box" behaviour of an ANN and the result can be manually optimised.
– In contrast to conventional fuzzy systems, it is no longer necessary to develop each rule laboriously by hand and optimise it until the desired result is achieved. Particularly for large systems with many input variables and correspondingly many rules, this can rapidly become confusing. The system can learn the rules independently from example data.

Neuro-fuzzy models are predestined for the evaluation of tunnelling data since the advantages of both systems are combined and also enable automatic process control through connections to conventional control systems. For the present example of settlement analysis, the high number of input quantities can be selected with the ANN and then passed to a fuzzy controller for process control.

4.2.5 IT systems for the recording and evaluation of geotechnical data

IT is particularly useful both for data transmission and for the evaluation and monitoring of geotechnical data. Quite apart from the rapid recording, the systematics, continuity and comparability with rapid access back to previous measurements can be mentioned as advantages. Systems have therefore been developed in recent years for the automatic recording, processing and visualisation of the measured data.

Even with such ample information being made available on the computer screens of contractors, site supervision, employers and specialist consultants, the staff responsible for site management and supervision still has to make decisions on site. Particularly modern tunnelling procedures, with the measurements being part of the verification of structural safety, demand evaluation and making of decisions on site. Fig. 4-10 shows an example of a system for automatic recording of measured data in real time (in conventional tunnelling) with graphical processing and saving of the data. This is accompanied by simple calculations like, for example, temperature compensations for the measured readings. The data can be transmitted by modem to external locations (site supervision, specialist consultant) [254].

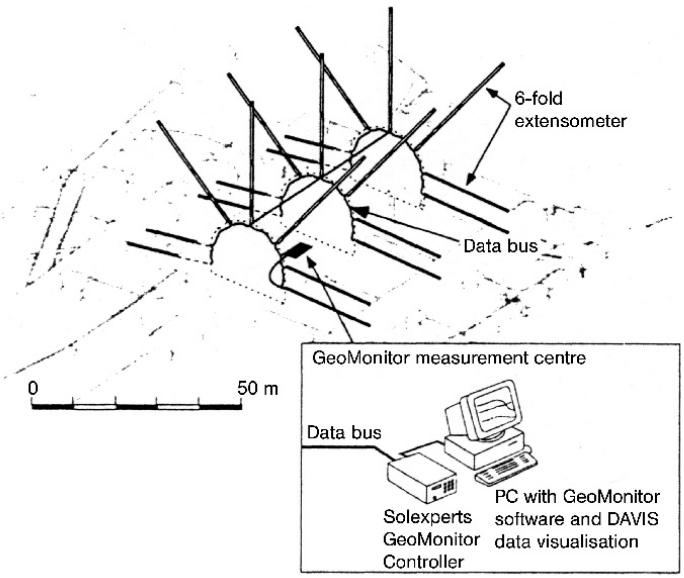


Figure 4-10 Example of a data logging system [254].

4.3 Measurement processes, instruments

The next section describes measurement processes and instrumentation that are currently in use. Table 4-2 gives an initial overview of the purpose and example applications of the instrumentation used.

Table 4-2 Overview of the purpose of measurements, applications and instruments [25].

No.	Measuring instrument (e.g.)	Purpose	Example
1	Convergence (KV) – measuring tape Geodesy	Demonstration of a new equilibrium after the driving of the tunnel	– Convergence measurements – Location surveying
2	Extensometer and inclinometer	Comparison with calculations	– Measurement of deformation in surrounding ground
	Pressure cells	--> for improved tunnel design	– Stress measurement in the shotcrete layer
3	Tunnel scanner DIBIT deflectometer	Quality assurance of individual construction projects	– Excavation profile – Directional precision of boring
4	Electronic hose levelling instrument Electrolevel measuring chain Motorised digital level	Control measurements of entire construction sequences	Soilfrac ® process in inner-city tunnelling (heave/settlement)

4.3.1 Deformation measurement

Continuous observation of the behaviour of a tunnel at every phase of construction through simple and reliable measurement of displacements is an essential aid to safe and economic construction. The observation of deformation and displacements is particularly significant because these can be relatively simply measured and interpreted. From deformations caused by changes of construction states and their behaviour with time, it is possible to reach conclusions about the properties of the rock mass, the development of ground pressure and the extent of surface settlement. The information has a quantitative character and can provide a sound basis for important decisions. Measurements also often fulfil a control function, with possible collapses being detected before being noticed visually to enable immediate action to reduce damage, for example by strengthening or propping the support.

4.3.1.1 Geodetic surveying

Considered on its own, geodetic surveying is time-consuming in performance and evaluation. A considerable part of this time, however, is already necessary for regular tunnel surveying as the tunnel advances, so the additional time taken to determine deformation is not particularly long.

Measurements. Geodetic instruments can be used to determine the alteration of the location of points on the support relative to other points on the support. The absolute level of points in the tunnel above sea level can also be determined by levelling from bolts, which have been checked once or more by levelling from known benchmarks. In this way it is possible to record the absolute heave or settlement of support elements.

The situation with the vertical location of foot points is similar. When more than one adjacent tunnel is excavated, not only the change of the horizontal diameter of the first tunnel during the excavation of the adjacent tunnels is important but also the change of vertical position of the right- and left-hand benches.

Measurements of settlement on the surface (terrain or buildings) also belong to geodetic surveying and are performed by precision levelling.

4.3.1.2 Convergence measurements

Convergence measurements determine the relative displacement between measurement points situated at the perimeter of the cavity. An example of the layout of the measured distances shown in Fig. 4-11. The measuring of diagonals can cause considerable obstruction to construction operations.

Measurements. Immediately after the exposure of the perimeter of the cavity, convergence measurement bolts are set at various points and a first reading is measured (zero measurement). The measurement is repeated, initially at short intervals and later as the support becomes effective at longer intervals according to the specified measurement programme. The measurement of diagonals is often impossible without considerable obstruction of construction operations. Fig. 4-11 shows as an example a diagram of a classic convergence measurement.

Convergences are measured with special measuring tapes, which are constantly tensioned to a reproducible degree. According to the quality of the measuring tape and the type of connection to the measurement bolts, a precision of 0.01 to 0.1 mm can be achieved and thus much more precise than optical or geodetic methods [25].

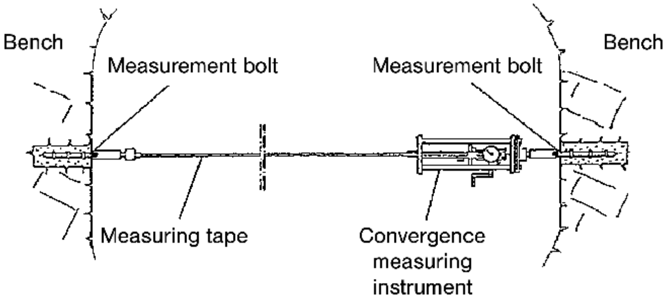


Figure 4-11 Diagram of convergence measurements in a tunnel (Interfels GmbH).

Convergence measurements are evaluated with a displacement-time diagram (Fig. 4-12), whose coordinates start from the zero measurement and with the repeat measurements being entered as difference from the zero measurements. Optimisation of the support entails the best possible adaptation to the final displacement, which means a (nearly) horizontal curve in the diagram.

In the example shown here (Fig. 4-12), the crown settlement F_S is approx. 100 mm. According to experience, the horizontal convergence is about 10 to 70% of the vertical, although it can also be zero or even negative depending on the ground conditions. Statements about the absolute displacements of the individual measurement points can be made by providing additional extensometers (see Section 4.3.1.5) or by optical surveying of displacements (see Section 4.3.1.3), which enable the measurement of vertical displacements over longer distances from survey points on the perimeter of the tunnel or on the ground surface.

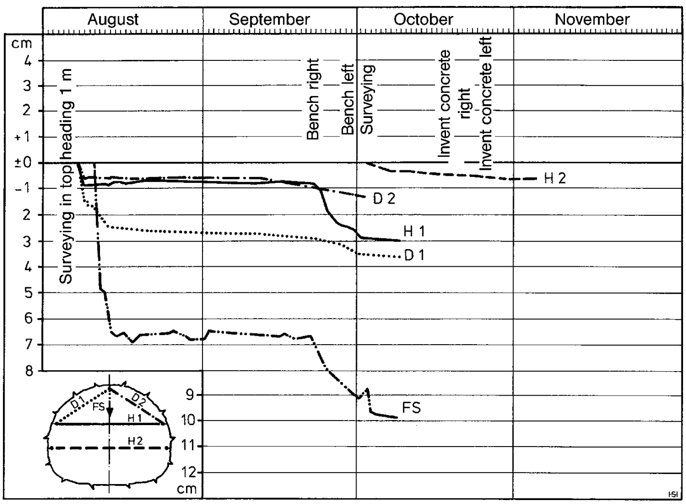


Figure 4-12 Graphical display of convergence measurement in a time-displacement graph.

The primary sources of errors are incorrect handling of the wire or measuring tape – kinks for example can lead to false readings – and failure to compensate the effect of temperature.

Obstruction. Since the zero measurement has to be undertaken as soon as possible after excavation, this can easily obstruct tunnelling work. For this reason, other methods of measuring displacement (for example optical) have become established. This is indeed less precise, but does not cause any obstruction at the face and can also cover a greater number of survey points permanently in less time [24].

4.3.1.3 Optical surveying of displacement with electronic total station

Using the digital surveying instruments and powerful computer programs for data processing that are now available, displacements in three dimensions of any survey points can be determined in absolute coordinates, which means a reference system more stable than the tunnel. This makes it considerably easier to detect local and large-scale deformation than with relative surveying methods like convergence measurement or levelling. A three-dimensional display of the results can enable comparison of the displacements of all the survey targets in a selected area. One essential feature of optical surveying of displacements is the free positioning of the survey station (Fig. 4-13). The position can be chosen flexibly to provide the best sight lines and the least obstruction of construction work. The accuracy is determined essentially by the geometrical relationships and the precision of the surveying instrument and the targets.

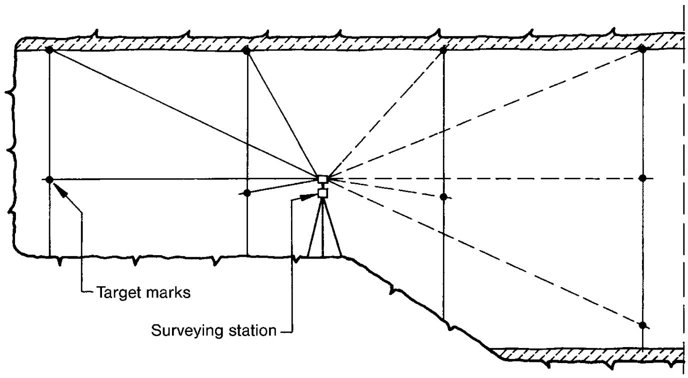


Figure 4-13 Free positioning of the total station for optical 3D displacement surveying [136].

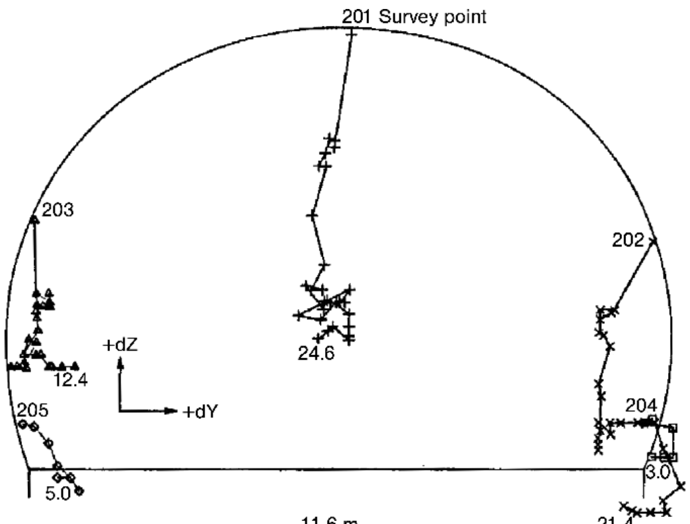


Figure 4-14 Surveying of the absolute position of convergence targets with display of their movement vectors [187].

Electronic total stations with coaxial electronic distance meters are used for optical surveying of displacements.

An example of a display of the absolute surveying of the location of convergence targets with their movement vectors is shown in Fig. 4-14.

4.3.1.4 Surface surveying

When shallow tunnels are constructed, particularly for inner-city underground railways, the surveying of settlement on the surface is of particular importance to evaluate the construction and to record evidence in case of any legal disputes. The observations normally consist of precise levelling of the ground surface. Steel bolts are concreted into the terrain before the start of construction and levelled; if there are existing buildings, level marks will be established on the structure, for example in the cellar. Inside buildings, levelling can also be continued with water levels.

Evaluation. Further instruments for the recording of surface settlement to give information about movement vectors are extensometers and sliding micrometers. An example of this is the construction of an urban rail tunnel with only 4 m overburden under a built-up area in Dortmund-Lütgendortmund (Fig. 4-15).

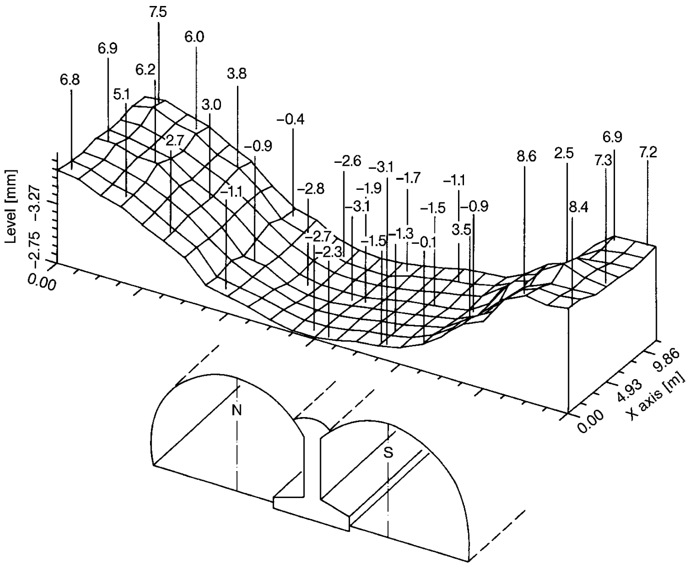


Figure 4-15 Surface surveying for the construction of the Dortmund-Lütgendortmund Tunnel; Interfels GmbH.

In the interpretation of the results of the levelling, which should always be done in combination with the relevant convergence and extensometer measurements, the various causes of settlement can be analysed, for example settlement due to groundwater lowering, due to excessive lining deformation, the influence of the construction process like too fast an advance rate or inadequate grouting of the annular gap, in order to be able to introduce immediate countermeasures in case of unexpected settlement.

4.3.1.5 Extensometer measurements

Convergence measurements and levelling can only record the movement of a surface (tunnel walls or ground surface), but not deformation in the ground or the distribution of deformation with depth. In order to record these effects, single or multiple extensometers are installed in boreholes drilled from the tunnel (Fig. 4-16 left) or – for shallow tunnels – above ground (Fig. 4-16 right).

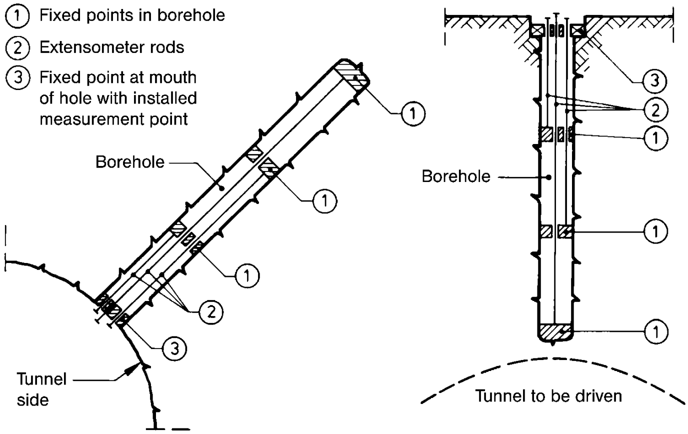


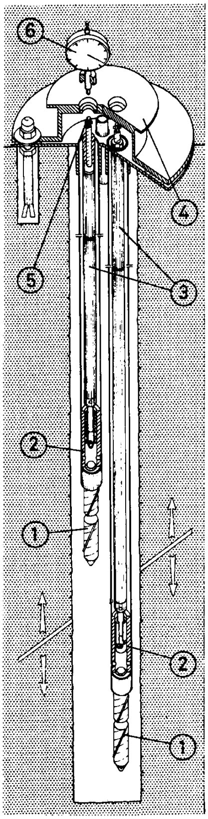
Figure 4-16 Extensometer to monitor deformations in the direction of a borehole. Borehole drilled from the tunnel (left) or from above ground (right).

Single extensometer. A single extensometer consists in principle of a rebar, which is cemented into a drilled hole (fixed point), the measuring rod and the extensometer head, which is fixed into the opening of the hole with anchors (rod guide and measuring instrument). The movement of the rock mass is shown by the change of length of the free length of the measuring rod projecting from the extensometer head.

Multiple extensometer. Multiple extensometers (Fig. 4-16 and Fig. 4-17) or a number of single extensometers of various lengths can be used to determine the movements of the rock mass to give – presuming that the fixed point anchored at depth does not lie within the loosened zone – a picture of the extent and deformation behaviour and the absolute displacement of the perimeter of the cavity (Fig. 4-18). Details of the installation of a multiple extensometer can be found in the relevant product information.

In addition to extensometers with measuring rods, telescopic tubes (rope) and wire extensometers are also used. The use of the latter type is recommended when simultaneous horizontal displacements are to be expected.

The borehole diameters vary according to the number of fixed points from 25 to 100 mm. Measurement accuracies of about 0.1 to 0.01 mm enable early recording of displacement tendencies. The measured values can be read at the location from mechanical dial gauges, but the installation of an electronic transducer at the extensometer head for fine measurement and registration is often more practical.



1. Rebar anchor, fixed by mortaning into borehole
2. Anchor chuck with measurement and calibration position to check the function of the device
3. Measuring rod in plastic protection sleeve, locking spur on anchor side, dial gange contacts head
4. Extensometer head with rod guide and rock screw to fix in mouth of borehole
5. Measurement contact point for dial gauge or electrical sensor
6. Dial gauge with contact, mechanical dial

Figure 4-17 "Rex" multiple extensometer from Interfels GmbH.

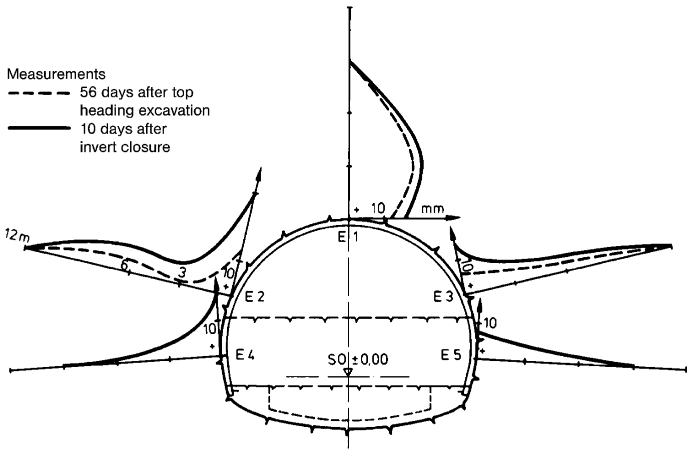


Figure 4-18 Example of extensometer measurement.

In shallow tunnelling, extensometers set at ground level can be used to record the settlement behaviour of various soil strata and also the particularly interesting course of settlement above the crown at each phase of construction (Fig. 4-19). The surface settlement itself must, as already explained in Section 4.3.1.4, be measured by levelling.

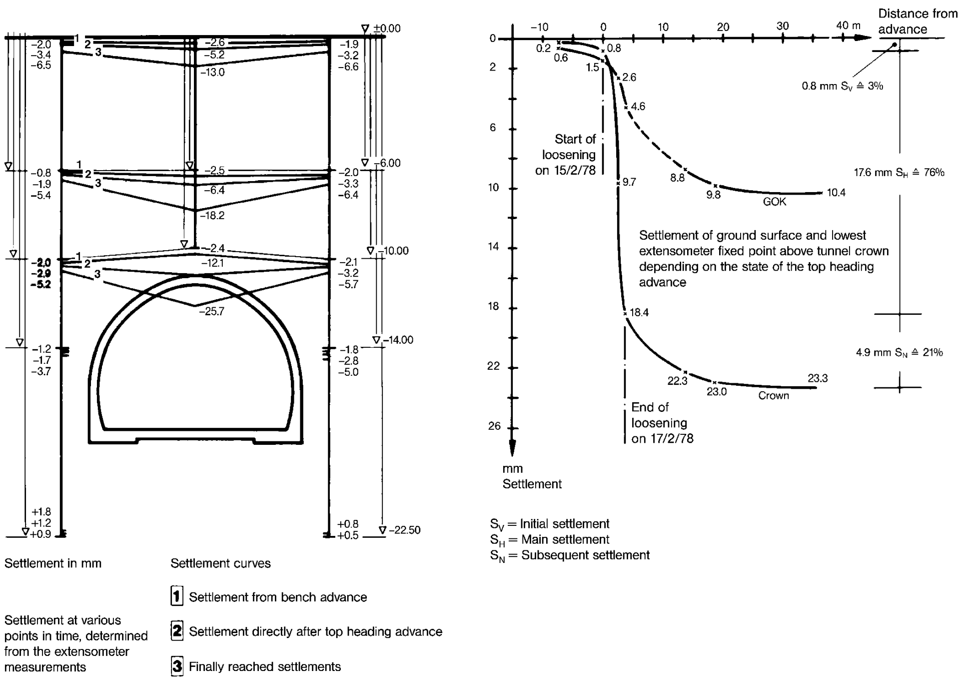


Figure 4-19 Extensometer measurements at the construction of the Stadtbahn, Mülheim an der Ruhr [69]. Settlement in transverse profile (left) and behaviour above the crown with time (right).

Layout and installation. The positioning of extensometers should consider the following points:

– In stratified or foliated geology with anisotropic stress-strain behaviour, extensometers should be installed as far as possible both normal and parallel to the bedding or foliation in order to investigate the effect of deformation in different directions.
– Extensometers should generally be installed before the tunnel advance or, if this is not possible, immediately after the tunnel advance. If there is already an investigation heading in a planned rock cavity, initial displacements can be recorded with extensometers that can be disconnected. When the excavation approaches, the endangered components can then be removed, leaving the remaining components unaffected in the borehole.
– For the arrangement of fixed points at various depths of the borehole, it is sensible to take into account the prevailing bedding conditions from the borehole logs (if possible from cores).

Since the behaviour of the fixed points can no longer be controlled in a filled borehole, measurements are subject to uncertainty whether the actual behaviour corresponds to the assumed behaviour, i.e. whether the fixed points remain at the intended locations or come loose from the rock mass together with the mortar or are pulled out of the mortar.

Length measuring anchors. Another proposal for the layout of extensometers is made by F. Schuermann [214] under the description “length measuring anchor process”. Four single rock extensometers are set in a monitoring section as shown in Fig. 4-20, in pairs with two horizontal and two vertical. In order to record the formation of a load-bearing ring in the surrounding rock mass, the extensometers should be as long as the tunnel is wide and high. As shown in Fig. 4-20, the lengths to be measured are in the following relationship:

Vertical $LI = LA + LH + LL$

Horizontal $LB = LM + LO + LW$

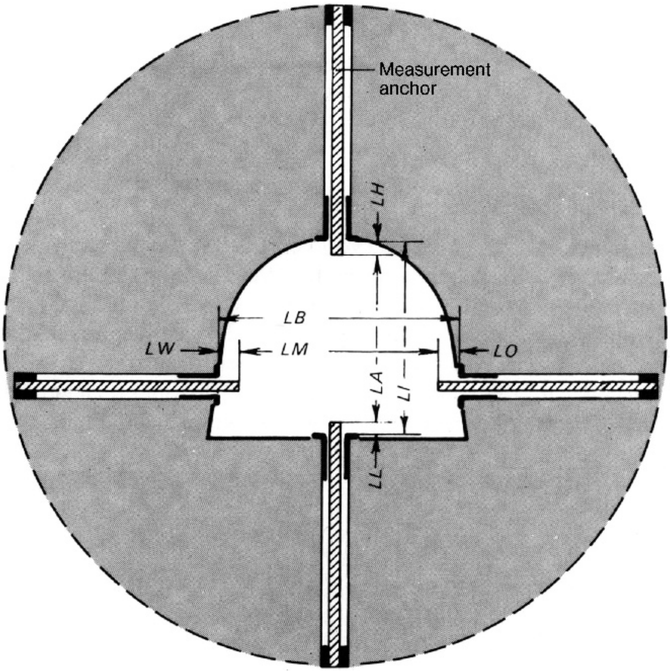


Figure 4-20 Diagram of the principle of the length measuring anchor process, from F. Schuermann [214].

The lengths are measured with measuring tapes. In the crown and in one of the sides, the ends of the measuring rods are fitted with hooks to hang the measuring tape. This makes it possible for one surveyor to undertake the measurements. The advantage of this process is that the measurements can be checked directly by checking the sum with the equations given above. If this is wrong, the measurement can be repeated.

The intermediate displacements calculated by comparing the newly measured lengths with the original lengths can be calculated, due to the principle of measurement, as follows:

Vertical $I = A + H + L$

Horizontal $B = M + O + W$

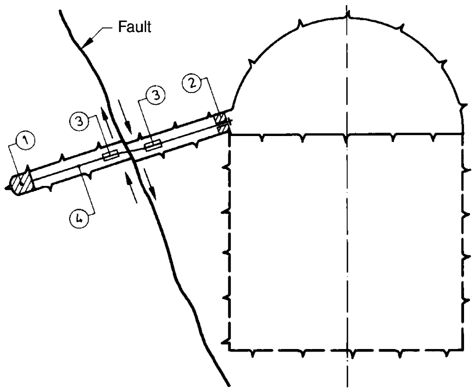
These eight measured values enable the following statements:

- Readings A and M give information about the stressing of the load-bearing ring at its periphery, that is from outside. Alterations of these values mean that the fixed points of the extensometers have moved in the holes.
- Displacements I and B show the vertical and horizontal convergences of the tunnel.
- Values I, A, B and M show the deformation state of the load-bearing ring. The quiescent state can then be recognised in that I, A, B and M are initially much smaller (less than 20 mm). The length alterations of I and A and for B and M respectively are nearly identical [126].

– If on the other hand $I > A$ and or $B > M$, then the load-bearing ring already has failure deformation. At failure deformation, volume increases occur because the broken pieces fracture further and the broken material takes up a larger volume than the original compact rock. The volume increase at failure deformation is associated with the loss of load-bearing capacity of the ring (loosening). The location and type of failure deformation can be read from the extensometer measurements, since each extensometer gives the displacements occurring in its sector (*H*, *L*, *O*, *W*).

4.3.1.6 Inclinometer / deflectometer measurements

In order to measure displacements transverse to a borehole, deflectometers are used, also called inclinometers. Transducers, which are generally connected to each other by a tight wire and measure the relative displacement at right angles to the axis in a polygonal traverse, are set at defined spacings into a lined borehole. In this way, they can deliver the components of the movement of the rock mass, which are not detected by extensometers, which measure changes of length along the axis of the hole (Fig. 4-21).



1 End piece
2 Head piece with tight wire device and connections for electrical display
3 Electrical transducer
4 Wire

Figure 4-21 Deflectometer to measure displacements transverse to a borehole.

A simple deflectometer (Fig. 4-22) consists of a head piece 1, a measuring element 2 and an end piece 3, which are each mounted at three points inside the casing of the borehole 4 and connected to each other with spacing tubes 5, which have universal joints. In the three measuring elements are precision blades 6 for the exact support of the tight measuring wire 7. In the central measuring element 2, an electrical inductive transducer with actuating motor 8 is mounted so that it can measure changes of the length of the measuring wire in one direction both sides of the blades. If, for example, displacements occur along the length head piece – measuring element or measuring element – end piece, then this movement is transferred to the borehole casing and thus to an arm of the deflectometer, and then displayed by the inductive transducer.

The chain deflectometer is an extended form of the simple deflectometer and can contain up to eight members connected in a row.

Installation. Deflectometers are installed in the borehole at the head piece with a guide rod and can be reused after the completion of measurements. The diameter of the device is relatively large due to its electrical components. Measured lengths of up to 60 m are possible, and the accuracy is 0.1 to 0.01 mm according to the spacing of the measuring elements.

Drilling and the installation of the measuring equipment should be completed before driving the tunnel in order to be able to record the first deformation of the rock mass (Fig. 4-21).

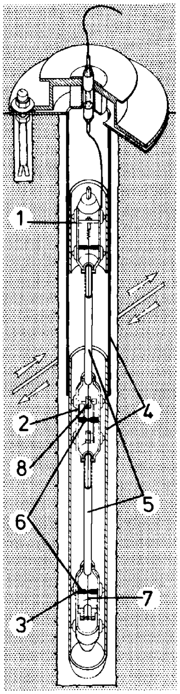


Figure 4-22 Simple deflectometer from Interfels GmbH (see legend in text).

Settlement measurement. Another use of the deflectometer, apart from installation in a borehole to measure transverse displacements, is to set them horizontal at ground level to measure settlement. This procedure was used, for example, on contract A2 of the Stadtbahn Bochum [98]. In addition to a dense grid of levelled points on the surface, an open chain of deflectometers was installed at right angles to the direction of advance, which recorded the vertical displacements at short intervals with automatic registration and thus the settlement rates (Fig. 4-23).

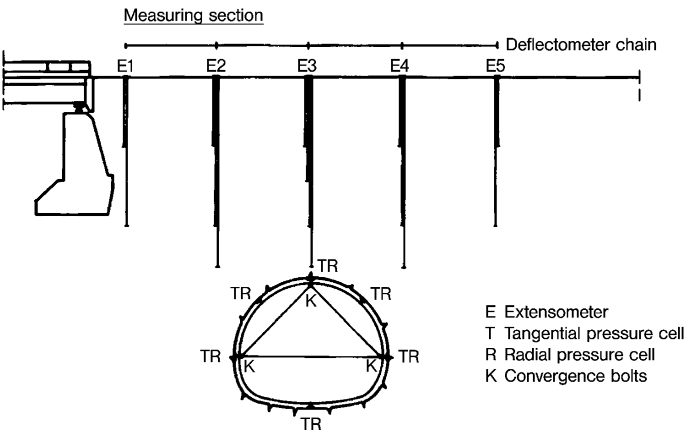


Figure 4-23 Use of an open deflector measuring chain on contract A 2 of the Stadtbahn Bochum [98].

4.3.1.7 Sliding micrometer measurements

The sliding micrometer is a highly precise instrument for measuring strain. The device is used for the complete determination of axial displacement components along straight lines in rock, concrete or soil. The high degree of precision is based on the bracing of the portable probe in the measuring marks on the ball-and-cone principle.

Installation, function. Metallic measuring marks, which are connected to each other with a plastic protection pipe, are fixed by grouting in a borehole of about 100 mm diameter (Fig. 4-24). To perform a measurement, the probe is inserted at the operating rod in the protection pipe stepwise to the measuring marks, which are spaced at 1 m from each other. The two ball-shaped measuring heads at the end of the probe pass through the measuring marks (sliding position). By turning it through 45° and pulling the operating rod or the cable, the probe is braced with the measuring heads between two adjacent measuring marks (measurement position). In vertical or heavily inclined measuring tubes at depths of up to 50 m, the probe is brought into the measuring position and braced with the aid of the operating rod alone, or with a reel if the depth is more than 50 m. With horizontal or slightly inclined tubes, straight distances of up to 100 m can be measured without a reel.

The extremely high setting accuracy of $\pm 10^{-6}$ m in the calibration device, or $\pm 2 \times 10^{-6}$ m in place in the measuring tube, is achieved through the exact setting of the probe in the cut-outs (ball-cone). With regard to strain, the device has a measurement sensitivity of 10^{-6} m and the measurement range is 20 mm.

Fig. 4-25 shows the monitoring of ground loosening with a sliding micrometer through the example of an urban rail tunnel in Dortmund. The change of direction of the displacements after the tunnel has passed the measuring point can be easily recognised. During the arrival of the tunnel, negative displacements are measured in the direction of the borehole, or compression of the ground (left-hand branch), while after the passage of the tunnel, positive displacements are measured in the direction of the borehole, or pulling (right-hand branch).

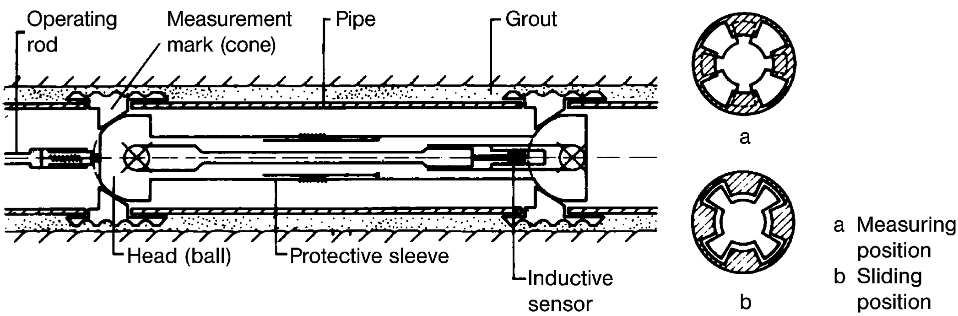


Figure 4-24 Sliding micrometer for the determination of the axial displacement component in the direction of the borehole. Section through the measuring probe in the borehole (Solexperts).

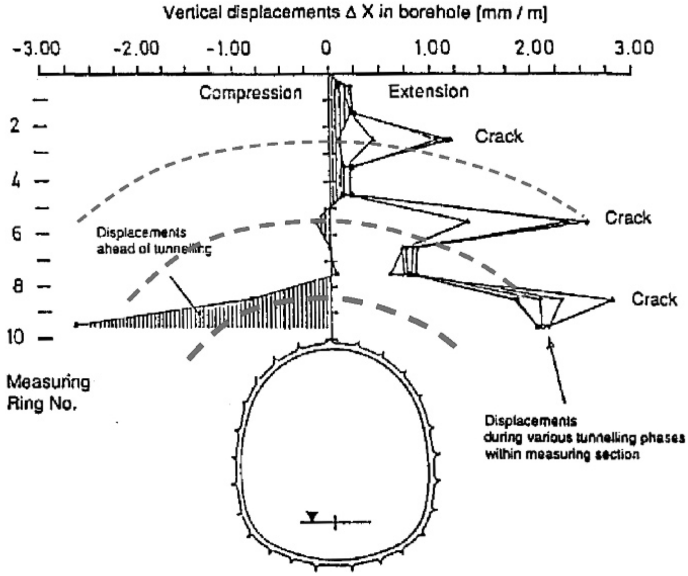


Figure 4-25 Monitoring of ground loosening in the crown of a tunnel [25].

4.3.1.8 Trivec measurements

In contrast to measurements, in which for example a displacement component or a strain is measured, the concept of linewise measurement offers great advantages [114]. In linewise measurement with the Trivec instrument, the distribution of displacement quantities along a measurement line, for example a borehole, is completely measured.

Installation, function. The device is set and mortared in a borehole with a PVC casing (Fig. 4-26). This is provided with measuring marks at regular intervals of 1 m. Since the measuring tube is connected to the rock, the measuring marks adopt the displacements of the surrounding medium. During the measurement of the borehole, the probe slides through the measuring tube and is set stepwise metre for metre in the measuring marks until all the measured distances have been read. If the measuring marks experience a relative displacement between two successive measurements, then this shows as the difference between two readings. The Trivec is essentially a sliding micrometer, which is additionally equipped with two inclinometer sensors for measuring the displacements transverse to the borehole direction.

4.3.2 Profile surveying

4.3.2.1 Photogrammetric scanner

As part of the increasing requirements for quality assurance and environmental protection, an increasing amount of control measurements are made in tunnelling [24].

A new development in measurement instrumentation is the photogrammetric scanner. In contrast to conventional instruments, this does not record individual points on the tunnel lining but the complete surface. The device is thus capable of recording not only oversized and undersized profiles but even layer thicknesses and to a certain extent convergences by comparing images at different times.

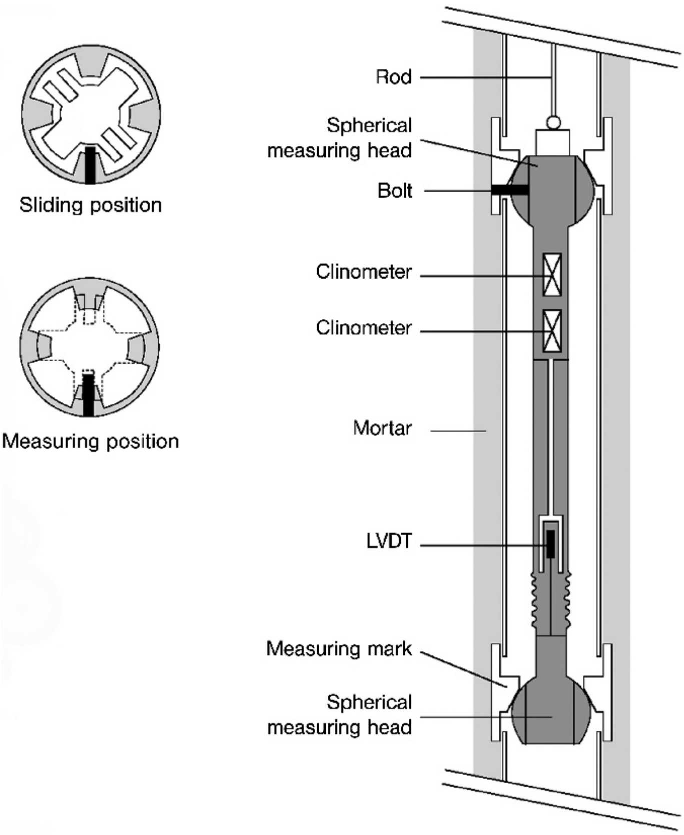


Figure 4-26 Trivec device for the measurement of three displacement components along a vertical measurement line; Solexperts AG.

The principle of a tunnel scanner is based on a “stereo-photogrammetric” image of the tunnel surface, which means that all areas of the tunnel surface are scanned simultaneously from two different positions to enable 3D reconstruction. The position of the instrument is fixed by surveying, for example with auto-targeting theodolites. An example of such a system is shown in Fig. 4-27.

The scanner works with two high-resolution digital cameras (Charge Coupled Device, CCD), which are mounted on a frame and successively record the tunnel wall (Fig. 4-27). The recorded data can be digitally further processed into a digital model of the tunnel support layer. This system offers the following advantages in tunnelling [254]:

– Short recording times.
– Simple handling by one person without special training.
– Robust nature of the system.
– Precise results (± 1cm for 3D images).
– Simple and rapid processing.
– Direct digital further processing.
– Flexible display of results.

The DIBIT scanner (Fig. 4-27) was used in the driving of the Hellenberg Tunnel to record the profile of the shotcrete layer. Since it demonstrated a complete over-profile, it was possible to alter the geometry of the inner lining and save concrete. Fig. 4-28 shows an example of an image of a shotcrete layer recorded by the tunnel scanner.

Further applications ensue from the comparison of various images, for example a shotcrete support layer can be imaged at various time and the deformations can be deduced or the layer thickness can be determined [254].

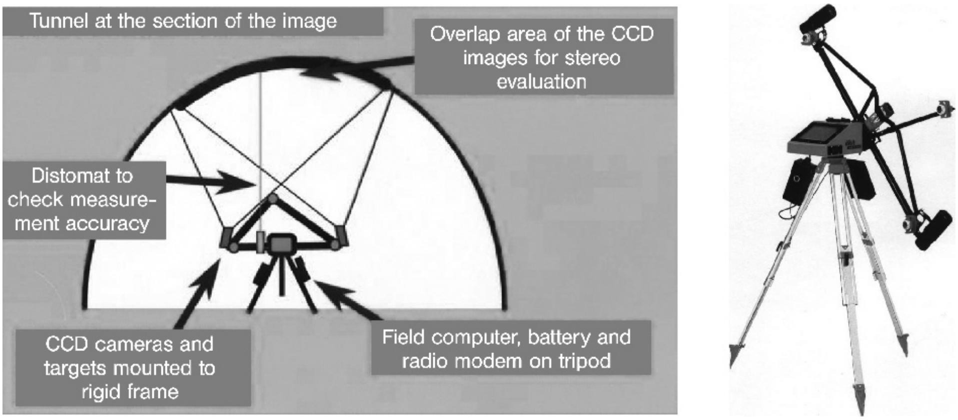


Figure 4-27 Photogrammetric tunnel scanner (DIBIT) [25].

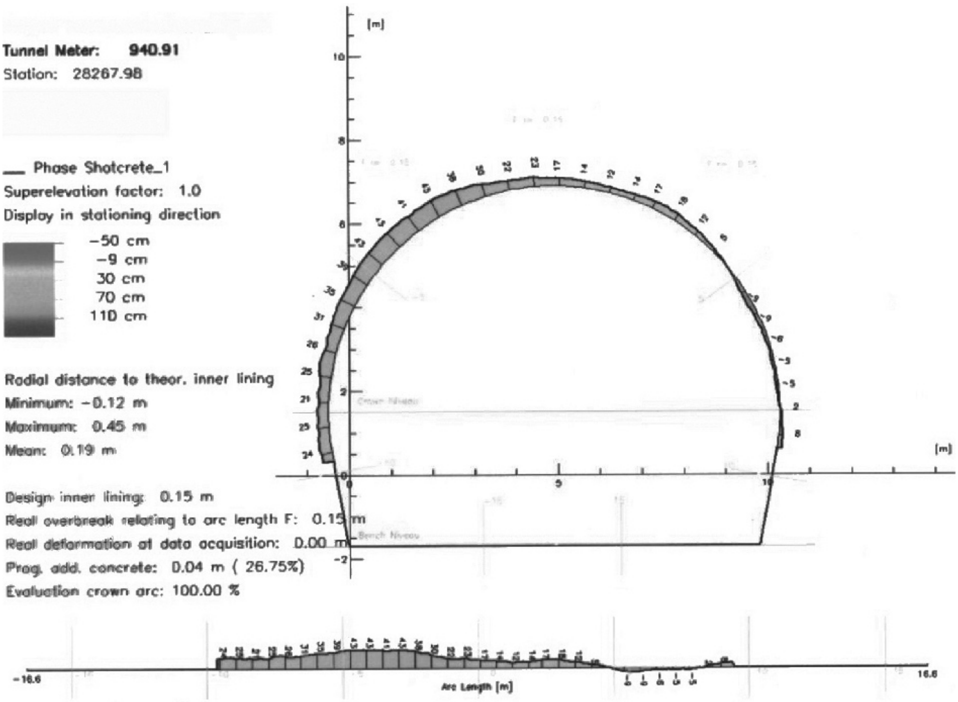
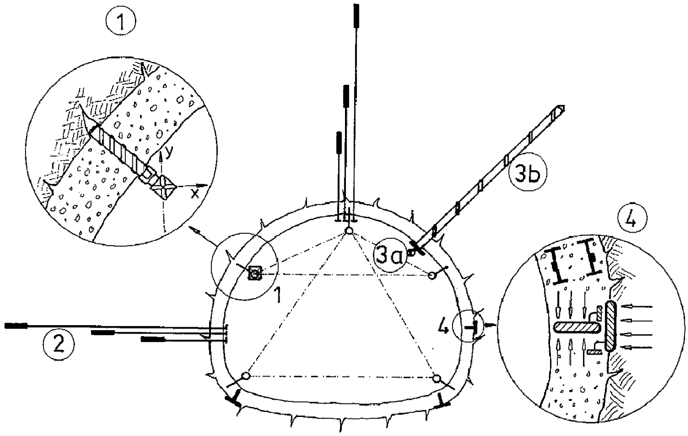


Figure 4-28 Image of a shotcrete layer [25].

4.3.3 Stress and strain measurements in the support layer

Comparison of the actual state with the assumptions in the design calculations, which has already been mentioned in Table 4-2, requires not only the measurement of displacements but also stress and strain in the support layer. This enables a complete mechanical description of the system of tunnel-support-ground. Fig. 4-29 shows the instrumentation normally used for this purpose ("Standard instrumentation for tunnelling") from [24].



1 Measurement of displacements of the supported tunnel perimeter (convergence tape or geodetic distance measurement, see Section 4.3.1.1 or 4.3.1.2)
2 Measurement of displacements in the ground around the tunnel (extensometers or inclinometers, see Section 4.3.1.5 or 4.3.1.6)
3 Monitoring of the anchors (anchor force sensor, measuring anchor, see Section 4.3.4)
4 Monitoring of the shotcrete (pressure cells, vibrating wire sensors, see Section 4.3.3.1)

Figure 4-29 Standard instrumentation for the monitoring of deep tunnels; Interfels GmbH.

4.3.3.1 Radial and tangential stress measurement in concrete

Stress measurements are mainly undertaken in the support, mostly in concrete. There are two basic types:

1. Instruments using the hydraulic principle.
2. Instruments using the vibrating wire principle.

The latter type of system is used in many applications due to its universal measurement principle, for example in extensometers, anchor force measurement and steel strain measurement.

Principle of pressure cells working on the hydraulic principle. The investigated stress σ is compensated by a hydraulic or pneumatic pressure that builds up independently in the sensor and in the supply pipe (Fig. 4-30). This is possible because the sensor is designed as a pressure relief valve, which is loaded and controlled with the investigated stress. The back-pressure p

can be measured at the end of the hose, i.e. outside the concrete, and is equal to the stress σ, apart from a slight pressure less in the supply pipe of about 0.01 N/mm^2 per 100 m pipe length.

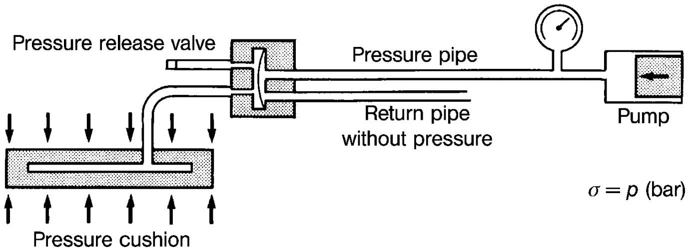
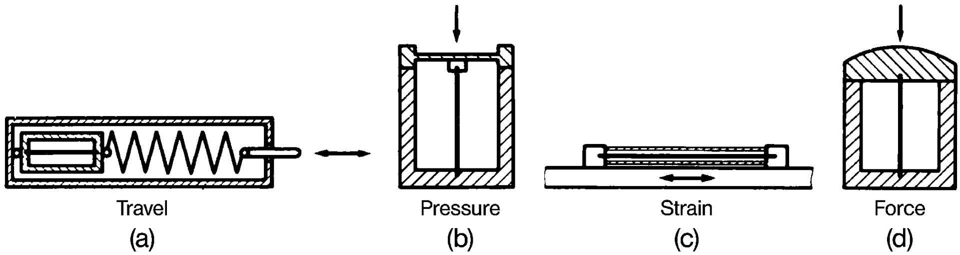


Figure 4-30 Diagram of the principle of a hydraulic pressure cell; Glötzl System, Gesellschaft für Baumesstechnik mbH.

Principle of electrical vibrating wire sensors. Changes of a measured quantity (for example travel, pressure, strain, force) cause changes of strain and thus the resonance frequency of a measurement wire, which is fixed in the sensor but capable of vibrating (Fig. 4-31). The wire vibrates in the magnetic field of an electro-magnet system and induces an electrical oscillation of the same frequency, which is transmitted through a cable to the receiver and there processed to give the measured value. The measuring wire is stimulated by the same electro-magnet system as the receiver. The wire frequency can be calculated according to the formula

$$f = C \cdot \sqrt{\varepsilon}$$

with measurement constant C determined by length, mass and elasticity of the wire. The change of strain ε of the wire is proportional to the change of the measured value and thus determines the frequency change of a constructively fixed sensor.



(a) Used in convergence measuring instruments and extensometers
(b) Used for radial and tangential stress measurement in concrete
(c) Used for strain measurement in steel arches
(d) Used to check anchor forces

Figure 4-31 Basic system of a vibrating wire sensor, System Maihak AG.

Measurements. The two types of measuring instrument can be used both to measure radial ground pressure acting on the concrete layer (contact stress ground-concrete) and to determine the tangential pressure distribution in the concrete layer itself as pressure cell. Pressure cells (vibrating wire process) are installed for this purpose before spraying the shotcrete and are thus lost. Fig. 4-32 shows a typical arrangement of pressure cells for radial and tangential measurements in tunnelling.

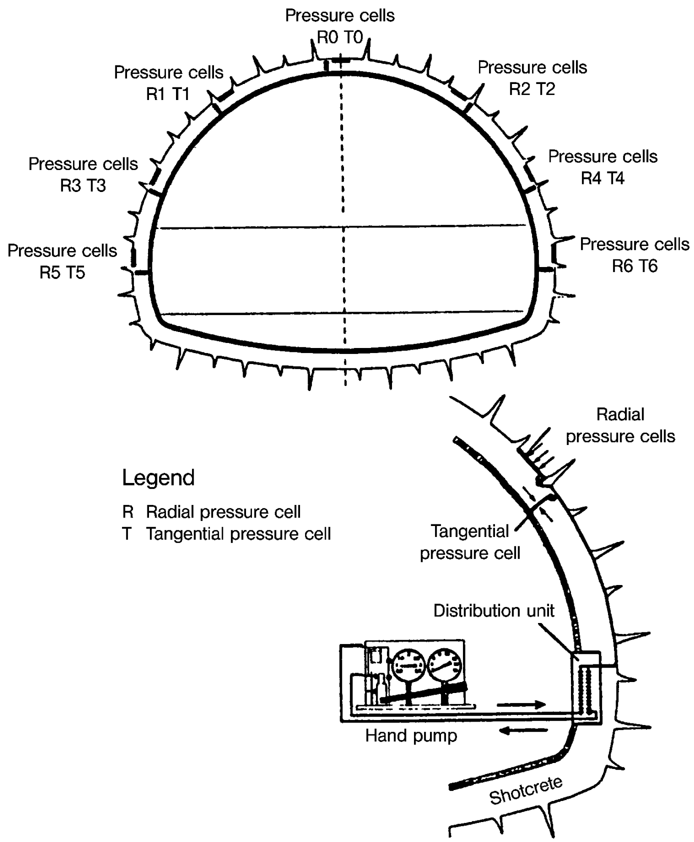


Figure 4-32 Arrangement of radial and tangential pressure cells (Geodata GmbH).

Stress measurements in concrete and at the interface between ground and concrete only deliver limited information, which encounters the following difficulties of interpretation [69, 119, 161, 221]:

- Radial pressure is only measured at points.
- Heart of hydration, creep and shrinkage also cause changes of stress, which have to be recognised and eliminated in the interpretation.
- The thickness of the shotcrete layer can vary around the perimeter of the tunnel.
- Any bending stresses, which may occur, cannot be reliably recorded due to the variable thickness of the shotcrete layer and the variable location of the neutral axis.
- The measurement range of the devices has to be decided before installation. Some information for this purpose can be derived from the structural calculations. If these forces do not occur or are exceeded, the sensor will be unusable.

4.3.3.2 Measurements in steel arches

Strain measurement using vibrating wires. The sensors work on the principle already described in Section 4.3.3 (Fig. 4-31). In order to be able to calculate the stresses at a location from the strains, a device must be bolted or welded onto the arch externally and internally. The external device is covered by a cap and sprayed with shotcrete and thus cannot be reused.

Such systems have become less popular in recent years.

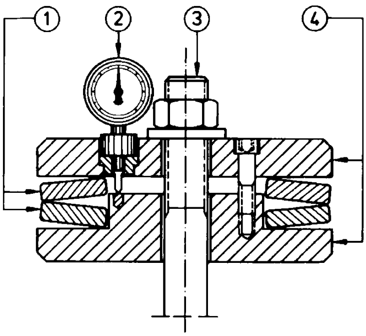
4.3.4 Measurements of the loading and function of anchors

4.3.4.1 Checking of anchor forces in unbonded anchors

To check the force originally applied to an unbonded anchor and its change depending on events at the installation location and with time, sensors are installed at the anchor head. The following systems can be used:

- Mechanical measurement plates.
- Vibrating wire devices.
- Hydraulic measurement process.
- Load cells with strain strips and inductive sensors.

Measuring plate. This method of measurement is based on the squeezing together of two load-distribution plates integrated into the anchor head, which have calibrated plate springs between them. The effective anchor force can be determined from the change of length of the plate springs through the strain ε and the stress σ [195]. Fig. 4-33 shows a diagram of the function.



1 Plate springs
2 Portable reader
3 Tightening anchor with nut
4 Load distribution plates

Figure 4-33 Checking anchor forces with a mechanical measuring plate; Interfels GmbH.

Vibrating wire sensor. In a vibrating wire sensor, the axial anchor force to be measured is introduced into a ring-shaped pick-up, where it produces an elastic deformation of the measurement body and thus changes the resonance frequency of the installed steel wire. In the ring-shaped measurement body, which is fixed to the anchor head, the introduced axial force is distributed to three, or for higher measurement ranges six sectors, each of which has a vibrating wire measurement system. The sum of the individual values gives the total force (Fig. 4-34).

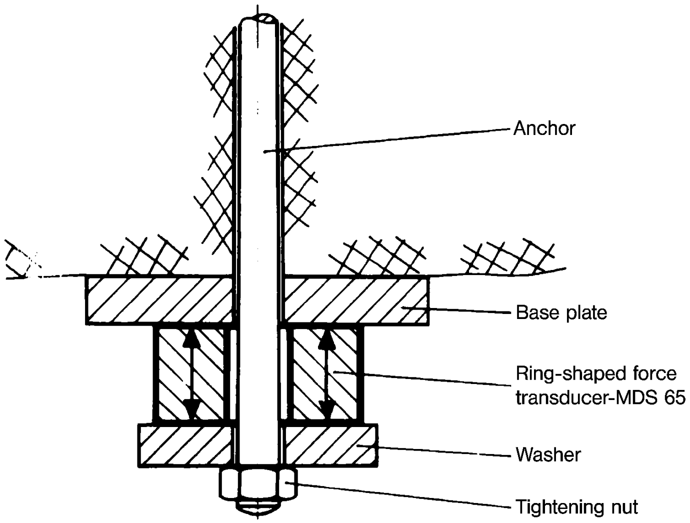


Figure 4-34 Checking anchor forces with a vibrating wire sensor; Maihak AG.

Hydraulic valve sensor. Fig. 4-35 shows a section through a hydraulic anchor force measuring system (Glötzl-Solexperts). The equipment permits the removal of the force sensor without unloading the anchor and can therefore be used any number of anchors. The anchor force itself is transferred through an external anchor nut and a spreader plate to the load cell.

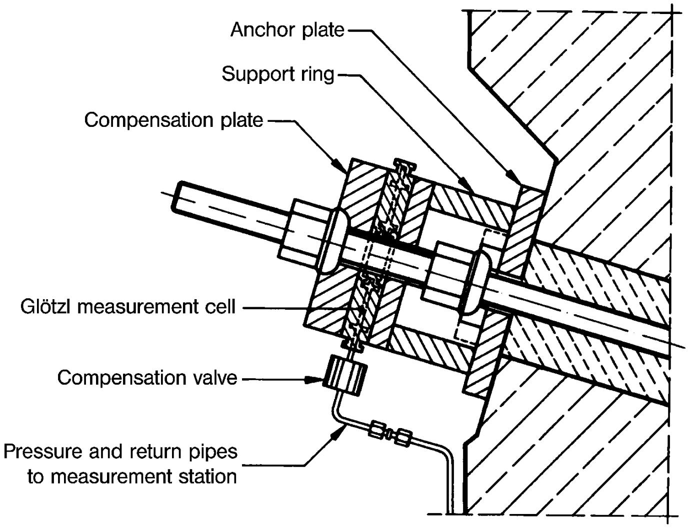


Figure 4-35 Checking anchor forces with a hydraulic valve sensor [243].

Example. The following example shows the evaluation and development of anchor forces during a tunnel drive.

In order to measure anchor forces during the excavation of an investigation heading for the Engelberg Base Tunnel on the Heilbronn – Stuttgart autobahn in Germany, a 4 m long unbonded anchor was used as shown in Fig. 4-36. Anchor I was set after the completion of the first partial excavation and – just like anchors II and III set in the subsequent phases of excavation – prestressed with 30 kN. Its anchor force increased as a result of the excavation of the second phase by 55 to 85 kN, and the opening of the third partial excavation led to a further increase of load to altogether 93 kN. Anchor II, which was set and prestressed in the second phase, took an additional load of 33 kN, which increased in the subsequent fourth phase of excavation once again by 7 to altogether about 70 kN. Anchor III, after an initial decrease of loading, also reached its maximum load of 40 kN in the fourth phase, while anchor I experienced a continuously decreasing load in this phase. The load on the other anchors only started to decrease after the invert was closed; the final state was then reached after about 22 d: anchor 1 was the highest loaded with 30 kN more than the prestressing force, altogether 60 kN, anchor II maintained approximately its pre-stressing force of 30 kN and the force in anchor III decreased to 10 kN.

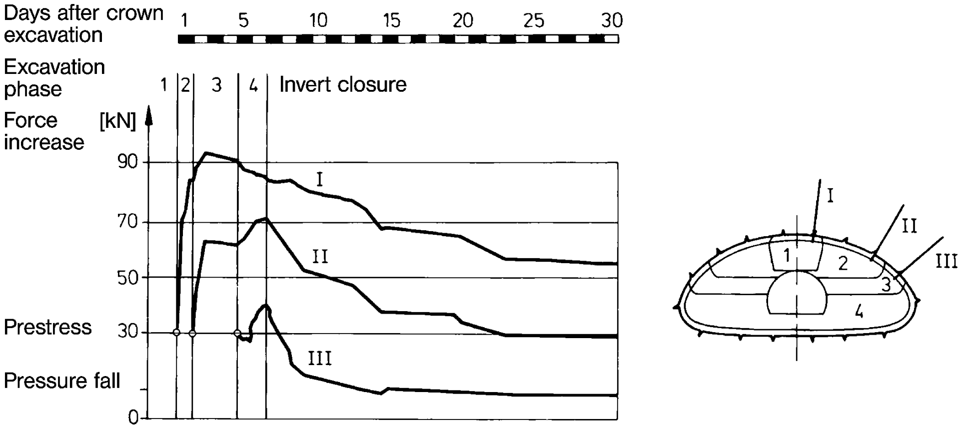


Figure 4-36 Log of anchor forces on the site of the Engelberg Base Tunnel (autobahn Heilbronn – Stuttgart).

4.3.4.2 Checking of anchor forces with mechanical measurement anchors

While the anchor force can be measured with a sensor at the anchor head, mechanical measurement anchors can determine the function of the anchor in detail, with the curve of loading against depth and time. They correspond to the SN anchors used for pattern bolting and have the same cross-sectional area with the difference that they are hollow in the middle; inside their hollow shafts, measurement rods are fixed to the anchor shaft at various depths. The length changes between the fixed points and the anchor head are measured (Fig. 4-37). Mechanical measurement anchor 1 contains several partial lengths I to IV, with extensometer points 4 to 7 fixed at their ends in the internal bore. The extensometer rods 8 to 11 from the extensometer points to the measurement stop 2 permit the measurement of the change of the partial length under load and thus the determination of the force acting there [87]. The difference of the displacements between the individual fixed points is a measure of the loading of the anchor in the individual sections; these figures can be converted to stress since the E modulus is known. It has to be considered that the results can be falsified, for example by bending. The 2nd derivation of the curve of the displacements gives the shear transfer between anchor and rock mass, from which the action of the anchor can be estimated.

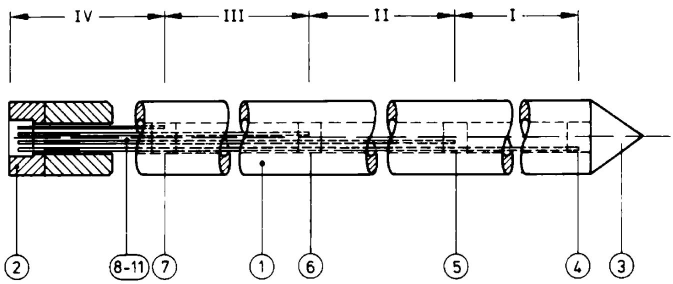


Figure 4-37 Mechanical measurement anchor from Interfels GmbH (see legend in text).

Example. The loading on the anchors used in the construction of the Landrücken Tunnel were determined by mechanical measurement anchors. The results are shown in Fig. 4-38.

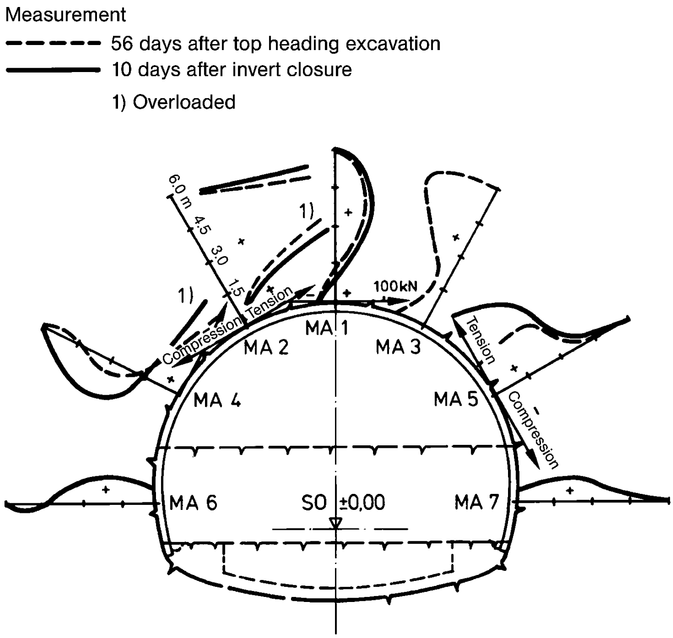


Figure 4-38 Readings from mechanical measurement anchors for the determination of the anchor loading at the Landrücken Tunnel (measurement anchors MA 1 to 7).

4.4 Geophysical exploration ahead of the face

Difficulties during the driving of a tunnel are normally caused by unexpected changes of geology. This makes advance probing ahead of the tunnel ever more significant. The already known geophysical methods have recently been developed further. The basic methods and the associated geophysical parameters are summarised in Table 4-3.

Investigations in recent years have demonstrated that none of these methods should be used independently since even slight differences in the individual properties of the rock or rock mass and measuring inaccuracy can lead to faulty interpretations of the measured results. One particular problem remains calibration and interpretation. Promising are methods, which combine two or three geophysical methods. The following section briefly describes the function of the individual methods.

Table 4-3 Geophysical investigation methods and the associated petrophysical parameters [231].

Geophysical method	Petrophysical parameter
Seismic	propagation of seismic waves
Geoelectrical	specific, electrical resistance
Gravimetric	density
Magnetic	susceptibility
Radar	relative permittivity (dielectric constant)
Geothermal	specific thermal conductivity

4.4.1 Seismology

Applied seismology investigates the propagation of artificial waves. The propagation velocity of the waves, the absorption coefficient and the sound resistance are the parameters used for evaluation. After an initial calibration, conclusions can be drawn about the rock. The following procedures are available:

– Direct determination of elapsed time.
– Refraction seismology.
– Reflection seismology.

Reflection seismology is particularly suitable for tunnelling because in this case transmitter and receiver are at the same location. The direct determination of elapsed time, on the other hand, requires additional boreholes from the surface or another already existing cavity in the vicinity of the tunnel.

4.4.2 Geoelectrical

Geoelectrical methods such as electrical resistivity tomography (ERT) or electrical resistivity imaging (ERI) investigate the spatial distribution of the electrical properties of the ground, using either natural electrical fields or with artificial supply into the ground. The rock parameters that can be measured are the electrical conductivity, resistance and relative permittivity (dielectric constant). Above all radar is suitable for advance probing, which also works on the reflection principle.

4.4.3 Gravimetric

Gravimetric methods can be used to detect regional of local anomalies in the acceleration due to gravity from normal values. If there is a sufficiently large difference to the surrounding rock, the presence, contour and depth of geological structures can be determined and conclusions made about anomalous geotechnical situations.

4.4.4 Geomagnetic

Geomagnetic measurements can be used to record regional or local anomalies of the Earth’s magnetic field and thus reach conclusions about the structure, contour and depth of geotechnical situations.

4.4.5 Geothermal

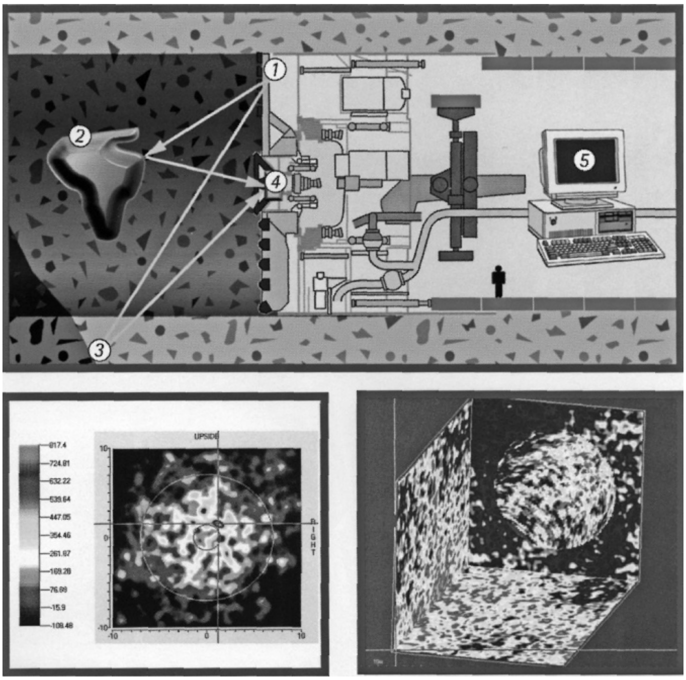
Geothermal methods use temperature measurements on the surface, underground and in boreholes to describe the temperature distribution with place and time. It includes the study of natural and artificial sources of heat, heat transfer processes inside the Earth and geothermal parameters of rocks and rock mass to enable the consideration of mineral deposit processes and structural features.

4.4.6 Examples and experience

There now follow two practical examples of the scope of application of geophysical methods.

4.4.6.1 Probing with SSP (Sonic Softground Probing)

Despite intensive geological investigations before the start of tunnelling, unexpected obstructions along the tunnel alignment cannot be fully ruled out before starting. Since these obstructions are a particular problem for a shield tunnelling machine, systems have been developed to probe the ground in front of the cutting wheel. The SSP advance probing system (Fig. 4-39) developed by Herrenknecht can display contrasts of density in the ground up to 40 m ahead of the machine, which enables the detection of any obstructions in the ground. Ultrasound transmitters mounted on the cutting wheel send signals, which are differently reflected according to the properties of the ground.



1 Transmitter
2 Boulder
3 Strata boundary
4 Receiver
5 Display

Figure 4-39 Diagram of the SSP system from Herrenknecht and display on the monitor [91].

The reflected signals are received by special microphones and then evaluated fully automatically. A three-dimensional profile of the ground is calculated from the signal elapsed times and amplitudes and displayed for the shield driver in the form of a colour 3D graphic display. When the system was tested on tunnel drives (4^{th} bore of the Elbe Tunnel, Pannerdenschkanal Tunnel), however, it turned out that support by a specialist was essential for the evaluation and interpretation of the complex data and graphics, and clear results are not always achieved. The use of the gathered data and graphic displays by the shield driver as the tunnel advances is not yet fully practical. This innovative technology will have to be developed further and adapted with the experience from current projects.

4.4.6.2 Probing karst caves

Tunnels in karst regions and areas susceptible to sinkholes are a great challenge for employers, consultants and contracting firms, since caves are often difficult to locate. The following example from the new DB line Nuremberg – Ingolstadt illustrate the combined use of direct (for example boreholes) and indirect (geophysical) methods of probing for karst caves.

Effect on the tunnel of karst features. Tunnels are a composite structure of rock mass and lining. Both load-bearing elements are affected by karst features (Fig. 4-40), with stress distributions that are very different from tunnels in unaffected areas.

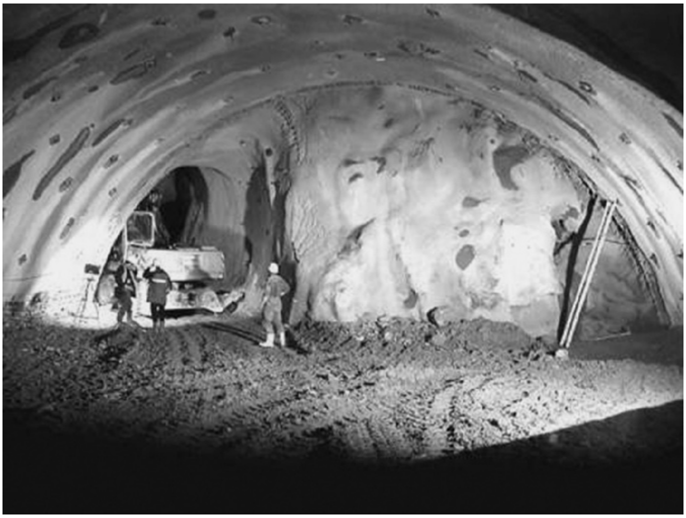


Figure 4-40 Karst cave, Irlahüll Tunnel (source: site photo BÜZ Mitte).

The rock mass is an essential load-bearing element in tunnel construction and its stability is a decisive factor for the verification of structural safety. The effects of karst features can essentially be differentiated according to their location in the side, invert or crown [154]. When karst features are present at the sides, a pillar of rock remains between the karst anomaly and the tunnel structure. The highest stress occurs in this rock pillar, whose stability determines the overall stability of the tunnel cavity. The decisive point for the evaluation of structural stability is thus not local peaks of stress but the overall load-bearing behaviour of the rock pillar. This is done by determining the average loading factor in a horizontal section through the pillar. This represents the ratio of the actual stress to that which can be borne using the Mohr-Coulomb failure criterion (Fig. 4-41). If a karst cave is below the tunnel, the rock body suspended between tunnel and cave is loaded by its self-weight, the tunnel lining and live loads from rail operation. If its thickness is insufficient, a shear or bending tension failure can occur.

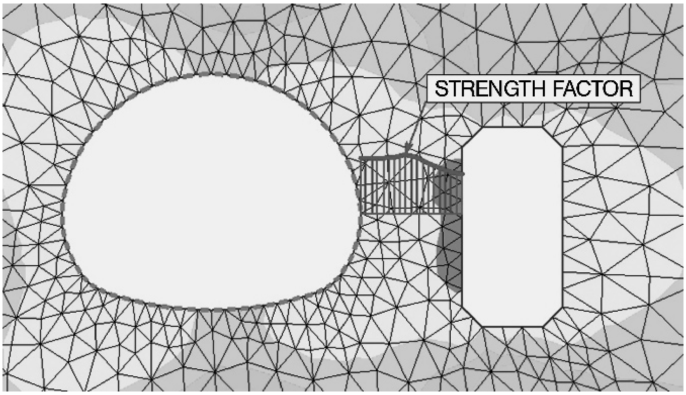


Figure 4-41 Loading on the rock pillar.

In addition, the stress state in the lining can also be affected by karst anomalies. If the outer support layer fails, the load that it originally resisted is transferred to the rock mass and the inner lining. Karst structures cause asymmetric loading and altered subgrade reaction conditions (Fig. 4-42).

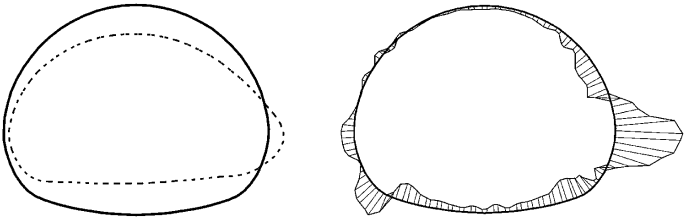


Figure 4-42 Deformation of the tunnel lining and curve of bending moments with a karst cave beside the tunnel.

In order to quantitatively evaluate the influence of a karst cave on the inner lining, the location of karst structures must be known. Karst anomalies have to be detected by various methods of investigation and their geology described. This information can then be used to verify structural stability including consideration of the identified feature.

Small karst features can only be detected by indirect and direct methods at relatively high cost, which may even not be technically feasible. It is therefore economical and technically more sensible to use a generalised, overall approach to the performance of the verification of structural safety and serviceability. The limit line for the investigation of individual features is any cave or anomaly, which causes an increase of the maximum edge stress in the inner lining of more than 10 % compared to a location without karst features. From the effects that have already been described and the selected limit line, it is possible to derive the size of the caves or voids that have to be investigated depending on the distance from the tunnel (Fig. 4-43) and thus the specification for the site investigation in order to ensure permanent structural and operational safety of the tunnel.

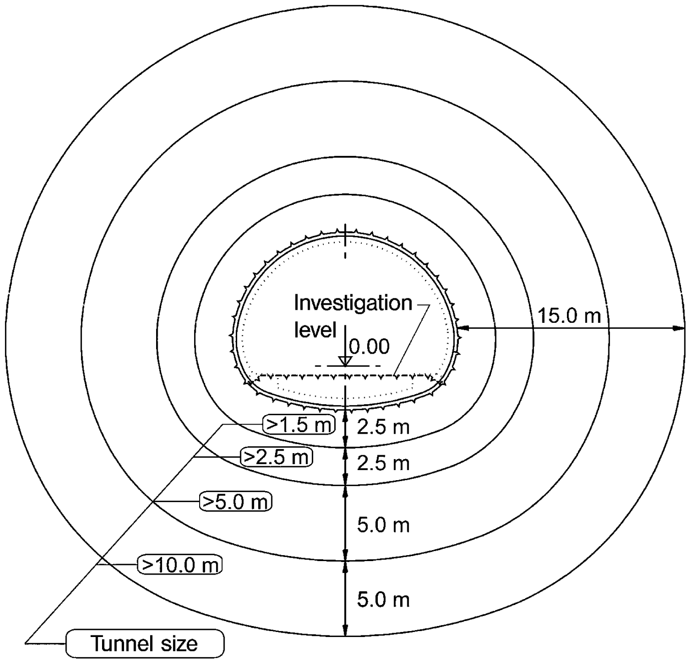


Figure 4-43 Specification for the investigation of karst features / anomalies.

Investigation concept. The investigation required to provide an adequate description of the ground is carried out in stages [181] (Fig. 4-44), with the investigation before the start of construction and the investigation before the construction of the inner lining being differentiated. The objectives of investigation are based on this concept:

The first phase of investigation before the start of construction, and the tunnelling concept derived from it, enable the tunnel to be constructed safely. Using field investigations, mapping of karst morphology and historical research, scenarios are produced of relevant dangers to the construction works. In order to secure the tunnel drive against caves in the immediate surroundings of the tunnel, angled holes are drilled (Fig. 4-45).

It can be sensible for shallow tunnels to use geophysical methods coupled with drilling in the first phase of investigation in order to provide full coverage for the geological model. These investigations serve to better determine the model of the rock mass or improve it further.

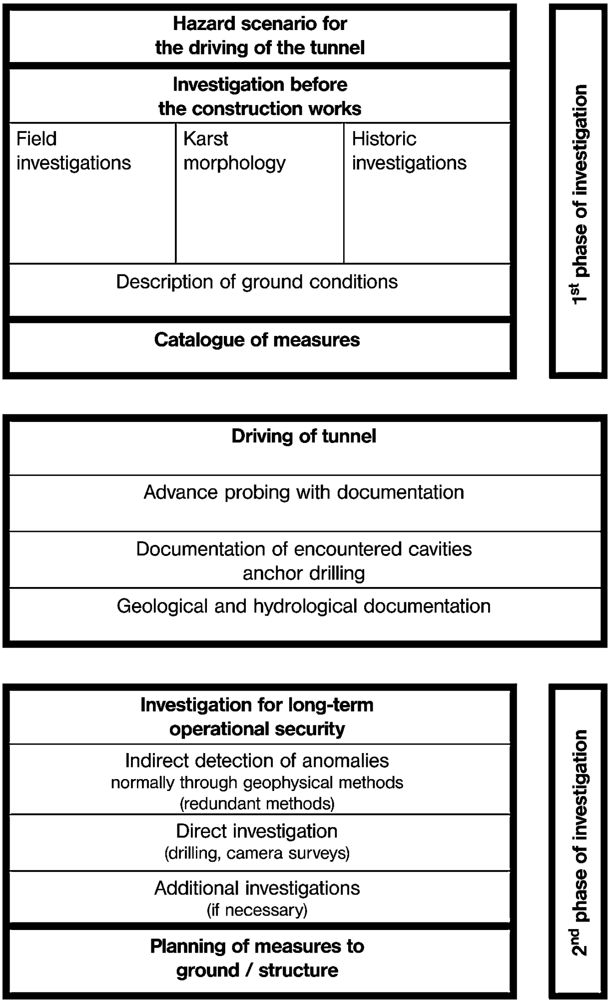


Figure 4-44 Staged concept.

The result is to detect the shape of the rock surface as it affects tunnelling, weak zones and anomalies in the rock mass. The resolution capability of geophysical investigations falls with increasing depth and in heavily faulted zones. Deeper karst features, which are associated with tectonically stressed zones on the rock mass, are therefore often difficult to detect with geophysical methods.

The second phase of investigation has the objective of investigating karst features in more detail so that appropriate measures can be derived to ensure the permanent operational safety of the tunnel. In addition to the geological-hydrogeological documentation during the driving of the tunnel, probe holes drilled during the drive in the profile and the invert and drilling for systematic anchoring should be documented and evaluated so that larger caves are directly explored or caves in the immediate vicinity of the tunnel can be ruled out. Additional investigation is necessary to discover any anomalies outside the tunnel, which could have an effect on operational safety.

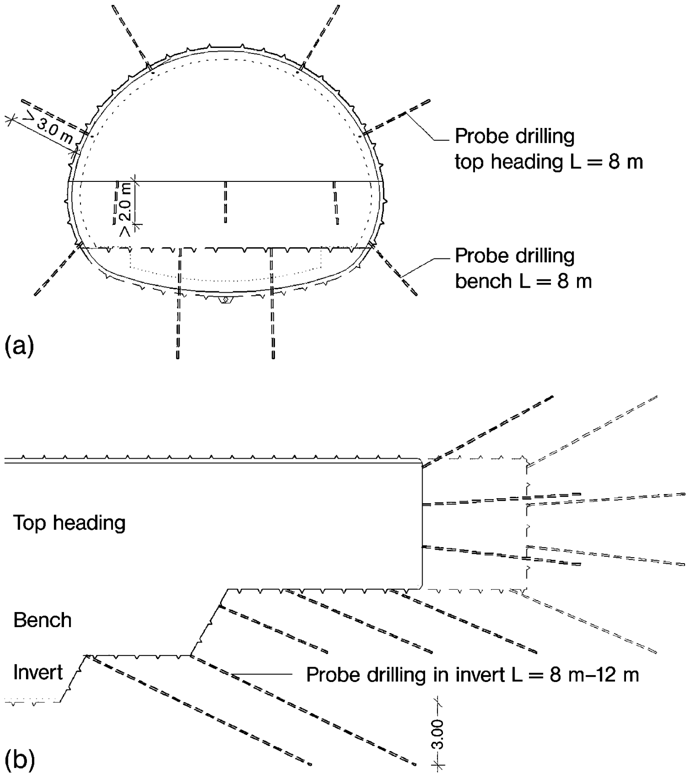


Figure 4-45 Plan of probe drilling immediately in front of the advance.

In order to decide the aims of further investigation, not only the information gained during the driving of the tunnel is used but also numerical parameter studies to evaluate the relevance of anomalies for the structure. If an overall approach is used to consider anomalies, then it is assumed in the verification of structural stability that they could be present at any location in the surroundings of the tunnel and reach the limit line size for anomalies to be detected according to the specification.

The specification of the limit line sizes is a technical and economic decision, which has to consider:

- The distance of the anomaly from the tunnel and the location relative to the tunnel.
- The cost of direct and indirect methods, which increases over-proportionately with decreasing limit line size.
- Technical constraints for the application of various methods.
- Technical limitations of various direct methods under the given local conditions: geology, vibration etc.
- Cost of overall coverage of the influence of undiscovered voids, which lie below the limit line size.

For the second phase of investigation, indirect geophysical methods in combination with direct methods are practical. Indirect methods are used to provide complete coverage of the entire area of the tunnel and deliver suspicious areas (anomalies), which are indirect indications of karts features (Fig. 4.46). Each of these areas of suspected anomalies is explored by direct methods, and the type and characteristics of the feature are determined and evaluated for their structural relevance.

The geophysical methods mentioned in Section 4.4 have proved suitable as indirect investigation methods. A geophysical method should be selected according to the geological-hydrogeological conditions, the constraints such as implementation of the geophysical procedure after the installation of the reinforced outer support layer of the tunnel, and the aim of investigation. When the risk potential is high, it is sensible to implement two different geophysical principles as complementary/redundant.

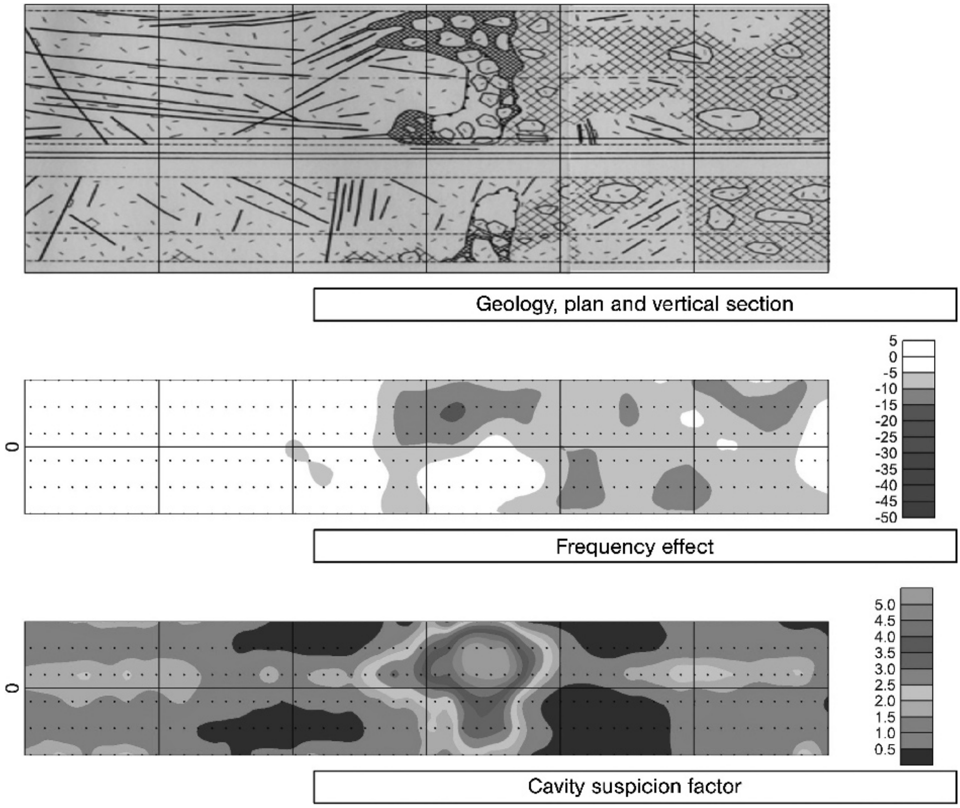


Figure 4-46 Geological documentation during tunnelling and graphical result of geophysical investigations (frequency effect according to BEAM and seismic cave suspicion factor) near an open karst cave [184].

During the performance of geophysical investigations, the conditions encountered in the tunnel should be continuously checked to examine whether they restrict the capability or reliability of the measured results and whether the preconditions for the measurements intended in the design are fulfilled and whether the achievement of the specification is guaranteed. For example, anchors and metal support elements can have a significant effect on the result of methods based on electrical principles, and seismic results can be affected by nearby construction operations.

In order to ensure an application of geophysical methods that is satisfactory for all contract parties, the performance capabilities and technical limits of the method used must be known in advance and an appropriate specification decided. One essential factor is the resolution capability and thus the size of features, which can be detected geophysically depending on distance. Fig. 4-47 shows the size of the anomalies that can be detected by a geoelectrical process [104]. This limit line has been determined empirically and takes into account the current geology. As long as the geology is homogeneous, the limit line is lower so that smaller anomalies can be detected, and vice-versa. The reliability of the described process is negatively influenced by metallic support elements, which have no contact to the reinforcing steel in the outer support layer, so the limit line rises.

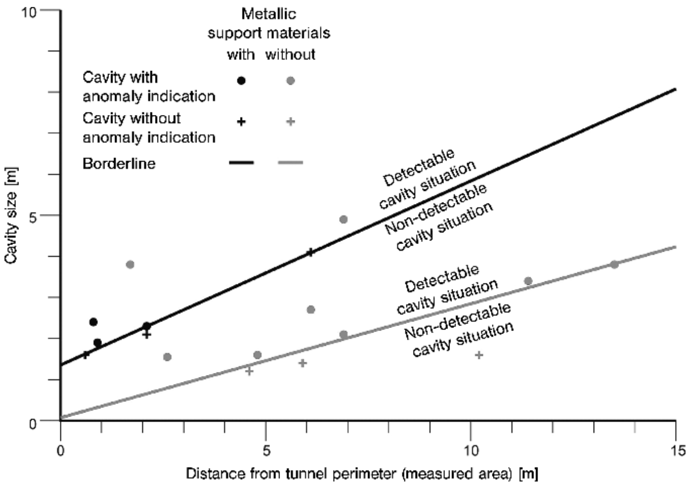


Figure 4-47 Limit line of detectable caves and voids (determined from experience).

With seismic methods, the lateral and vertical resolution and the resolution of individual features depend on the signal frequency and the propagation velocity of the waves, the bandwidth of the signals and the depth of exploration. The lateral resolution is determined by the size of the Fresnel zone and the distance of the underground scanning. Guideline values for the lateral and vertical resolution capability at various depths, assuming a variable velocity distribution, are compared in Table 4-4 independent of frequency.

Table 4-4 Seismic – resolution capability.

Depth interval [m]	Wave velocity [ms]	Signal frequency [Hz]					
		50	100	150	50	100	150
		Resolution capacity [m]					
		vertical			lateral		
< 25	1500	3.8	1.9	1.3	6.9	4.8	3.8
25 to approx. 40	2000	5	2.5	1.7	10	6.9	5.5
40 to approx. 100	3000	7.5	3.8	2.5	18.9	13.1	10.7

In order to interpret the results of direct and indirect methods, the results from the first phase of investigation like historical research, aerial photos, geological documentation and other knowledge should also be considered.

Tunnel with pilot heading. The investigation stages and the objectives of investigation are the same as for tunnelling without a pilot heading. One advantage of a pilot heading is that its smaller cross-section has to make it easier in principle to overcome any unexpected karst features than the larger section of the final tunnel.

It is advantageous to place the pilot heading inside the section of the final tunnel. Considering the small size of a pilot heading and the resolution of the geophysical process, which decreases with increasing distance, and the costs which rise over-proportionately with increasing investigation depth, it should be considered in each case whether the pilot heading can already deliver sufficiently accurate investigation results for the final structure. Experience shows that investigation and treatment performed from the pilot heading cannot rule out further undiscovered anomalies being found when the main tunnel is driven. This requires additional investigation and treatment measures as the main tunnel advances.

In cases of heavy karstification, which demand correspondingly extensive ground improvement measures, economic considerations could also argue against complete advance ground improvement from the pilot heading, if for example extensive drilling and grouting works have to be carried out in the restricted space of the pilot heading.

4.5 Monitoring and evidence-gathering measures for tunnelling beneath buildings and transport infrastructure

4.5.1 General

Tunnelling underneath buildings and roads and railway lines is subject to additional monitoring requirements and normally demands a survey of the existing situation to secure evidence. The condition of buildings and transport infrastructure is mostly determined by an independent expert before the start of the works to gather evidence and after completion. In order to restrict damage, the effect of tunnelling on buildings is monitored and immediate measures are undertaken when the deformations or forces specified in advance are exceeded. The following section describes the procedure proposed by N. Klawa [109].

4.5.2 Monitoring and evidence-gathering measures

The condition of the building is established in a protocol with exact recording of existing cracks and damage (drawings, photos). Patches of plaster are applied to larger cracks or joints to indicate movement. The gathering of evidence and monitoring of building settlement during the driving of underground tunnels is normally performed with levelling bolts on corners of buildings above the tunnel and adjacent buildings, possibly also with one or more intermediate points on walls. Settlement of the ground surface between buildings (along and across the tunnel axis) is also surveyed, since the movement of buildings and the ground surface are not identical. From the difference between the settlement of buildings and the ground outside, it is possible to reach conclusions about the stresses on the building. All levelling points should be levelled before the start of construction from settlement-free and reliable bench marks with closed precise levelling (reading accuracy 0.1 mm) to determine the zero levels.

The systematic observation of building and ground levels has the following objectives in addition to the determination of the absolute settlement plan:

- Determination of the effect on settlement of each individual phase of construction.
- Extrapolation of the curve of movements with time, the nature of their consolidation and the probable final results.
- Early determination of dangerous situations in order to be able to introduce any necessary changes to the construction process or support measures.

The frequency of measurements is based on the progress of the tunnel advance. Observation intervals of about one day are normal above the tunnel advance. After the tunnel has passed by, typical measurement intervals are one to three months. Measured settlements of buildings above the tunnel are displayed graphically for evaluation. If the tunnel passes beneath a densely built-up area, a settlement plan (showing lines of equal settlement) is an important aid to comprehension because not all the buildings in the area of the settlement trough can be provided with levelling bolts. Other common means of displaying the results are time-settlement curves for individual points with details of the working phases and diagrams of settlement against time in transverse and longitudinal profiles.

4.5.3 Noise and vibration protection

When the tunnel is shallow and the requirements for noise and vibration protection are stringent, special measures may be necessary for tunnelling beneath buildings, for which trackbed mats or mass-spring systems are mostly used. This is dealt with in detail by H.W. Koch [111].

4.5.4 Permissible deformation of buildings

When an underground tunnel is driven below an existing building, the danger results not only from linear settlement but above all from the shape and extent of the settlement trough, which itself is influenced decisively by the stiffness of the building. The settlement of buildings and the ground outside are normally not identical (Fig. 4-48).

The magnitude of the permissible settlement difference or the permissible angular distortion depend on the material and construction of the building and the nature of the tunnel alignment (for example central or at the side). Buildings can generally survive larger settlements than can be verified through calculation. Fig. 4-49 shows permissible deformations according to various authors [207] for conventional buildings with masonry with reinforced concrete floor slabs. The values vary widely, and it should also be borne in mind that they only apply to trough locations. Experience shows that hogging locations lead to worse loading on the building and thus to cracking damage at considerably smaller settlement differences.

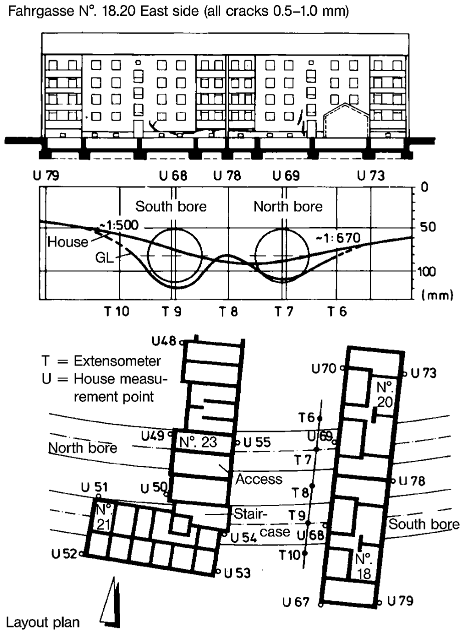
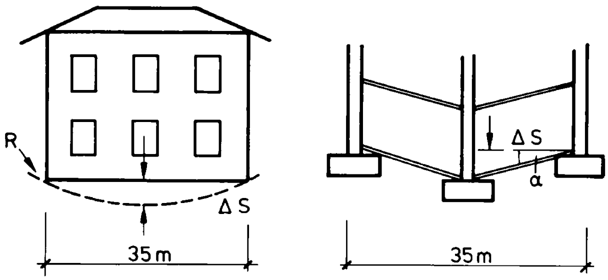


Figure 4-48 Settlement and damage to the east side of the building at Fahrgasse No. 18/20 in Frankfurt, the result of a shield machine passing beneath [38].



		R_{km}	ΔS_{cm}	$\tan \alpha$
Terzaghi	1948	2.5-6.0	5.0-2.0	1/350 -1/875
Leussink	1954	2.5-6.0	5.0-2.0	1/350 - 1/875
Russian standards	1955	1.0-2.5	12.0-5.0	1/146 - 1/350
Meyerhoff	1955	2.2	6.0	1/292
Skemton	1957	2.7	4.0	1/438
Rausch	1955	4.0	3.0	1/583

Figure 4-49 Permissible deformations of masonry buildings with reinforced concrete floor slabs located in a settlement trough; curvature radius *R*, settlement difference *S* and angular distortion *a*, according to J. Schmidbauer [207].

For reinforced concrete or steel framed buildings, the effect of a trough or hogging location is not so serious since these materials can be stressed in tension as well as compression. J. Kramer [116] gives the following damage limits for reinforced concrete framed buildings depending on the angular distortion tan α:

- No damage up to 1/1000.
- Damage possible from 1/1000 to 1/600.
- Slight architectural damage predominant from 1/600 to 1/400.
- Medium architectural damage predominant from 1/400 to 1/300.
- Heavy architectural damage predominant from 1/300 to 1/200.
- Slight to medium structural damage predominant from 1/200 to 1/150.
- Heavy structural damage predominant from 1/150 to 1/120.
- Collapse or necessary demolition predominant from 1/120 to 1/90.

Fig. 4-50 shows the angular distortion with the damage limits depending on the degree of damage in percent of the newbuild cost of the damaged building entered as a curve. The curve shows that architectural damage can be repaired relatively cheaply, but that costs increase in leaps and bounds for angular distortions of 1/250 to 1/200. Regarding the given values, it should be considered that angular distortions of 1/400 to 1/500 are assumed as a result of settlement during and after the building of a reinforced concrete framed building. If a 1 cm settlement difference occurs at a spacing of 5 m, for example, then the critical boundary between architectural and structural damage has already been reached. A forecast or analysis of damage therefore also has to include consideration of loading that has already occurred and possibly also that, which is to be expected. The following general points apply to the behaviour of buildings subjected to settlement:

1. A free-standing single building reacts more favourably to a tunnel being driven beneath it than a structure or wing of a building connected to adjacent buildings. The common junction of two buildings can cause structural damage due to the different settlement behaviour of the two buildings.
2. If the tunnel does not pass beneath the buildings at right angles, then this causes not only a skew position but also a distortion, which worsens the risk of damage.
3. If two adjacent tunnels are driven one after the other, then the buildings on the surface can be forced into a "swing movement".

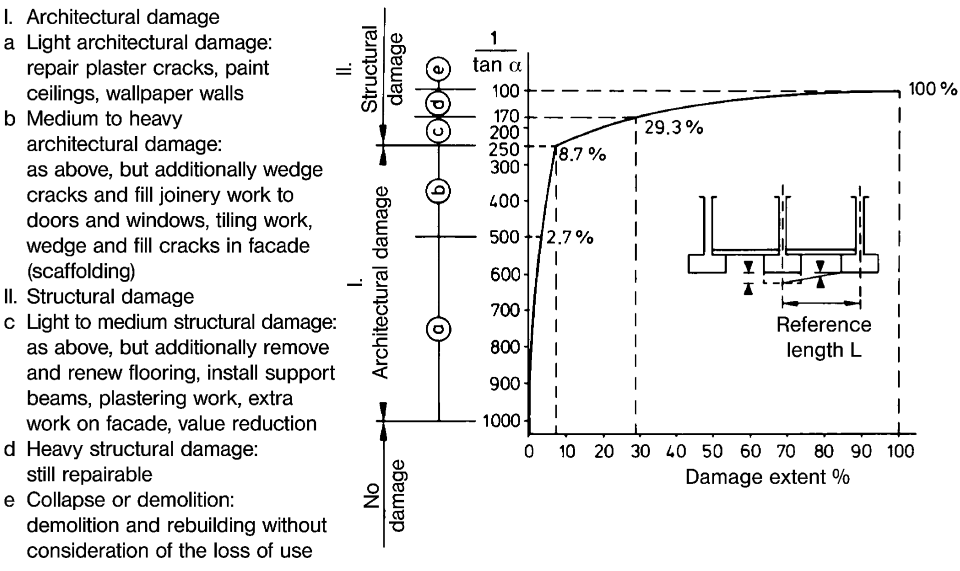


Figure 4-50 Damage to reinforced concrete framed buildings due to the effects of settlement [80].

Permissible deformation of transport infrastructure. The settlement difference or the angular distortion are also of great interest for roads and railway lines. While the permissible magnitudes are not of significance for roads, railways can be much more sensitive, depending on the permissible maximum speed. Particularly when a tunnel passes close beneath a railway line, correction of this settlement, for example by packing underneath, is only possible with difficulty. Locations with softer subgrade reaction can have a considerable impact on rail traffic.

5 Dewatering, waterproofing and drainage

5.1 General

Groundwater has always been one of the most challenging problems for engineers in the history of tunnelling. The protection of structures against damp or water ingress is of great technical significance since it is critical for the survival and function of a tunnel.

During underground construction work, water ingress from the disturbed ground has to be expected. Water has an influence both on construction works and the operation of the completed tunnel. The removal of groundwater through practical dewatering remains one of the most important tasks in the construction of underground structures in order to avoid dangerous water inflows and enable quick and safe tunnel advance with as little obstruction as possible.

For the protection of the structure, not only the ingress of water but also its aggressiveness is significant, since concrete and steel – the most common materials used in modern tunnel construction – can be attacked and destroyed by the aggressive content of the water.

Due to the great significance of groundwater, tunnellers have long devoted their attention to dealing with water (Fig. 5-16, page 209). Much of this knowledge now seems to have faded. Only in recent years has the subject again been paid more attention due to the many appearances of damage. The measures undertaken to deal with the problem have also become more varied.

5.2 Dewatering during construction

In order to be able to plan the nature and extent of dewatering measures in advance, geological, hydrological und chemical investigations have to be undertaken to determine as completely as possible the quantity and properties of water ingress to be expected. The water that enters the tunnel as it is excavated then has to be collected and drained away until the measures intended to deal with water the completed tunnel are functioning.

5.2.1 Water quantity and difficulties

Water can be present as groundwater, groundwater, slope water or in the joints and only investigation of the ground to gain detailed knowledge about the conditions can enable a rough forecast in what form and in what quantities the water will enter the artificial cavity, that is the tunnel.

The hydrostatic and flow pressure effect, the dissolving and rock conversion effect and aggressiveness (in the acid range, pH < 6.5) are all important factors. It can be stated in general that flowing water is always a greater problem than blocked, static water. The feasibility of a construction project and its constructional operation methods may also be influenced by the effect of water on the construction works.

5.2.1.1 Water flow in the ground

In order for water to penetrate underground, connected cavities are necessary, which can be differentiated into pores in soil or joint gaps in rock. Accordingly, tunnels in loose ground or fractured rock will normally encounter groundwater or groundwater and tunnels in solid rock will encounter joint water.

Aquifers and aquitards. Well permeable strata that store water are described as aquifers, and impermeable strata that contain water are called aquitards. Knowledge about the geological formation of the ground forming the groundwater reservoir is not sufficient to fully understand groundwater conditions. The bedding of the reservoir rock, the type of facies and the geological structure of the region are of decisive importance. For example, a good potential aquifer located in an arched fold (anticline) may contain no or very little water whereas one located in a trough or basin (syncline) can store considerable quantities of water. There are also various different bedding forms in aquifers and aquitards, which – even though frequently altered – repeat in their basis types.

Pressure conditions. Unpressurised groundwater is found where the aquifer can form an open groundwater table, which is termed an unconfined aquifer. If this is not possible because the aquifer is covered by overlying impermeable strata, the aquifer is confined. If the pressure head is higher than ground level, then the groundwater conditions are described as artesian.

Water storage capacity. The volume of voids determines the water storage capacity (porosity) of a rock mass. According to K.F. Busch and L. Luckner [35], such voids can be classified according to Table 5-1.

Table 5-1 Classification of voids in a rock mass [35].

Primary voids	Secondary voids
1. Bulk porosity of consolidated and unconsolidated sediments 2. Voids through inclusions in the formation of chemical and biogenic sediments 3. Voids due to gas emission from the magma during eruption	1. Joints, cracks and bedding surfaces 2. Voids in fractured zones 3. Dissolved voids due to a) solution of soluble minerals or b) chemical weathering of individual rock components 4. Voids due to organisms and crystallisation bursting

Permeability. The permeability of a rock mass, which is typically much higher than that of the solid rock, is determined by the size, form and linking of the joints. Water passages through pervious rock is mostly provided by favourably aligned permeability, which is the result of anisotropic orientation of the jointing system. Table 5-2 summarises the hydrological properties of a rock mass in relation to groundwater.

Table 5-2 Properties, which characterise the behaviour of a rock mass in relation to groundwater, from L. Müller [160].

Property	Definition	Increasing factors	Decreasing factors	Determined by
Water storage capacity	Capability of the rock mass to admit water	Fractured zones; faults; open joints; closely spaced joints; loose bedding	Low degree of separation of joints; dense joint filling	Analysis of rock mass structure; wished-in-place and large-scale tests
Permeability	Property of the rock mass to permit water flow through the joints	High density of joints combined with high degree of separation of joints; loosening; flat joint faces	Loam in joints; loam in faults; mylonites; grouting; low water temperature	Settling or pumping tests in shafts or tunnels; wished-in-place tests in boreholes
Water passage	Property of the rock mass to develop a particularly high permeability in certain directions or provide the water with concentrated, often bundled passages	Anisotropy of the joint system; fractured zones; solubility of the rock substance; opening of individual joints sets or large joints, i. e. the jointing network; removability of joint filling; softness of the water	Joint filling; low degree of separation of the joints	Analysis of the structure of the rock mass; large-scale tests with dyed liquids or gases

Construction works will be endangered by groundwater or groundwater when the tunnel passes through rocks, whose jointing is filled completely or to a large part of its layer thickness with contiguous quantities of water (Table 5-3). Inflow out of the joints can also occur when water quantities are only small but under high hydrostatic pressure.

Table 5-3 Overview of water transport through rocks [35].

Rock type	Water passage type	Water flow
Igneous		
Intrusive, for example granite, syenite gabbro, diorite	joints, fissures, stratum interfaces, fractured zones	dependent on jointing, mostly low
Extrusive, for example basalt, phonolite, quartz porphyry	bubble voids, cooling joints (extensive in basalt)	dependent on jointing, mostly low
Tuffs, for example basalt tuff, diabase tuff, pumice tuff	relatively high porosity, often jointed	dependent on jointing, mostly good
Metamorphic, for example marble, slate	cleavage planes, fractured zones, joints	dependent on jointing and cleavage, mostly low
Sedimentary		
Clastic sediments, for example gravel, sand, silt, clay	pore channels	dependent on the pore size, very good to low
Chemical sediments, for example gypsum, salt, limestone, dolomite	caves, chimneys, pipes, splits, fractures, fractured zones	depending on jointing and dissolved voids, seldom large quantities
Biogenic sediments, carbonate sediments such as chalk	mostly low porosity, jointed	very variable, mostly low
Siliceous sediments, for example siliceous shale	low porosity dependent on diagnesis, jointed	variable, seldom significant
Organic origin		
for example peat, lignite, coal, anthracite	porosity according to the degree of coal diagnesis (peat 85 %, lignite 50 %), joints in anthracite, coal, but also lignite	mostly low

Karst water. Karst water deserves particular mention because it is acutely dangerous to tunnelling. Chemical dissolution and mechanical erosion by percolating rainwater in joints and bedding planes can create various patterns of branched underground sinkholes, which can join to form a complicated system of caves and permeate the karst rock mass in various stories (Fig. 5-1). Cave channels run along weak zones (for example faults), since the rocks in fault zones are more fractured and susceptible to dissolution and erosion.

The fine veins turn into main veins, which are fewer in number but indicate much larger caves (Fig. 5-1 a). Karst water networks are mostly found in dolomite and limestone because this type of connected water vein can only be found in soluble rocks. Karst formations can be high-level or low-level (Fig. 5-1 b, c). In high-level karst formations, the water only remains in the rock mass for the time it takes to percolate and then appears from a spring. Low-level karst holds the water percolating from the surface in an underground basin. If a tunnel is driven through such a low-level formation, the water can pour into the tunnel at high speed under great hydrostatic pressure and in great quantities of many 100 m^3/min (which happened for example in the Mont d'Or Tunnel [238]). The water also often carries suspended particles, bed load and sand.

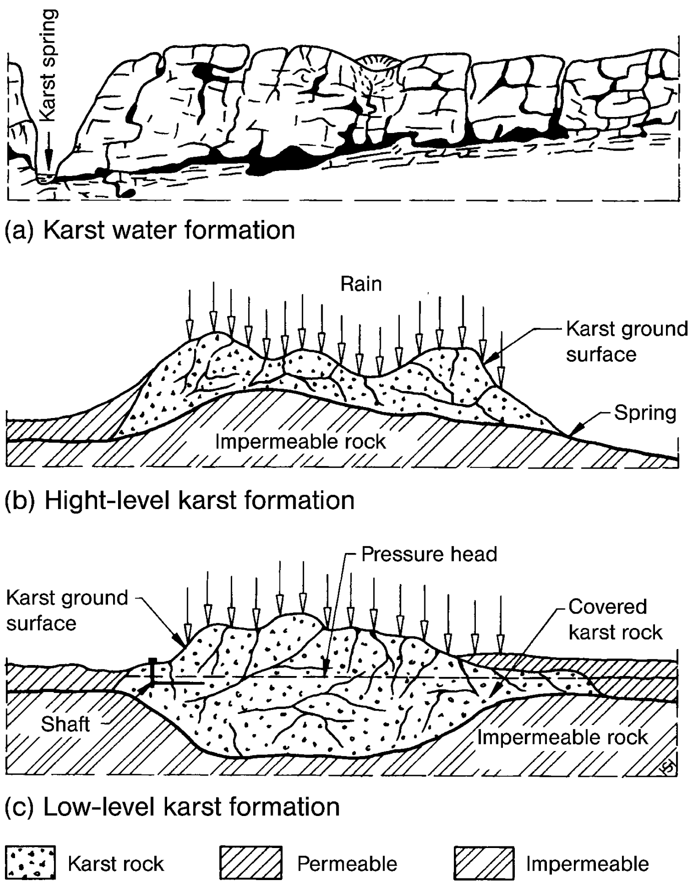


Figure 5-1 Water conditions in a karst rock mass [238].

5.2.1.2 Forms of underground water

After the percolated rainwater has infiltrated through the unsaturated zones of subsoil, part of the water flows above the groundwater table into natural discharges (interflow). The remainder reaches the saturated zone and enters the groundwater. This process is called percolation, the formation of new groundwater.

Water inflow. According to L. Müller [160], underground water can enter an artificial cavity in very varied forms (Table 5-4).

Table 5-4 Forms of water ingress into the cavity [160].

Form	Properties
Sweating	water appearing at the perimeter of the cavity; dripping from the crown (similar to condensation from the rock)
Percolating water	water emerging from cracks and joints (isolated or distributed around the entire perimeter)
Water ingress (strained water)	water spraying in jets from individual cracks (Fig. 5-2); in thin jets, this is described as springing; concentrated flows which can have particularly large flow sections in karst, are described as joint water
Water inflow	massive, often sudden inflow of water, often carrying suspended particles, mud or broken rock

All these forms can continue for a long time or reduce after a short time or even stop. The intensity, with which the water can enter the tunnel, is a result of the high pressure heads and flow gradients that are possible in a deep tunnel but is primarily due to the fact that the permeability of the rock mass in certain directions can often be eight to twelve times as high as that of the solid rock. C. Louis gives water permeability coefficients for selected rocks, which are shown in Table 5-5.

Figure 5-2 Water inflow at the face of the Schönrain Tunnel near Gemünden.

The quantity and point of appearance of the inflowing groundwater can also change with time, if the tunnel is only being driven partially below the groundwater table or if the groundwater table fluctuates with time [130, 160].

Table 5-5 Water permeability of rock (k_G) and rock mass (k_F) from C. Louis [125].

Rock		**Rock with one joint per metre**	
Rock type	k_G **cm/s**	**Joint width mm**	k_F **in the joint direction cm/s**
1. Limestones	(0.36 to 23.0) 10^{-13}	0.1	$0.7 \cdot 10^{-4}$
2. Sandstone Carboniferous Devonian	 (0.29 to 6.0) 10^{-11} (0.21 to 2.0) 10^{-11}	 0.2 0.4	 $0.6 \cdot 10^{-3}$ $0.5 \cdot 10^{-2}$
3. Mixed rocks sandy-calcareous clayey-sandy calcarious-clayey	 (0.33 to 33.0) 10^{-12} (0.85 to 130.0) 10^{-13} (0.27 to 80.0) 10^{-12}	 0.7 1.0	 $2.5 \cdot 10^{-2}$ $0.7 \cdot 10^{-1}$
4. Granite	(0.5 to 2.0) 10^{-10}	2.0	0.6
5. Shale	(0.7 to 1.6) 10^{-10}	4.0	$0.5 \cdot 10^{1}$
6. Limestones	(0.7 to 120.0) 10^{-9}		
7. Dolomite	(0.5 to 1.2) 10^{-8}	6.0	$1.6 \cdot 10^{1}$

5.2.1.3 Payment and quantity measurement

The reference quantity and measurement point of the incidental water quantity in l/s must be clearly regulated and specified in the contract between the employer and the contractor in order to avoid payment difficulties. Depending on the conditions at the face, it should be specified whether the reference quantity is measured “per advance face”, “per 10 m of tunnel”, “per 30 m behind the face” or “total water quantity at tunnel exit” [130].

DIN 18 312. German construction contract procedures (VOB) - Part C: General technical specifications for construction contracts (ATV) - Underground construction work” (12/2002) [59] includes the following statements to describe dewatering measures during tunnel construction in Section 0.2:

0.2.18 Type and extent of measures to collect, drain and if required treat groundwater during the construction period.

0.2.19 The limit water quantity for groundwater.

The area for the determination of the limit water quantity is given in DIN 18312 as maximum 50 m from the face (Section 4.1.4 of the standard). All costs arising from water ingress below this limit quantity during excavation and support works are deemed included in the estimation of unit prices for excavation in the relevant tunnelling class and are not paid separately. Only when this limit is exceeded can the contractor claim extra payment. The water quantity for payment purposes is then determined from the quantity drained from the tunnel less the quantity of service water supplied (Section 5.2 of the standard).

For the construction of the Euerwang Tunnel on the new line from Nuremberg–Ingolstadt for the Deutsche Bahn AG, the tenders from the company are based on an estimation quantity of water ingress according to DIN 18 312, that is measured about 50 m behind the face, with the following water quantity limits being defined:

– short-term water ingress up to 4 hours: 100 l/s
– long-term water ingress over 4 hours:
  a) in the top heading: 10 l/s
  b) in the bench and invert: 20 l/s

SIA standard 198. The Swiss SIA standard 198 (issue 2007) [218] states the following under clause 19.2 "Obstruction resulting from water for tunnelling in rock" about the drainage, payment and measurement of incidental water during tunnelling in hard rock:

19.2.1 Tender

19.2.1.1 The crew daywork rates deducted from the bill of quantities for obstruction resulting from water are to be taken into account proportionately in the construction schedule.

19.2.1.2 The number of planned and payment programme days for obstruction resulting from water is determined with consideration of reduction factors on crew daywork rates.

19.2.1.3 The reduction factors on crew daywork rates for the determination of the planned and payment construction time are to be laid down on each project according to the hydrological conditions. If this is not specified in the tender documents, the values in Table 17 are considered contractually agreed.

19.2.1.4 Water measurement in caverns and shafts is to be regulated for each project in the tender documents.

19.2.2 Payment regulations and remeasurement provisions

19.2.2.1 Water ingress results in reduced work performance (working in water or extra costs for drilling and repairs to construction site facilities). This reduced performance (obstruction) is paid according to crew daywork rates.

19.2.2.2 Payment for obstruction is dependent on the tunnel gradient (rising or falling), the excavation cross-section and the water quantity less service water, which occurs at the face during a period of at least eight hours in the relevant section.

19.2.2.3 Obstruction due to water quantities, which do not reach the lower figures given in Table 17, are included in the excavation prices.

19.2.2.4 The water ingress will be measured by the employer and the contractor with suitable instruments and at an appropriate location. Measurement instrumentation is to be regularly maintained and sporadically calibrated.

19.2.2.5 The relevant section behind the face for the determination of water ingress is 100 metres.

19.2.2.6 In case of immediately consecutive partial excavations, the face that is furthest back counts as the face.

19.2.2.7 For partial excavations, which lie more than 100 m from each other, the relevant section for the determination of the water quantity is measured from the relevant face.

19.2.2.8 For the variation of contractual deadlines as a result of altered quantities compared to the crew daywork rates deducted from the bill of quantities, the difference expressed in programme days between the invoiced programme and planned schedule applies (for example see Appendix D).

19.2.2.9 Payment for obstruction due to the effect of water on tunnelling works is paid according to crew daywork rates. The crew daywork rate includes all incurred wage costs for the delay. Installation and machinery costs are compensated through extended provision periods.

19.2.2.10 The obstruction starts with the exceeding of the lower limit and ends when it is no longer exceeded.

19.2.2.11 The start of obstruction is to be agreed in writing between the contractor and the employer without delay.

19.2.2.12 Regular water measurements are to be carried out for the duration of the payment.

19.2.2.13 Expenses for the production, operation and maintenance of measuring instrumentation will be paid. The assistance of the contractor with the water measurements will not be paid separately.

19.2.2.14 Obstruction during concreting works in the invert or for setting the invert segment will not be separately paid.

19.2.2.15 Measures to transport the water will be separately paid.

Table 5-6 Reduction factors for the performance values in case of water ingress (Tab. 17 of the SIA standard 198 [218]).

Tunnel with theoretical excavation area $A \leq 25$ m² or $\varphi \leq 5$ m		Tunnel with theoretical excavation area $A > 25$ m² or $\varphi > 5$ m		Shafts		Reduction factor for daywork hours
rising [l/s]	**falling [l/s]**	**rising [l/s]**	**falling [l/s]**	**rising [l/s]**	**falling [l/s]**	
10 – 20	5 – 10	10 – 20	5 – 10	2 – 5	1 – 2	0.2
>20 – 30	>10 – 20	>20 – 40	>10 – 20	>5 – 10	>2 – 5	0.4
>30 – 40		>40 – 60	>20 – 30			0.6

ÖNorm B 2203-1. The Austrian standard B 2203-1 “Underground works – Works contract – Part 1: Cycling driving” (issue 12/01) [170] requires project-related contractual regulation of obstructions due to water ingress.

Obstructions due to the ingress of groundwater are compensated by additional tunnelling time (payment units), which are determined using reduction factors. For this purpose, there must be, in addition to the items for time-dependent costs of the construction site, items for wage costs for the mining crew per time unit (including other time-dependent costs, which are not included in the time-dependent costs of the construction site).

1. The bidder is to provide details for water obstructions categorised according to rising and falling advance depending on the groundwater quantity and water obstruction classes (Table 5-8).
   The different effects of water on the rock mass and the location of water occurrence in the tunnel cross-section are to be described and assigned to water obstruction classes according to Table 5-7 for each project.
   In case the partial cross-sections bench and invert are not on the critical path (meaning they are excavated parallel to the top heading), only the items for the mining crew and the time-dependent costs are to be provided for these partial sections. A separate determination of the payment units for the bench and invert is not provided, but rather the number of invoicing units is to be taken from the top heading.
   In case the partial cross-sections bench and invert are just as time-critical as the top heading, and thus have their own advance rate details and items for time-dependent costs, the payment units for water obstruction are separated analogously to the top heading. In this case the reduction factors (Table 5-8) are also to be requested for these partial cross-sections.
2. The maximum water quantity to be considered in the estimation is to be determined for each project. If required, the lower limit of water quantity, up to which any obstructions are included in the excavation prices, is also to be laid down for each project. If no figures are agreed for the lower and upper limit values, the figures in Table 5-8 apply.
3. The employer is to lay down a number of working days with water obstruction for the individual partial cross-sections and assign these to obstruction classes and water quantities. The number of invoicing units is calculated from the number of working days laid down by the employer and the reduction factors according to Table 5-8 to be stated by the contractor within the limits laid down by the employer. The number of working days laid down by the employer are calculated from the tunnel sections with water obstruction and the advance rates without water obstruction that are assumed by the employer.
4. The payment units for water obstructions that determine the construction time are to be considered in the sum of the payment units for tunnelling.
5. The type, location and frequency of measurements of groundwater quantity are to be given separately for rising and falling advances for the individual working areas.
6. Payment items for water obstruction to the installation of support measures and inner lining are also to be provided for each project if required.

Table 5-7 Obstruction classes according to ÖNorm B 2203-1 [170].

Influence of water on the ground	Location of water ingress in the partial cross-section	
	invert	sides and face
negligible	1	2
medium	2	3
heavy	3	4

REMARKS
In this table, the relevant location is defined as follows:
tunnel sides and face is the area from 1.0 m above the actual invert (top heading invert) up to the crown; invert is the area from the relevant actual invert (top heading invert) up to 1.0 mm above it.

Table 5-8 Sample table of reduction factors for water obstruction [170].

Obstruction classes	Favourable				Medium				Unfavourable				Very unfavourable			
	1				2				3				4			
Water quantity	**FILLED IN BY CLIENT**			**FILLED IN BY CONTRACTOR**	**FILLED IN BY CLIENT**			**FILLED IN BY CONTRACTOR**	**FILLED IN BY CLIENT**			**FILLED IN BY CONTRACTOR**	**FILLED IN BY CLIENT**			**FILLED IN BY CONTRACTOR**
		min.	**max.**			**min.**	**max.**			**min.**	**max.**			**min.**	**max.**	
l/s	days	%	%	%	days	%	%	%	days	%	%	%	days	%	%	%
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
0.5 to 2	■	■	■	■	■	■	■	■								
over 2 to 5	■	■	■	■												
over 5 to 10																
over 10 to 20																
over 20 to 40																
over 40																

REMARKS:
Working days and min./max. reduction factors are to be given by the client for the specific project, depending on the formation water quantity and water obstruction classes. The reduction factors to be given by the contractor must be higher for unfavourable combinations than for favourable combinations. A reduction of 20 % means that the advance rate under the conditions of water obstruction is only 80 % of the advance rate without water obstruction

Proposal for quantity measurement. For excavation and support works, the water should be measured about 100 m behind the face, or the relevant advance location. The total water quantity should also be measured at the start of the excavation of the round (Fig. 5-3). It is assumed here that cost of excavation and support obstructions is determined from the water quantities about 10 m behind the relevant advance location (top heading or bench). Water ingress is measured in l/s. Limit values should be given in the specification for various degrees of obstruction, sub-divided according to the different geological conditions and their effect on the obstructions.

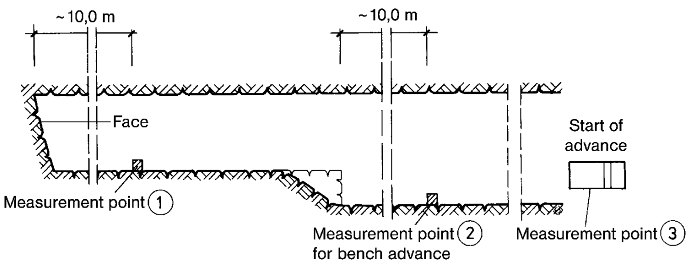


Figure 5-3 Water measurement points during tunnel excavation [130].

5.2.2 Measures to collect and drain groundwater

Possible operational measures to maintain the construction schedule are explained in the following section. The dewatering measures described in Table 5-9 can be used individually or in combination according to the construction process, local conditions and ground conditions.

Table 5-9 Dewatering measures.

Measure	Purpose / properties
Drainage channel	Removal of groundwater, percolating, service and surface water from the tunnel excavation (open dewatering)
Borehole drainage	Relief of static water pressure
Drainage tunnel	Can be provided both inside and outside (for slope and joint water flows) the final cross-section so as to obstruct the advance as little as possible
Vacuum lances	Application of negative pressure to provide the pressure gradient required to ensure flow; removal of water held in the soil by capillary action
Compressed air working / hydrostatic support	Stabilisation of water in the soil
Grouting / ground freezing	Sealing of the prevailing water

No further details of dewatering with „vacuum lances“ or compressed air are given here. The following literature can be recommended: [130, 153].

5.2.2.1 Measures to collect water

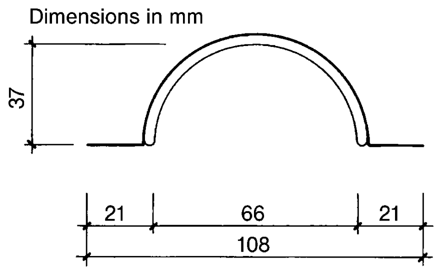
In order to keep the obstruction to tunnelling works from the groundwater as slight as possible, the water should be collected at the tunnel sides and fed into the already described drainage. The methods summarised in Table 5-10 are available depending on the intensity and appearance location of the water.

Table 5-10 Methods of water collection [251].

Point collection	Linear collection	Surface collection
– Drilling, – Push-fit pipes – Drip channel	– Oberhasli process (purely manual, outdated) – Filter pipes (holed with filter surround) – Semi-circular channel drainage (much manual work, blockage through pointing) – Filter strips – dimpled sheeting or ribbed foil – drainage mats – structured mats	– No-fines outer support layer (elaborate tunnel construction method from the 1960s) – No-fines concrete fill instead of construction material sluicing, possible with multi-layer tunnel lining – U-profile precast element drainage – Dimpled drainage sheets, ribbed drainage foil and structured mats – Protective fleece as drainage and protection element for foil waterproofing membrane – Invert filter according to the type of frost protection layer

The most important processes are described in the following section.

Plastic gutters and channels with various profiles (Fig. 5-4) are used to collect localised water ingress. Gutters made of soft PVC and stiffened longwise and crosswise with steel wire are laid by hand with rapid-hardening cement mortar or temporarily fixed with forked pins until they have been covered with a shotcrete layer a few centimetres thick.



Form U: 66/108 mm wide semi-circular section with 19 cm^2 flow section; roll: 25 m, 240 g/m

Figure 5-4 Drainage channel of plastic; Aliva AG.

Protective fleece. The main purpose of the fleeces that are normally provided between temporary support and final lining is the protection of the waterproofing membrane against mechanical damage, for example perforation due to the rough surface of the shotcrete substrate. Protective fleeces should also enable surface collection and drainage of groundwater. In order to ensure that this function is effective, qualitative and as far as possible quantitative verifications should be undertaken of the flow quantities and measures to avoid sintering.

Drainage or dimpled mats. Drainage mats of plastic (Fig. 5-5) are used when large quantities of groundwater are expected to appear from large areas. The dimpled side should ideally be installed to the outside – against the rock. Since the application of shotcrete onto the dimpled sheet is impractical, they are often laid on the shotcrete support layer and an in-situ concrete inner lining is then concreted, with or without waterproofing.

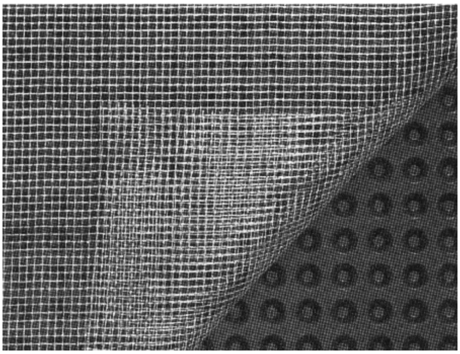


Figure 5-5 Plastic dimpled mat; Aliva AG.

5.2.2.2 Measures to drain water, open dewatering

Open dewatering is used to remove water, which enters the tunnel, and also surface and service water. Drainage channels collect the water at the sides of the tunnel [130]. These can be fixed in various ways according to the type of ground. In practice, it must be said that too little attention is often paid to tidy water collection.

Channels and pipes. The German standard book of bill items StLB 007 "Underground works" [228] provides drainage channels bedded in mortar or sand. A mortar bed is recommended when heavy water ingress from the invert is expected. Channels can be made of precast concrete elements, stoneware, in-situ concrete, steel or plastic. The clear opening and the clear flow height should not be less that 0.3 m. The fall should be at least 2 % in order to ensure a flow speed of at least 0.5 m/s so the channel will keep itself clean. PVC pipes can also be used instead of channels, and should be protected with filter concrete. It is important that pipes or channels are not damaged by the chemical properties of the incoming water. Prefabricated concrete channels as a composite with semi-circular gutters of polyester resin concrete or polyester resin can be used when the water is very aggressive or to enable cleaning.

Pumps. When the fall is insufficient or the tunnel is advancing downhill, the water has to be collected at pumping stations and pumped away with drainage water pumps. The pumps switch and off automatically according to the water level.

5.2.2.3 Drainage boreholes and drainage tunnels

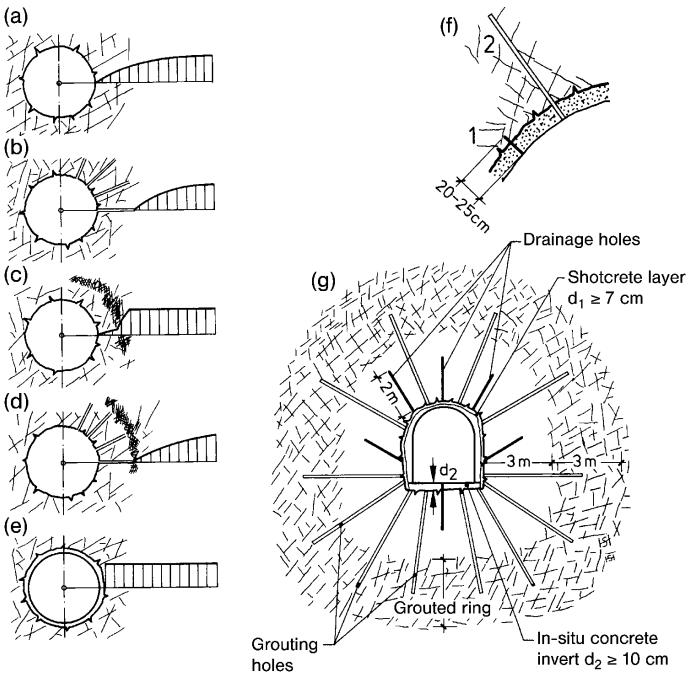
Drainage boreholes following the advance. Waterproof shotcrete on the sides of the tunnel will hold water and close water passages. This can lead increased flow gradients and thus pressure in the immediate vicinity of the tunnel. Drainage boreholes drilled after the advance can prevent this and also have the advantage of reducing static pressure on the support. They can be used for targeted water collection, ideally localised at joints. The extent to which water pressure is relieved by drilling depends on the following factors:

- The location of the borehole in the aquiferous horizon.
- The direction of the holes related to the position of the joints.
- Spacing, length and diameter of the holes depending on the jointing structure and water pressure.

Complete coverage of pressure relief around the tunnel is only ensured when the pressure-relieving effect of two adjacent holes overlaps. The holes are connected to the open dewatering system and normally remain in operation to relieve the shotcrete support layer until the inner lining has hardened. They should be laid out so that tunnelling operations are not obstructed.

The most favourable combination of spacing, length and diameter of the holes has to be discovered experimentally, although economic aspects also have to be considered. It is certainly cheaper to drill more or longer holes with the available drills than to drill fewer holes of larger diameter if this entails importing new machinery.

Fig. 5-6 shows the effect on the water pressure distribution of measures like borehole drainage, grouting and waterproof linings.



a Unaffected condition
b Pressure altered by drainage boreholes
c Pressure altered by grouting
d Pressure altered by grouting and drainage boreholes
e Pressure altered by waterproof support layer
f 1 Boreholes for contact grouting
f 2 Boreholes for ground consolidation and waterproofing grouting
g General layout of grouting and pressure-relief holes in the Plöcken heading for the Trieste-Ingolstadt pipeline

Figure 5-6 Groundwater pressure distributions around tunnels [160].

Drainage boreholes ahead of the advance. (Horizontal drains) are sensible if

– heavy water inflow is expected,
– sudden but not predictable water inflows can be expected (for example when there is standing water or when large-scale advance drilling in karst conditions is also intended to detect open joints containing water under pressure),
– advance drainage can be expected to considerably simplify the subsequent excavation and support works (for example in gravel),
– flow pressures can considerably impair structural stability (quicksand, running ground),
– water lenses may be present and are to be detected,
– high joint water pressures are to be relieved.

Drainage drilling ahead of the tunnel should be as deep as possible so that tunnelling works are not interrupted too often by the drilling of new holes. Holed or slotted hard PVC pipes or perforated plastic drainage pipes with glass wool filters are often inserted into these 35 to 100 mm diameter holes when the hydrogeological investigations suggest a risk of retrogressive erosion. The pipes are normally extended with a flexible plastic hose at the exposed end of the hole to drain the water into the side drainage channels or into a temporary drainage pipe hung from the wall.

Drainage tunnel. The provision of a drainage tunnel or heading is practical when water inflow is heavy or above-all one-sided, and it may also be useful for other purposes such as ventilation, investigation or as a pilot heading.

As the tunnel is excavated, the drainage heading can be used to improve water drainage. Further drainage holes 2 to 5 m long can be drilled from the heading. These holes should penetrate as many aquiferous joints as possible and drain the rock mass.

As an example of this, the measures undertaken during the construction of the 7864 m long Karawanken Tunnel (built 1987 to 1991) can be mentioned [140]. The dewatering measures during the construction period were based on thorough draining of the rock mass, which was also intended to relieve any high pressure. Systematic advance drilling was used both for geological investigation and to drain the rock mass. During the tunnel drive, a collapse occurred at the face of the top heading, with large quantities of water and about 4000 m^3 of material pouring into the tunnel. In order to overcome this collapse, drilling was carried out from a drainage heading and from the main tunnel to drain water and to grout a zone wider than the presumed fault zone (Fig. 5-7). In addition, the top heading area of the main tunnel was drained through further holes drilled from the bypass tunnel.

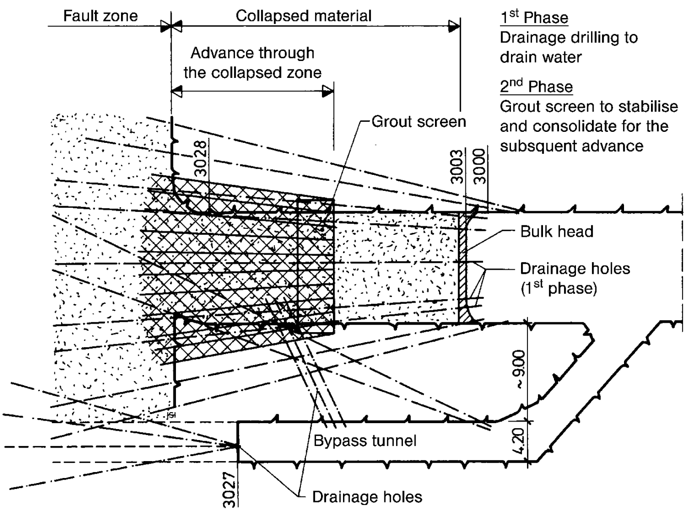


Figure 5-7 Measures to overcome a collapsed area in the new Karawanken Tunnel (1987) [140].

5.2.3 Obstructions and reduced performance

5.2.3.1 General description

Before the rock mass is disturbed by the driving of an underground cavity, the water stands in the joints or cracks in the rock mass or moves according to its potential gradients. This slight movement of water does not risk washing out fines; the water does not have sufficient carrying capacity. The situation changes dramatically with the driving of a tunnel. The pressure and speed gradients and thus the carrying capacity of the water are suddenly increased so that fines are eroded from the grain structure and transported. The water now represents an acute danger to the driving of the tunnel. The charges can be ejected from the blasting holes by water pressure unless special measures are undertaken [160]. When rock bolts are being set, there is a danger of the anchor being flushed out when a vein of water is drilled into and the water pours into the tunnel through the drilled hole. Particularly shotcrete spraying is affected by water that occurs over an area, since the adhesion and hardening are negatively affected [132]. The overall effect is an obstruction, as has already been described in Section 5.2.1.3.

Dewatering. Water entering the tunnel always represents an additional obstruction to construction works. If the tunnel is rising, then the water can be drained as open dewatering down drainage channels at the sides, which of course have to be constructed as the tunnel advances and kept free of mud, from the inside of the tunnel to a discharge point.

If the tunnel is being driven downhill, the water collects at the face and causes considerable obstruction to the works. This demands careful construction of dewatering measures with intermediate pumping sumps, from which the collected water is pumped through pipes at the sides of the tunnel to the discharge. Particularly for falling tunnels, it is essential to investigate any change of the characteristics of the rock mass due to the action of water.

Investigation of the effects of water. This can be done by digging trial pits and subjecting them to uninterrupted water action for about a month. Such tests were carried out, for example, in the middle Bunter sandstone of the Landrücken-Nord Tunnel to evaluate settlement effects at the top heading feet. The rock samples taken from the trial pits showed that the middle Bunter sandstone experienced no geomechanical changes due to the action of water such as weathering effects or changed rock strength due to softening. No punching failure of the top heading feet due to the action of water was to be feared under normal rock and pressure conditions, except in the vicinity of any chimneys. These areas had to be protected against water, for example with concrete or plastic channels.

5.2.3.2 Influence of groundwater on the advance rate

Groundwater affects the duration of the round cycle and thus the advance rate depending on the nature and volume (Table 5-11).

In order to estimate the effect of groundwater on the advance rate, six groundwater classes BWKL (BWKL 1 = no water, BWKL 2 = 0 to 5 l/s etc., up to BWKL 6 ≥ 60 l/s) are defined [233]. While groundwater class 1 produces no obstruction to the advance and groundwater class 2 and 3 only slight obstruction, the advance rate is clearly slowed by water ingress of more than 15 l/s in the round area. The following tables and illustrations illustrate the type of obstructions and estimate the extent of the obstruction by groundwater on the advance rate.

Table 5-11 Type of obstruction to the advance due to groundwater [233].

Activity in the round cycle	Type of obstruction
Drilling	Difficulties for drilling works are caused by water spurting from the drilled holes, by water emerging from areas of the face and by erosion occurring at the face. The moving of holes can lead to their collapse. Localised washing-out of material from joints can also occur. Delays in withdrawing the drill stem and the general worsening of working conditions can lead to a further reduction of drilling performance.
Loading charges	The time taken to load the charges can be increased due to moved holes and the worsened working conditions.
Mucking	The softening of the invert and any extra excavation can increase the time taken to clear the muck.
Support works	The duration of shotcrete spraying increases due to the increased rebound. The anchors require longer to install due to the reduced drilling performance and increased anchor manipulation time. During the performance of support works, extra dewatering work such as fitting hoses, waterproofing areas and digging sumps have to be undertaken. Support works are also slowed by the worsened working conditions.

The extent of obstruction due to groundwater on the individual activities of the round cycle is shown graphically in the following illustrations, using the rock mass type definitions from the Austrian standard ÖNorm B 2203 [170] as percentages of the normal duration of each activity. There is a basic difference between two rock mass types, B1 and C2. Rock mass type B1 describes an unstable rock mass with rapidly decreasing deformation and slight loosening of the crown and the upper bench area. Rock mass type C2 describes a squeezing rock mass with pronounced deformation, fracture events and plastic zones, in which systematic installation of support is required in the crown and bench (see Section 2.4 “The ground and its classification”).

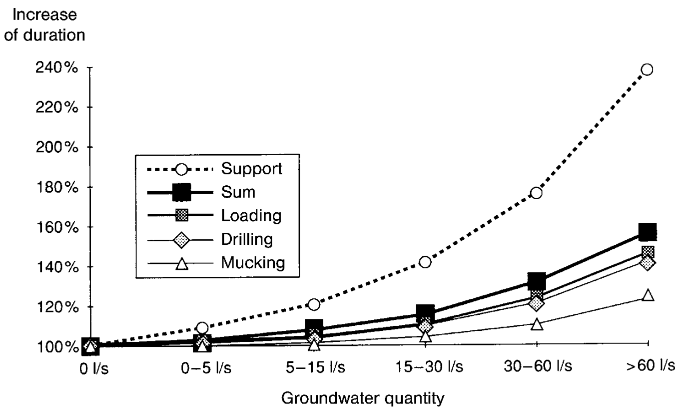


Figure 5-8 Extent of obstruction by groundwater (rock mass type B1 according to ÖNorm B 2203 [170]).

In the worse rock mass type (C2), the percentage increase of the duration of support works is less than in the better rock mass type (B1), since the support work in the worse rock mass is already extensive without groundwater. The support works require about 50% of the round cycle time in rock mass type C2.

Fig 5.10 shows the result of investigations into dependence on the quantity of groundwater [120].

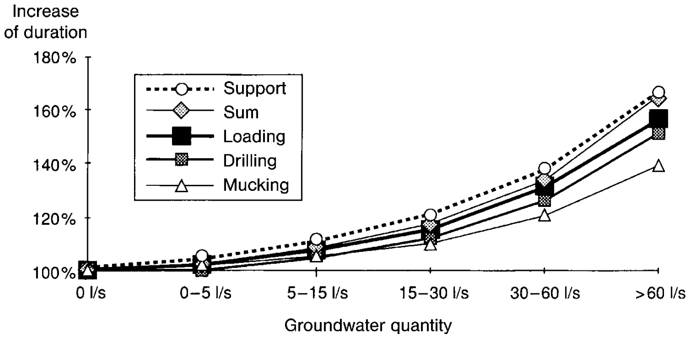


Figure 5-9 Extent of obstruction by groundwater (rock mass type C2 according to ÖNorm B 2203 [170]).

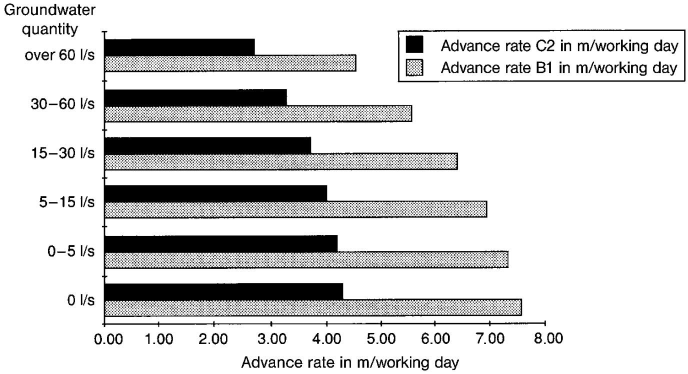


Figure 5-10 Average advance rate for rock mass types B1 and C2 depending on quantity of groundwater [120].

5.2.3.3 Influence of groundwater on tunnelling costs

In addition to the effect on the advance rate, groundwater also has a drastic effect on overall tunnelling costs. The factors that influence costs are as follows:

Machinery costs. The slower advance rate with water ingress increases the average machine time per metre of tunnel advance. Worse machine wear and higher repair costs also have to be expected. Pumps, for example, have to be considered as additional machinery.

Material costs. In addition to the increased shotcrete consumption due to increased rebound, the materials for dewatering measures of drainage and waterproofing also have to be considering.

Personnel costs. Due to the significantly worsened working conditions with water ingress, reduced performance has to be expected. The various obstructions caused by water also slow the advance rate, which increases the personnel costs per metre of tunnel advance. The proportion of personnel costs amounts to about 40 % of overall cost and is also a decisive factor for the extra costs due to water and can be up to 60 % more than the original proportion.

5.2.4 Environmental impact and cleaning

The environmental impact of construction works is becoming ever more significant due to increased environmental awareness. In order to avoid any possible criticism of tunnel construction, which can actually be described as "environmentally friendly", the effects on the surrounding groundwater should be paid great attention.

5.2.4.1 Effect on the groundwater system

The collection and drainage of groundwater lowers the groundwater table down to a persistent state and changes the flow conditions inside the rock mass.

This intrusion into the groundwater system can become noticeable at ground level if the hydrogeological conditions are unfavourable and large quantities are drained. The possible consequences include smaller springs drying up, foundation damage or at worst massive impact on ecological systems. The worst effects do, however, only apply to regions with particularly sensitive groundwater conditions. Long-term observations as part of tunnelling projects show no impairment of the ecological equilibrium on the surface under normal conditions [143].

Detailed studies of environmental impact, considering the overall situation of the tunnel project, have now become a normal part of the construction approval process.

Groundwater lowering. Groundwater lowering can be temporary or permanent.

Temporary groundwater lowering. Temporary groundwater lowering lowers the groundwater table for the duration of the tunnel drive, and is technically necessary when the prevailing groundwater would obstruct the tunnel excavation very greatly or even make it impossible. This is normally the case in groundwater-saturated soils with high pore volumes (30 % and more) or high *k* value. The limits to the application of drill and blast are, for example, tunnels more than a certain length in water-saturated soil or also solid rock with high water inflow quantities, where closed shield machines are more economic [142].

Permanent groundwater lowering. The tunnel is drained permanently, both during the construction period and after completion and the groundwater table is lowered in the long term. This intrusion into the groundwater regime and the high cost of cleaning and operational interruption due to the flushing of sintered drainage in some rail tunnels [42] have led to considerations, for example by German Railways at the moment, that tunnels should generally be constructed watertight and without drainage of groundwater [84]. Such a decision poses new demands on the feasibility of waterproofing measures and the economic design of the inner lining under water pressure. Depending on the cross-section size, inner linings reach the limit of practical design for a water pressure more than 7.5 bar.

The decision between temporary or permanent groundwater lowering is always produced in steps as a compromise, which has to be adapted to the given and the chosen conditions of each tunnel project. Particularly for tunnels with high water ingress and high pressures, combination solutions will be necessary in the future, such as for example the use of a pressurised water limitation system. In this case the groundwater is drained under control and only in such quantities that the water pressure does not exceed the design pressure on the lining and the waterproofing. The first experience with this sort of system was the construction of the 7 km long Freudenstein Tunnels (built 1987 to 1990) on the new Mannheim – Stuttgart railway line [106].

Groundwater monitoring concept. A monitoring concept can be produced for the affected springs, surface water and groundwater conditions, taking into account data for the catchment area (for example location, area, average height). The duration of monitoring depends on the distance of the catchment area from the tunnel and the permeability of the ground. It increases with decreasing permeability of the ground and increasing distance from the tunnel. The duration of observation of the spring and groundwater system after the completion of construction works is calculated, depending on the type of tunnel waterproofing, with flow time calculations using hydraulic models [37].

5.2.4.2 Effects on groundwater quality

Drainage channels not only carry the groundwater but also the service water pumped from the surface. This water is heavily contaminated by construction operations. The wastewater contains mineral solids from grated or pulverised rock and its chemistry has been altered. Increased pH values have to be expected with the production of shotcrete or in-situ concrete. This is due to leached contents of cement or accelerator.

Bulletin A 115 from the wastewater technical association [171] prescribes the following guideline values for the discharging of water into public drainage systems (drains, treatment plants):

- Waste water temperature < 35 °C.
- pH value between 6.5 and 8.5.
- Minimisation of salting by the neutralisation reaction.

Preventive measures. Operational methods should be modified to attempt to avoid contamination of tunnel water.

If it is not possible to avoid the softening of the unsupported invert by wheeled vehicles next to open drainage channels at the sides, then it is advisable to pave the invert with ballast. The ballast must either be delivered or a suitable excavated material can be used instead.

The potential to contaminate water should be considered in the storage and processing of all construction materials (cement, accelerator, grout etc.). For example, it is advisable to clear rebounded shotcrete before drilling the following round in order to avoid soluble substances being washed out.

Reduction of the pH value. Hardened cement paste also contains contents, which can be dissolved out of the cement matrix by any natural water. This applies particularly to calcium hydroxide produced by the hydration of cement and potassium and sodium, which are also mainly present as hydroxides and derive from the use of accelerators.

Dissolved calcium hydroxide is precipitated on contact with carbon dioxide as calcium carbonate and this can cause sintering of the tunnel drainage. The much more soluble sodium and potassium hydroxides are responsible for the increased pH value of drain water.

The following measures are possible to counter these mechanisms [112], [149]:

- Reduction of water flow at the concrete surface (collection of the water within the rock mass).
- Increasing the density of the internal structure of the concrete (optimised production, use of Flugacal or microsilica).
- Reduction of contents susceptible to leaching (use of lime-free aggregates, shotcrete without accelerator, alkali-free accelerators) (see also Section 5.4.2).

After-treatment to neutralise alkaline water can be achieved by adding CO_2, such as for example gasification in a basin 1.5 to 2 m deep (Solvocarb-B process). The use of carbon dioxide has many environmental advantages over the use of mineral acids (HCL, H_2SO_4, HNO_3, H_3PO_4). It does not increase the salt load in the water, no storage of acids is required and the use of CO_2 prevents the danger of over-acidification.

Waste water treatment. A certain contamination of the tunnel water cannot to be avoided. Depending on the discharge conditions into the relevant waterway, treatment measures may be necessary. The facilities for this can also be installed underground (Table 5-12).

Table 5-12 Water treatment equipment [62].

Underground	Above ground	
Transportable settling basin	Mechanical process:	settling basin
Intermediate pump sump as a settling basin	Chemical process:	precipitation, neutralisation
Main pump sump with upstream treatment basin for settlement	Sludge treatment:	chamber filter press
Use of a vacuum suction vehicle	Monitoring equipment:	pH meter, quantity measurement

Underground treatment facilities. Fig. 5-11 shows a possible layout of an underground treatment plant. The water collected at the face passes through many successive transportable settling basins in order to make use of the potential for separation of solids in the tunnel. The intermediate pumping sump and the main pumping sump also provide settlement, and a simple sheet steel container can be equipped as an initial settling basin. The settled sludge must be cleaned from the basins daily by a vacuum suction vehicle.

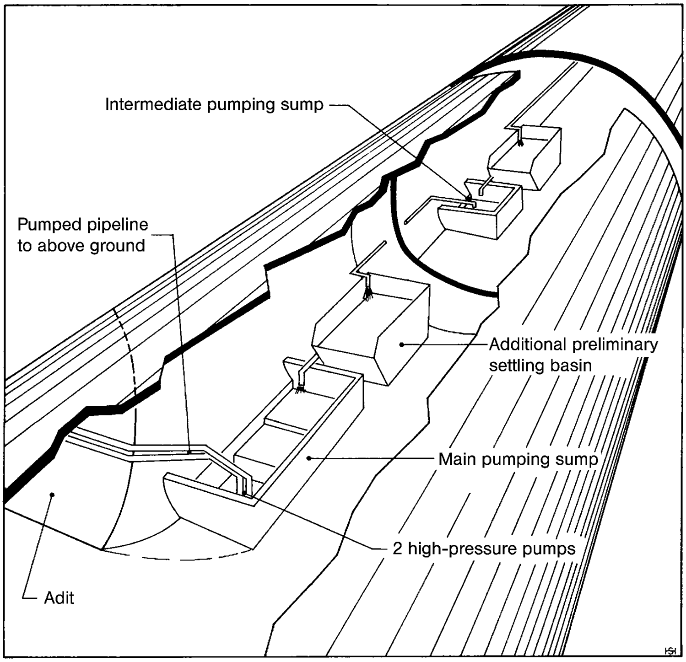


Figure 5-11 Main pump sump underground [62].

Treatment facilities above ground. The design of a treatment plant above ground (Fig. 5-12) should be based on the following constraints:

- Max. water inflow in l/s.
- Stay time at max. throughput.
- Construction of at least two settling basins, which can be operated independently (in order to clean one of the basins).
- Consideration of other process stages depending on the required cleaning effect (precipitation, neutralisation, sludge dewatering).

A submerged wall at the end of the basin can retain any light fluids, which can then be skimmed. The outflow channel should be fitted with a toothed weir to ensure that flow comes from the entire width of the basin [62]. A chamber with measuring instruments should be provided to check the cleaned water and measure the quantity.

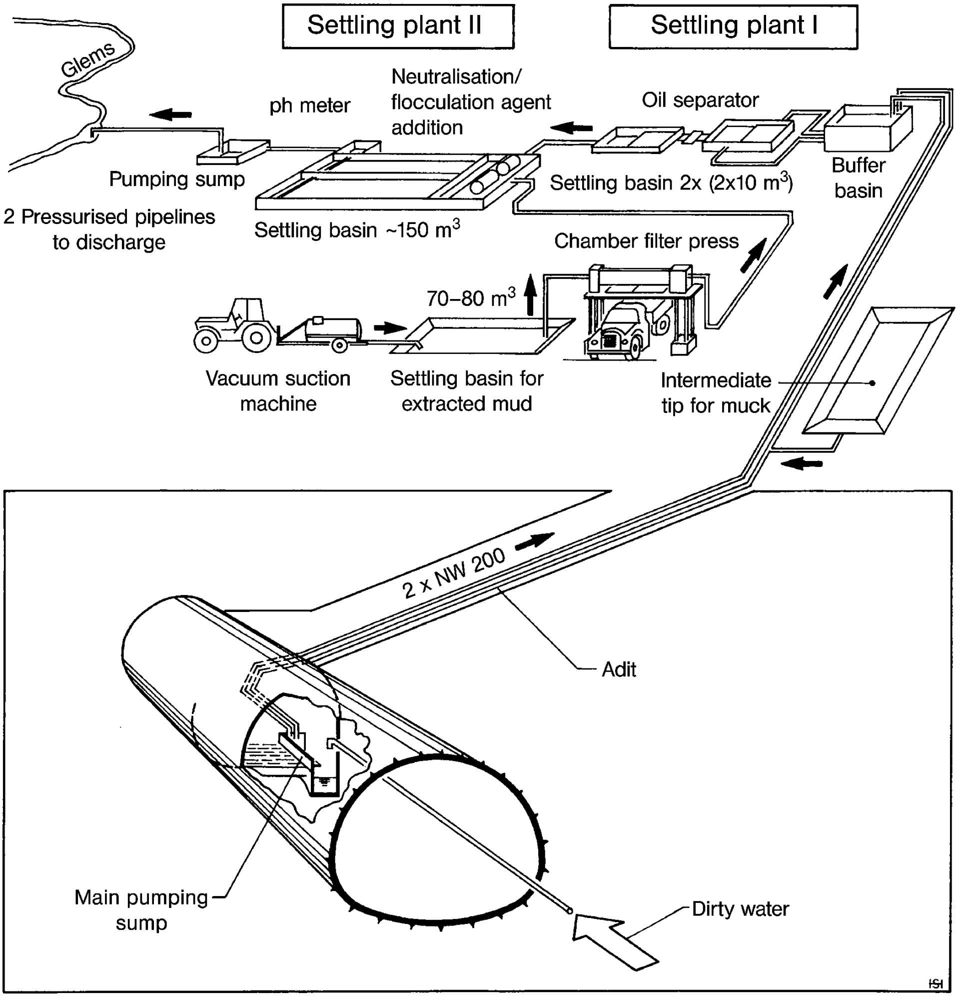


Figure 5-12 Water treatment and discharge above ground [62].

5.2.5 Sealing groundwater

Beside the collection and drainage of groundwater during the construction period, it is also possible to undertake sealing measures, which could involve grouting or ground freezing.

5.2.5.1 Grouting process

In the grouting process, the voids in the rock mass are closed by injecting suitable liquids. This reduces the water permeability and simultaneously increases the stability of the ground. According to the type of grout, this can be categorised into cement grouting, chemical grouting or resin grouting.

The suitability of the ground and the grout material for grouting depend on the following geological, hydrological and chemical factors:

- Mineralogical composition of the soil.
- Grading.
- Stratum thickness and location of the boundaries.
- Joint network of the ground.
- Water permeability of the ground.
- Hydrostatic pressure of the groundwater.
- Direction and speed of groundwater flow.
- Chemical properties of the groundwater (for example salt content).

Research and development tendencies show that the use of grouting seems particularly suitable when water pressure in the joints is very high and their volume is small. This process also enables the surrounding ground to be exploited for load-bearing, which is especially significant for tunnels under very high water pressure. The further development of this basic idea is promising as a future method of economic construction of tunnel linings under high pressure.

A grouted body in the form of a closed ring is created ahead of the advance. Due to the increased resistance to flow, the potential gradient in the grouted ground produced by drainage inside the ring is increased, which leads to the grouted ring resisting hydrostatic pressure (Fig. 5-13).

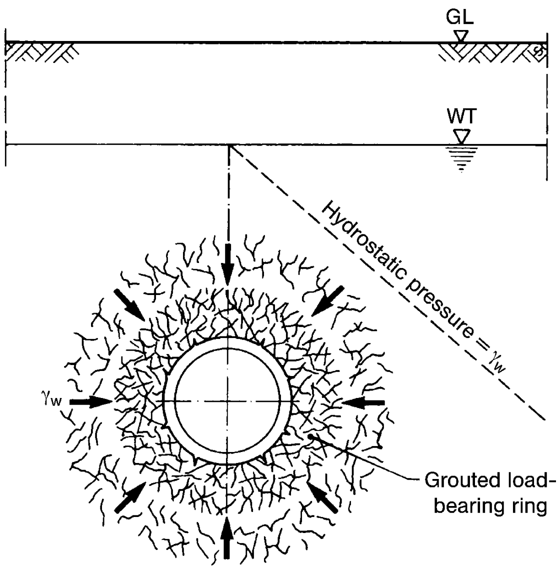


Figure 5-13 Grouted load-bearing ring under hydrostatic pressure, from [70].

Experience with the use of grouting under very high pressure was gained during the construction of the 23.5 km long drill and blast underwater section of the Seikan Tunnel (overall construction period 1964 to 1984) in Japan [130]. A hydrostatic pressure of 24 bar and the danger of seawater breaking in demanded extensive grouting with cement- and waterglass-based grouts (Fig. 5-14).

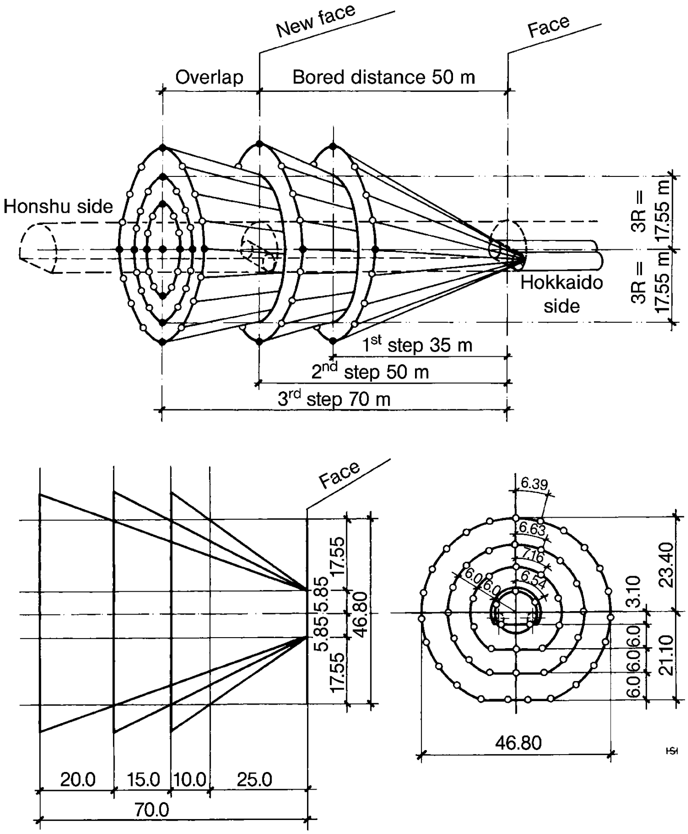


Figure 5-14 Grouting pattern in the undersea section of the Seikan Tunnel [158].

5.2.5.2 Ground freezing

Ground freezing can temporarily seal and consolidate the ground under conditions that are water-bearing but not suitable for grouting. In contrast to grouting, the process only affects the groundwater conditions during the construction period.

The process can be used with a soil water content of more than 156 % and a groundwater speed of less than 1 to 3 m/d [130].

More detailed information about groundwater sealing can be found in [130, 153, 160].

5.3 Tunnel waterproofing

Since it was not technically possible to waterproof underground tunnels against water under pressure until the middle of the 20^{th} century, the measures used were mostly based on the principle of draining water, collecting the groundwater and feeding it to a discharge. Drainage of a tunnel has the effect of lowering the groundwater table to a new steady state (Fig. 5-15).

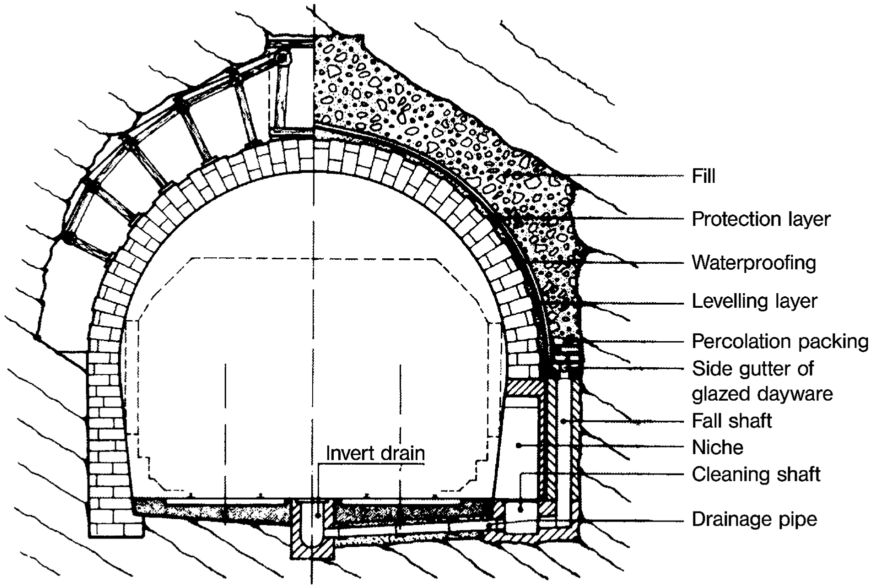


Figure 5-15 Drainage system of an old rail tunnel.

Waterproofing technology for underground tunnels has been developed from many years of experience with roof waterproofing, waterproof troughs for buildings and industrial facilities and the waterproofing of cut-and-cover tunnels. Materials and installation procedures were sometimes used without paying sufficient attention to the special features of tunnel construction, the characteristics of the materials used and the restricted space in a tunnel. The limited possibilities of checking quality, problems locating leaks and the difficulty of repairing tunnel waterproofing systems in comparison with the applications mentioned above also led to difficulties. Manufacturers of construction materials often saw the waterproofing of a tunnel as packing without taking care how this packing was to be installed, and that there are many points of detail, which have to be solved such as how to guarantee that this packing would fulfil the waterproofing requirements after its installation. Many of these questions were left to the company responsible for waterproofing, which normally works as a subcontractor to the main contractor. It is beneficial that it has now become standard practice that the waterproofing is designed and detailed as a constructional element within the overall structure.

Due to the frequent problems and difficulty of repairing them, it was urgent to find an alternative solution. The development of technologies and construction processes, however, was also towards construction processes without waterproofing. Fig. 5-16 shows the basic principles of protecting a tunnel against damp or water ingress.

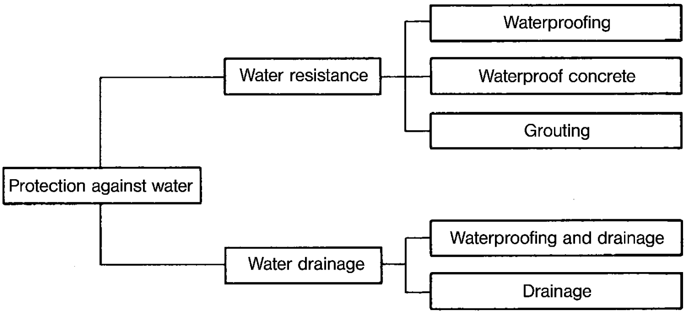
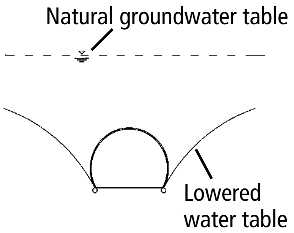
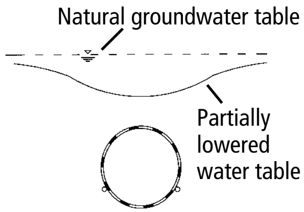
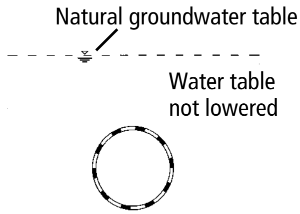


Figure 5-16 Basic principles of protecting a tunnel against damp or water ingress.

There are altogether three possibilities (Table 5-13). The tunnel can be constructed

– to resist water under pressure without drainage (watertight),
– with pressure-regulated drainage (drained) or
– with free drainage (drained).

Table 5-13 Basic systems.

Free drainage	Pressure-regulated drainage	Without drainage
		
Water drainage until complete pressure relief Drained water quantity cannot be regulated Waterproofing against unpressurised water Structurally unlimited scope of application	Water drainage until selected limit water level or water drainage until selected or permissible water drainage quantity Waterproofing against reduced water pressure Structurally unlimited scope of application	No water drainage Waterproofing against natural water pressure Structurally limited scope of application

The decision between these basic principles has far-reaching consequences for the cross-sectional shape, the constructional details of the inner lining, the excavated cross-section, the waterproofing system and possibly also the drainage system and is thus one of the most important decisions to be taken by a design team. All three variants have their advantages and disadvantages so there is no simple right or wrong solution.

Construction **without drainage** has the advantage of no permanent impact on the natural groundwater regime. The groundwater also plays no role in the operation and maintenance of a watertight tunnel. The disadvantage of this principle is the higher construction cost due to the larger excavated sections since an invert arch is always necessary, also thicker linings with more reinforcement and elaborate waterproofing measures. As demonstrated by many practical examples, it is difficult to ensure that the construction will really be waterproof despite the elaborate construction. The measures, which may have to be undertaken to cure leaks, can be of a considerable extent and can even cost more than the original construction.

The principle of **pressure-regulated drainage** is that the groundwater table is only partially lowered and the level of the groundwater is regulated either by a regulating valve or by a raised outlet opening. One advantage of a pressure-regulated system is that the impact on the groundwater regime remains within specified bounds [90]. The drainage systems of the tunnels constructed until now on this principle have also proved to require relatively little maintenance. The cost of construction of such a tunnel is very high due to the pressurised water and the elaborate drainage system. Altogether, there is no conclusive experience with this drainage principle, since only two pressure-regulated tunnels are in operation in Germany.

Most tunnels until now have been constructed with **free drainage** according to the principle "drained tunnel with umbrella waterproofing". In contrast to the two types already described, the construction cost of a drained tunnel is relatively low, since elaborate pressure-tight waterproofing and an inner lining designed to resist pressure are not required. In addition, such tunnels function relatively reliably. The disadvantages that have become increasingly apparent with the construction and operation of many tunnels in recent years are the sometimes impermissible permanent impact on natural groundwater conditions and the expense of maintaining the drainage due to sintering. Depending on the specific design, construction and geological conditions, the cost of maintaining drainage affected by sintering can reach alarming sums. Worries about the effect on vegetation above drained tunnels on the surface have not been confirmed in investigations [143]. In these investigations, operators, responsible authorities, associations, interested groups and local inhabitants were questioned about the environmental impact of tunnel drainage, but no long-term effects were known. Satellite photos have also been evaluated regarding this question without any effects being determined.

5.3.1 Requirements

The requirements on a tunnel waterproofing system derive from the actions on the structure and the requirements for its use. Compliance with the waterproofing requirement, which has to be individually specified for each tunnel, is the main purpose of every tunnel waterproofing system. The specification of a waterproofing system should basically be decided so that it meets the needs of the construction and use periods. The waterproofing must also represent the solution with the best possible cost-benefit relationship.

5.3.1.1 Required degree of water-tightness

The definition of a required degree of water-tightness can cause misunderstandings. In order to avoid this, the "Additional technical conditions of contract and guidelines for engineered structures" (ZTV-Ing.) [263], published by the German Ministry of Transport, Building and Town Development, (BMVBS) gives a classification of water-tightness classes according to [263].

Table 5-14 Classification of the water-tightness of road tunnels according to ZTV-Ing. [263].

Waterproofing class	Damp characteristics	Waterproofing requirements
1	completely dry	The sides of the lining must be so waterproof that no damp patches are detectable on the inner face
2	largely dry	The sides of the lining must be so waterproof that only a slight dampness (e.g. noticeable through discoloration) is detectable on the inner face in isolated locations. When the slightly damp patches are touched with the hand, no trace of water should be left on the hand. Blotting paper or absorbent newspaper laid against the patch may not discolour due to moisture absorption.
3	capillary moisture penetration	The sides of the lining must be so waterproof that only isolated and localised patches, which are wet to touch, occur. Patches, which are wet to touch, are defined in that moisture penetration of the tunnel sides is noticeable and blotting paper or absorbent newspaper laid against the patch discolours due to moisture absorption, but no water drips occur.

No explicit mention is made of the requirements of German Railways here because these now almost correspond to the relevant content of the ZTV-Ing, and the relevant Ril 853 now actually refers to the ZTV-Ing concerning waterproofing questions.

The definite requirements on tunnel waterproofing used as the basis for this classification derive from:

- The geological and hydrological conditions.
- Material requirements,
- Construction process,
- Design detailing,
- Balancing of the risks of production and the associated quality assurance,
- Maintenance,
- User requirements,
- Environmental and waterways protection requirements and
- Cost-effectiveness and maintenance.

5.3.1.2 Requirements resulting from geological and hydrological conditions

This demands the best possible detailed and binding statement of limit values by the employer regarding the following exposures:

- Type, quantity and aggressiveness of the water acting on the tunnel structure:
  a) Groundwater.
  b) Strata or joint water.
  c) Dammed water.
- Maximum water pressure on the waterproofing and the structure.
- Maximum surface pressure in the construction and completed states.
- Possible deformations of the rock mass and tunnel.
- Highest and lowest temperatures in the construction and completed states.
- Tendency to sintering.

5.3.1.3 Material requirements

The general material requirements on the waterproofing material (membrane, waterstops etc.) have to be appropriate for the given constructional and environmental actions. The next section states general criteria, which can be tested for the various materials according to regulations. In the further course of this chapter, these requirements are specified in more detail for plastic waterproofing membranes specifically:

– Permanent resistance against the prevailing mix of soil and water, including all chemical contents.
– Resistance in the tunnel climate against chemical substances, e. g. vehicle emissions or de-icing salt.
– Resistance against all adjacent construction materials. This includes chemicals used for the shotcrete support layer, for ground improvement or grouting of fractured rock or in connection with grouted anchors.
– Resistance against the expected static and dynamic loading in the construction and completed states and against the resulting deformations.
– Adequate mechanical strength at all temperatures, which could occur during the construction period and after completion. Particular attention should be paid to ramp and portal areas, where extreme summer and winter temperatures have to be assumed.
– When different materials are used, their compatibility with each other should be checked.
– Environmental compatibility, which means the materials used should not contaminate percolating water or groundwater.
– Resistance in case of fire. Attention should be paid not only to the flammability but also the release of poisonous fumes.
– Overall, care should be taken that all requirements can be met for the entire period of use of the structure.

5.3.1.4 Requirements for the construction process

Waterproofing must comply with the requirements from the construction process of the overall structure and the installation of the waterproofing:

– Surface features of the waterproofing support in temporary and completed states; the shotcrete normally follows the geometry of the overbreak and this can result in shapes, which the waterproofing cannot adapt to.
– Feasibility of checking.
– Bedding between protective layers. Protection against mechanical damage.
– Division of the waterproofing into compartments in order to be able to localise and repair any leaks.
– Repair capability, in order to be able to remedy any defects occurring during the construction period.
– Multi-layer construction of the waterproofing, or if there is one layer, reliable feasibility of checking its function.
– Simple and defect-free installation feasibility. When systems are built up from roll material, the joining of the individual strips at the seams (in both directions) can be performed reliably and easily, even overhead.

- Adaptability to the structure, for example at edges, grooves and corners.
- Simple construction of working and movement joints.
- Possibility of prefabrication and simple and clear process for hanging and applying; simple installation on the construction site.
- In order to be able to work cost-effectively, the possibility of organised working, repeating the same activities without interruptions if possible.
- Use of laying devices and semi-automatic or automatic devices (for example welding devices).
- Feasibility of testing with appropriate testing procedure.
- It must be possible to clearly describe the waterproofing system and its technical and material specification in the contract and perform regular remeasurement with the employer.
- It must be possible for the contractor to comply with the guarantee requirements of the specification.

The requirements must be adapted to include the development of new technologies in tunnel construction like the construction of inner linings of shotcrete or the use of fibre shotcrete for the outer and inner support layers, but also waterproofing systems for tunnels bored by full-face machines without a closed shotcrete support layer being used.

5.3.1.5 Requirements for design and detailing

A tunnel waterproofing system is much more dependent on and thus integrated into the construction and the construction process than the waterproofing of other structures. The nearest to the requirements are cut-and-cover transport structures that are constructed without working space. Also in this case, the waterproofing support changes position from the temporary construction state to the completed state from inside to outside. Some comments and detailing requirements:

- Construction detailing cannot be left to the material manufacturers or the waterproofing subcontractor alone, since neither of them has enough understanding of the construction, the structural system or the actions on them. They have no information about the special local features of the tunnel.
- Waterproofing subcontractors do not know the details of the construction process. The interactions between construction states, which are inherent in the construction process and influence the construction detailing, are also insufficiently known.
- Waterproofing subcontractors often do not have the necessary engineering capacity available.

It is unavoidable to specify not only the laying instructions, which are generally valid, but also to solve all details constructively and have them tested [129]. This testing cannot be limited to the material properties, which are undertaken by testing organisations nowadays, but also has to be extended to cover the structure to be constructed and the constructive installation of the waterproofing appropriate to the construction process and the construction states. Particularly the construction process has a great influence on the loading, for example the actions of shrinkage, at joints, at butted connections with partial excavations and others. Similar guidelines should actually apply to waterproofing as for acoustic or thermal tests. The aim should not be perfection, but the practice until now, with these details being dealt with by personnel on site without even plans, cannot be allowed to continue. Detail drawings with the complete design and specification of all details and their testing are essential.

5.3.1.6 Maintenance

The following interests of the operator with regard to the maintenance of the tunnel can have a considerable influence on the waterproofing requirements:

- Possibility of checking for leaks.
- Localisation of the damage when leaks are detected.
- Practicality of leak repair.
- Design for the planned use lifetime.
- Protection of the tunnel equipment against water.

5.3.1.7 Requirements of the users

Accidents in road and rail tunnels can have very serious consequences, as has been shown recently by the examples of Mont Blanc and St. Gotthard. The safety and comfort of the users is therefore also of central importance for tunnel waterproofing [82]:

- Prevention of ice formation on the road surface.
- Prevention of icicle formation.
- Prevention of the walls becoming dirty with the resulting darkening of the road conditions.

5.3.1.8 Requirements of environmental and waterways protection

At a time of increased environmental awareness, the requirements to protect the environment and waterways are ever more important. The requirements of environmental authorities are often decisive for the fundamental design of the drainage and waterproofing concept (pressure-resistant, pressure-regulated or with free drainage). The following points have to be considered in the design of the waterproofing system:

- Emission of pollutants from the tunnel into the surrounding ground must be avoided.
- Drainage of groundwater has to comply with the conditions of the planning approval.

5.3.1.9 Requirements of cost-effectiveness

The waterproofing can amount to at least 5 to 10 % of the total construction cost of a tunnel and has to function correctly to protect 100 % of the structure and the goods within it. Defects in the waterproofing have a direct and noticeable effect on maintenance costs. This results in the following cost-effectiveness requirements:

- Minimised construction costs.
- Minimised maintenance costs.

5.3.2 Waterproofing concepts

5.3.2.1 Categorisation

A useful categorisation of waterproofing measures was given in 1964 by A. Peduzzi [264]. He differentiated

- preliminary waterproofing,
- main waterproofing and
- repairs to waterproofing.

This categorisation is still useful because is makes clear that measures to deal with groundwater in tunnelling are necessary during excavation, in support work until the inner lining is complete and also in the operation period.

5.3.2.2 Preliminary waterproofing

As the tunnel is driven in the first pass, the preliminary waterproofing has to ensure that the tunnel can be driven without interruption and that the subsequent main waterproofing can be installed with the required quality. The measures taken at this stage have already been described in Section 5.2.5.

5.3.2.3 Main waterproofing

The main waterproofing denotes the installation of the actual waterproofing construction.

The waterproofing effect can be provided by various methods:

- Waterproof concrete construction. Not only the specification of waterproof concrete, but the detailing of the block and working joints have to be paid attention. A sliding layer should also be provided to reduce crack formation due to toothing between the outer and inner support layers.
- The densest possible shotcrete construction, as is defined and specified for secondary parts of tunnels (cross-passages, escape tunnels, caverns) of German road tunnels according to the “Additional technical conditions of contract and guidelines for engineered structures” (ZTV-Ing.).
- Plastic waterproofing membranes. Waterproofing with glued or hot-applied roll material of bitumen or polymer bitumen is no longer usual today, although it was used on occasions in the 1960s.
- Sprayed waterproofing. Liquid plastic waterproofing systems have come into consideration again, and some products are available on the market. Their limits of application do, however, correlate very strongly with the constraints on site (air moisture content, ventilation etc.).
- Metal waterproofing. The use of metal for waterproofing is rare in transport construction. One example would be the steel armouring that is commonly used in power station construction.

For the selection of a waterproofing system for two-layer tunnel structures, various possible combinations are described in the “Recommendations for waterproofing systems in tunnelling”, published by the German Geotechnics Society in collaboration with the BMVBS and the DB AG Table 5-15.

Table 5-15 Selection of a waterproofing system depending on the groundwater conditions [EAG-EDT].

No.	Waterproofing membrane geometry	Hydrostatic pressure head above the tunnel invert in m	Chemical concrete attack[1] (exposure class)		Required additional measures		
			weak, moderate (XA1, XA2)[2]	heavy (XA3)[3]	Waterstops		Integrated grouting system[4]
					internal	external	
1	umbrella	without (no standing water permissible)	membrane 2 mm[5]	–	no	no	no
2	umbrella		waterproof concrete WUB-KO[6]	–	present depending on system	no	no
3	all-round		–	membrane 2 mm[5]	no	no	no
4	all-round	up to approx. 30	membrane 3 mm[5]		no	yes	yes
5	all-round		waterproof concrete WUB-KO[6]	–	present depending on system	no	no
6	all-round	from approx. 30 to approx. 60	waterproof concrete WUB-Komod[6] + membrane 3 mm		no	yes	yes
7	all-round		two layers membrane[5] (rock side 3 mm + air side 2 mm)		no	yes	yes

[1] Classification of concrete-aggressive environments according to DIN-Fachbericht Beton 100
[2] Corresponds to „weak", „strong" according to DIN 4030
[3] Corresponds to „very strong" according to DIN 4030
[4] Normally to be provided under water pressures from 10 m head
– for single-layer membrane waterproofing, compartments between plastic membrane and inner lining, groutable
– for two-layer plastic membranes, chamber element between the two layers, groutable
[5] Concrete specification requirements for the inner lining according to Ril 853 Modul 853.4004 or ZTV-ING
[6] Waterproof concrete WUB-KO according to ZTV-ING corresponds to WUBK according to Ril 853, concrete specification requirements for the inner lining according to Modul 853.4004 of Ril 853, WUB-Ko$_{mod}$ see Section 2.3
[7] Under higher water pressures, special measures may have to be taken, which are to be specified for each project

5.3.2.4 Repair of waterproofing

This can denote a planned addition to the main waterproofing or repair works to leaks in a tunnel. The most common repair measures undertaken in tunnel refurbishment are:

– Grouting. If the waterproofing is specified with one or two layers of plastic membrane, grouting hoses can be provided for the repair of any leaks. This should enable full-surface filling of the space behind the inner lining with grout in case of defects. The grouting system should always be installed in combination with a compartmentalisation, in order to prevent uncontrolled transverse penetration of the grout and should be suitable for repeated grouting.
– Shotcrete and sprayed mortar. Combined with water drainage channels, this is the most commonly used method of refurbishing masonry tunnel linings [264]
– Additional complete lining, as installed in the Munt La Schera to Livigno Tunnel [156] or at the Brenner [163].

5.3.3 Waterproofing elements and materials

Waterproofing systems can be categorised according to their mechanical behaviour into:

– Rigid systems.
– Flexible systems.

This division is primarily according to the material properties. An overview of waterproofing systems is given in Fig. 5-17.

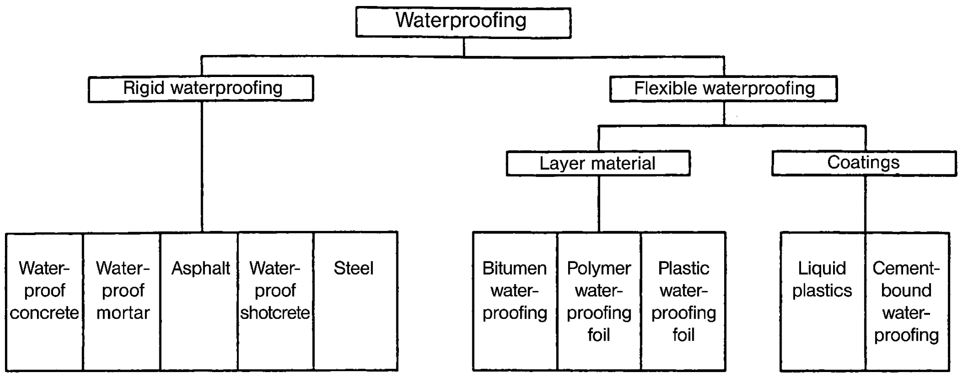


Figure 5-17 Differentiation of waterproofing systems according to materials and material properties [82].

5.3.3.1 Waterproof concrete

Definition

Waterproof concrete is concrete, with composition and processing that make it capable of permanently holding water acting from one side under limited pressure. According to DIN 1045, Section 6.5.7.2, the maximum permissible penetration depth must not exceed 5 cm in a test according to DIN 1048 for elements with a thickness of between 10 and 40 cm. Waterproof concrete can be placed in-situ or precast as segments. Employers often require more stringent specifications for structures of waterproof concrete.

Construction

Requirements. Any crack formation impairs the water-impermeability. The construction must be designed to exclude any danger of crack formation due to settlement, movement of the structure or static or dynamic loading. Waterproof in-situ concrete is often provided with a waterproofing protection layer because any leaks can then be repaired more easily. When it is used as a final lining without an additional waterproofing layer, DB guideline 853 requires the following:

- The water head may not exceed 30 m.
- Crack formation due to structural loading can be securely prevented to more than the usual degree in a verification of crack width limitation.
- The surrounding ground and groundwater are free of content that is aggressive to concrete, or the degree of attack is at the worst "weakly attacking" according to DIN 4030.
- The type of construction delivers economic or other advantages over the use of a separate waterproofing layer.

An additional waterproofing layer of appropriate waterproofing material should be provided for tunnel sections susceptible to frost penetration, for tunnels with special equipment, for external walls with greatly different stiffnesses (like corners, box-outs, weak points, strengthening) and for tunnels beneath roads with shallow cover. Waterproof concrete without additional waterproofing may only be used under higher water pressures than 30 m head when a special exception has been issued.

Structural design. The forces and moments acting on a section are determined for waterproof concrete as for normal concrete according to DIN 1045. Since the occurrence of cracks endanger the serviceability and endanger the structural safety in the long term due to increased risk of corrosion of the reinforcement, tension stresses in the concrete are to be reduced to less than the tension strength. The crack width is limited through an appropriate selection of the degree of reinforcement, steel stress and bar diameter. Additional verifications are to be preformed for the reduction of crack formation or the limitation of crack width according to DIN 1045, Section 17.6.2.

Ground. If the geological investigation states that larger settlements are to be expected and cannot be contained, then a structure with a waterproof layer is more appropriate. This also applies to areas of mining settlement, for large cross-sections and for uneven temporary support as connection surfaces.

Concrete sections, block joints. The cross-sections should have a simple, clear shape. In order to avoid the formation of shrinkage cracks, block lengths less than 8 m should be selected, in other words half the length of block lengths when waterproofing is provided. Sections should by thick enough to enable successful placing and compaction of the concrete, normally more than 30 cm. Special low-shrinkage cements are often used in tunnel construction. In order to ensure unrestricted shrinkage of the inner lining, large irregularities in the outer layer are to be levelled and when the surface is rough, an appropriate separating layer should be provided. Measures known to have been used include limewash, bitumen paint, foils, fleece and mortar levelling layers.

Working joints are unfortunately unavoidable. Joint surfaces should be prepared according to specification before concreting. Steel waterproofing strips or waterstops should be cast into the joints. Grouting hoses are necessary to enable subsequent grouting in the crown.

In order to take into account the different shrinkage behaviour of various construction elements (transition invert – wall), the nominal reinforcement in the distribution direction of horizontal working joints is to be doubled up to 1.5 m above the joint.

The construction of a section without working joints requires an expensive formwork unit.

Concrete cover. Corrosion increases the volume of the reinforcement and can cause concrete spalling. Since the penetration of water can never be completely prevented, the minimum concrete cover inside the tunnel should be nominally 50 mm and on the outside 60 mm.

Concrete technology

Water-cement ratio. According to DIN 1045, the water-cement ratio of elements 10 to 40 cm thick may not exceed 0.6 or for thicker elements 0.7 (Fig. 5-18). In order to consider the variation of site mixing, the *W/C ratio* should be set about 0.05 less for site batching, although lower values are desirable. Concrete is tested according to DIN 1048.

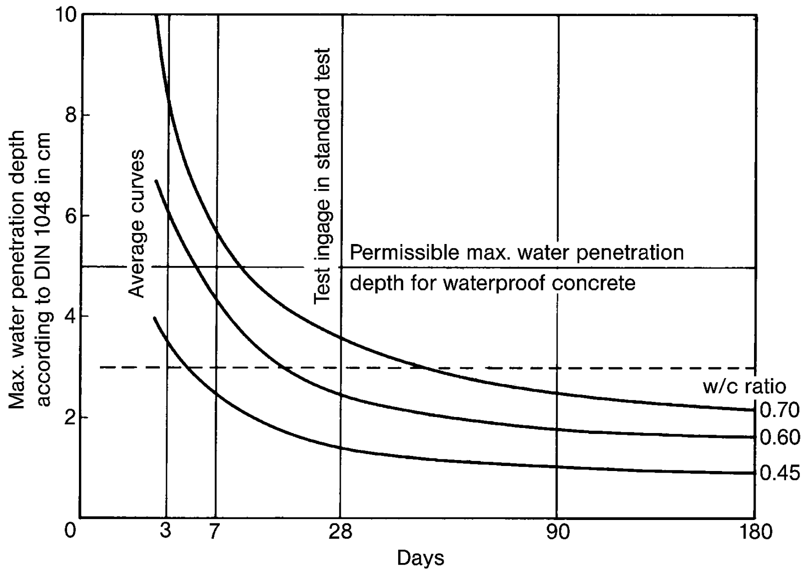


Figure 5-18 Waterproof concrete. Reduction of the maximum water penetration depth with increasing hardening duration [26].

The *aggregates* should be as dense as possible and have a compact shape, and the grading curve should lie within the most favourable zone.

A *cement* should be selected, which only develops low heat of hydration, in other words a slow-hardening cement; this requirement is in contradiction to the required short striking time. The fines content (passing 0.063 mm sieve) should be between 350 and 400 kg/m^3 solid concrete.

Production as B I. Waterproof concrete with a lower strength class than B 35 may also be produced under the conditions for B I. The cement content in this case with 0 to 16 mm aggregates should be at least 400 kg/m^3, for 0 to 32 mm aggregates at least 350 kg/m^3 and the grading curve should lie within the most favourable zone.

Production as B II. According to guideline 853 of the Deutsche Bahn AG: waterproof concrete should be produced as concrete of strength class B II. The minimum thickness should be:

- For single-layer linings $d = 0.40$ m.
- For the inner lining of two-pass support $d = 0.30$ m.

The bulletin "Impermeable construction elements of concrete" from the German Concrete Association should be complied with. The rules laid down in the DS 804 Anlage 10A of DB AG are to be applied as appropriate.

Concrete admixtures. Waterproofing admixtures (DM) close the pores and reduce the capillary suction capacity, plasticizers (BV) and air-entraining agents (LP) reduce the water demand, improve workability and produce denser concrete. The concrete properties can be improved by the addition of admixtures, although this effect is often regarded by employers as not effective on its own.

Processing

Formwork. Since the concrete should be kept damp for as long as possible, the use of formwork with a sealed skin, for example made of multi-layer boards, is advantageous. This ensures that the concrete can move while it hydrates and lowers the risk of cracking. The formwork ties shown in Fig. 5-19 are fitted with plastic cones, which are unscrewed after concreting and pointed with waterproof concrete plugs. The central part of the tie remains in the concrete.

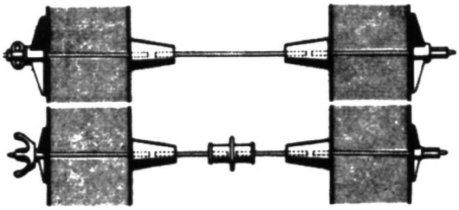


Figure 5-19 Formwork ties for waterproof concrete.

Placing. Particular attention should be paid to placing the concrete correctly. Mistakes made during concreting can lead to separation of the mix and result in leaks. Uniform compaction of the poured layers with a vibrator is important. For closed tunnel formwork units, formwork vibrators are mostly used. Inhomogeneities are to be avoided and the hardening concrete should be vibrated through once again as late as possible.

After-treatment. The construction element should be protected against drying out too fast, which generally means keeping it damp for at least seven days or the provision of other appropriate measures like spraying a sealing agent, and it should be exposed to water as late as possible. Permanent air draughts in the tunnel should be avoided, although heat build-up should also be considered when the striking times are very long.

5.3.3.2 Water-resistant plaster, sealing mortar, resin concrete

Synthetically modified mortars and concretes are produced by adding, for example, polymers or artificial resin to the mortar mix. So-called resin-modified mortars or concretes are made by the addition of water-compatible reaction resin to the fresh mortar or concrete with the intention of improving certain material properties, such as the waterproofing. The reaction resin used is normally epoxy resin, the hardening of which has to be matched to the hydration of the cement in order that both binders can work together. The resin content can be altered to have a targeted effect on the properties of the mortar. Resin concretes and mortars can be placed in formwork or sprayed.

In tunnelling, there is scarcely any experience of the use of synthetically modified concretes or mortars as sealing materials. Such an application would, however, be conceivable for single-pass lining [82]. Tests at STUVA in collaboration with the industry have shown that resin-modified mortars are suitable for tunnel waterproofing [213] and have potential for further development.

The advantages of resin concrete waterproofing are:

– The first layer of shotcrete can be completely installed in the conventional way as excavation support.
– Sprayed concrete application achieves a relatively quick installation of the waterproofing.
– Conventional shotcrete machinery can, with certain alterations, be used for application.
– The waterproofing is relatively insensitive to damage from following works.

The disadvantages of waterproofing with resin concrete are:

– The quality of the waterproofing is essentially determined by the quality of the application.
– Cracks can occur in the waterproofing layer through static or dynamic loading.
– Resin concrete is not fire-resistant.

5.3.3.3 Bituminous waterproofing

General. Loose-laid waterproofing as sheet material has been common in tunnelling for many years. It is usually installed followed by a concrete inner lining.

Ready-made plastic waterproofing membranes are now used almost exclusively in tunnelling, although bituminous waterproofing membranes can sometimes be used to waterproof cut-and-cover tunnels. For the sake of completeness, these are now considered in more detail.

Materials. Bitumen is made from heavy crude oil. It is a dark-coloured, semi-solid to resiliently hard, meltable mix of hydrocarbons with good resistance against most inorganic acids, salts and alkalis. The number (such as B 45) of bitumen according to standard denotes the penetration depth in a penetration test, and for blown bitumen a double number (such as 85/25) denotes the softening point in a ring or ball test (85) and the penetration depth (25). The addition of mineral fillers or rubber can raise the softening point and lower or at least hold the combustion point (filled bitumen). Bitumen is used in various forms in construction:

– Bitumen solutions are solutions of bitumen in solvents, which vaporise after application.
– Bitumen emulsions are mixtures of finely distributed bitumen in water, which evaporates after application [79].
– Asphalt is a mixture of bitumen and minerals, most frequently with 22 % bitumen for better plasticity.

Bituminous masses have a very good sealing and waterproofing effect but little strength. A backing medium is therefore normally provided in the form of fibrous organic material (wool felt board, jute), inorganic textiles (fibreglass mat), bitumen roll material with metal inlays such as aluminium or copper foil or corrugated copper band as reinforcement. Bitumen polymer waterproofing membranes (predominantly polyisobutylene (PIB) according to DIN 16 935). All organic fibre materials must be impregnated with bituminous material on order to provide a waterproofing effect and against rotting.

Commonly available products are:

1. Felts: felt impregnated with bitumen but without a bituminous covering layer is called “roofing felt”. This material is described according to the weight of the unimpregnated raw felt (500 denotes 500 g/m²).
2. Hot-applied bitumen (FSK-Bahn, DIN 52130): Bitumen waterproofing material with a bitumen covering layer on one or two sides each about 1.5 to 2.5 mm thick; Inlays of glass fibre, jute or polymer mats, metal bands or plastic foils; applied hot.
3. Bitumen latex (BL): Bitumen mixed with 15 to 20 % by volume addition of latex rubber, mostly based on chloroprene, to improve the mechanical properties of the bitumen. BL waterproofing is normally sprayed as an emulsion in many layers. Plastic sheet material, for example CSM, is used as a carrier insert [86].

Waterproofing layers and application process. Waterproof coating: Applied with brushes or brooms or sprayed. Primer (normally liquid without heating) followed by many layers of waterproofing (applied hot or cold, trowelled as a paste). Waterproof coatings cannot achieve permanent protection and are thus not of significance in tunnelling [127].

Trowelled: Mixtures of bitumen and rock dust/natural asphalt raw dust and also possibly asbestos fibres or similar as filler are applied hot or cold as a viscous mass in one or more layers with wooden spreaders or trowels. Each layer should be at least 8 mm thick. This is only of significance in tunnelling for repair work.

Brushed application according to DIN 4122: The support and waterproofing membrane are painted with bituminous adhesive at 180 to 200 °C, the material is rolled out and pressed down. The consumption is about 2 to 4 kg/m² per layer. In tunnelling, this is nowadays only interesting for cut-and-cover construction.

Poured and rolled-in process according to DIN 4122: Bitumen adhesive at 200 °C is poured between the layers and a roller is used to press down. For overhead application, the adhesive mass is poured onto the hanging end of the roll. This process is laborious but reliable when mixed correctly. The process is only be used for cut-and-cover tunnels today.

Welded process (also flamed melting adhesion or flamed process): requires waterproofing roll material with a sufficiently thick layer of bitumen (1.5 to 2 mm each side). After the application of a primer coat, the material is heated with a propane torch to make it sticky and rolled out. There are conditions for the application of this process in closed rooms. Due to the fire risk in closed rooms, the method is only permitted as hot air welding.

Bitumen latex emulsions are sprayed in many layers each 2 to 3 mm thick to give an overall thickness of 5 to 15 mm. The emulsion and a precipitant are sprayed separately so that they mix in the air, which forms a closed layer on the wall. The emulsion water runs down or has to be sucked up. The rubber provides high elasticity and good stretching and ageing properties. Despite good experience with this process in the Czech Republic, it has not become established. Good results have been achieved in combination with plastic foils.

Evaluation. Bituminous waterproofing materials are now hardly ever used in underground tunnelling. The reasons for this are:

- bituminous waterproofing materials need a mostly dry and flat support, which would require profiled levelling of the tunnel sides, often with extensive profiling at a high cost in time and money,
- the installation of the reinforcement for the inner lining was found to easily damage the waterproofing layer,
- groundwater that cannot be completely collected leads to the waterproofing not adhering to the support with a risk of defects,
- fire risk of using torches with open flames,
- the application requires more skilled personnel than, for example, ready-made plastic membranes.

On the other hand, the application in many layers is of course a great advantage.

5.3.3.4 Plastic waterproofing membranes

Materials and requirements. Plastic waterproofing membranes are normally of thermoplastic sheets with the advantage of good mechanical properties (high tension strength, high failure strain, good flexible behaviour). Many different materials have been used in the past for the production of membranes, which may still be encountered in refurbishment work. The following materials are possible:

- Ethylene copolymer modified bitumen (ECB).
- Polyethylene (PE).
- Polyisobutylene (PIB).
- Chlorosulphonated polyethylene (CSM).
- Soft polyvinyl chloride (PVC).

In addition to these thermoplastics, thermosetting plastics have also sometimes been used. The most common thermosetting plastics for waterproofing are:

- Unsaturated polyester resin (UP).
- Epoxy resin (EP).
- Polyurethane (PUR).
- Polysulphide.

Thermosetting plastics have been used a few times but have not become established due to their brittle behaviour. Particularly the subsequent transfer of the waterproofing support from outer support layer to inner lining led to damage.

Today only PVC or polyolefin foils are used for waterproofing tunnels. Numerous methods of testing the quality of these materials have been developed in recent years in order to check their suitability for application and durability. All the necessary tests and the associated standards have been collected in the EAG-EDT. Due to the enormous extent of this, only the most important tests are stated in the following tables:

Table 5-16 Summary of the most important quality tests for plastic membranes from [EAG-EDT].

Property	Testing according to		Requirements	
	standard	EAG-EDT section	polyolefin-based	PVC-P-based
General condition	DIN EN 1850-2	4.12.3	Free from bubbles, cracks, casting defects and inclusions of foreign matter, complete bonding of the signal layer with the base material	
Straightness (g) Flatness (p)	DIN EN 1848-2	4.12.4	$g \leq 50$ mm $p \leq 10$ mm	
Dimensions Thickness without signal layer	DIN EN 1849-2	4.12.6.1	Nominal thickness: 4.0 mm; 3.0 mm; 2.0 mm Average value: ≥ nominal thickness Smallest value. ≥ average value – 5 % Largest value. ≤ average value + 5 %	
Behaviour under tension loading Secant modulus ($E_{1\text{-}2}$ modulus) in longitudinal and transverse directions Tension strength in longitudinal and transverse directions Failure strain in longitudinal and transverse directions	DIN EN ISO 527-1 & 3 Test bodies 5	4.12.8.1	≤ 100 N/mm² ≥ 15 N/mm² ≥ 500 %	≤ 20 N/mm² ≥ 12 N/mm² ≥ 250 %
Piston penetration force	DIN EN ISO 12 236	4.12.10	determine value	
Curved arc strain in burst test	DIN EN 14151 Test body Ø 1.0 m	4.12.11	≥ 50 %	
Water permeability (sealing against liquids)	DIN EN 14150	4.12.13	waterproof	
Behaviour in perforation test	DIN EN 19956 Appendix G 500 g	4.12.17	for nominal thickness 2mm: no leaks under 750 mm fall height for nominal thickness 3mm: no leaks under 1250 mm fall height	

Table 5-16 continued

Property	Testing according to		Requirements	
	standard	EAG-EDT section	polyolefin-based	PVC-P-based
Long-term resistance behaviour after heat alteration (70 d, 80 °C)	DIN EN ISO 11925-2 DIN EN 13501-1	4.12.21		Alteration of the tension strength and failure strain compared to delivery state ≤ 20 % Behaviour when folded cold (-20° C): no cracks
Fire behaviour	DIN EN ISO 11925-2 DIN EN 13501-1	4.12.23	Class E	
Behaviour of joint seam in shear test Short-term factor of joint seam Behaviour of joint seam in peel resistance test Peel resistance	DIN EN 12317-2 DIN EN 12316-2		Tear outside the joint seam ≥ 0.6 Peeling is permissible as long as peel resistance is achieved ≥ 6.0 N/mm²	

The above table does not include any reference to the recently developed autoclave tests, which can give information about the ageing process of plastic membranes and enable a theoretical lifetime to be determined. More information can be found in the EAG-EDT.

Waterproofing layers. The following sequence of layers is currently usual for waterproofing with plastic membranes (Fig. 5-20) [143]:

– Shotcrete outer support layer as waterproofing support.
– Geotextile protection layer.
– Plastic waterproofing membrane.
– Inner lining.

Laying the waterproofing. A 204 g/m^2 synthetic fleece is first fixed by shooting to the tunnel sides. This serves to protect the waterproofing membrane against mechanical damage, for example perforation due to the uneven surface of the underlying shotcrete. Due to the danger of hydrolysis of polyester, geotextiles made of this material should be avoided. The fleece also prevents any toothing or shear transfer between the tunnel sides and the waterproofing membrane in sections under compression loading, and avoids areas of excessive tension stress in the membrane due to deformation of the rock mass or shrinkage movement. The fleece layer also serves as a drainage layer to collect any residual water and drain it down to the invert.

The fleece is fixed at points with steel washers (Fig. 5-21) of 20 mm diameter. The spacing of the nailed fixings reduces towards the crown to ensure that the fleece does not hang down. When the tunnel vault is uneven, the nails must be shot at the deepest points, as otherwise there would be a danger that fleece and waterproofing membrane could tear when the inner lining is concreted. The inner lining requires the best possible void-free contact with the outer support for rock mass structural reasons. The external fleece does not need welded seams, but should be overlapped.

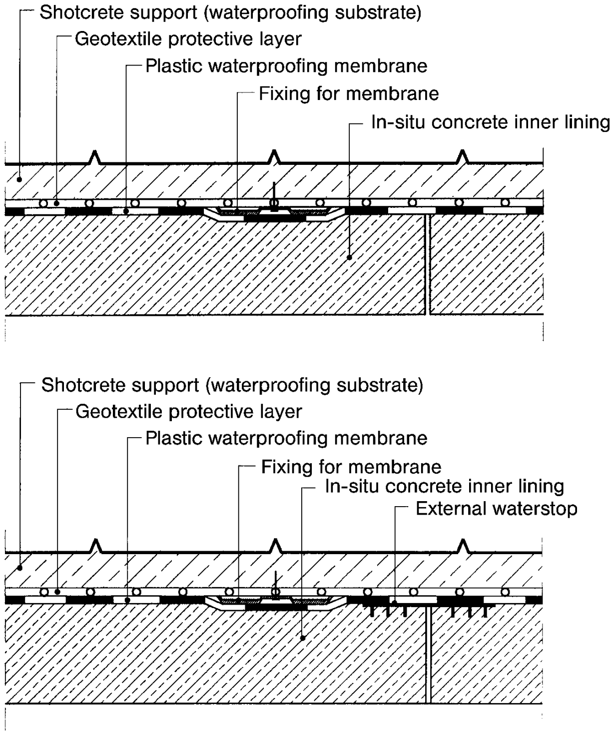
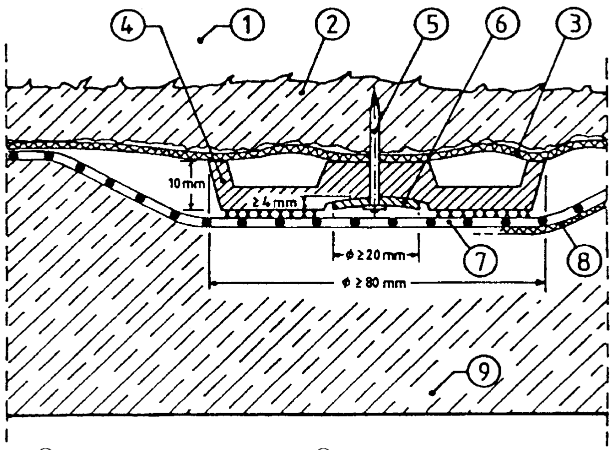


Figure 5-20 Waterproofing detail with single-layer plastic membrane at the sides and in the crown (top) and in the invert (bottom) [EAG-EDT].



① Rockmass
② Waterproofing substrate (outer support layer)
③ Protective layer
④ Fixing element with stiffening ribs
⑤ Shot nail
⑥ Metal washer
⑦ Plastic waterproofing membrane
⑧ Protection for membrane if required (e.g. in invert)
⑨ Inner lining

Figure 5-21 Point fixing of protection layer and plastic waterproofing membrane to the support [193].

Soft PVC waterproofing membranes have become predominant in recent years. In order to ensure adequate resistance to damage during installation and in the completed state, the thickness should be 3.0 mm. Thicknesses of more than 3 mm are not recommended due to the increasing stiffness of the material. The roll or sheet material is fixed at points, which are either the same as those already used for the fixing of the fleece as shown in Fig. 5-21 or newly shot for the purpose, but always at the deep points of uneven tunnel vaults. The number of fixing points per square metre should be at least 0 to 0.5 in the invert, 1 at the sides and 2 to 3 in the crown.

The individual strips or pre-fabricated sheets should be overlapped at their edges by at least 5 cm. The seams are welded as double seams with two 15 mm wide single seams suitable for testing and are normally welded by an automatic welder. Groundwater emerging from the rock mass and building up behind the waterproofing must be prevented from sagging the membrane during the construction phase. The danger of breaking the skin of the membrane is at the greatest before the inner lining can support it (Fig. 5-22).

Figure 5-22 Plastic waterproofing membrane.

Concreting imposes the first significant loading on the waterproofing. When the space behind the formwork is filled, the weight of the pumped concrete acts fully on the waterproofing and causes tension loading, which can separate the membrane from the fixing elements. This separation means a localised defect in the waterproofing membrane, which cannot be allowed to lead to tearing of the material.

The waterproofing should preferably be laid from travelling scaffolding, often equipped with mechanical laying aids. This enables the installation of prefabricated sheets with a width of up to 6 m. The length of the sheet corresponds to the developed length of the tunnel vault, or at least half. Such large sheets cannot be moved by hand due to their weight, above all overhead. Nonetheless, they are to be preferred to strips of 1 to about 2 m width since this reduces the number of seams, which have to be welded on site, often under difficult conditions.

Laying machines are mounted on a tracked scaffolding, which is adapted to the tunnel profile (Fig. 5-23). Considering the uneven tunnel sides, it is important that the already laid membrane is not damaged by forwards or sideways travel of the laying machine. The provision of rubber-wheeled spacing rollers mounted on springs is strongly recommended.

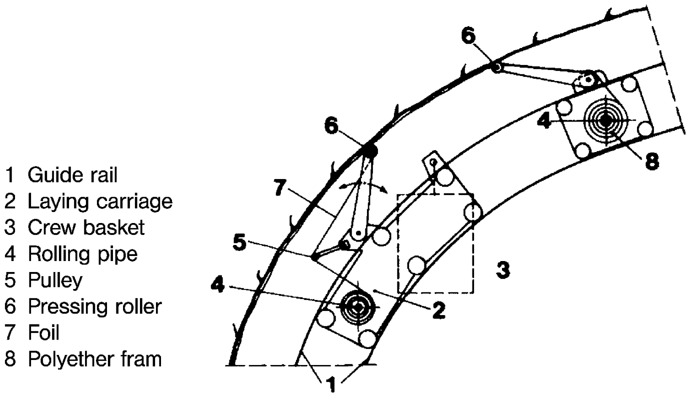


Figure 5-23 Laying machine for the waterproofing of the Seelisberg road tunnel.

When plastic membranes are used to waterproof against water under pressure, compartment seals should be provided at every concrete block joint as ring bulkheads with connection strips (Fig. 5-24 and Fig. 5-25) [193]. The connection strips should be welded watertight to the loose-laid waterproofing membrane and to their welded seams.

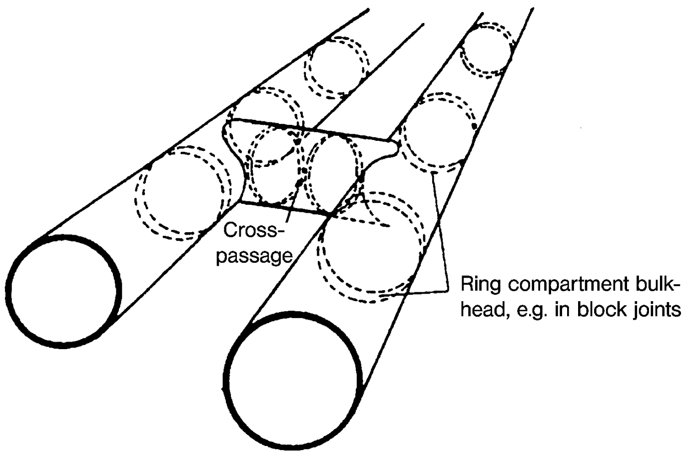


Figure 5-24 Principle of a compartment system [193].

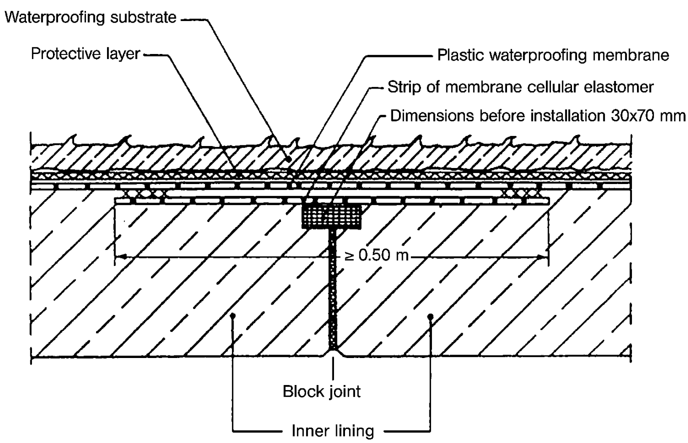


Figure 5-25 Bulkhead sealing at a block joint with single-layer waterproofing membrane [EAG-EDT].

Finally, voids remaining in the tunnel crown are grouted with cement grout. Special measures need to be taken for the adequate ventilation and filling of the locking anchors of the connection strips. The installation of additional slightly tapered concreting pipes in the crown, as shown in Fig. 5-26, has proved successful. The pipes provide ventilation while the concrete is poured and are later used to inject cement suspension “wet-in-wet”.

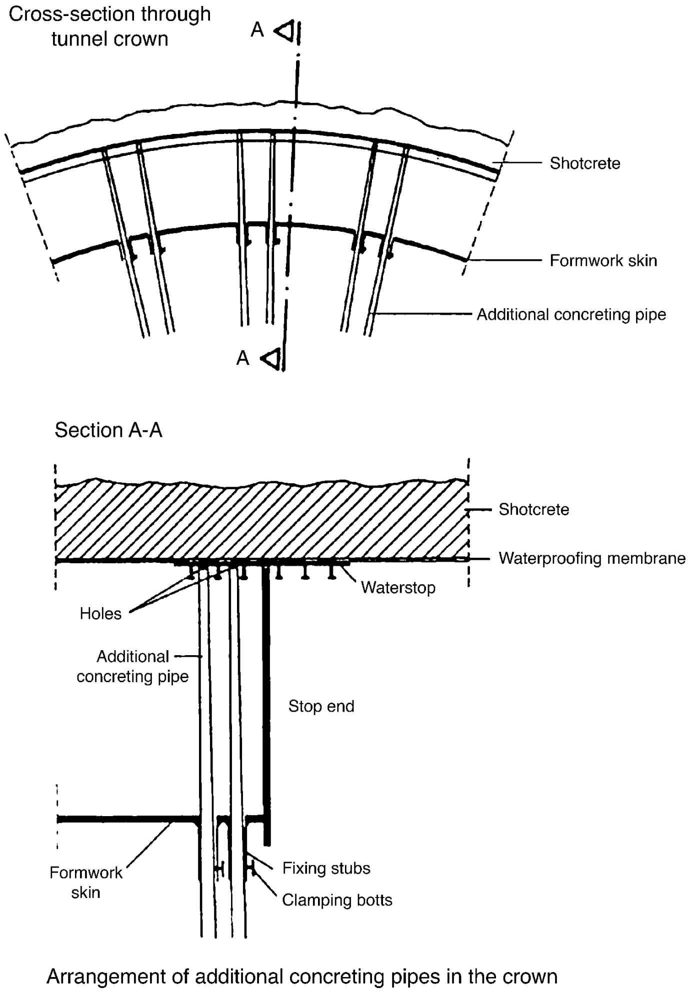


Figure 5-26 Arrangement of additional concreting pipes in the crown [193].

Waterproofing support. The loading in the construction state is different from that in the completed state, which often provides scope for mistakes. The waterproofing support in the construction state, the temporary support layer, is for economic reasons very dependent on the construction process, which itself depends on the geological conditions and the constructional requirements. The first purpose of the temporary support is to support the rock mass and its function as a support for the waterproofing layer is only secondary; this secondary task is often considered of little importance [129]. The temporary support layer typically consists of steel profiles, rock bolts and shotcrete; nonetheless, larger overbreak occurs in all type of rock. The overbreak can have a magnitude of metres and is seldom completely filled and thus often provide an inadequate waterproofing support.

There can also be considerable local deviations in the geometry of the temporary support, redistribution of the ground pressure, shrinkage of the inner concrete construction. However it also has to be considered that a perfectly flat waterproofing support would often be impossibly expensive.

The waterproofing support must be constructed so that its surface complies with the requirements of the following process regarding geometry and its material properties for fixing. Regarding the requirements to be met for geometrical imperfections of the surface, two cases have to be differentiated [193]:

a) Areas, which are designed with a completely or nearly constant cross-sectional shape with a slight curve of the waterproofing in one direction.
   In such areas, the laying and joining of the waterproofing membrane sheets can normally be carried out without any special measures as long as the following requirements are complied with (Fig. 5-27).
   - Adjacent high and low points of the waterproofing support should not differ by more than 20 cm in their distance from the tunnel centreline.
   - The deviation *a* measured in this way should not be more than 10 % of the relevant spacing *l* of the adjacent high and low points.
   - The radius of curvature *r* of the surface of the waterproofing support should not be less than 5*a*.

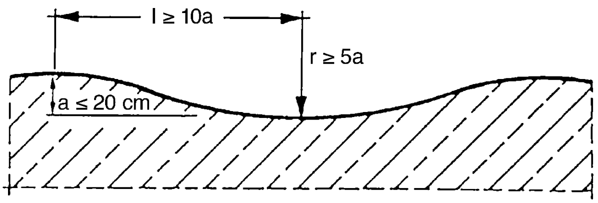


Figure 5-27 Permissible deviations of the waterproofing support [193].

b) All cases, which do not fit the description in a), especially niches and changes of section.
   In these areas, the standard method of laying the membrane with deviations being simply passed over is not permissible. The membrane has to be adapted exactly to the unevenness of the support and its final location by appropriate cutting to shape, fixing and welding.
   The criteria under a) for geometrical imperfections of the waterproofing support do not have to be complied with on a large scale where ridges and valleys are inevitable due to planned niches and changes of cross-section.
   Ridges and valleys in the waterproofing support should be rounded as far as possible, with a minimum radius of 20 cm (Fig. 5-28).

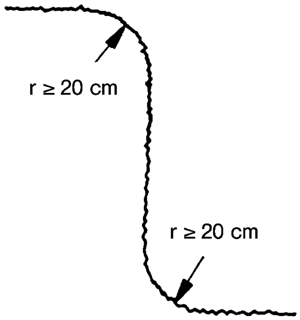


Figure 5-28 Minimum radii at niches and changes of section [193].

In general, the temporary support layer should be levelled with shotcrete or sprayed render to remove ridges and peaks, fill larger local overbreak and cover anchor heads before the installation of the waterproofing. Two examples of details of levelling layers on the waterproofing support are shown in Fig. 5-29.

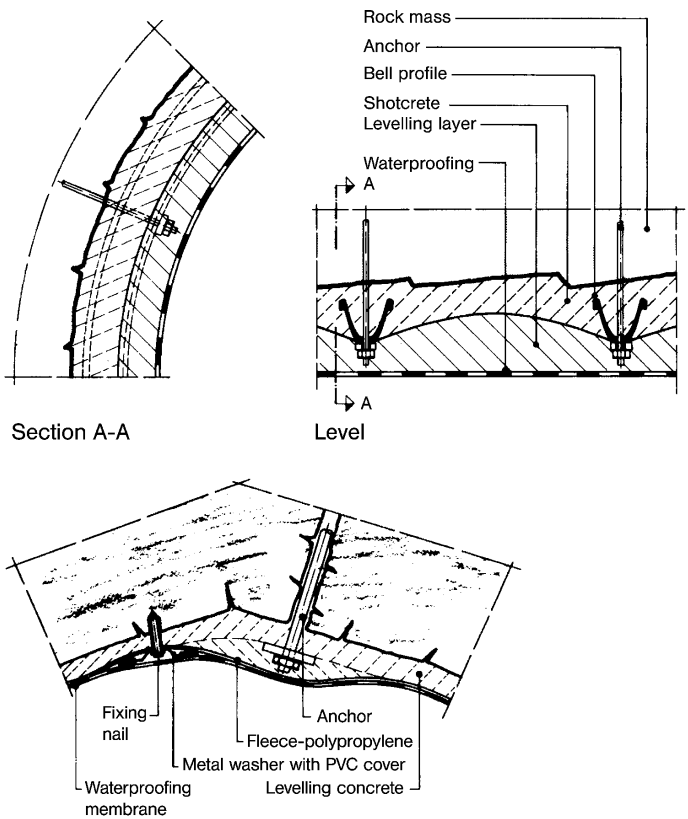


Figure 5-29 Waterproofing with levelling layer. Geometrically precise levelling layer (top), adapted levelling layer (bottom).

5.3.3.5 Sprayed waterproofing

The basic idea of spraying waterproofing is to produce the waterproofing where it is required. The waterproofing material is sprayed in liquid form directly onto the support. The following materials are suitable:

- Reaction resins.
- Cement-plastic combinations.
- Polymers.
- Bitumen-plastic combinations.

The addition of about 20 % by weight of glass fibre (rovings) 3 to 5 cm long can also improve the properties.

The main advantages of sprayed waterproofing are the absence of seams and the reduced amount of work to install the waterproofing. In the ideal case, the cost of providing waterproofing will be reduced.

The main problem with successful application is the requirements for the surface of the support; the waterproofing support cannot have any local bumps larger than the layer thickness (Fig. 5-30). The conditions in a tunnel (dampness etc.) also cause problems for the application. Above all high humidity can delay the setting process of sprayed waterproofing materials.

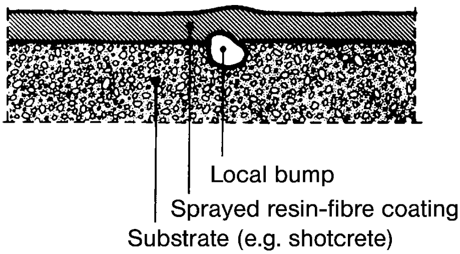


Figure 5-30 Resin-fibre waterproofing with a local bump.

No guideline that is supported by public clients is yet available for the application of sprayed waterproofing in any German-speaking country. At the International Tunnelling and Underground Space Association (ITA) a first design guidance is prepared by a group of industry partners. For this reason, additional investigations will have to be performed for each specific structure or project to establish its suitability in the individual case.

5.3.3.6 Metallic waterproofing materials

Metals are seldom used as pure waterproofing material. Only a few metals are technically and economically conceivable as waterproofing materials. They can be used as thin foils (aluminium, copper, steel) or as thin plates a few millimetres thick (steel). Aluminium and copper foils are used as a mechanical strengthening for bitumen waterproofing materials. Steel plates can be welded watertight at their joints. Welded steel plates, at least 2 mm thick, are used in pressure tunnels. Internal steel armouring of larger thickness is normally intended to resist high water pressures in addition to waterproofing.

5.3.4 Testing of seams in waterproofing membranes

Continuous testing during installation is important since later patching and repair is practically impossible. The following testing methods are used in practice or are regulated in Germany by DIN standards and further guidelines:

1. Testing by visual inspection; additional determination of the seam geometry according to [DVS guideline 2225-5] for waterproofing membranes.
2. Tapping to sound multi-layer waterproofing for voids. This demands particular experience.
3. Testing the behaviour of a welded membrane seam in a shear test according to DIN EN 12317-2.
4. Scratching the seam with a nail or scoring tool.
5. Testing the behaviour of a seam in a peeling test according to DIN EN 12316-2.
6. Blowing the seam from the side with compressed air.
7. Spot checks at some points.
8. Electrical testing with high-voltage. The testing area is probed with an electrode and a spark occurs at defective locations. The precondition is that the support is conductive and the waterproofing layer to be tested has a lower electrical conductivity than the support.
9. Compressed air or compressed water testing of joints: the joint is made as a hose and filled with compressed air or water. Loss of pressure indicates a leak. References like [EAG-EDT] state the necessary testing pressures, durations and permissible pressure loss.
10. Electrical testing of joints: a metal strip is laid in the joint and tested with a high-voltage device.
11. Vacuum testing of joints: similar to pressure testing, but the air is sucked from the outside.

5.4 Tunnel drainage

The use of drainage to reduce groundwater pressure in road and rail tunnels is a functionally reliable and in many cases also economic construction principle in order to be able to construct tunnels under high water pressures. Drainage measures have to be provided for the temporary construction state and the completed state. Drainage measures in the temporary construction state have already been dealt with under 5.2.2.1.

The disadvantages of tunnel drainage that have become increasingly apparent in tunnelling work in recent years are the permanent intrusion into the natural groundwater system, which has already been discussed in the introduction, and the sometimes very expensive maintenance of drainage systems due to sintering (Fig. 5-31 and Fig. 5-32).

Depending on the design, construction and geological conditions, the cost of maintenance due to sintering for cleaning tunnel drainage can be considerable, and can in some circumstances exceed the extra cost of the alternative of a completely waterproofed tunnel. Investigations at the Institute for Tunnelling and Construction Management at the Ruhr University, Bochum have however shown that the basic principle of a drained tunnel still remains an optimal concept even considering the high maintenance cost in many areas due to sintering [209].

Figure 5-31 Sintering of seepage slots.

Figure 5-32 Sintering of a pipe invert.

Many new guidelines and regulations have been published in German-speaking countries in recent years based on this and other investigations, and also experience from large projects where drainage was used such as the new Gotthard Base Tunnel. These are intended to enable the design, construction and operation of low-maintenance drainage systems. There is general agreement in Germany, Austria and Switzerland about the basic construction principles of drainage system design – except for some anomalies. There are noticeable differences of opinion regarding the maintenance and repair of systems and the associated strategies to reduce maintenance costs.

After a brief introduction to the causes of sintering, the design, construction and maintenance of drainage systems are dealt with separately and the central differences between the two most prominent approaches – German and Austrian – are discussed.

5.4.1 The origin of sintering

The chemical and physical processes, which finally lead to deposits in drainage systems, are complex. Sintering consists 95 % of calcium carbonate ($CaCO_3$) with small quantities of magnesium compounds.

The carbonate is a form of the carbonic acid, which is produced by the reaction of water with carbon dioxide (CO_2). Increased carbon dioxide concentration is often a characteristic of groundwater.

The calcium in the compounds comes either from the rock mass encountered or from the construction materials used such as shotcrete, and is dissolved and transported by the groundwater on its way from the surface down into the tunnel drainage (Fig. 5-33). Due to processes, which will be described in more detail below, the calcium dissolved in the drainage water precipitates in the pipes as calcium carbonate and forms a crystalline structure.

Calcium carbonate is thus a product of lime and a form of carbon dioxide and the chemical process of sintering is thus mainly influenced by the lime-carbon dioxide balance.

As the rainwater percolates down from the surface to finally arrive in the tunnel drainage, numerous local conditions can affect the lime-carbon dioxide balance. Chemical and physical conditions can be differentiated according to whether the water tends to dissolve or precipitate lime.

Precipitation processes. The water in the drainage contains water containing either dissolved calcium hydroxide $Ca(OH)_2$ or dissolved calcium hydrogen carbonate $CaH_2(CO_3)_2$. As the water enters the drainage, the conditions change and additional water with different properties is sometimes mixed in. This causes the precipitation along the route from the gully to the discharge point.

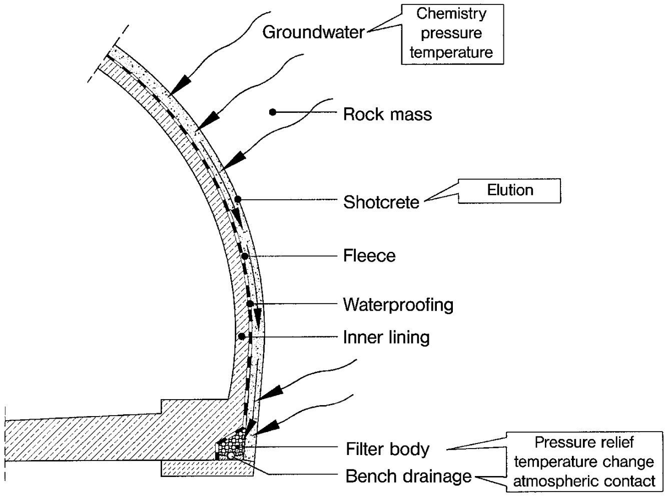


Figure 5-33 Route of groundwater into the tunnel drainage.

Drainage water containing calcium hydroxide. Two possible processes can lead to precipitation from soft drainage water with increased calcium hydroxide content.

The first possibility is that the drainage water comes into contact with air in the pipe and absorbs carbon dioxide. The calcium hydroxide and the carbon dioxide react to form calcium carbonate and water.

$$Ca(OH)_2 + CO_2 \quad \rightarrow \quad CaCO_3 + H_2O$$

When carbonated water mixes with water containing calcium hydroxide in the drainpipe, the calcium hydroxide reacts with the carbonic acid to form calcium carbonate and water.

$$Ca(OH)_2 + H_2CO_3 \quad \rightarrow \quad CaCO_3 + 2H_2O$$

Drainage water containing calcium hydrogen carbonate. Calcium hydrogen carbonate requires stabilising carbon dioxide in the environment in order to be stable. The solubility of CO_2 in water and thus the carbonic acid content is thus dependent on the physical conditions of pressure and temperature.

The dependence of the water solubility of carbon dioxide on temperature and pressure is illustrated in Fig. 5-34. The effect is that an over-proportionate amount of carbon dioxide remains dissolved in the water at low temperatures. The relationship of pressure to CO_2 content, on the other hand, is proportional at constant temperature.

With the entry of the water into the drainage system, its temperature normally increases and the pressure reduces. The consequence is that CO_2 gases off, and the calcium hydrogen carbonate breaks down into calcium carbonate, carbon dioxide and water.

$$Ca(HCO_3)_2 \quad \rightarrow \quad CaCO_3 + CO_2 + H_2O$$

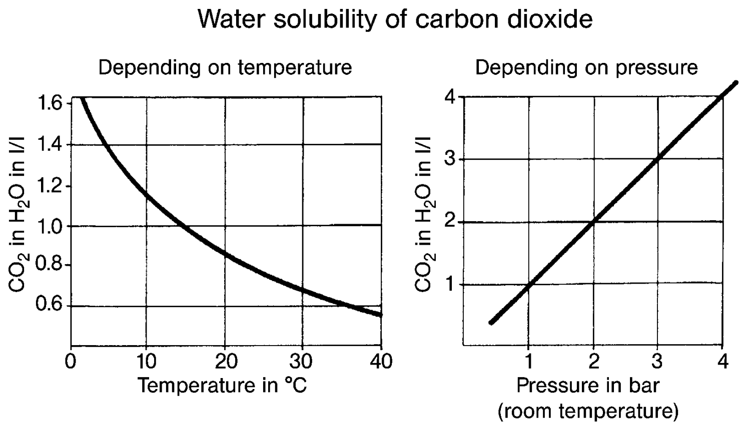


Figure 5-34 Water solubility of carbon dioxide [117].

An increased pH value due to mixing with water, which has been in contact with alkaline construction materials, also leads to lime precipitation. Possible sources of hydroxide ions in water are concrete in the form of shotcrete or in-situ concrete and accelerators with alkaline contents.

The calcium hydrogen carbonate reacts with the calcium hydroxide to form calcium carbonate and water.

$$Ca(HCO_3)_2 + Ca(OH)_2 \quad \rightarrow \quad 2CaCO_3 + 2H_2O$$

Accelerator admixtures containing alkali are normally based on easily soluble potassium and sodium aluminates. Sodium hydroxide, for example, reacts with calcium hydrogen carbonate to form calcium carbonate, sodium hydrogen carbonate and water.

$$Ca(HCO_3)_2 + Na(OH) \quad \rightarrow \quad CaCO_3 + NaHCO_3 + H_2O$$

Not one but many of these processes take place in a typical drainage system. In different locations in the drain run, the processes can combine and result in heavier precipitation and finally to sintering. The form of the sinter can range from milky (easily transported by the water flow) through sugary (with a fragile structure) to the most commonly encountered hard sintering.

5.4.2 Design of tunnel drainage for low sintering

Tests in the design phase. In order that the risk to a tunnel drainage system can be estimated in advance, it is necessary to make use of geological and hydrogeological investigations to estimate and interpret sintering problems.

According to DIN 4020 [54a], the structure and properties of soil and rock in the ground or in quarries for construction materials and also the ground water conditions should be adequately understood. Samples of soil and rock are evaluated as spot checks and permit probable conclusions about the areas between them..

In order to be able to estimate the danger of sintering, the geological investigation should include the approximate determination of the lime content with dilute hydrochloric acid according to DIN 4022 [55].

Chemical analysis of water samples was limited until recently to just the determination of the concrete aggressiveness according to DIN 4030 [56] and the steel aggressiveness. In order to determine the sintering potential, the calcite saturation should also be determined. The determination of the lime solution capacity as part of the tests according to DIN 4030 [56] is not sufficient and must be supplemented by the determination of the calcite saturation according to DIN 38 404-10 [60].

Information about the hardness and hydrogen carbonate hardness can supplement the evaluation of the degree of attack denoted by the lime dissolving capacity (CO_2). Soft water with a hardness of less than 30 mg CaO/l can dissolve calcium hydroxide out of hardened cement paste.

The information gained in this way can be used by an expert to interpret whether a drainage system can be designed according to the standardised procedure given by the relevant guidelines or whether additional measures need to be provided. The possible types and potential extent of such measures are described below.

Design of a basic system for low sintering. As has already been explained, samples of soil and rock obtained as part of a geological and hydrological investigation programme can only provide probability statements for the areas between the sampling locations. Varied experience makes clear that unrecognised water properties can have considerable consequences for the sintering behaviour of a drainage system.

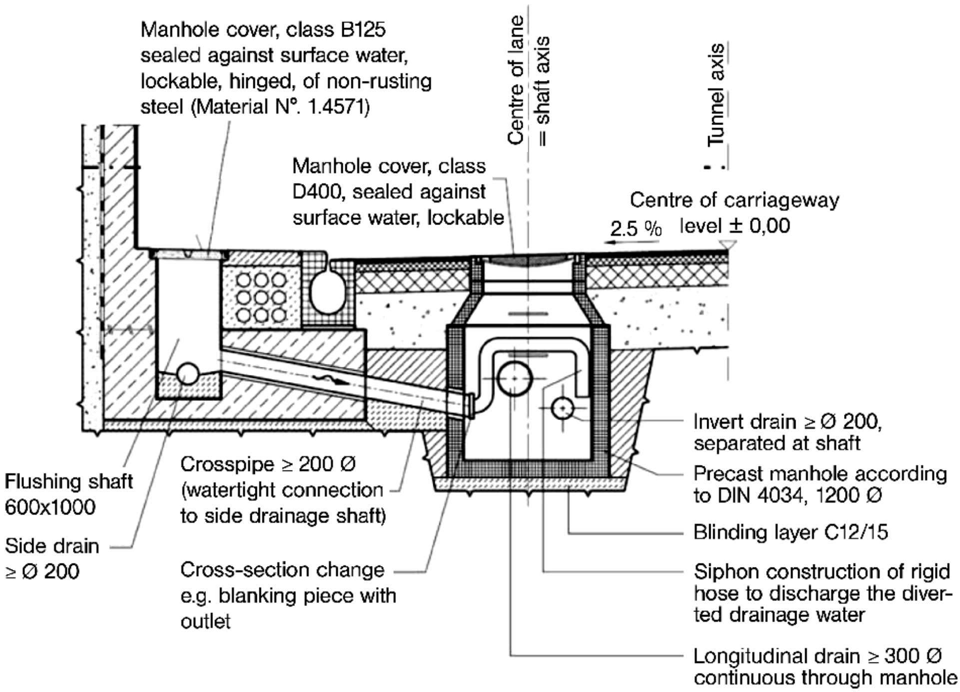


Figure 5-35 Basic system for tunnel drainage.

In order to be able to counter such a risk, a basic system conforming to minimum requirements for the water collection and drainage components should be the starting point of the design of any tunnel drainage system. Such a basic system consists of two side drainage pipe runs and a non-central and deeper invert drainage run (Fig. 5-35). The layout of the drainpipes of the basic system thus corresponds to the guideline detail drawings already provided in the BMVBS. An additional basis for the design of drainage systems for reduced sintering is also provided in Germany by the RI-BWD-TU. This describes various defined design elements and properties, which – backed up by experimental results – have proved to result in less sintering. Alternatively, the guideline "Tunnel Drainage" published by the Austrian Association of Concrete and Construction Technology can also be consulted.

The minimum requirements for a basic system according to all sources are:

1. **Shotcrete** with low susceptibility to leaching should be used for rock support. This means the general use of alkali-free accelerator or spraying cement. No aggregate of limestone or dolomite should be used unless its leaching behaviour has been tested and approved.
2. All **drainpipes**, which are laid to collect and drain groundwater, should have a smooth inner surface, a minimum diameter of 200 mm and any slots should have a width of 5 to 6 mm. Drain pipes should be highly durable and have good abrasion resistance against high-pressure flushing.
3. For **load-bearing and filter layers,** the use of 100 kg cement CEM III per m^3 filter material as binder is to be specified as a minimum requirement, and the material, installation process and the stability of the filter layer should be suitable for the size of the inlet openings of the drainage elements. Aggregates of limestone or dolomite should not be used [107]. For further reduction of sintering potential through elution of calcareous binder contents, filter bodies without binder can also be used in the presence of aggressive groundwater. This has to be balanced against the risk of damage to pipes followed by collapse of the filter medium and blockage of the drainpipe.
4. In order to ensure that water passages of **surface drainage** in the annular gap, a filament fleece with a weight of at least 1,000 g/m^2 should be specified and a transmissivity of 1×10^{-5} $m^3/m*s$.

The exact layout of the individual water-collecting and -draining components in cases of high sintering potential depends greatly whether the potential is due to the leaching of construction materials or the natural lime content in the groundwater. The guidelines mentioned do not make any statements about this.

Improvements to the basic system. In cases of acute risk of sintering, denoted by a high sintering potential of the groundwater or derived from other hydrogeological information, the developed basic system can be improved further. The German guidelines provide in this case for a bypass construction, which can be used to reduce the sintering exposure of the side drains in operation. If the sintering potential is forecast as high, bypass constructions according to the following illustration should be included in the design. The decision whether to design an improved system with a bypass construction is made by the consultant in discussion with the geotechnical consultant and the employer.

In Austria and Switzerland – as already explained – a different approach is followed for the operational state. This uses so-called hardness stabilisers to reduce the extent of sintering potential and the resulting maintenance costs. Hardness stabilisers are chemical agents, which can be added to the drainage water either solid in the form of depot stones which the groundwater flows through, or liquid in large dosage systems. The released or added agents are intended to condition the water to reduce the tendency to sintering and thus reduce maintenance costs. The following examples can be given of the use of depot stones:

– Placing of hardness stabilisation stones with a dissolving lifetime of two years, preferably in the percolation packing or at the pipe inlets, in order to moderate the typical initial sintering potential.
– Laying of solid hardness stabilisation agents in manholes and cleaning shafts.
– Installation of hardness stabilisation plants with liquid hardness stabilisation agents at critical locations in the drainage with constantly high water flow and uniform chemistry (pH value).
– This type of measures can also be provided at the design stage of the drainage system if aggressive groundwater is found to be present and integrated into the tender documents.

5.4.3 Construction of tunnel drainage to reduce sintering

In order to be able to ensure functional drainage in the long term, recording of data about the drainage system and the conditions that affect it must already start in the construction phase. This can include hydrogeological data recording, investigations of and in the drainage and camera surveying to ensure that a defect-free system is supplemented by quality assurance measures.

Hydrogeological data recording during the construction phase. In order to provide a basis for all further investigations and measures, the hydrogeological investigations from the design phase should be updated with complete documentation of water ingress occurring during the construction phase. The following parameters should be measured and documented as part of the geological engineering documentation:

– precise location of ingress of groundwater,
– quantities of groundwater entering the tunnel,
– concrete aggressiveness of the groundwater according to DIN 4030 [56],
– temperature, pH value and electrical conductivity of the groundwater.

The exact location and the incoming quantity should be documented for each ingress location; the concrete aggressiveness and all other values should be measured every 50 m. The calcium content and the total hardness only need to be determined in cases of increased sintering potential. The documentation should be updated again before the installation of the waterproofing and drainage system. When taking samples, care should be taken that no contamination from outside materials leads to false results. In order to determine the concrete aggressiveness according to DIN 4030 [56], pre-prepared reagent sets, called compact laboratories, can be used. Temperature, pH value and electrical conductivity can be determined immediately with measuring devices at the sampling location. When using the equipment for measuring the pH value and the electrical conductivity, it should be ensured that these can compensate the temperature. The calcium content and the total hardness can be determined, for example, with prepared titration testing sets.

The sintering behaviour of drainage systems should be documented during the construction period with water analyses, inspections of the manholes and camera surveys. This can start in completed sections. Experience shows that sintering can be already recognised at an early stage. The complete sintering behaviour can however only be analysed when the drainage system is in operation. The already recorded data are also of assistance to obtain information about the behaviour of the drainage system as soon as possible.

In the manholes, the following should be evaluated or measured:

- degree of sintering,
- water quantity and
- the calcite precipitation tendency of the drainage water according to DIN 38 404-10 [60].

5.4.3.1 Camera surveys of the pipe runs between the manholes

The condition of the drainage system should already be checked during construction as part of normal quality assurance measures. This means that individual sections of drain runs should be checked with a camera survey immediately after their construction in order to be able to detect any defects as soon as possible. All these surveys should be undertaken independently of the camera surveys undertaken as part of the acceptance of the drainage system.

The images fro the camera survey can then permit conclusions about the exact position and intensity of individual water ingress locations and sintering. The RI-BWD-TU provide their own evaluation model, which is based on the ATV (General technical specifications for construction contracts) system for the recording of damage to pipe runs.

5.4.3.2 Data processing and administration

The protocols from the manholes can be used to enter the parameters

- water level, sintering magnitude,
- pH value,
- water temperature and
- saturation index

into plans of the drainage pipes. It is then easy to recognise areas with heavy sintering and significant changes from the sintering behaviour, temperature curve, and the curves of the water temperature and saturation index.

In order to evaluate the camera surveys, it is recommended to digitise the video images every 5 m and still closer when anything interesting is discovered. Using the digital images and result sheets using a classification of the code system, a plan can be produced showing the parameters:

- sintering of slots,
- sintering of pipe invert,
- water ingress locations,
- special features and
- damage.

The plan can then be used to identify any possible improvements. On the other hand, the plans also deliver the first information about the condition of the drainage system in general and the tendency to sintering in particular. The causes of problems in the operational phase have often been shown to derive from loss of information between the construction phase and the operational phase. Therefore an information flow based on such processed data is advisable. This enables the information from the construction phase to be used to optimise the maintenance plan in the operational phase.

5.4.3.3 Other quality assurance measures during the construction phase

The contracting firm has – independent of the detailed design – the largest influence on the correct mounting and installation of the groundwater drainage. The contractor orders and checks the materials being installed and is directly involved in the process of constructing a fault-free drainage system with personnel and construction process. Defects caused in the construction phase and the quality of construction are often the main causes of damage to drainage systems, the resulting long-term maintenance costs and the fact that such defects can provoke sintering. Correct installation and a responsible contractor can even reduce or remedy defects, which are due to incorrect design.

The possibilities for action during the construction phase are in many areas. It is essential to use an appropriate process for the construction of the groundwater drainage, to display the necessary care in the selection of materials and their installation, and constantly check the quality of the completed system. It is also possible to undertake targeted measures on site, like for example directed water diversion or additional dewatering to react to special events or local conditions. The quality assurance aspects can be differentiated into

- optimisation of the construction process and
- supervision of construction.

The latter point applies to both internal supervision in the contracting firms by the responsible site management and external supervision by the responsible supervisors on site. According to the [Ri-BWD-TU], the following points are particularly important:

- Drain lasers should be used to check the direction and exact invert level of the drain pipes and should therefore be available in sufficient quantities.
- The drainage should generally be constructed in runs. The repeating sequence of work activities (manhole, run, manhole etc.) should be checked by the site supervision. The works should proceed continuously and the construction of the following permanent element (pavement at the sides and road construction in the invert) should if possible follow directly.
- Laying against the invert gradient of the tunnel is – even if a possible process – to be avoided. Otherwise, special measures have to be taken to react to any encrustation in the drain run.
- Drainage runs that are under construction or already completed must be protected with appropriate measures against external actions and damage. This can be ensured either by the completion of the following, permanent element (pavement, invert construction) or – in case construction progress stagnates due to other reasons – through appropriate protection measures.
- After the final completion of the drainage, its correct construction must be documented with an acceptance protocol and a camera survey.

5.4.4 Operation and maintenance of drainage systems to reduce sintering

Not only the actual flushing costs but also the development of the expenditure or the tendency to sintering have to be considered in the analysis of costs resulting from sintering. Ideally a comparison of water parameters over a period of years should be performed. Water analysis results from the construction period can offer a basis for comparison. In order to estimate the development of expenditure or the tendency to sintering, the personnel who carry out the flushing should also be questioned. The data gained can then be used to perform a cost-benefit estimate and make decisions whether constructional or operational improvement measures are economically justified. The specific quantity of data required for such an exercise is based on the parameters already listed under 5.4.3.1. The necessary rhythm of data collection, and the decision when to perform general inspections of the tunnel, depend on the extent of the sintering problem. [Ri-BWD-TU] and [RiLi Tunneldrainage] give detailed information about the exact determination of the specific inspection requirement.

5.4.4.1 Concepts to reduce maintenance through improvements to systems

Germany. The results of operational inspections are used to analyse the sintering behaviour of the drainage system and investigate the possible bypassing of the water and the resulting savings potential for maintenance work.

Bypassing keeps less exposed runs of the side drainage free of deposits caused by groundwater percolating from above. In addition, the flow of less concentrated water from the side drains into the invert drain should result in a reduction of the calcite saturation.

In order to implement the bypass concept, points must first be identified, at which bypassing of water with a high tendency to precipitate calcite from the sides into the invert is possible. The results of this investigation are then used to design suitable constructional measures to modify the drainage system, the intention being a considerable reduction of the maintenance work due to sintering at relatively little cost. The basic principle is the bypass construction already described in Section 5.4.2.

Austria and Switzerland.

Maintenance works can have a preventive or correctional nature according to the Austrian standard ÖNORM EN 13306. [ÖNORM EN 13306]:

The following preventative measures are recommended by [RiLi Tunneldrainage]:

– hardness stabilisation of the groundwater in the so-called secondary drainage (all non-accessible areas such as filter packing or surface drainage),
– low-pressure flushing (for example with the tunnel water supply as part of functional tests),
– periodic high-pressure flushing at low cost.

Corrective maintenance measures should be carried out after the formation of hardened deposits. The measures recommended by [RiLi Tunneldrainage] include:

– high-pressure flushing with large quantities of water and additional equipment,
– the targeted use of hardness stabilisers to avoid new formation of hard deposits and possibly to loosen existing accretion.

The general intention should be shorter maintenance intervals or preventative maintenance measures in order to prevent consolidation or too much growth of the deposits.

5.4.4.2 Cleaning of drainage systems

In the course of maintenance work, the pipes must be cleared of deposits without damage to the pipes. In the high-pressure flushing-suction process, pressures of 15 to 40 MPa at the pump or 10 to 20 MPa at the flushing head are used with total water quantities of 300 to 600 l/min in order to remove the loosened deposits. Very high-pressure cleaning with jet pressures of over 40 MPa should not be used for the maintenance of tunnel drainage, but rather for repair works.

When flushing drains, the local flow direction of the drain should be observed to rule out flushing against the flow direction. The flushing pressure should be suitable for the pipe material and the use of chains or hammer jets should generally be avoided unless cleaning is impossible without these methods. The maximum flushing distance should be restricted to two runs between manholes in order that the flushing pressure does not exceed a reasonable value due to the long distance.

6 New measurement and control technology in tunnelling

6.1 General

The development of tunnelling machinery has led to a particular degree of automatic control. This has many reasons, of which the most important are:

– Automation can accelerate working processes and improve machine utilisation.
– The precision of machine control is essential for subsequent costs, for example the lining of the tunnel. The precision requirements for water and sewer tunnels with little fall are particularly stringent.
– The working conditions for the operation of manually controlled tunnelling machinery often lead to worsened health risks and mistakes due to obstructed view and noise. Automation leads to an optimisation of control and often permits remote control, particularly in small-section tunnels.

On the other hand, there are limits in all fields to the degree of automation, and these have sometimes been reached. This is mostly due to the inadequate capability of automatic controls to react to extreme conditions. Almost all systems therefore allow manual override in order to overcome particularly difficult situations. For the surveying of an excavated section of tunnel, devices have been developed, which can cope with the conditions better than formerly, particularly the urgency and inaccessibility of the areas to be surveyed.

6.2 Measurement instruments

6.2.1 Gyroscopic devices

A gyroscope is a spinning rotor that maintains its axis of rotation and revolution speed as long as no external torque is applied. Technical gyroscopes are flattened and symmetrical bodies, which rotate about the principal axis with the largest moment of inertia; the moments of inertia for the other two principal axes are equal. The rotation speed of technical gyroscopes is between 3,000 and 48,000 rpm.

Free gyroscope. A gyroscope that can freely rotate in two further axes in addition to its spinning axis is called a free gyroscope. Such a gyroscope has three degrees of freedom, which is made possible by mounting it in gimbals (Fig. 6-1 left). Since no external torque acts on it, a free gyroscope maintains the spatial location of its axis of rotation in space. It can therefore be used to display the direction of movement (as a directional gyro or heading indicator) or vertical (as a vertical gyro). The rotation of the Earth also leads to movement of such a gyroscopic device and to a relative deflection, which has to be considered or corrected. Free gyroscopes are not capable of determining their absolute direction.

Important applications of gyroscopic devices are characterised by the limitation of their movement. If the mounting only permits rotation about one more axis in addition to the spinning rotation, then forces act on the gyroscope when it is moved. The gyroscope then attempts to orient its axis of rotation in the direction of the axis of the external rotation.

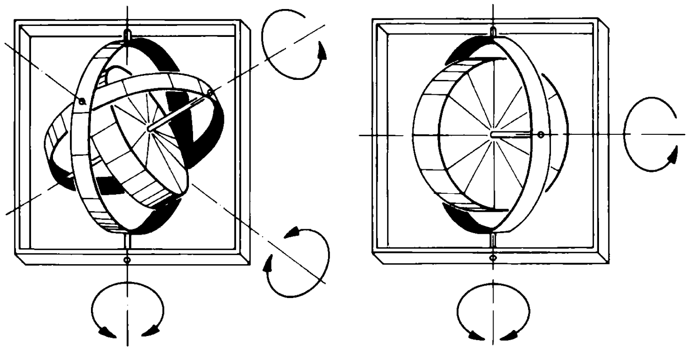


Figure 6-1 A free gyroscope mounted in gimbals (left) and the principle of the mounting of a gyrocompass (right).

Rate gyro. When a gyroscope is mounted to permit rotation about the horizontal axis, the force applied by the gyroscope is proportional to the rate of turn of the carrier and can be measured. Such an instrument is called a rate gyro or turn indicator; it does not display the magnitude of a turn but the rate of turn, for example of a vehicle.

Gyrocompass. A gyroscope, which also rotates about a horizontal axis, but with a second degree of freedom about the vertical axis, is subjected to an external torque by the rotation of the Earth, which forces the axis of the gyroscope to point to true north. This type of device is called a gyrocompass or meridian pointer (Fig. 6-2 right). A gyrocompass attempts to orient its axis of rotation as far as possible parallel to the axis of the external rotation, which is the Earth's axis, so its direction of rotation agrees with that of the Earth (Fig. 6-2).

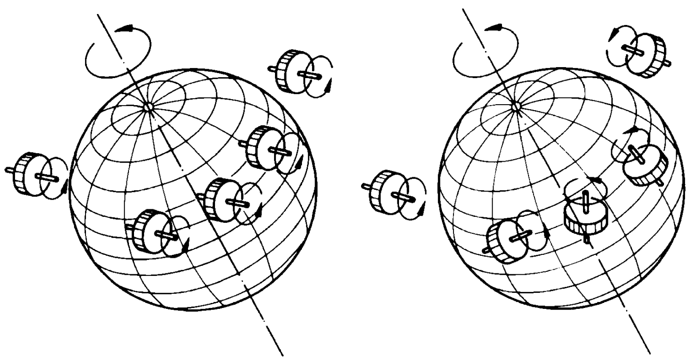


Figure 6-2 A free gyroscope (left) maintains its location in space; the rotation of the Earth changes its location relative to the surface of the earth. The north-seeking gyrocompass (right) orients itself so that its axis of rotation is a parallel as possible to that of the Earth, and both rotate in the same direction. After settling, it maintains its location relative to the surface of the earth.

There are two types of meridian pointer:

– The tape-suspended meridian gyroscope is normally installed in gyrotheodolites. Instead of being mounted in a gimbal, the spinning rotor is suspended on a very thin tape (Fig. 6-3 left).
– The north-seeking gyroscope is mounted in one gimbal in air bearings, which results in an elastic fixing to the vertical as with a tape-suspended meridian gyroscope (Fig. 6-3 right).

The spinning rotor oscillates as it seeks north, and this has to be damped. The energy of oscillation can be removed by delayed operation of the upper tape clamp; this can also be done manually. The north-seeking gyroscope is normally fitted with electric damping. The method of damping is also determined by the method of reading the north direction. The following processes can be used:

– Determination of the location of the settled gyroscope.
– Calculation of the centre point of the oscillations.
– The gyroscope is held in position electrically and the compensating north-seeking moment is measured. This is proportional to the deviation from north.

All meridian gyroscopes can only determine the north direction if set up in a fixed position and kept free of vibration. In addition to the drift, which is partly due to imperfect agreement of the rotational axis of the rotor with the axis of inertia of the suspension frame, the influence of the mass of the gimbals and bearing friction, there are a range of mechanisms by which vibration could lead to reading deviations. Even slight vibration from machinery, vehicles or general seismic activity can disturb readings. The cause is that vibration can produce erroneous torques, which can exceed and disturb the north-seeking moment.

The tape-suspended meridian gyroscope reacts particularly sensitively to horizontal translations, but rotations about the vertical are not a problem. The north-seeking gyroscope, on the other hand, does not react much to translations but is very sensitive to rotations.

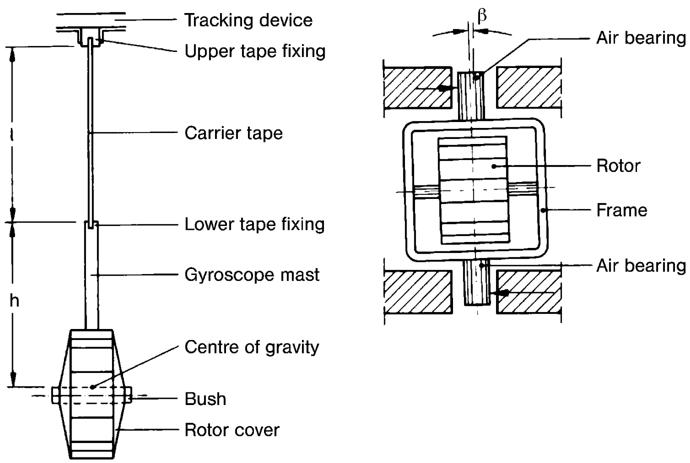


Figure 6-3 Tape-suspended meridian gyroscope (left) and north-seeking meridian gyroscope with air bearings [72] (right).

In general, vibration causes more disturbance, the lower its frequency, and this should always be considered for installation. Disturbance torques normally oscillate at higher frequencies than the north-seeking moments. In order to avoid disturbance from vibration, north-seeking gyroscopes are fitted with elastic supports, as a short-term outside jolt then only causes a short-term disturbance moment. On a fixed support, on the other hand, disturbance can take the form of rotational speeds, which would lead to much more serious errors. The reduction of errors is a filter problem, which can be approached mechanically through the provision of a suitable support or electrically through eddy current damping [72].

Ship's gyrocompass. The gyrocompass used in ships is also described as a gyrocompass. This is indeed a form of meridian gyroscope, but far more elaborate. Since a ship's compass has to cope with vibrations and oscillations with very variable periods, the time taken to settle is designed to be very long, actually 84.4 min, since this minimises the errors due to the Earth's rotation. Numerous other features (supported by floating in a fluid, the use of two acutely angled rotors, suitable matching of all influential parameters), the disturbances are sufficiently minimised.

In a ship's gyrocompass are two rotors connected to each other by springs at an angle of about 90° (Fig. 6-4). This arrangement suppresses the errors that occur during the rotation of the system about the vertical axis (streaming error). The two rotors are installed in the rotor ball, as shown in Fig. 6-4. The rotor ball floats in an electrically conductive fluid (mixture of water, glycerine and benzoic acid) in the housing ball, with the floating uplift compensating the effective weight to a few grams. The remainder of the necessary uplift is provided by the so-called blowout coil: Alternating current in the bottom of the housing ball induces phase-delayed current on the surface of the rotor ball, which achieves a repelling effect. The rotor ball thus floats without contact in the housing ball. The power supply to the rotors is though the support fluid. In order to avoid friction when the rotor ball turns, the housing ball is turned with it electromagnetically when it turns, even through a very small angle. The location of the housing ball is thus identical with that of the rotor ball and serves to show the direction of north.

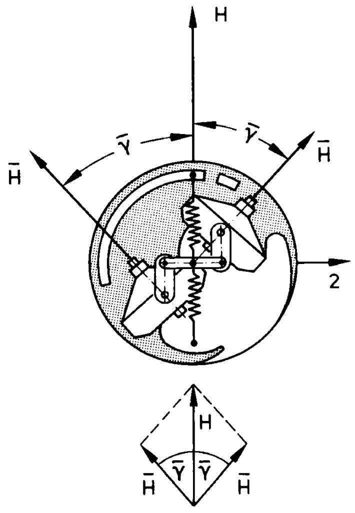


Figure 6-4 Arrangement of two rotors in a ship's gyrocompass. The hinged lever ensures that the rotor caps can only rotate in opposite directions about the vertical axis relative to the rotor ball. The direction of the resulting angular momentum, which is the sum of the angular momentums of the two rotors, is thus fixed in the rotor ball [72].

The long settling time of a ship's gyrocompass would be a great disadvantage for the control of machinery. It is however the only north-pointing gyroscope system, which also works while in motion and its application in tunnelling is practical where continuous slow movements have to be controlled (pipe jacking, shield tunnelling).

6.2.2 Lasers

Since the construction of the first laser in 1960, many laser devices and applications have been developed at a rapid rate. The types that are useful in construction will be described here. Laser light differs from normal light in the following properties:

1. Temporal and spatial coherence, which means that there is a fixed phase relationship between the individual rays of light; they oscillate their emission in unison and maintain this property as they propagate through space.
2. Monochrome, which means the light is exclusively of one wavelength, or displays a very narrow spectrum.

Laser light can also have the following additional properties:

– Bundling: laser light is tightly bundled (but not in every case).
– Many laser sources emit polarised light.

Construction and function of a laser source. Every laser device consists of a gain medium, which can be a gas (for example argon) or gas mixture (for example helium-neon), a solid body (for example ruby) or a semi-conductor (for example gallium-arsenide). This is excited by an energy source (pumped) so that the energy state of the atomic shells is not in thermal equilibrium. The most important types of excitation are optical (flash lamps) and electrical (voltage supply). The decay of the excited atoms to their normal level, in other words that corresponding to the temperature, results in the emission of radiation, which is used directly as laser light or excites another part of the gain medium, which then emits laser light while "jumping back".

The emission of light is partially spontaneous, but mostly stimulated; only the latter part leads to the creation of laser light. The stimulation is performed by the laser light itself: this is reflected back and forth between two reflectors and equal-phase emissions are produced with each passage through the medium. These contribute to further equal-phase emissions as they pass through the medium. This process is called optical resonance and is the precondition for the creation of laser light.

The laser light can be radiated continuously or in short pulses. Pulse operation is normally only used in laser systems, in which the pulses are exactly controlled by external action, in the simplest form by the excitation (for example in a semiconductor laser). In surveying laser technology for construction, the helium-neon laser has been prevalent for a long time but semiconductor lasers have become more common recently.

Helium-neon laser. The construction of a helium-neon laser is shown in Fig. 6-5. The helium-neon gas mixture is enclosed in a glass tube; the excitation occurs at the anode and the cathode through the application of a voltage of many kV. The angled ends permit loss-free passing of the radiation through the windows for linear vertically polarised light (Brewster windows). The convex outer mirrors reflect the light so that resonance can occur. The laser light passes through one of the two concave mirrors, which is for example 1% partially transparent. The internal beam is then about 100 times stronger than the emitted beam. The radiation of a helium-neon laser has extreme coherence and frequency stability; the most-used wavelength is 632.8 nm (red light; other oscillation modes with different wavelengths are also possible). The light is linearly polarised.

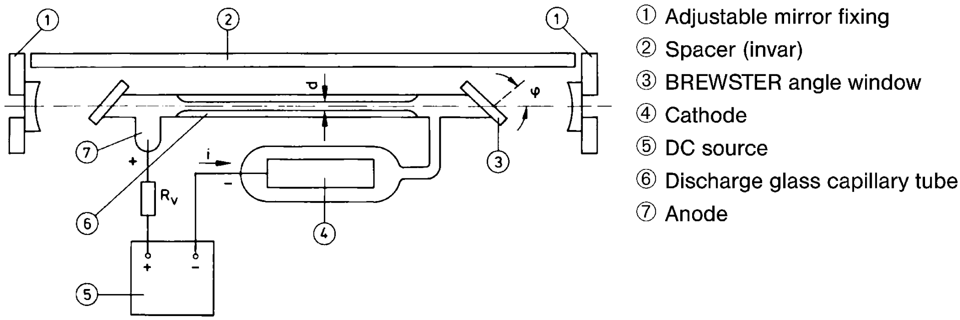


Figure 6-5 Principle of a helium-neon laser, from [29].

Helium-neon lasers deliver continuous light with powers of 0.5 to 50 mW, and the efficiency of the systems is 0.1 %. The wide prevalence of this type is due to the long lifetime of the tubes with low manufacturing costs, and also that no special cooling is necessary.

Semiconductor lasers. The semiconductor laser (also called diode laser or injection laser) has a much more simple construction although its manufacture demands highly developed technology. In an isolating base material are various elements, some of which emit electrons and some of which can absorb electrons. With the application of an external voltage, the electrons are recombined, which reverses spontaneously leading to the emission of radiation. The construction of a semiconductor laser is shown in Fig. 6-6.

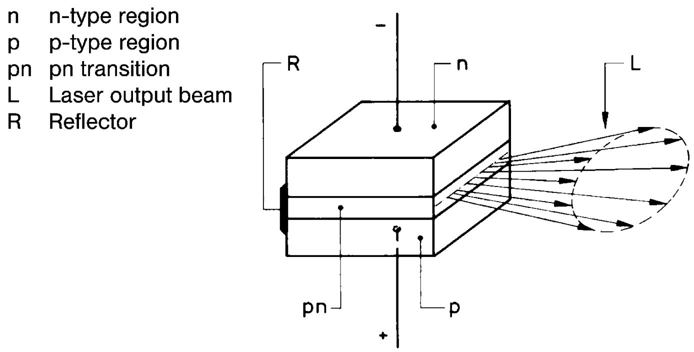


Figure 6-6 Construction of a semiconductor laser.

Optical resonance occurs through partial reflection at the outer edges. Semiconductor lasers emit solely in the invisible infrared range. The other properties are:

– Simple excitation of extremely short pulses (less than 200 ns) while simultaneously permitting continuous operation,
– Extremely small dimensions of the element,
– Low coherence,
– No bundling of the light.

Powers of up to 10 kW are normal in continuous operation, and with short pulses with relatively long pauses, peak pulse powers of up to 100 kW are possible, although the average power has to correspond to that of continuous operation (danger of overheating). The supply voltage is only a few volts (in contrast to the helium-neon laser with many kV).

The possibilities of pulse modulation have led to applications of semiconductor lasers in optical data transmission and distance measurement.

Laser light is subject to the same laws as natural light of the same wavelength. In stable air layering, beams can be deflected, which can be much more significant in tunnels than in the open air due to temperature differences. Deflection of a laser beam by 5 cm at a distance of 100 m has been demonstrated for pipe jacking [47]. Air turbulence causes the beam to spread, which can also hinder surveying. Nonetheless, when appropriate methods are used, the bundle of rays can be concentrated at the intended contact point. Dust and water droplets lead to scattering, which also results in beam spread and energy loss. The wandering of a beam, described as drift, is essentially due to temperature dependency of the oscillation of the optical resonator. The magnitude of this is insignificant for surveying technology in construction.

6.2.3 Optical components for laser devices

Beam-widening optics reduce the divergence of the laser beam exactly to the degree to which they enlarge the beam diameter near the device. The construction is similar to that of a refracting telescope, with the beam entering at the eyepiece side. The widening of the beam from 1 to 10 mm can theoretically reduce the spot size at a distance, for example from 20 to 2 mm. Fig. 6-7 (top) shows this conversion qualitatively.

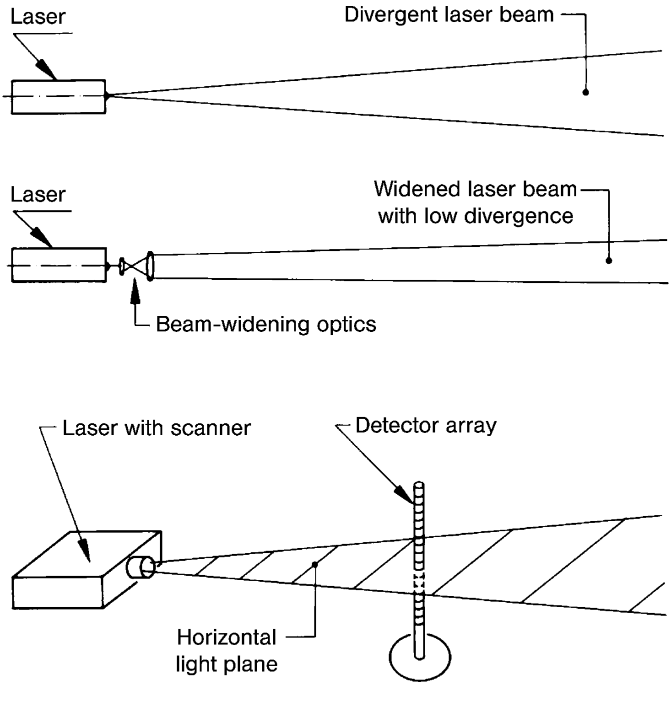


Figure 6-7 Effect of beam widening optics (top). Horizontal scanning (bottom) creates a reference level for levelling with a detector array.

There are also widening optics, which can be focussed. These permit the bundling of the rays to be set for the relevant distance, so that the spot size is reduced to the same magnitude as the source (1 mm) or the residual beam widening is due to refraction and spread. The helium-neon devices that are common in construction have non-focussing beam widening to enable simple operation.

Scanning technology with periodic beam deflection. Scanning is used to define a plane in space and exploit it for surveying purposes. The beam covers a plane with high frequency. Fig. 6-7 (bottom) shows an application of scanning. The deflection of an oscillating reflector is mostly used but the sideways movement of lenses can also be used. Crossed scanning in a horizontal and a vertical plane also creates a point in addition to the planes at the intersection line of the planes. Suitable detectors can then determine not only the position of a machine as the distance from the beam but also all three coordinates.

Similarly to scanning, cylinder lenses can also be used to spread a beam in a plane. In this case the energy is however greatly reduced so this is seldom used with optical-electrical receivers.

6.2.4 Optical receiver devices

Photodiodes are light receivers, which are operated with applied voltage; the entry of light increases the current flow. Photo elements are components, which are operated without voltage and produce a photocurrent when light enters. Photodiodes and elements are fitted with filters, which only admit the wavelength of the desired signal (typically the red light of a helium-neon laser).

A number of photodiodes in a row form a photodiode array. This can be used, for example, with a light plane emitted by a rotating laser to determine position. Photodiodes arranged over a surface form an active target, which can detect the light of an impinging laser beam. If the data is appropriately recorded, the contact point can be made visible on the screen or also calculated and given as coordinates or used as a control impulse.

CCD cameras. While in the processes mentioned so far, the receiver is moved equivalent surveying is also possible with stationary receivers. Particularly CCD (Charge Coupled Devices) cameras are used for this purpose. The light from a (moving) source is imaged with a lens on the approx. 1 cm^2 sensor, which is formed of a matrix of pixels (picture-elements) in arrays. The arrays are arranged horizontally and consist of isolated semiconductor elements and electrodes. Through changing voltage to the electrodes adjacent to the semiconductors, the electrons liberated by incoming light are transferred vertically into arrayed storage sections, which are not exposed to light.

An appropriate voltage change then transfers the charges horizontally in rows to an output section, where they are amplified and outputted as standardised voltage-modulated video output. The sequence of this image sampling process corresponds to the scanning of tube cameras and television screens. The refresh rates range from 50 images per second (television-compatible) to 300 images per second for special applications. The resolution of CCD cameras can exceed that of tube cameras, but a CCD camera cannot compete regarding grey-scale and light sensitivity. While the functioning of tube cameras can result in distorted images, an image from a CCD camera is geometrically stable due to the fixed arrangement of the sensors on the silicon base material, which makes the principle useful for surveying applications.

The data can be fed to a computer, which can calculate the light areas from the coordinates and brightness values of the illuminated pixels. In addition to the CCD camera, there are other types of semiconductor cameras, which in principle could give comparable results in surveying.

Corresponding to the photodiode array, there are also linear cameras based on CCD technology. These have up to 4,000 photo elements with lengths of 1 to 2.5 cm. The use of line sensors is appropriate, for example, when only one coordinate of a light source has to be recorded, which is then imaged on the sensor by a cylindrical lens as a frame line. If a line camera is periodically rotated about a certain angle (scanned), this can produce a two-dimensional image similarly to a matrix camera, except with lower refresh rate and up to ten times higher resolution. The data can be read in lines and saved in a computer. In order to considerably reduce the enormous quantity of data, only the coordinates of the illuminated pixel will be saved in practice. In combination with a laser beam, the line camera is also used for distance measurement using the trigonometric principle.

6.2.5 Hose levelling instruments

Hose levelling instruments work like a water level work on the principle of communicating liquids and can be used to measure relative level differences. There are two basic measurement principles:

1. Sensing of the liquid level in a measuring cylinder with a measuring tip, which is held electrically exactly at the surface of the liquid. The position of the measuring tip is the signal, which is electronically queried. Levelling instruments working on this principle are very precise (to fractions of millimetres) although they cannot be used to continuously record moving processes due to the inertia of the liquid (Fig. 6-8).
2. The liquid remains enclosed in the hose and there is no flow, but the pressure change at one end of the hose is measured. This process is less exact (to a few millimetres) but if there is suitable damping of the liquid against oscillation and the signal is filtered electronically, it is also suitable for moving processes (Fig. 6-9).

A number of such instruments can be combined to record the location of shield machines.

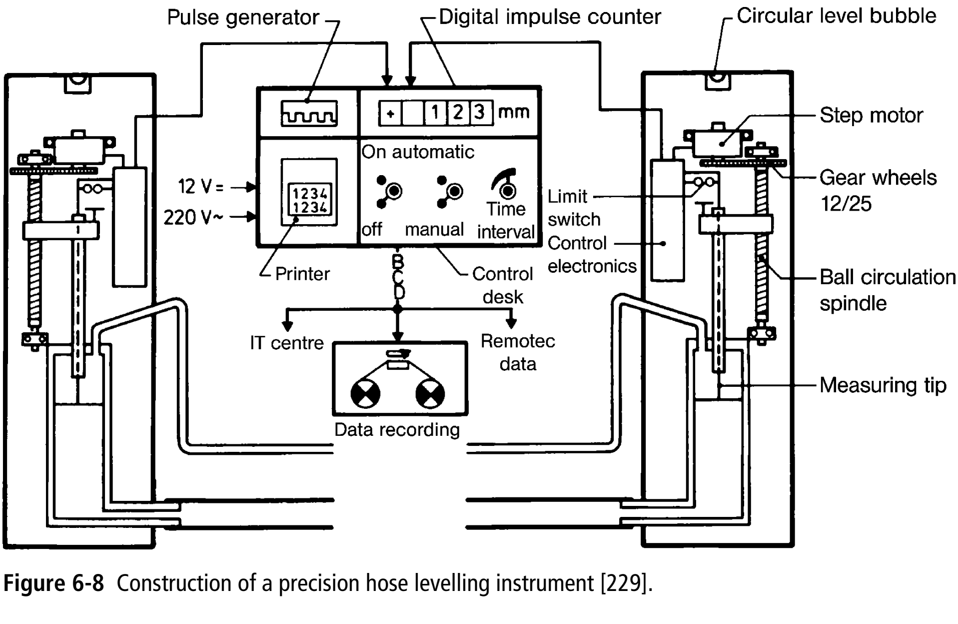


Figure 6-8 Construction of a precision hose levelling instrument [229].

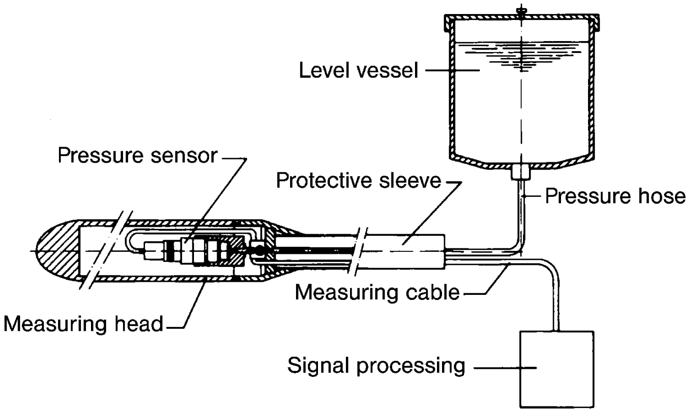


Figure 6-9 Hose levelling instrument based on pressure measurement, from H.-J. Collins [39].

6.2.6 Inclinometer

Inclinometers can show, for example, the inclination of a tunnelling machine or ground deformation. They are normally constructed with a pendulum rotating about an axis and held in position by an electrical control circuit. The current necessary to compensate the torque resulting from an inclination is proportional to the angle of inclination and delivers the measured signal. Two inclinometers arranged at 90° to each other can deliver the complete deviation of a machine against the vertical.

6.3 Control in drill and blast tunnelling

The drilling rig, drilling booms and drilling process in drill and blast tunnelling were all originally manually controlled. Attempts were then made to display the arrangement of holes and the blasting pattern on the face with an aligned projector, but this process did not become established. Since the overbreak, extra concrete and the geometry of the support layer essentially depend on the precision of the holes set out by the drillers, the trend for some years has increasingly been the development of drilling jumbos with computer control and computer assistance. The most significant innovation is the determination of the position of the drilling boom and the automation of the drilling process. The developments extend from the automation of individual working steps through full automation with repositioning of the drilling boom to fully automatic drilling during the entire round cycle. In order to achieve this, the machine only has to be aligned once with the tunnel laser and thereafter the drilling of the entire face only has to be watched by the operator. Computer-assistance and -control of the drilling jumbo are thus a significant aid for quality assurance in tunnel driving.

6.3.1 Drilling jumbo navigation

In order to determine the starting position in drill and blast tunnelling, firstly the location is determined. The drill jumbo is levelled on jacks. Then two sighting targets are mounted on one of the drill feeds. This feed is then aligned so the beam from the directional laser passes through both sighting targets (Fig. 6-10). The exact position is determined from the line of the laser beam and the tunnel chainage of the jumbo and the drilling of the next round can begin.

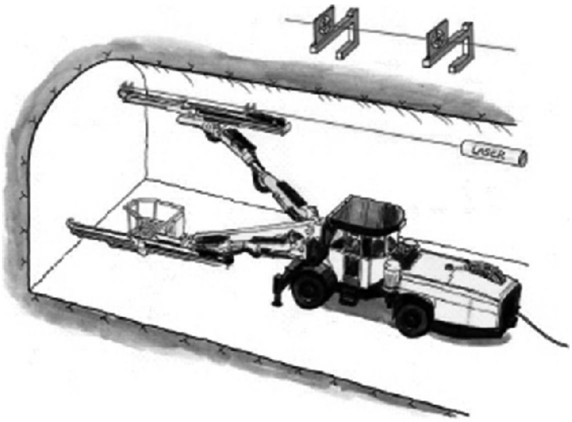


Figure 6-10 Setting the position of a drill jumbo with a tunnel laser; Atlas Copco Deutschland GmbH.

6.3.2 Determining the position of a drilling boom

One simple example for the surveying of the position of drilling feeds is the Tunnel-Angie system from Ilmeg System AB, Nyköping. Inclinometers on the drill feed determine their inclination against the vertical and this is displayed in the control position. This gives the inclination of the drill holes, which is particularly important in the crown and the invert (Fig. 6-11). The horizontal angle of the drill feeds is determined by another separately installed system, for which angle sensors are mounted on the hinges of the drilling boom. The sum of the two angles of the drilling boom hinges is displayed in the control position and this gives the total angle between drill feed and base vehicle.

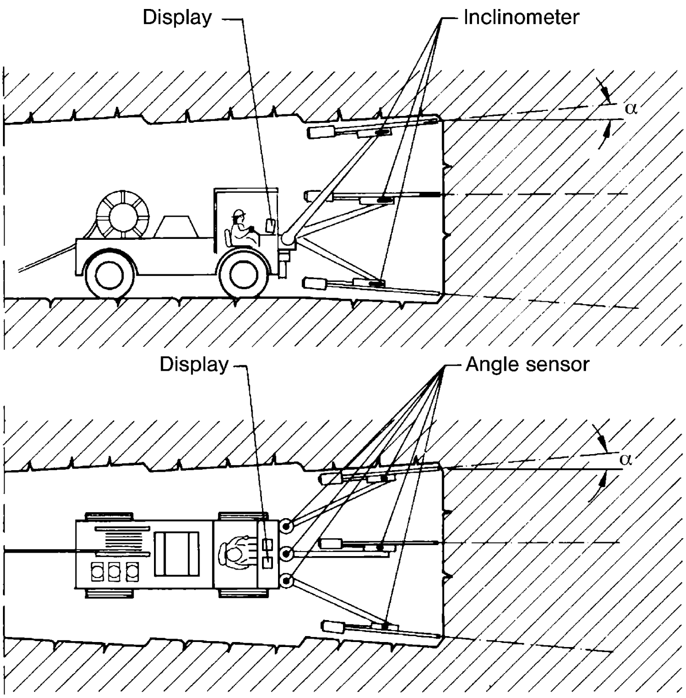


Figure 6-11 Recording of the vertical (top) and horizontal (bottom) positions of the drill feed using inclinometer and angle sensor. Tunnel-Angie System from Ilmeg System AB [95].

6.3.3 Hydraulic parallel holding of the feeds

In order to simplify the operation of a drill jumbo, a hydraulic system has been developed that enables the sideways transfer and changing of the angle to be performed with separate commands independently of each other, which means that the drill guide is held parallel to the set alignment when moved to a new hole. This is enabled by a special arrangement of the hydraulic cylinders, as is shown in Fig. 6-12. The machine can also rotate the feeds by 360° to minimise the drill shadow and thus the over-profile when drilling contour holes [11].

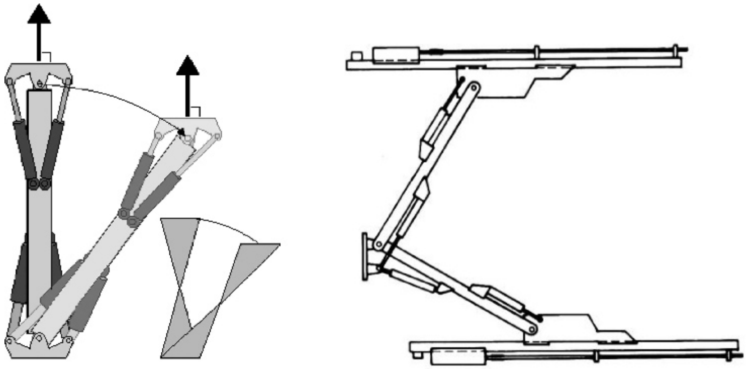
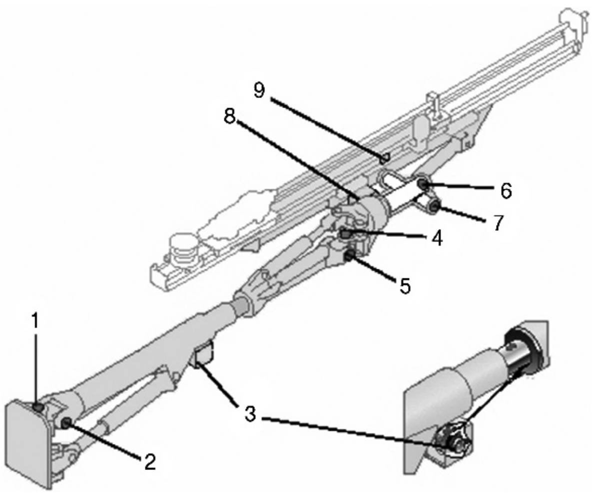


Figure 6-12 Fixed parallel alignment of the drill feeds, BUT 30 System; Atlas Copco Deutschland GmbH.

6.3.4 Control of drill booms by microprocessors

In the interest of faster and more precise tunnel advance, fully automated drill booms have also been developed. Control of the drill pressure and torque, which is usual nowadays, is a precondition for this. In addition, all data is recorded, which is necessary for the control of the drilling process. This includes the angle of the hinges, the telescopic extension of the drilling boom and the position of the drill in the drill feed (Fig. 6-13).



1. Slewing sensor, drill boom
2. Angle sensor, drill boom
3. Length sensor, drill boom
4. Slewing sensor, drill feed
5. Angle sensor, drill feed
6. Rotation sensor, drill feed
7. Inclination sensor, drill feed
8. Extension sensor, drill feed
9. Recording of hole depth

Figure 6-13 Sensor functions of the drill boom control system; Atlas Copco Deutschland GmbH.

6.3.5 Hydraulic drill booms

Bupec 36 and Bupec 50 from Montabert GmbH are suitable for fully automated drilling. The drill feed position in this case is determined with angle sensors at the drill boom. These transmit the digitised angles to the central computer, which calculates the position of the drill boom. In order to permit free rotation of the feed by 360° (rollover), the electrical signals are connected through 20-circuit electrical hinged connections, which avoids the need for additional cables at the drill boom hinges (since the drill boom can be rotated, inclinometers would not be suitable). In order to fully determine the position, the position of the base machine would have to be determined from a stationary device. This is dealt with simply by manually entering the tunnel axis: the feed is aligned manually parallel to a laser beam, and a command to the computer saves the drill feed position as tunnel axis and automated drilling can begin. This process is called learning. The principle is independent of the location of the drill jumbo. There are three operating modes with this system:

– Fully automated cycle: height and location of all holes are selected according to programming or a stored drilling pattern and drilled. Prismatic and fan-shaped cut patterns are also possible. Altogether 90 different drilling patterns can be stored and called up.
– Semi-automatic: the height and location of the holes are manually adjusted, in which case the drill feed can still be kept parallel by the control system (electronic parallel holding).
– Manual: All four types of movement are controlled independently of each other.

The advantage of microprocessor control is not the increased advance rate but particularly the precision in holding the configured drilling pattern.

6.4 Control of roadheaders

The control of roadheaders demands elaborate measurement and control technology due to the number of degrees of freedom and the method of working. The working conditions are very difficult and the face is often hidden by dust development. The danger to the driver sometimes demands remote control of the machine, which is not practical without at least some automation. On the other hand, the precision of excavation is subject to stringent requirements, which are now practical to comply with. Geometrical deviations from the design profile or the design axis increase support costs and also have an effect on the load-bearing behaviour of the support.

6.4.1 Movement parameters determined by the control system

A roadheader consists of a tracked vehicle with loading conveyor and control station and a cutting boom with the cutting head. In order to completely capture the movements of the cutting head, which is not possible optically due to the dust development and other conditions, the movements of the cutting boom and of the machine have to be captured.

Movement parameters. The machine essentially has six degrees of freedom; three describing its spatial movements, in other words its position, and three individual degrees of freedom in its alignment. The parameters to be determined are (Fig. 6-14):

– Height divergence above axis.
– Side divergence from axis.
– Travelled distance.
– Pitch angle.
– Yaw angle.
– Roll angle.

These parameters have to be determined in relation to a locally defined coordinate system. The movements of the cutting boom each have one degree of freedom: lifting – lowering, slewing and the length (extension) of the cutting boom. These are first measured as machine-related coordinates and converted into the locally defined tunnel coordinate system using the data describing the machine position (Fig. 6-14). The movement parameters of machines with in-line and cross-cutting head mounting do not differ. In practice, the determination of the excavated distance is not integrated into the machine control system but undertaken by the personnel. When a curve is driven, this is undertaken at a close spacing with the laser instruments installed in the tunnel being moved forward.

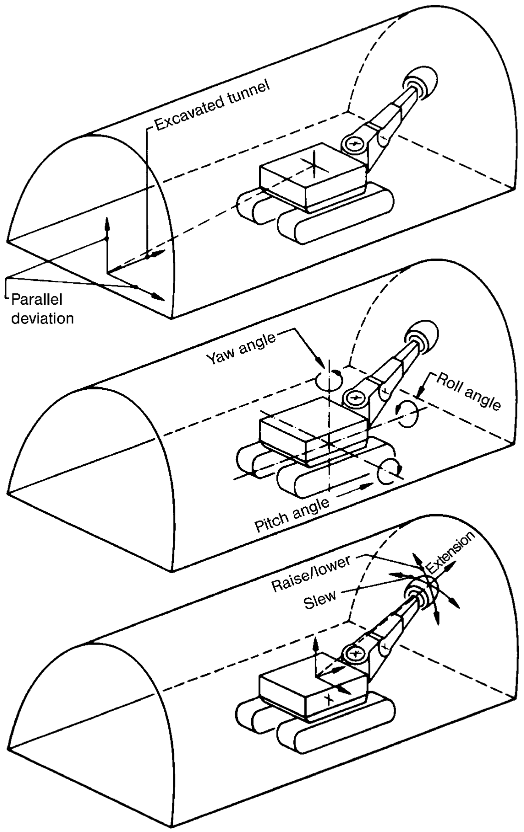


Figure 6-14 Degrees of freedom of movement of a roadheader with in-line cutting head.

For the control of a rail-mounted roadheader, the movement parameters to be measured are reduced to the three degrees of movement of the machine.

The simplest way of determining the position of the machine is to observe a target fixed to the machine from a theodolite set up at an appropriate location. Since the machine not only moves forward as it works but also changes its parallel divergence, this demands the continuous presence of a surveyor. Automatic control of the excavator boom control would scarcely be practical since this would require updated data about the position of the machine to flow to the control computer. The automatic control system described below is based on the following principles:

1. The machine position is determined using a system, of which some parts are installed separately from the machine in the tunnel.
2. The cutting boom position is determined with a machine-mounted system of sensors.

6.4.2 Roadheader control system from Voest Alpine

VOEST-ALPINE Bergtechnik Ges.m.b.H. (VAB) offers the integrated control system AMOR as an option for its machines. The objective is a considerable improvement of the productivity and safety of the machines. Machines from VAB have been fitted with integrated control systems since 1990 (Fig. 6-15) and more than 250 roadheaders equipped with these control systems were in use worldwide in 2003 [248].

The automation includes:

– Control of direction.
– Display.
– Automated cutting.
– Remote control.
– VOEST ALPINE Monitoring and Reporting System.
– Safety and overload protection system.
– Tele-operation and underground communication.
– Electronic operating manuals and maintenance aids.
– Remote maintenance.

The determination of direction, as with the company Eickhoff (see 6.4.3), is mostly based on the PPS system and is not described in more detail here. Similar systems are also offered and applied by the companies VMT and Geodata.

Control of direction. VAB has tried out and put into practice control of direction using either a theodolite or a north-seeking gyroscope. The control of direction enables cutting of the correct profile to an accuracy of centimetres related to the design tunnel axis. This reduces costs by avoiding later reprofiling and unnecessary overbreak and also saves shotcrete quantity.

Figure 6-15 ALPINE MINER AM 105. Driver's cabin with display of directional control and the operating mode of the machine in December 2002.

Display. The display visualises the operational conditions of the machine for the driver. If the visibility is poor, the cutting process can also be controlled from the display on the screen (Fig. 6-16).

Various types and sizes of screen can be used according to requirements, including touch screens

The display of the operational conditions and the associated aids like operation manual and instructions for fault finding increase the efficiency of the machine and the associated maintenance (Fig. 6-17).

Figure 6-16 Visualisation of the cutting process of an ALPINE BOLTER MINER ABM 14 in the USA. The machine driver operates the machine by radio remote control and can observe the progress of cutting and the operational conditions on the display screen. The display, which the operator observes, is shown enlarged at top left.

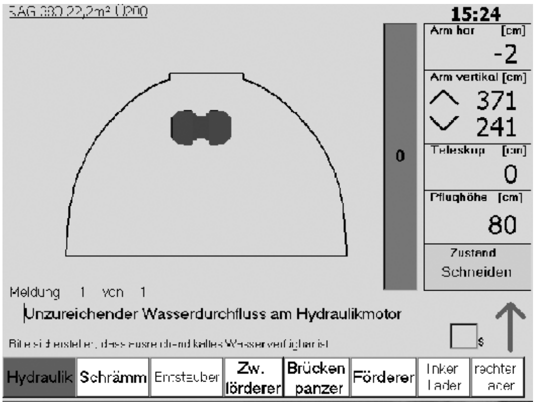


Figure 6-17 Display of the cutting pattern and the machine condition in the driver's cabin or at the remote control operating position. Shown here are the position of the cutting head and an error report. The machine driver can call up various menu pages with further, more detailed information. The function "electronic hand-book" can also be installed if required.

Cutting automation. The use of an appropriate system of sensors enables automated cutting. Depending on the particular application, the machine driver can decide between the functions profile display, profile control, partially automated excavation and fully automated excavation.

These options are:

– *Profile display:* The design profile is displayed on the screen and the machine driver stops at the edge of the profile, with the approaching of the profile edge being announced by a visual and/or acoustic warning signal. The system does not take over control.
– *Profile control:* The control system stops the horizontal/vertical movement of the cutting head when the edge of the profile is reached.
– *Semi-automatic:* The control system stops the horizontal/vertical movement of the cutting head when the edge of the profile is reached, and the next cut is automatically prepared, with the load on the cutting drive being included in the calculation.
– *Fully automatic:* The control system cuts a complete round inside the edges of the profile, if required with a defined cutting pattern. The control system takes into account parameters like position, cutting load, forces and acceleration in the control of the cutting sequence. At the moment (2003), an experienced and practiced machine driver can still cope better with heavily inhomogeneous ground conditions than fully automatic operation.

The automation of the cutting process is of considerable assistance to the machine driver and the movements of the cutting head are not dependent on the capability or tiredness of the operator.

Remote control. Radio or cable remote control (Fig. 6-18) makes it possible for the machine operator to control the position of the machine from a position where he has the best overview. This is of particular advantage when the machine has to be moved frequently or for safety reasons.

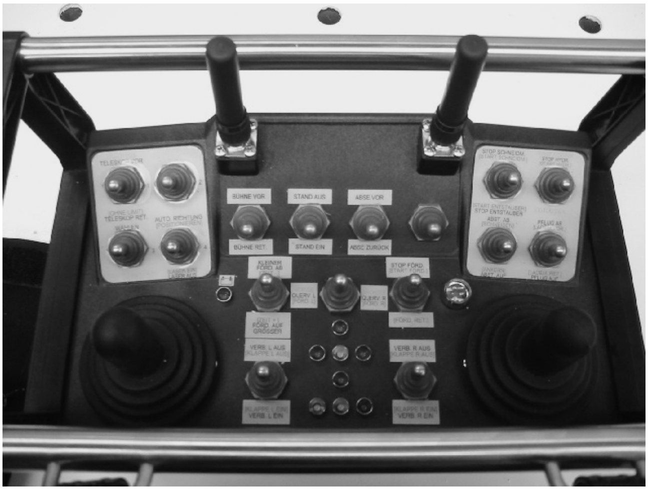


Figure 6-18 Radio remote control for ALPINE MINER AM 105 with return information.

VOEST-ALPINE Monitoring and Reporting System. All relevant machine and operational data can be measured and recorded on board of the roadheader (Fig. 6-19). This data is then sent, either through the 1,000 V supply cable or a data cable, to above ground where the current operational conditions are displayed on a screen. Reporting software is provided to generate shift reports over longer periods of time, and can also include the detection of trends. The advance rate and machine condition are monitored and recorded.

This offers considerable cost-saving potential in maintenance and site organisation.

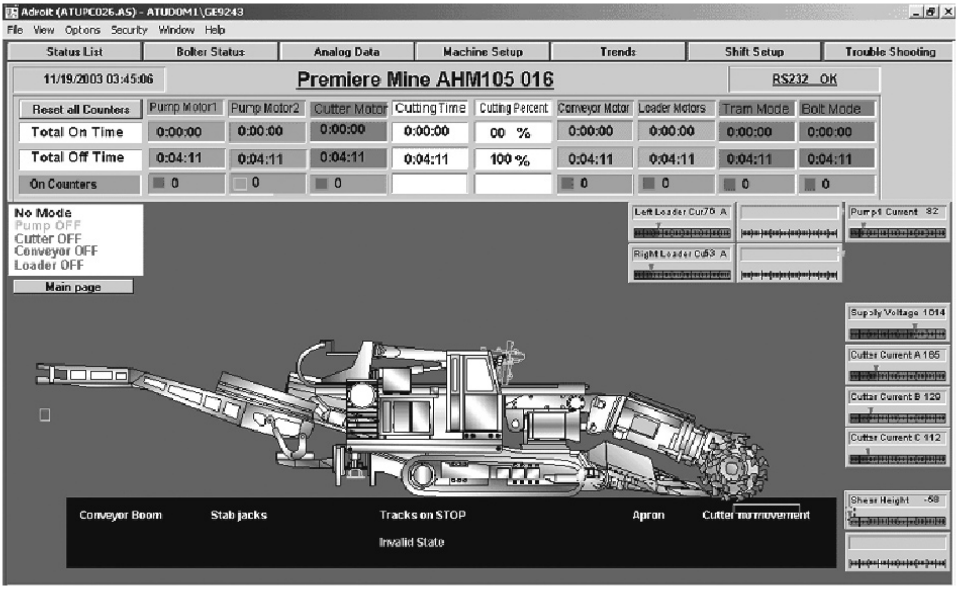


Figure 6-19 Display of operating condition of an ALPINE HARDROCK MINER AHM 105 016 in South Africa.

Safety and overload protection functions. Sensors are used to detect dangerous operating conditions and activate appropriate countermeasures through the control system, in an extreme case the activation of the emergency system (Fig. 6-20). Appropriate software controls the machine functions so that overloading of the machine structure and drive are limited. This means that the machine operator can use the product to the limits of its performance. The protection objectives of EN 954 and EN 61508 are complied with regarding safety.

Tele-operation and underground communication. Remote control of roadheaders is also possible through video systems (Fig. 6-21), but is only used in particularly restricted space or for safety reasons. This option is mainly used in mining.

Electronic operating manuals and maintenance aids. The AMOR control system is equipped with a self-test, and the maintenance aids are continuously updated. The maintenance aids shorten the service times. If required by the customer, the electronic handbook can also be installed and an electronic parts list is also available.

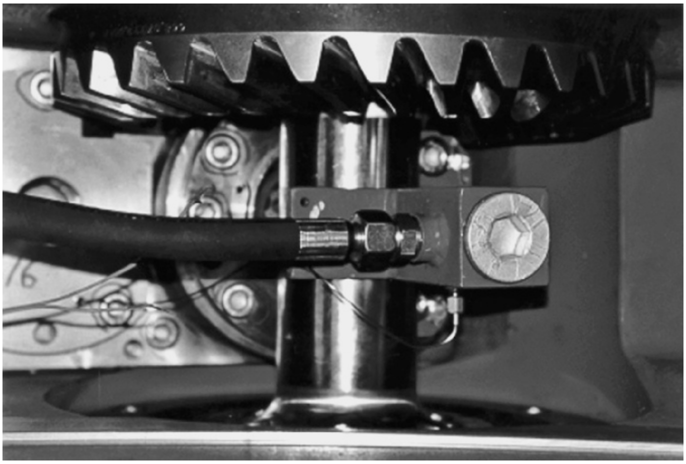


Figure 6-20 Cutter drive temperature monitoring.

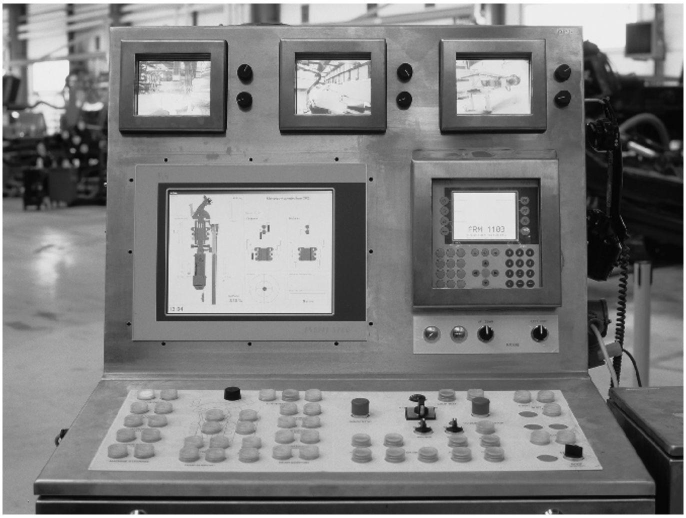


Figure 6-21 Operating position of the ALPINE REEF MINER ARM 1100. The machine in this case is being remotely controlled by cable from distances of 50 to 250 m, three video cameras monitor the operation of the machine.

Remote maintenance. Since 2001, VAB has provided remote maintenance systems for its products in agreement with the customers, to permit direct access to the machine on site by a service technician working at any location (Fig. 6-22). At the time of reporting in 2003, remote maintenance was being carried out in Mexico, Canada, India, Australia, Europe and South Africa from the production works in Zeltweg.

This leads to a considerable shortening of the maintenance times, since the maintenance operatives on site can be supported by specialists at any time. Long waiting times for the necessary expert to arrive at the site and expensive business journeys are both significantly reduced.

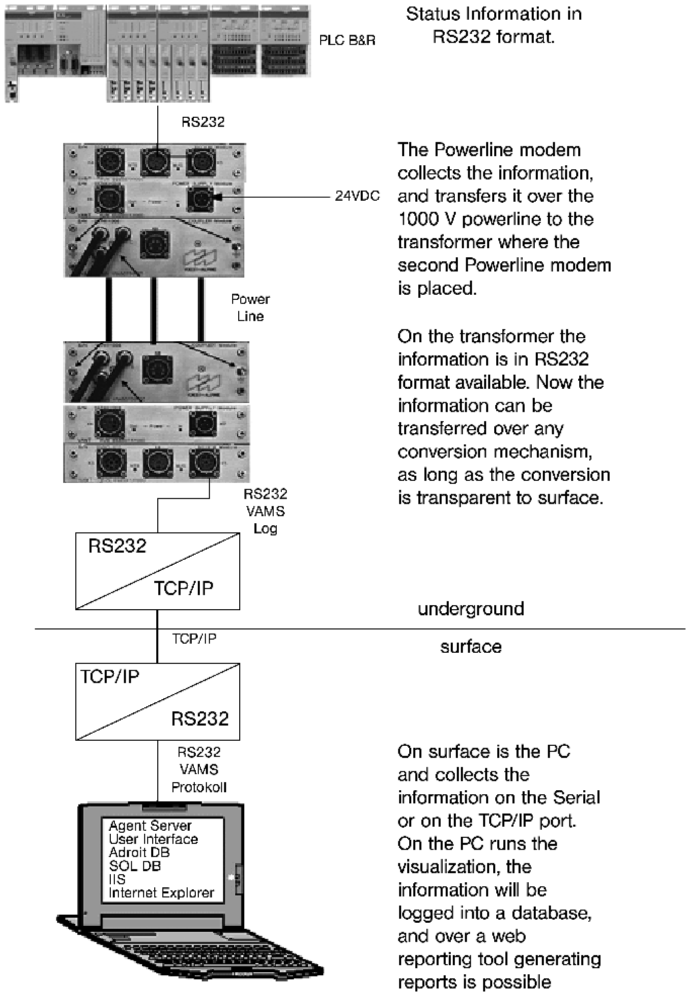


Figure 6-22 Data transfer from underground to overground. When a TCP/IP connection is used, worldwide access is possible for remote maintenance.

6.4.3 Roadheader control system from Eickhoff

The company Gebr. Eickhoff Maschinenfabrik und Eisengießerei GmbH & Co KG delivers roadheaders with a profiling system for use in mining and tunnelling [68]. This is based on a directional control system from PPS (Poltinger Precisions Systems).

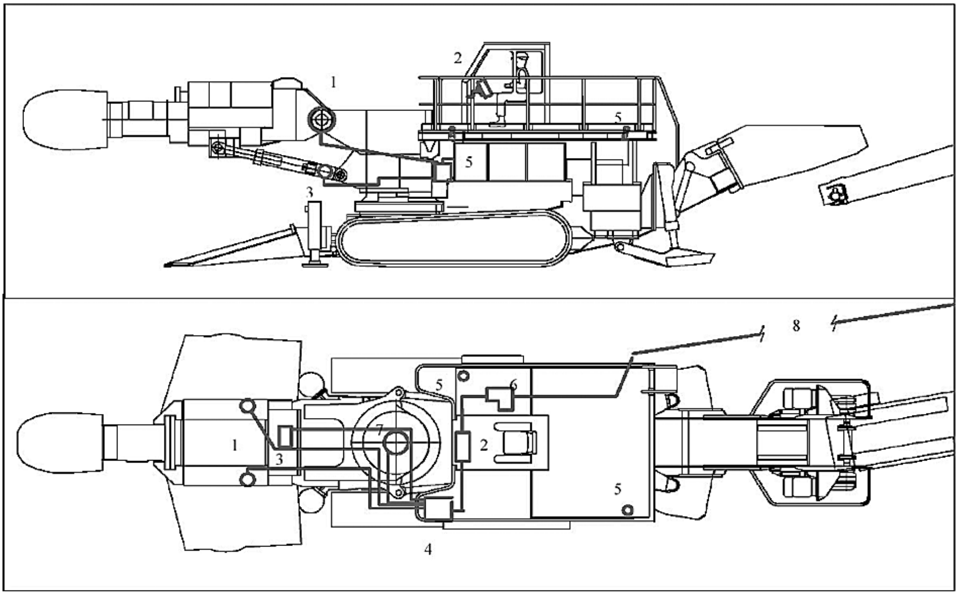
The machine driver can see on a screen the tunnel profile, the position of the machine and the cutting head. Even when the view is obscured, the driver can cut the entire profile continuously using the real-time display on the screen. According to information from the manufacturer, this has led to considerably improved performance compared to applications without computer assistance. At the same time, the profile accuracy can avoid underbreak and overbreak. The software itself is prepared for all the described control modes.

The functional mode "Directional control with profile management for roadheader" is described below:

Machine position. In order to unambiguously determine the position and orientation of the roadheader, at least two previously defined points on the machine are surveyed. A motorised theodolite is installed in the tunnel, which automatically sights two prisms installed on the machine (Fig. 6-23). The location of the theodolite and its orientation are part of the normal tunnel traverse. In order to check and determine relative position changes of the theodolite due to deformation of the tunnel, known backsights further back in the tunnel are continuously sighted. Since the relationship of the prisms to the internal machine system is known, it is thus possible to continuously transmit the current position of the machine to a computer system.

The pitch and roll of the roadheader are determined by a precise electronic two-axis inclinometer. This is permanently installed on the machine and calibrated (Fig. 6-23).

As a result of the continuous measurements, the position, direction, pitch and roll of the machine can be determined exactly and made available continuously for calculation.



1, 7 Angle sensor
2 Screen
3 Cuttingarm extension
4 Data recorder
5 Prisms
6 Indinometer
8 Data radio connection

Figure 6-23 Example of the placing of roadheader control components [68].

Cutting boom position. In order to determine the position of the cutting boom, robust but sensitive sensors are installed. Angle gauges for horizontal and vertical slewing movements and a sensor for the telescopic travel of the extending boom send changes to the computer system.

Implementation. An industrial computer installed on the machine integrates the machine position and cutting boom data and calculates the three-dimensional coordinates of the cutting head. The data is then compared to the configured design alignment and the associated excavation profile. This means that the system is capable of displaying the position of the cutting boom relative to the design profile to the machine driver at any time.

There are three operating modes for cutting:

– In the mode “manual control”, any movement of the cutting boom is possible.
– In the mode “semi-automatic”, the hydraulics are limited through control of the hydraulic system so that cutting boom movements are only possible inside the configured profile; the relevant hydraulic is blocked at the profile boundary and the cutting head is switched off.
– In the mode “automatic”, a profile can be recut with full computer control. The cutting boom continuously cuts the tunnel profile and then returns to “semi-automatic” or “manual control” mode.

The fully graphic screen display (Fig. 6-24) shows the machine operator the position of the cutting head continuously relative to the currently configured design profile. If the cutting head is moved inside the design profile, it is shown green. If it comes within 10 cm of the design profile outline, the colour changes to yellow, which shows the machine operator that he is near the outline. If the cutting head goes outside the profile outline, the colour of the cutting head circle changes to red. This leads the machine operator to the exact profile through colour changes on the screen.

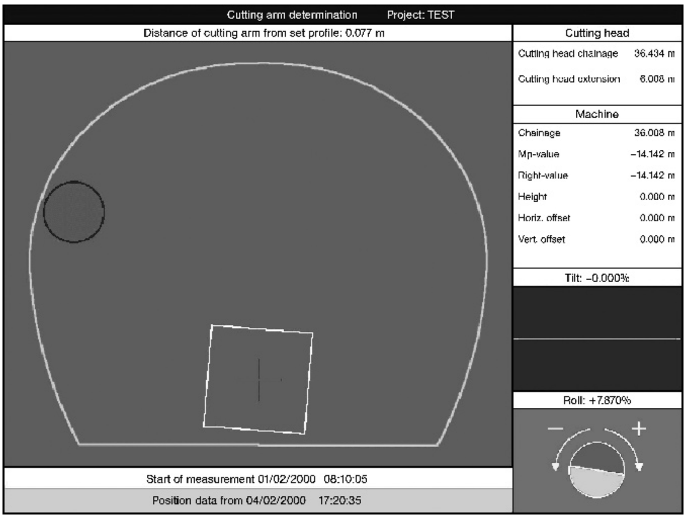


Figure 6-24 Screen display of the directional control from Eickhoff TSM [68].

The modes semi-automatic and automatic can basically be implemented by the software but are mostly not used since operation from the screen is reliable and uncomplicated. Manual control supported by the screen display is thus the current state of the technology.

6.4.4 Roadheader control system from ZED

The British firm ZED Instruments Ltd. builds a control system, which is suitable for the conversion of any roadheader and has also been used with tunnel boring machines. In a similar way to the already described systems, this system is capable of recording the design profile, cutting boom movements and the excavated part of the profile. The following only describes the system to determine the machine attitude and position.

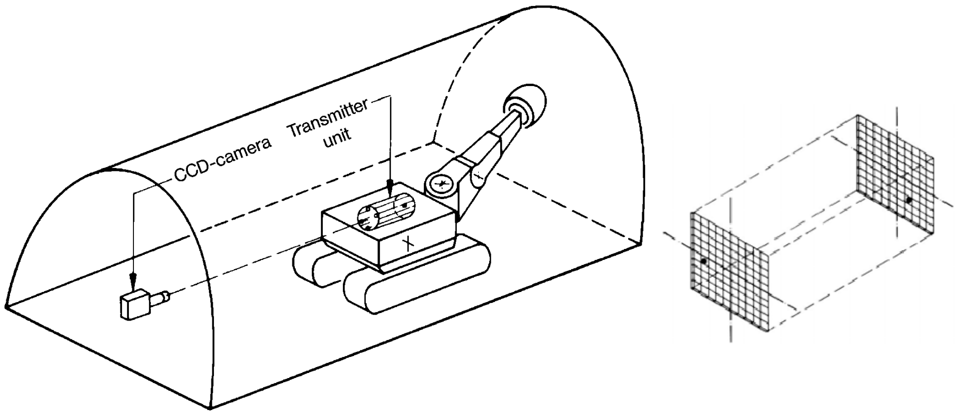


Figure 6-25 Recording the position of a roadheader (left) according to the principle of ZED Instruments Ltd. with two active targets and a laser beam. Right: details of the target.

A laser beam is aligned parallel to the tunnel axis and recorded by a target unit installed on the roadheader. The “intelligent” target consists of two targets, of which the back panel nearest the laser beam is partially transparent so that both targets can detect an impact point of the laser beam (Fig. 6-25). The target unit thus delivers two x-y coordinate pairs to the control computer, which can then calculate the parallel divergence and yaw angle of the machine. In order to reference to a global coordinate system, this system can also be combined with a motorised theodolite. This measures the distance, automatically points the laser at the target and transmits the relevant figures to the computer system [19]. In order to determine the roll and pitch angles, two inclinometers are used, which are installed on the machine at 90° to each other. The pitch angle could also be determined from the data of the target unit, but the use of an inclinometer is more precise. The position of the laser beam on the target unit is displayed to the machine operator as a control aid. As an alternative, the company ZED now also offers an infrared system, which uses an infrared beam instead of a laser beam.

6.5 Control of tunnel boring machines (TBM)

6.5.1 General

For the control of tunnel boring machines and shield machines (Fig. 6-26), the attitude and position of the shield have to be determined. The uniform movement of a shield means that systems can be used, which would not be suitable for a tracked roadheader. The control of shields through the hydraulic jacks demands careful working, since this process reacts slowly to steering commands and the shield continues still further from the alignment after a correction until the appropriate curve has been taken. Control commands therefore have to anticipate the sequence of movement. As the correct axis is neared, the machine has to be steered away again to straighten before reaching the axis.

The steering possibilities are limited, due to both the ground around the shield and the segments inside the shield tail. In order to avoid damage to the segments and unnecessary constraint forces, the steering should always avoid contact between the segments and the shield tail.

The shield has the following movements, which have to be recorded for control purposes:

– Pitch and roll angles are deviations from the vertical and are best determined with inclinometers.
– The horizontal deviation from the design axis is called the yaw angle. It can be determined optically using two targets in combination with a laser beam. Gyroscopic devices can also be used to determine it, normally meridian compasses, although these only deliver measurements when the machine is stopped between strokes. They are particularly suitable when the tunnel alignment is a great depth and the correct direction is difficult to transfer to the machine.
– The parallel divergence, that is the distance of the machine from the design axis in vertical and horizontal directions, can be determined as follows:

  1. A target on the machine is sighted with a theodolite.
  2. The entire parallel divergence can be determined using a laser beam in the design axis in combination with passive or active targets.
  3. The horizontal deviation can be determined with a meridian compass by taking bearings. A gyroscopic device in the shield indeed only delivers the yaw angle but can, when no sideways jumps of the shield movement are to be expected (correction steering in inhomogeneous ground), be used to calculate the horizontal locational divergence from the yaw angle and the travelled distance. When a gyroscopic device is used, the vertical divergence still remains to be determined. This can be measured with a laser level or a water level.

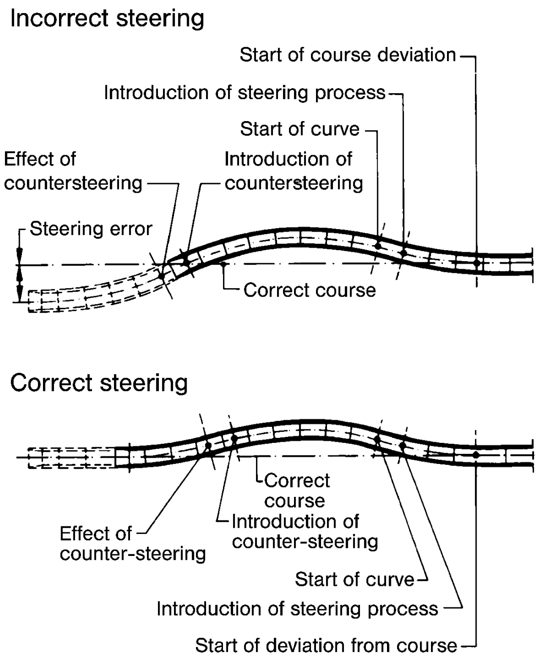


Figure 6-26 Control of full-face machines and shield machines. Steering, which only reacts to current divergence (top) and steering with anticipation (bottom) [197].

6.5.2 Steering with laser beam and active target

As an example of a modern steering control system for tunnel boring machines, the SLS-T (steering control system for tunnel boring machines with segment lining) system from the company VMT is now described:

The system consists of the following components (see Fig. 6-27):

– A laser theodolite with servostation.
– An active target (ELS) with integrated roll and inclination instrument.
– An industrial PC with monitor for the control position.
– A microprocessor interface box ("Yellow Box").

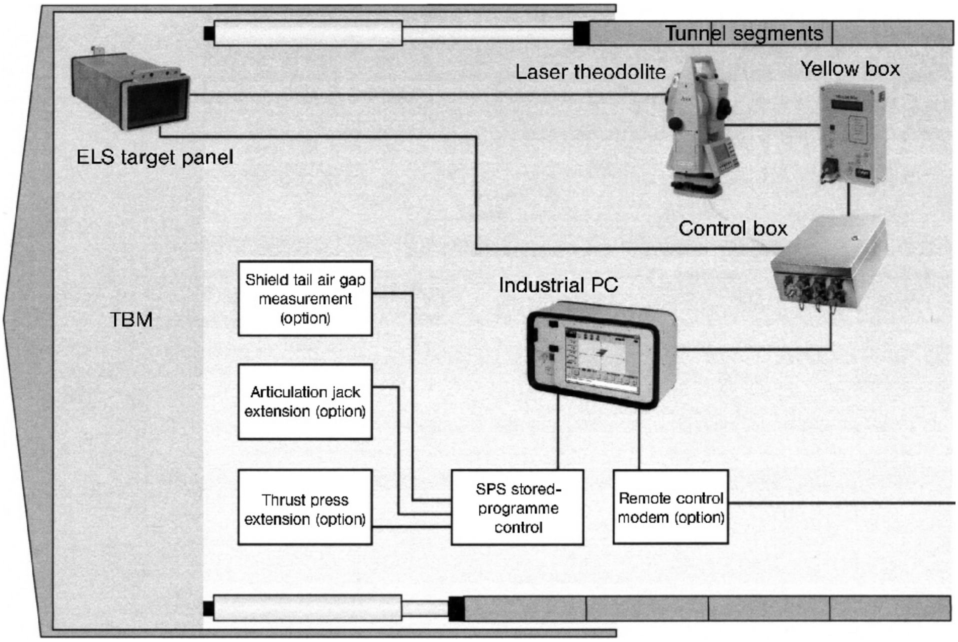


Figure 6-27 Components of the SLS-T system from VMT.

The system enables permanent monitoring of the position of a tunnel boring machine. The ELS has a light-sensitive sensor surface to localise the laser point and is fixed to the shield body. The position of the ELS is included in the construction of the shield machine. The point of contact of the laser beam on the target can be surveyed relative to the machine axis. A two-axis inclinometer for simultaneous measurement of the longitudinal attitude and roll of the machine is also integrated into the ELS. With an intelligent sensor system, the exact yaw angle is determined as the deviation between the longitudinal axis of the target and the laser beam. The offsets in horizontal and vertical directions are transmitted to the computer with the roll, yaw and pitch angles and used to determine the attitude and height. All values are determined every two to three seconds and the results are shown graphically and in numbers at the control position in relation to the planned tunnel alignment or the already calculated correction curve (see Fig. 6-28).

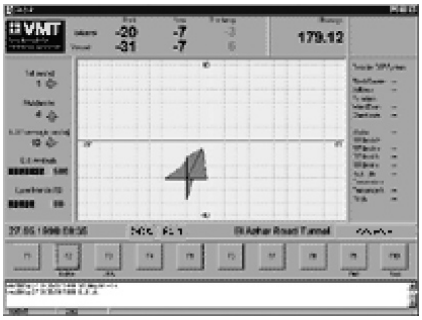


Figure 6-28 Display of the machine position and direction in relation to the tunnel axis or the planned correction curve at the control position.

This enables the machine driver to continuously see the reaction of the machine to the steering actions. This guarantees the appropriate steering commands for the situation with continuous driving along the design axis.

The distance between laser and target is normally automatically electro-optically measured and updated with a new tunnel survey after each move forward of the laser station. A normally available laser can be used, although it has to be moved forwards from time to time, especially when driving curves. The resulting alteration of the laser direction has to be inputted into the system manually. For fully automatic tracking of the laser, a servo-totalstation can be used. The laser is automatically tracked until a change of position is required. While a ring is being built, with the tunnel boring machine stationary, an automatic check of direction is made by sighting a target back down the tunnel. If any deviation of the position of the laser mounting is detected, an error is reported immediately.

When the location of the tunnel boring machine has been determined, after the most recent stroke of a gripper machine or after the most recently installed ring with a shield machine, the subsequent correct course for the machine can be calculated. If there is only a slight deviation, the course is set as the current tunnel axis. If larger deviations of some centimetres are detected, a modern system will calculate the appropriate correction curve, taking into account the practically possible steering corrections of a tunnel boring machine and the geometry of the segment rings (taper of the rings, see Volume 1, Section 2.8.5) in order to ensure installation of the segment rings without constraints.

The correction curve starts from the last actually driven curve and drives the tunnel boring machine at a feathered connection back to the design alignment. Excessive, sudden corrections can easily lead to leaving the design axis on the other side.

6.6 Steering of small diameter tunnels

6.6.1 General

For the driving of small diameter tunnels, particularly for pipe jacking, special conditions apply in many regards. The temperature differences between the heat from the machine and the cold pipe can form air layers and turbulence, which can make the use of a laser particularly difficult. On the other hand, the precision requirements are more stringent for two essential reasons: firstly, sewer pipes with little fall require high precision, and secondly the tunnel boring machine is steered hydraulically by tilting the steering head with the cutting wheel against the backup. This makes reaction to steering commands relatively sluggish. The exact maintenance of the design axis is only ensured when the steering reacts to the smallest deviations.

The most obvious and first method of manually steering a tunnelling machine is to sight a target marking with a theodolite. Despite the extensive new developments described below, this procedure is still used often in practice, for example when optical-electronic systems do not work. The same applies for the image of the beam from a laser installed in the launching shaft on a passive target. The spot of light from the laser beam on the target is imaged by a CCTV camera and displayed on a screen at the control position. The problems created by air layering and turbulence in small diameter pipes, specifically deflection and scattering of the laser beam, are significant. The magnitude of the errors can be worse than those with laser applications under unfavourable conditions in the open air by a factor of ten. Deflection of the beam by 5 cm and widening to 20 cm are not unusual over a distance of 100 m in a pipe. This situation can only be improved by an ample supply of fresh air released at the machine to create a uniform airflow back towards the starting shaft. This should be borne in mind in the evaluation of the new developments described below.

In order to avoid the difficulties mentioned above, the use of north-pointing gyroscopic devices in the machine control system has been proposed and tried out successfully. Gyroscopic devices deliver the direction with great precision and are very suitable for the surveying of curved alignments without visual contact and when the pipe is very deep, which makes the transmission of the direction to the surface difficult. The aim of automatic steering can only be achieved with very expensive ship's gyrocompasses. When a meridian compass is used, the machine has to be stopped for the determination of a reading (see Section 6.2.1). This procedure will thus only deliver a direction when a new pipe is being added. This does not comply with the requirement to react with steering commands to the smallest deviation as the pipe pushes forward. In addition, the level still has to be determined; the use of a water level is possible, but not unproblematic because the hose and device have to be passed through every new pipe. Nonetheless, the procedure has its uses, particularly considering the basic problems of optical systems.

6.6.2 Steering with a ship's gyrocompass

A ship's gyrocompass (see Section 6.2.1) is capable of pointing north even when in motion. However, the motion cannot lead to vibration or heavy acceleration, as is typically the case on large ships. In pipe jacking, this requirement is normally fulfilled.

A ship's gyrocompass housed in pressure housing with a weight of about 100 kg is installed in the leading pipe. Two repeaters are connected, one of which is clamped in a pipe further back and is read by a video camera. The second repeater is installed above ground at the control position. The repeaters only display readings and have no independent measuring function.

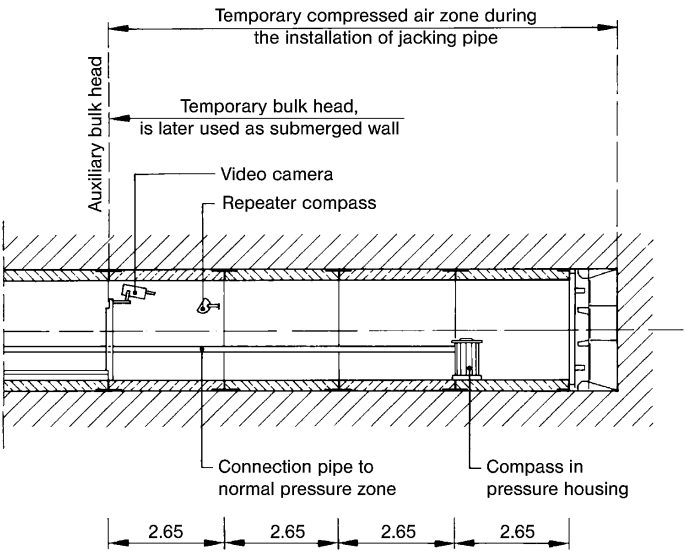


Figure 6-29 Arrangement of a ship's gyrocompass, repeater and video camera for the jacking of a sewer pipe in Hamburg-Harburg [9].

The display on the repeater is magnified six times by gearing; when the gyroscopic ball turns through 60°, the rose of the repeater shows a full revolution, which increases the reading accuracy to one minute of arc. The arrangement of ship's gyrocompass, repeater and video camera is shown in Fig. 6-29.

A successful application of this system is reported by M. Hunt [94]. The jacking of a sewer pipe in Hamburg-Harburg over 1200 m was controlled by an Anschütz gyroscopic system. The limits of the permissible horizontal deviation of 200 mm was maintained well with an error of 74 mm.

6.6.3 Pipe jacking steering with laser beam and active target

The construction of a fully automatic pipe jacking steering system, which works with an active target, is now illustrated by an example. The most important components of the steering control system for pipe jacking are the following:

1. Sensors. One active target, one inclinometer to measure the roll angle and one electrolyte spirit level to determine the inclination angle.
2. Control device, consisting of the control electronics for fully automatic steering, manual control and electric valve actuation.
3. Display device. This shows the parallel divergence, the roll and inclination angles, the deflection of the steering head and further information.

Fig. 6-30 shows the position of the target in the tunnelling machine. In order to calculate the current position, the roll angle must be included, since the target is not at the centre of the pipe but is fixed at the top of the machine. The data read from the target and the inclination sensors is displayed for the driver on a screen. In order to assist the machine driver and to avoid steering mistakes, fully automatic steering systems based on fuzzy controllers have been developed recently [92, 167]. The system otherwise corresponds in principle to the laser steering already described for tunnel boring machines.

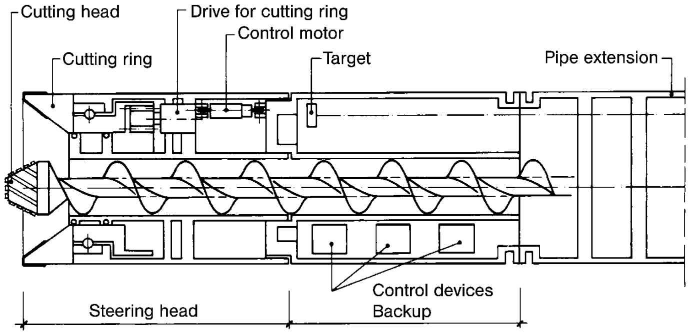


Figure 6-30 Pipe jacking machine with steering head, backup and active target. Witte Bohrtechnik GmbH/Stavia [260].

6.6.4 Steering with travelling total station

The jacking of small diameters along curved alignments, where the tunnelling machine can no longer be sighted from the starting shaft, poses new problems for surveying. Since the entire length of pipe is in movement in pipe jacking, the navigation of a pipe jacking machine is a great challenge when it is no longer possible from the starting shaft. In order to determine the position of the cutting head, no fixed points are available in the already completed tunnel. One possible solution to this problem is the steering control system for pipe jacking (SLS-RV) developed by VMT GmbH.

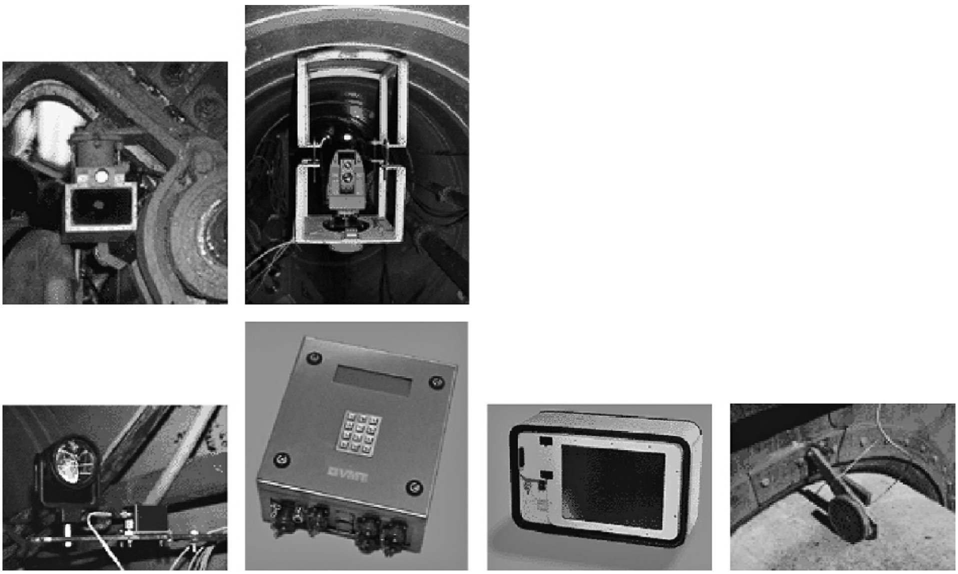


Figure 6-31 Components of the SLS-RV system from VMT GmbH. Top row from left: active target, laser theodolite. Bottom row: surveying prism, switching cabinet, control computer, measuring wheel; VMT GmbH.

The individual components of the SLS-RV (Fig. 6-31) are as follows:

- One active target as receiver for a laser beam and a simple prism to measure the distance in the pipe jacking machine.
- One servo-motorised laser theodolite on an automated tripod in the pipe run.
- Three surveying prisms (one connection target and two reference prisms for automatic surveying) in the pipe run.
- One uniaxial inclinometer in order to record the rolling of the tunnelling machine.
- One switching cabinet.
- One control computer at the control position of the machine.
- One measuring wheel to measure the distance jacked.

Fig. 6-32 shows the arrangement of the system components in the pipe run. This layout makes it possible to navigate even round horizontal and vertical curves. After a starting phase, the components are installed in the front part of the tunnel and move with the jacking of the pipes without any line of sight from the starting shaft. All hardware components are controlled by the control computer, the measured data is queried and saved in a database and the relevant information about the progress of the tunnel is displayed by the control computer.

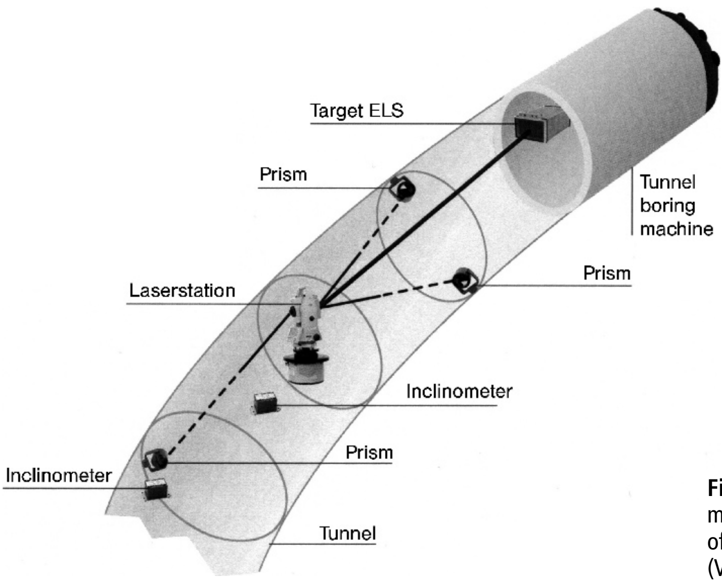


Figure 6-32 Arrangement of the components of the SLS-RV system (VMT GmbH).

Fig. 6-33 shows an example of the graphical and numerical display of the machine position and attitude at the control position. The machine driver can use this to steer the tunnelling machine.

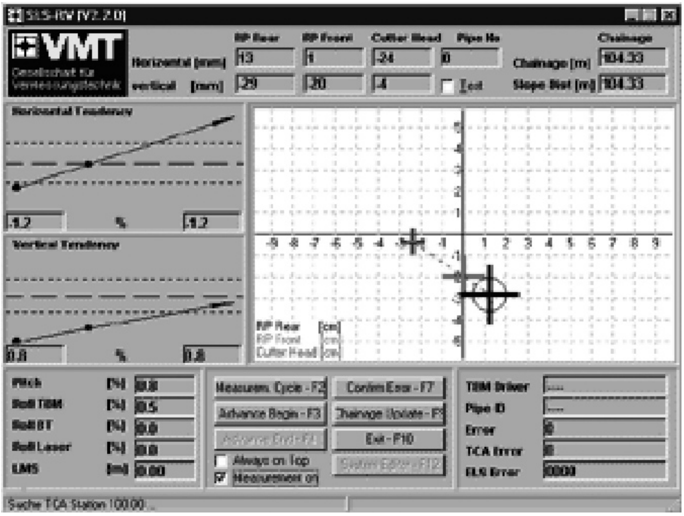


Figure 6-33 Graphical and numerical display of machine position and attitude at the control position.

7 Special features of scheduling tunnel works

7.1 General

The scheduling of tunnelling works has a few special features, which are discussed in this chapter together with the general procedure.

7.2 Historical overview

General. Detailed reports of the planning of operations and working sequence for completed tunnel projects have been available for more than 200 years. The names F. Foetterle, R. v. Gunesch, A. Lorenz and F. Ržiha make it clear that it were tunnel engineers, who had already recognised the importance of tight planning of the construction schedule [131].

This early recognition of the necessity of scheduling in tunnelling was not simply a coincidence. Increasing industrialisation in the 19th century demanded rapid development of transport, which could not be delayed by interminable waiting for the completion of tunnels. The limitations of working space (each group of workers created the working conditions for the following group of workers) did not permit faster working simply by employing more men. Construction periods of up to 20 years were no exception. The basic methods for quicker working were only invented in 1861 with the hammer drills used in the Mont Cenis Tunnel (Fréjus Rail Tunnel) and the invention of dynamite in 1867. This period also saw the introduction of the time-distance diagram, also called time-chainage diagram, which were then more accurately called work progress plans. Such plans also served as progress plans for invoicing and documentation for the employer, and also as insurance in the event of any legal disputes; scheduling overruns were subject to very high contract penalties at the time. The contractors had to accept most of the risk from geological conditions, with the effect on construction time and overbreak. Quite a few well-known tunnelling contractors became the victims of an unreasonable distribution of risk [131].

Examples for the planning of tunnelling works are given by the former Lupkower Tunnel in the Carpathian mountains, which was built by R. v. Gunesch in the then extremely short time of 2 ½ years [131], and the building of the 8,134 m Hauenstein Base Tunnel between 1912 and 1915 (Figs. 7.1 and 7.2). This was the first large mountain tunnel where the invert heading was drilled with compressed air hammer drills leading to improved and above all more uniform progress. In detail, the following advance rates were achieved:

Invert heading: average 300 m/month.

Enlargement: average 235 to 240 m/month.

The tunnel was actually completed 16 months ahead of the officially agreed deadline.

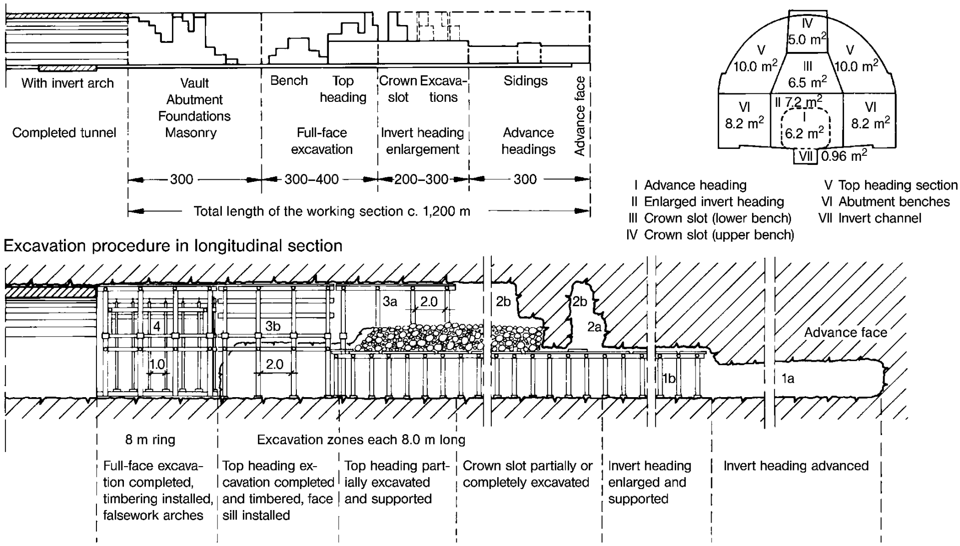


Figure 7-1 Diagram of the working method for excavation and timbering (support) in the Hauenstein Base Tunnel [257].

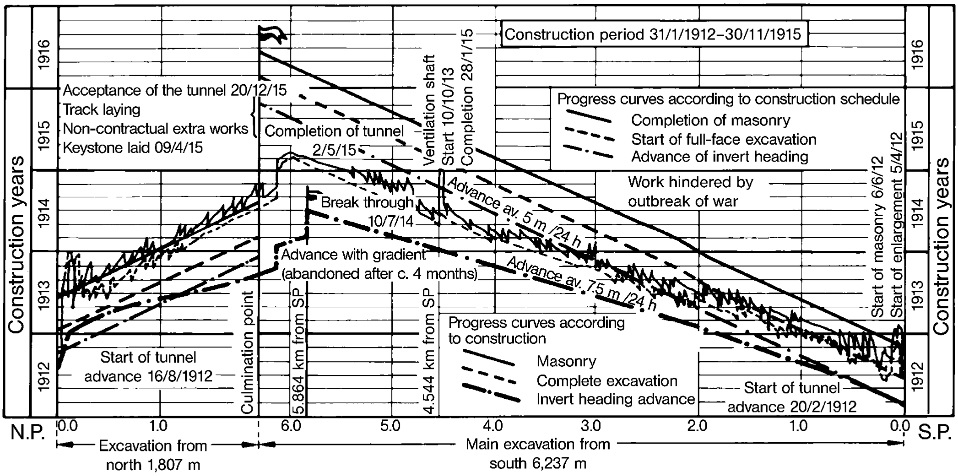
Fig. 7-2 shows a typical work progress plan with the planned and actually achieved deadlines.

Construction progress. At a time when tunnels were excavated by laborious manual labour with hammer and pick, or at best with relatively ineffective low-explosive gunpowder, the process of "breaking" had such overriding importance that other works were relatively insignificant. It was only possible to influence the advance rate by employing a larger number of hewers working at the face, which was of course restricted by the available space.

If we compare the monthly advance rates achieved in two classic tunnels of that time, the Mont Cenis and St. Gotthard Tunnels, then the considerably faster progress in the St. Gotthard Tunnel can be recognised:

1. Mont Cenis Tunnel, $v = 168.7$ m/month (working from both ends in October 1870).
2. St. Gotthard Tunnel, $v = 243.8$ m/month (working from both ends in October 1875).

It should be remarked that drills were in use in both of these tunnels, although in the Mont Cenis Tunnel, the use of drills was more in the nature of a trial and that also only traditional gunpowder was used. In the St. Gotthard, on the other hand, the tunnel site was better equipped with improved drills and high-explosive (dynamite) being used.



Contractual completion dates according to approved project planning		Construction progress of tunnel construction company	
Breakthrough of invert heading	13 Jan 1926	Breakthrough of invert heading	10 July 1914
Completion of full excavation	March 1916	Completion of full excavation	Feb 1915
Completion of masonry	01 Aug 1916	Completion of masonry	21 April 1915
Completion of tunnel incl. open-air line	13 Jan 1917	Completion of tunnel incl. open-air line	20 Dec 1915

Figure 7-2 Graphical progress plan for the excavation and masonry lining of the Hauenstein Base Tunnel [257].

The continued development of mechanical equipment in tunnelling and the demand for shorter construction times also led to further development of construction scheduling.

These schedules were intended to offer a certain pictorial representation of the sequence of construction works and also enable continuous control. Pictorial representation means the graphical illustration of the entire future construction sequence with visualisation of all planned activities, also their technical and operational dependencies to each other, in a clear and manageable plan. Particular value is placed on descriptive drawing. As the specific form of plan in tunnelling, the time-distance diagram still remains the most commonly used type. Fig. 7-3 shows an example for driving a mountain tunnel. The comparison of planned and actually achieved performance can be seen clearly, including consideration of the various difficulties in the form of rock mass classes. This form of illustration in a volume-time diagram enables quicker recognition of the various advance rates in the individually geologically defined sections from the different gradients of the progress lines. With the process speeds, which are normally determined from figures for time taken, it is possible to plan the production equipment in advance and match components. Fig. 7-4 shows how closely progress plans and operations diagrams are interrelated.

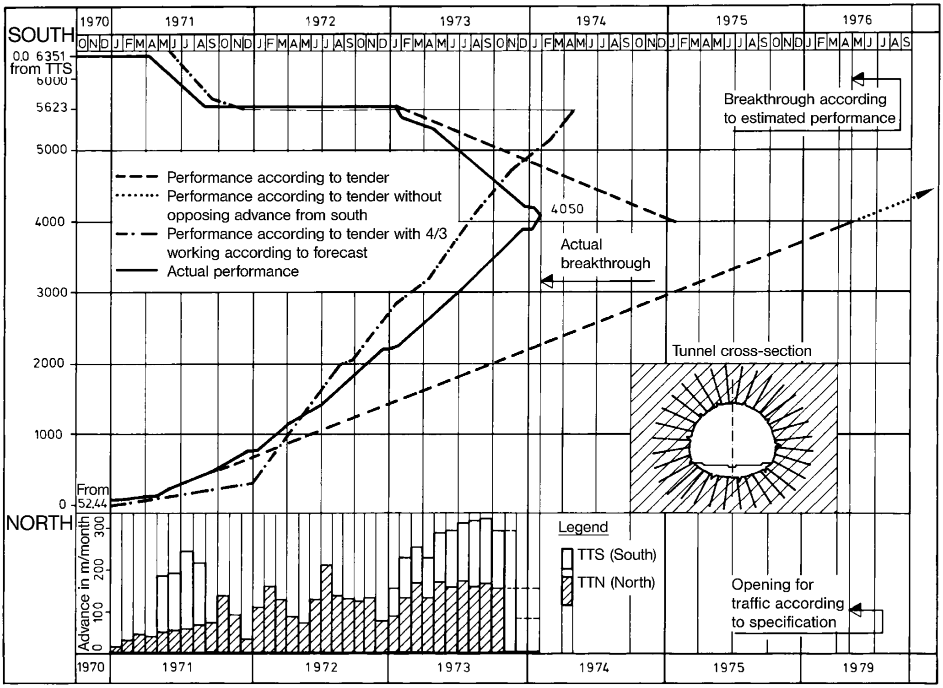


Figure 7-3 Progress plan for the Tauern Tunnel.

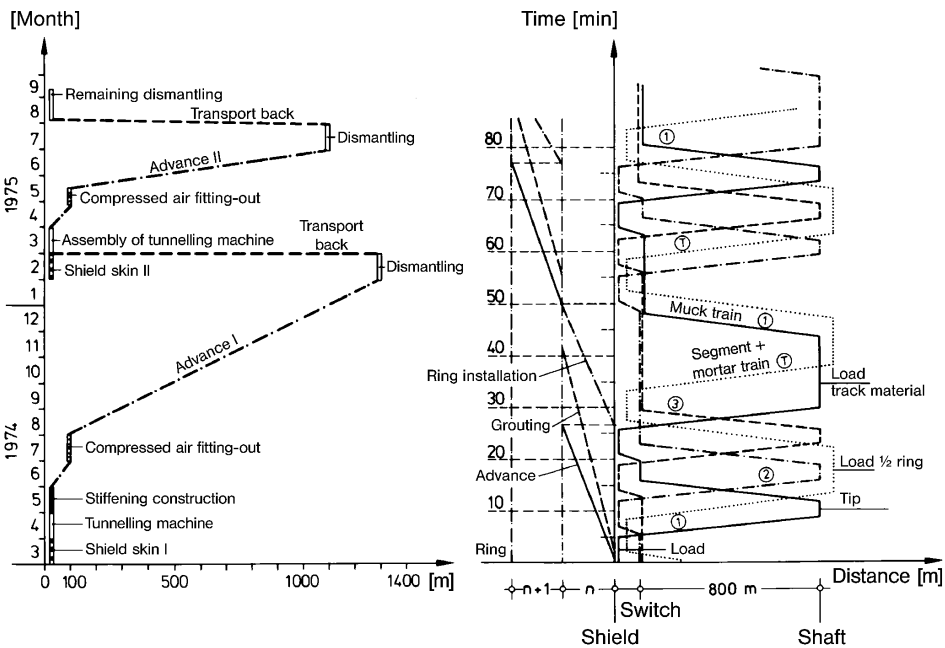


Figure 7-4 Time-distance diagram (left) and operational diagram for a shield tunnel drive, from H. Distelmeier [61] (right).

7.3 General planning of tunnel drives

General. A road or rail tunnel is always a complex structure, so its uninterrupted construction demands particularly careful planning of all activities. The construction schedule will arrange numerous individual objects, like for example cuttings, embankments, crossing structures for road or rail, drainage facilities, in the correct order in order that catenary, signal and communications equipment can be installed and finally construction and planting to fit the structure into the landscape should not be forgotten. W. Linkerhägner [123] writes: "The requirements of environmental protection have to be balanced against the principles of technical rail construction. Transparent planning and early contact with the local population is therefore unavoidable."

It is clear that planning cannot start with the start of construction of the project but the requirements of function, use, design and scheduling all have to considered and coordinated during the preliminary design phase. Above all the consideration of cost-effectiveness has to be carried out in relation to the required deadlines, which could lead to limitations for some construction processes. According to L. Wichter [255], the planning period before the construction of an inner-city construction project is about 10 to even 20 years, with six to ten years having to be included for the construction measure itself, depending on the extent and difficulty. The legal basis for construction in Germany is provided by:

– The Zoning plan or planning approval (road tunnel).
– The planning approval (rail tunnel).

Town development plan. The town development plan provides the legal framework for the regulation of town planning and is the basis for the issue of building approvals [244]. The town development plan is laid down and defined in Germany in the federal building law, § 8:

1. The town development plan contains the legal provisions for the regulation of town planning. It provides the basis for further measures necessary for the implementation of this law.
2. Town development plans are developed from zoning plans.

§ 9 of the federal building law regulates the content of the land development plan.

Planning decision. The planning decision is an administrative document and ends the planning process. The planning decision has the following preferential effects [244]:

– Concentration effect (the numerous legal matters of public administration are collected in one document and regulated, no further approvals are necessary).
– Configuration effect (decision cannot be appealed).
– Basis for compulsory purchase (claiming property of third parties).

In the course of the planning process, the construction process in the relevant construction sections is also defined, which makes it clear for the citizens whether, for example, a cut-and-cover tunnel or a mined tunnel will be built, or where additional measures like wells or cut-off walls will be installed. A summary of public participation in the various phases of design and planning can be seen in Table 7-1.

Table 7-1 Public participation in design and planning phases [244].

Design phase	Type of information or participation
1. Preliminary investigation (processing of variants)	General public information, explanation to the town council
2. Selection from the variants, combination of variants	Information and participation of the citizens affected by the design, group meetings, individual explanation
3. Design processing	Participation by the citizens affected by the design, essentially in individual meetings,
4. Planning process	Statutory participation of citizens
5. Planning decision	Legal remedies for citizens (action in the administrative court; appeal to higher administrative court; in fundamental cases, appeal to the federal administrative court)

New underground railways may only be built with the approval and the issue of a planning decision by the responsible state authority, so the planning decision can be regarded as the „actual building approval" for underground railway construction [201].

In the planning permission process for new lines of German Railways DB, the land use regulation process according to the federal land use law (ROG) and the appropriate state laws have to be fairly balanced with public and private interests [27]. The structure of the planning permission process under § 36 federal railway law and the planning guidelines of German Railways can be found in the reference [176] (Fig. 7-5).

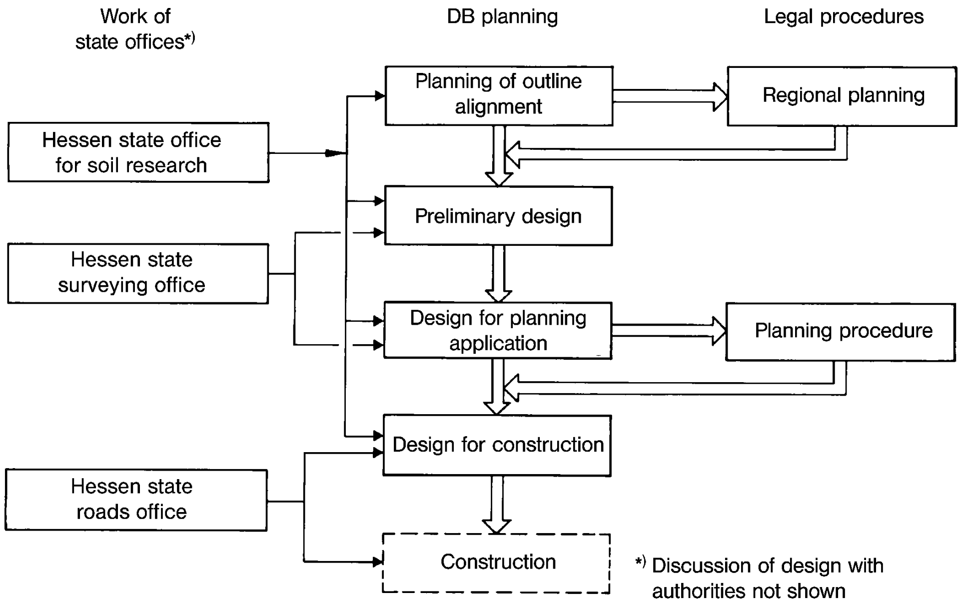


Figure 7-5 Example of planning systematic for a new line project in Hessen.

In summary, the town development plan and the planning decision differ in the following points [255]:

Town development plan

- The main emphasis of the town development plan lies in laying down various types of land use for a defined area (for example overground and underground transport areas, building and green spaces).
- Construction problems are only covered to a limited extent by the town development plan (for example tunnels passing beneath buildings, support with anchors).
- A town development plan is particularly suitable for the construction of transport facilities with heavy interdependence of the interests of urban planning use, like for example stops in densely built-up areas and where transport development requires intrusion into existing building, demanding a decision about replacement building.

Planning decision

- A planning decision includes the precise specification of a tunnel construction measure, which also includes construction aspects and regulates legal matters (for example water protection law).
- A planning decision is particularly suitable for pure transport infrastructure projects, which have little effect on other urban planning interests (for example building, green spaces).
- The planning decision is suitable for longer tunnels without major impact above ground.

The phase between the design of a construction project and the permission to start construction is nowadays the longest. In addition to the planning permission process, there are also political objectives, which can have a significant effect on the process.

The time taken for the pure construction work indeed seems short compared to the total duration but it is exactly in this area that faulty design work or the lack of stringent deadlines can lead to delays, both to the project and to connecting works, which could have been avoided by appropriately careful scheduling. Careful in this case means taking into account the number of parameters that have to be included in the planning of the construction schedule, a particularly large number for a tunnel project with the compounding problem that some influential factors are difficult to determine; although their effect on construction progress is known, their time of occurrence is not always foreseeable. Examples would be unpredictable heavy water ingress or changing geological conditions, for example karst features [157].

In order to produce a correct construction schedule, the evaluation of the preliminary geological investigation of the ground around the tunnel alignment is thus of significance. The determination of the duration of individual activities should be based on a calculation of performance, which is calculated for the most vital processes or at least should be taken from experience (values derived from analysis of costs from completed projects of a similar nature) [131].

7.4 Planning tools

General. For complicated construction projects, scheduling also undergoes a development process, which means that it is not possible at the start to assemble all the works that will be required and the associated durations of the actual construction works. Table 7-2 shows this process from conception through design to work preparation and construction. The planning tools are the bar chart (Gantt chart), precedence diagram or critical path diagram and the time-distance diagram.

Table 7-2 Stages of planning construction time [138].

Stages **Project phase**	**Construction programme** **Design phase**	**Construction schedule** **Before construction**	**Work sequence plan** **Construction phase**
Can be used as	rough plan	rough plan fine plan	fine plan
Process on the critical path leading activity supplementary activity	 calculated estimated	 calculated calculated	 calculated calculated
Process off the critical path leading activity supplementary activity	 estimated estimated	 calculated estimated	 calculated calculated
Consideration of the familiarisation time	no	no	yes
Planning tools	bar chart network diagram	distance-time diagram	speed diagram distance-time diagram

Bar chart. Bar charts, more specifically Gantt charts, have a time coordinate, on which the construction activities of a project are entered in coarse structure. No complete marking of the reciprocal dependencies of activities is normally undertaken. Considered overall, bar charts are only of limited use because of their coarse divisions and the approximate estimation of durations. Fig. 7-6 shows a bar chart produced as a construction schedule for the Westerschelde Tunnel.

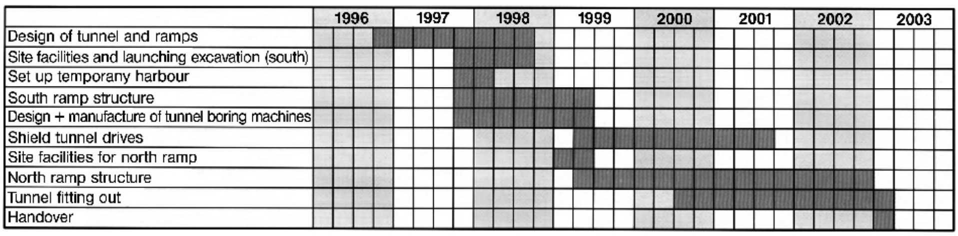


Figure 7-6 Construction schedule for the Westerschelde Tunnel [180].

Time-distance diagram. A tunnel is, except for branched caverns, normally a linear structure with the built volume expanding linearly, with the technical and operational dependencies being arranged in a clearly apparent row. For representation in a time-distance diagram, the tunnel length is shown on the x-axis and time on the y-axis, or sometimes the other way around (see Fig. 7-3). This depiction can simultaneously illustrate the temporal, spatial and capacity coordination of the activities, so the time-distance diagram is now predominant for the scheduling and control of the progress of tunnel construction.

Critical path diagram. A critical path diagram is a graphical depiction of the sequence of activities as nodes and arrows. This planning tool is predestined for the general planning of complex construction projects and for the planning of the sequence of complex processes. Critical path diagrams include the processes of analysis, description, planning, control and monitoring based on graph theory. Time, cost, equipment and other influential factors can be taken into account [138].

Critical path planning was first developed in 1956 and 1957 for the space industry and for production changeovers in chemical plants. The known methods “Critical Path Method” (CPM) and “Program Evaluation and Review Technique” (PERT) then spread quickly in the construction industry. After a few setbacks in practical application, attempts were made to develop systems that better reflected the conditions affecting construction production. These systems were also based on node-oriented networks, and some established names are the “Metra-Potential Method” (MPM), “block network planning” (BKN) and “precedence diagram”. Theoretical formulations and details of application are not given here and reference is made to the specialist literature.

7.5 Control methods

General. The tasks of progress planning include the coordination of construction time and capacity including consideration of cost, process technology, time and specific construction site factors. In the construction phase, the progress of construction is a variable quantity. This makes it even more important to continuously check the actual progress of the tunnel drive against the intended progress and against contractually agreed deadlines, and also to determine the actual cost position on the site.

7.5.1 Control of deadlines

It is safe to assume that the actual progress of construction will not normally coincide with the planned progress since either the design is changed, quantities are not as estimated, unforeseen geological difficulties have been encountered or also that not all conditions were known when the schedule was drawn up. In all these cases, however, the result is that measures to adapt capacities, dates and dependencies have to be planned and implemented as construction proceeds.

Only continuous checking of the schedules and deviations can deliver sufficient information. The system has to be able to process control data quickly. The results of the check should make the consequences clear and enable decisions, and measures to adapt to new circumstances should be introduced promptly, in order to be able to pass the new information to the site.

When the critical path planning method is used, the following data is required for the control of construction progress:

– Date of check.
– All completed activities since the last schedule check.
– All activities still being worked on at the date of the check and their remaining duration.
– All foreseeable alterations of dependencies.
– All foreseeable alterations of activities and quantities.

7.5.2 Cost control

The following data should be gathered on site for all running activities on the day of checking [138]:

1. Completed quantity of the activity V_1
2. Remaining quantity of the activity V_2
3. Current unit price of the activity k^V

with the following detailed points being considered:

– Running activities are those, for which the completed quantity V, and also the remaining quantity V_2 are greater than zero.
– Activities that have not yet been commenced ($V_1 = 0$) are only of interest for checking purposes if factors that will alter their costs can already be foreseen, such as the quantity and/or the unit price will be changed. This also includes activities, which will be introduced or completely omitted.
– If the final unit price has not yet been decided for an already completed activity, or if the quantities still have to be confirmed, then this activity can be continued with the expected quantity.
– Finally completed activities ($V_2 = 0$) should no longer be corrected.

The contractor's claim for payment is then calculated from the sum of the individual activity costs

$K_1 = V_1 \, k^V$

The current "actual" total cost $K = V \, k^V$ is compared to the total cost according to BOQ $K_o = V_o \, k_o V$, with the evaluation of the results of the progress check also having to be taken into account. If there is a critical path diagram for the site, then the checking procedure can be extended to deliver additional information. It should be noted that cost control can never be performed independently of progress control, since it otherwise delivers incorrect information.

One important factor that affects the costs arising should still be briefly mentioned. The objective is always to implement a construction project in the optimal construction time with optimal construction costs. This does, however, demand precautions and the provision of measures in case of problems. The development of costs resulting from an interruption for a limited time is illustrated in Fig. 7-7 and this shows how adjustments of capacity can lead to cost overruns.

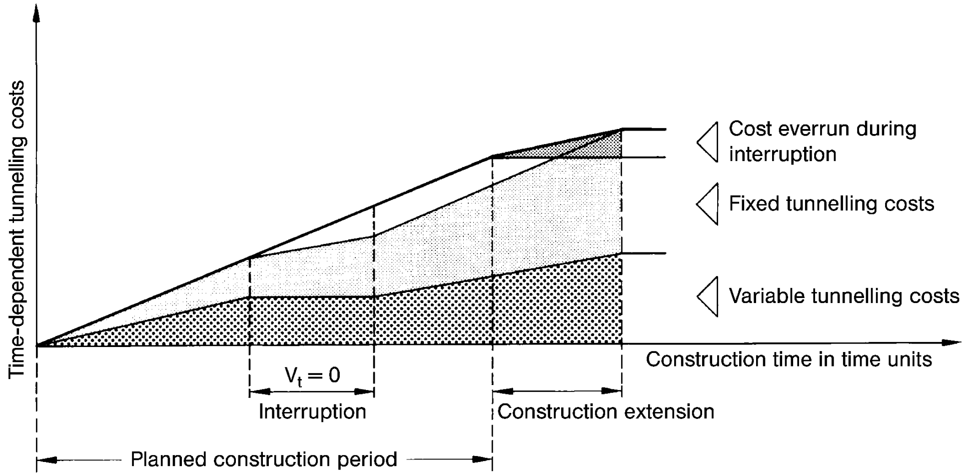


Figure 7-7 Cost development resulting from interruptions of the advance [122].

7.6 Examples of construction schedules

7.6.1 Construction schedule for the City Tunnel, Leipzig

The core of the future S-Bahn urban rail network in the metropolitan area of Leipzig/Halle is an altogether 4.1 km long north-south link through the inner city of Leipzig, which provides an underground connection between the main station, which was opened in 1915 and recently extensively rebuilt, and the Bayerischer Bahnhof station. Both these stations previously were termini and were linked underground by a 1.8 km long tunnel driven by a shield machine. This underground link consists of two single-track tunnel bores with single-pass watertight segment linings, bored by a shield machine. In order to ensure an optimised traffic utilisation of the connection, the tunnel was designed to provide two lines for both S-Bahn urban rail and express trains. The external diameter of the tunnel bores is thus 9.00 m. The tunnel cross-section is designed for a travel speed of 80 km/h.

The implementation of the mega-project is displayed clearly for all participants in a framework schedule. This contains the planning procedures, the permits, tender and award, the project management itself as well as acceptance and trial operation (Fig. 7-8).

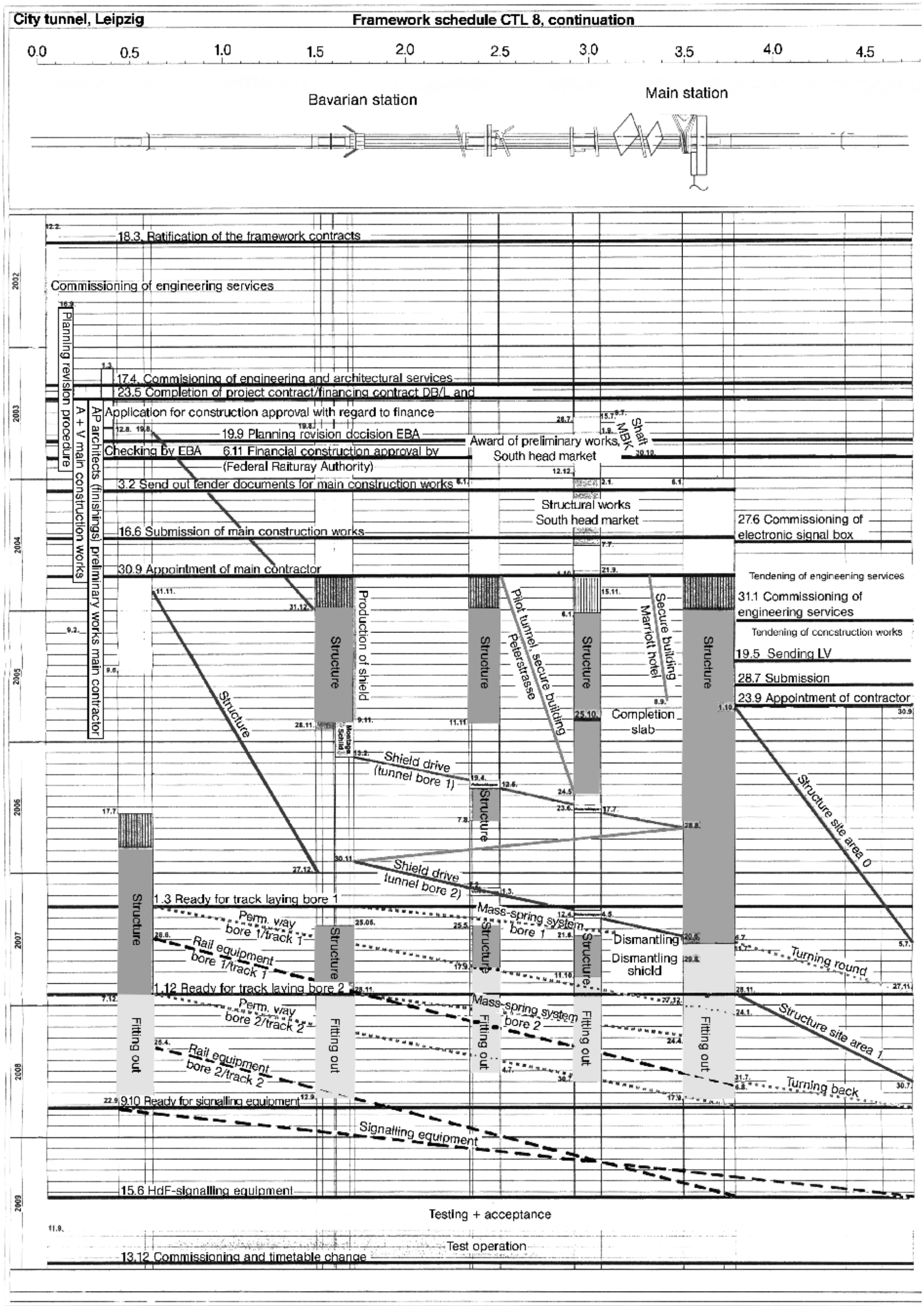


Figure 7-8 Framework schedule of the City Tunnel, Leipzig.

7.6.2 Scheduling of rail tunnels through the example of the Landrücken Tunnel and the particular question of starting points

General. The Landrücken-Tunnel is 10,779 m long and currently the longest rail tunnel in Germany. Both the planning of tunnelling works and the schedule demanded particular standards on this project.

Adit. In order to construct a mined tunnel, it does not always have to be practical or economically viable to start tunnelling directly from the tunnel portal. For example, a mined tunnel can be started from further into the mountain or additional starting points can be created within the length of the tunnel accessed through adits or shafts. Early tunnels were excavated as far as possible through a number of adits or shafts and other starting points, because these enabled even longer Alpine tunnels to be completed within a relatively short construction period.

Adits are normally provided to drive the main tunnel in both directions from additional starting points. According to L. Müller [160], this achieves the following:

- A number of faces can be excavated at the same time, so the number of miners and machines, which is limited by the restricted space, can be increased.
- Respectable shortening of construction times.
- Shorter transport routes lead to lower mucking costs, although the savings are often lost due to the additional machinery and site facilities required at the entrances.
- Much more simple operation, for example the number of switches, transformers or the dimensions of the pipework can be considerably reduced.

It can often be disadvantageous to select the shortest route for the access adit if this would slow the advance rate due to the prevailing geological conditions. In this case, it could work out cheaper to accept a certain extra distance (Fig. 7-9).

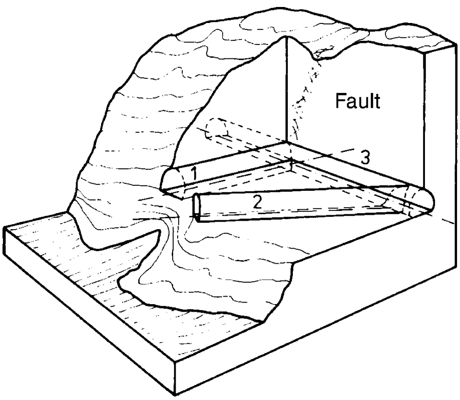


Figure 7-9 Access adit [160], variant 1: short, but difficult due to the fault; variant 2: longer than variant 1, but more advantageous since the rock is not faulted.

For the Landrücken Tunnel, which is described here as an example, the decision was made after detailed studies, and considering the length of the construction section, construction time and cost-effectiveness aspects, to split the section into three individual contracts with the accordingly required extra starting point for the middle section. The south contract section was also to be accessed through a side tunnel about 104 m long with an access adit. These additional starting points meant that nuisance from construction traffic for a built-up area extending to one of the portals could be greatly reduced. The overall situation with the division into contract sections and the intersection and three standard cross-section of the 650 m access adit for the middle contract section can be seen in Fig. 7-10.

The 650 m long access adit was started in September 1981 and completed about one year later, so the experience gained and information about geology and hydrology was available for the preparation of tenders for the main Landrücken Tunnel. Also well worth mentioning is the fluent contract boundary between the north contract and the middle contract, which the responsible joint venture agreed. The fluent contract boundary means nothing less than that the joint venture, which made better progress with their section, could also drive a longer distance of tunnel. This type of contract arrangement leads both directly to an optimisation of the construction schedule, and indirectly also leads to improved construction progress due to the competition to be expected between the contractors. Another factor that also turned out to be significant for construction progress was that the design was based on a tunnel section that just provided enough space for trucks to pass. The construction deadlines agreed with the joint ventures can be seen in Fig. 7-10.

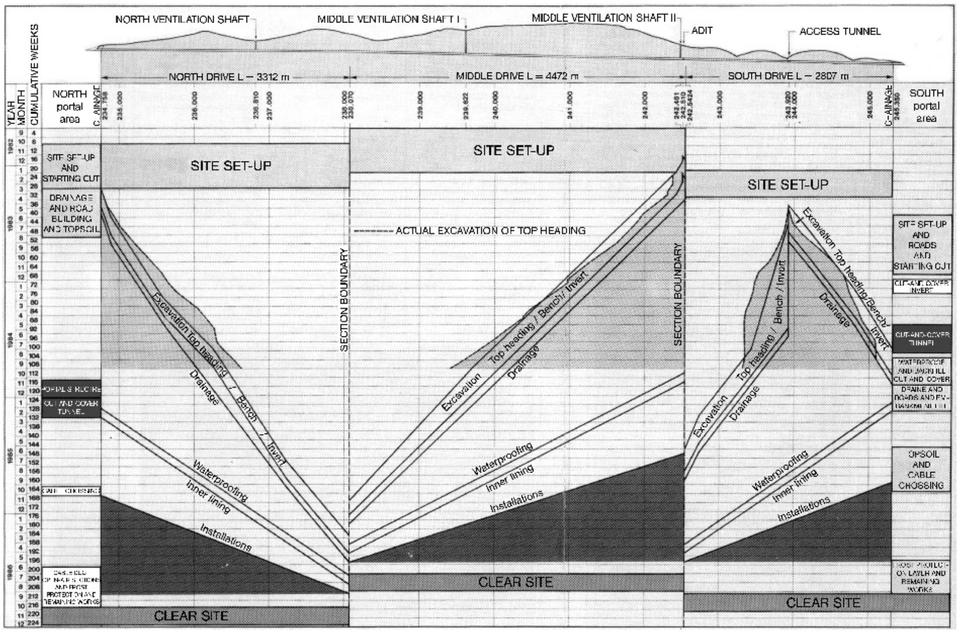


Figure 7-10 Construction schedule of the Landrücken Tunnel [31].

The breakthrough of the Landrücken Tunnel on 16 January 1986 was in advance of schedule despite the geological difficulties during the tunnel drive.

7.6.3 Scheduling of road tunnels through the example of the Arlberg Tunnel

General. The Arlberg Tunnel is described as an example for the scheduling of road tunnels. The tunnel was completed on 1 December 1978 after a construction time of four years and five months and handed over for opening. The tunnel provides a winter connection between the Austrian states of Tyrol and Vorarlberg.

The individual alignment proposals and variants, the cross-sectional sizes, ramps, starting tunnel, ventilation concept and further aspects will not be described at this point. This can be found in the relevant literature, for example the construction documentation of the Arlberg Straßentunnel AG [7]. What will be described is the interaction of the scheduling of the project and the construction of the works in the phases of

– tendering,
– bid and award,
– construction.

Progress according to tender. The concept of construction progress shown in Fig. 7-11 is based on the advance rates to be achieved in continuous operation. The following advance rates were demanded from the bidders:

– Starting tunnel 130 m/month
– Main Tunnel East 135 m/month
– Main Tunnel West 105 m/month

For the two shafts, advance rates of 50 m/month were intended.

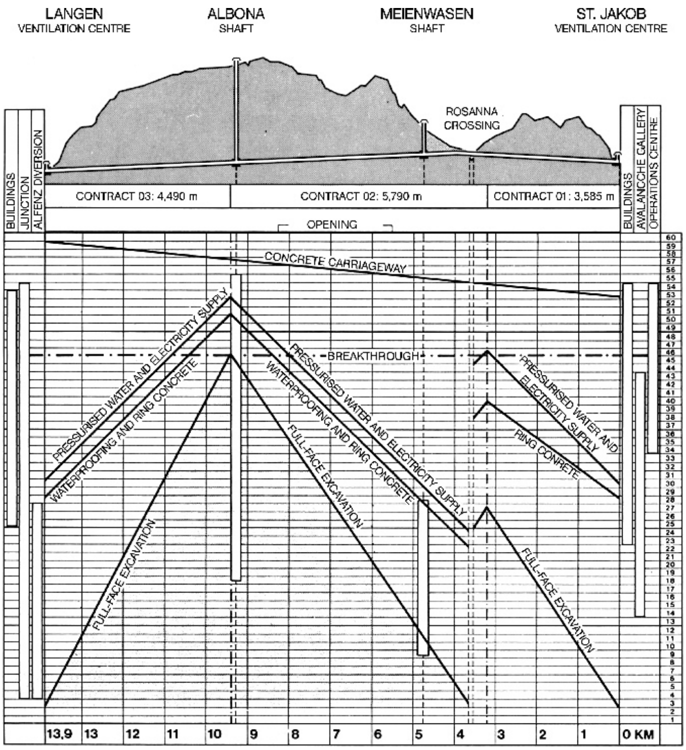


Figure 7-11 Construction schedule. Arlberg Tunnel tender [10].

Construction according to bid and award. After checking the shortlisted bids, which all offered faster construction than demanded in the tender, the combination of the tunnelling works in contract sections 01 and 02 with a total length of 8,348 m was decided. The constructions schedule, which the award was based on, can be found in Fig. 7-12. This is based on the following average advance rates:

- West Tunnel about 130 m/month
- East Tunnel (start) about 200 m/month
- East Tunnel (enlargement) about 165 m/month

A fluent contract boundary was also agreed on this project. The boundary could move 500 m without changing the price.

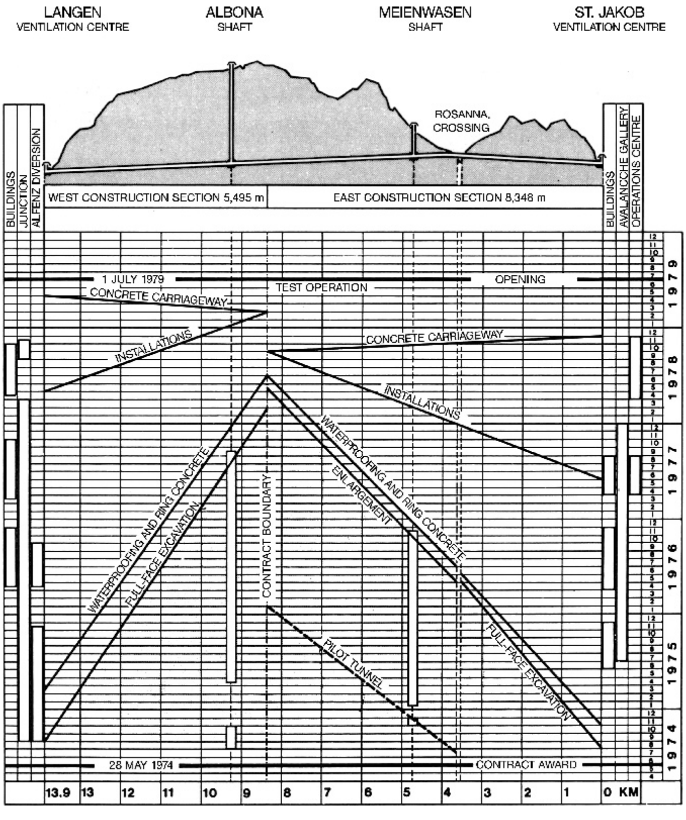


Figure 7-12 Construction schedule. Award of the Arlberg Tunnel [10].

Construction. Collapsed areas and shear fractures already had to be overcome shortly after the punctual start of the tunnelling works. In order to counter a threatened extension of construction time, as extrapolation of the encountered conditions would have led to a delay of 18 months, the following acceleration measures were agreed:

- Abandonment of the driving of the pilot heading in the east main tunnel (chainage 1,860 m), to be replaced by the driving of a pilot heading eastwards from the Rosanna gorge.

– Enlargement to the full profile between the Rosanna gorge and the Maienwasen cavern from two directions, which resulted in a shortened construction time by 3½ months.
– Contracts with the joint ventures to force the tunnel drive.

This enabled, in contrast to the deadlines originally demanded from the contractors, the construction time to be shortened by almost two years compared to the new situation (Fig. 7-13).

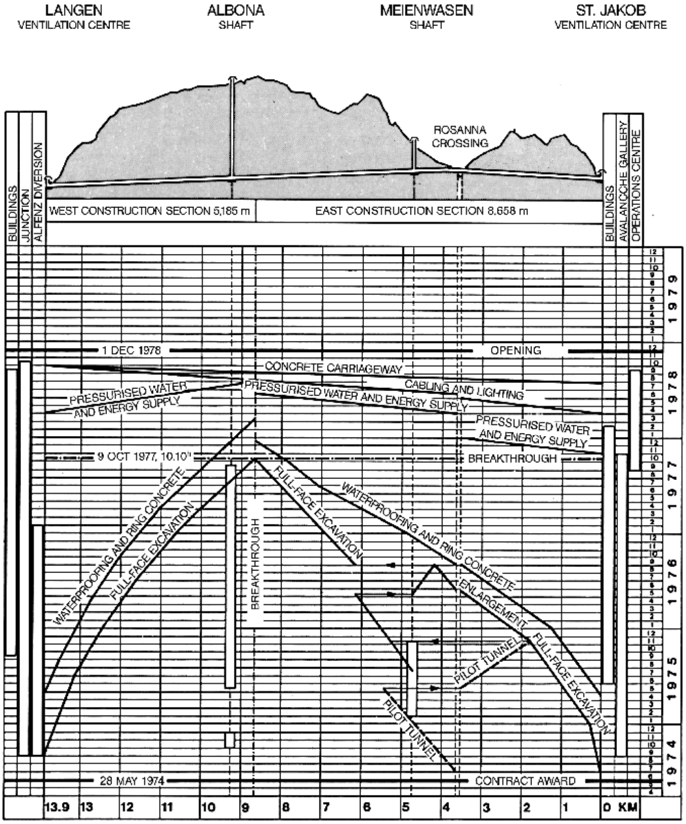


Figure 7-13 Schedule for the construction of the Arlberg Tunnel [10].

7.6.4 Scheduling of inner-city tunnelling through the example of the Stadtbahn Dortmund

General. The development of the Stadtbahn urban rail network was started in 1968 in the Rhein-Ruhr district as a supplement to the S-Bahn. This was intended to provide an alternative to private transport. Several examples of the methods used to construct and operate this rail network in Bochum, Dortmund, Duisburg, Essen, Gelsenkirchen and Mülheim have already been described in Volume I. The scheduling of a construction contract of the Stadtbahn Dortmund is now discussed in more detail.

Site description. Contract 22 (Fig. 7-14) of the Stadtbahn Dortmund includes the connection of the operations tracks to two main tracks, one station with tracks for turning and defective cars and the running tracks.

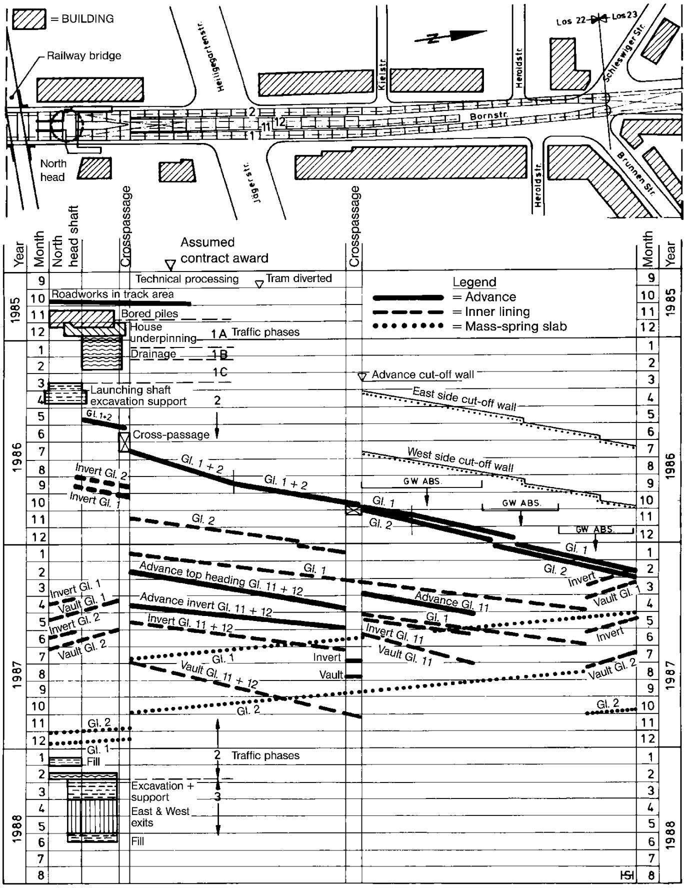


Figure 7-14 Construction schedule at the start of construction for the construction proposal. Stadtbahn Dortmund, contract 22.

The excavation and support of the sections used the shotcrete process with two-pass lining. The prevailing groundwater conditions were particularly critical. While water only had to be expected in the bedding and joints in the southern part, the northern part was below a continuous groundwater table. In contrast to the tender, which intended tunnelling below this groundwater table under the protection of ground freezing, the tunnel was driven inside cut-off walls produced by the soilcrete process. The remaining water inside this "box" was pumped out of vacuum deep wells. Another critical point of this Stadtbahn contract was passing below a railway bridge. The bridge had to be supported with additional measures before excavation of the tunnel.

Construction programme. One particular problem, which has to be faced in inner-city tunnelling, is the requirement in most cases to maintain the traffic flow of vehicles, trams and pedestrians. This considerably restricts the scope regarding the construction programme. The measures required according to the tender documents make clear what a sustainable influence this had on the scheduling and construction of the contract:

Phase 1a. This included the underpinning of the DB railway bridge and the installation of the eastern temporary bridges for the north and south heads. For this purpose, a two-track diversion was laid in what is now the track area.

Phase lb. After completion of the eastern temporary bridges, the north-south traffic was diverted to the track area. This was followed by the temporary diversion of the sewer and the installation of temporary bridges on the western side of the north and south station heads.

Phase 2. This was the main construction phase. While the traffic ran over the temporary bridges installed in Phases 1a and 1b, the construction site was set up between the tracks and the launching shafts were sunk at the south and north station heads. The entire tunnel was mined from these shafts. Meanwhile, the central bay of the DB bridge in the construction area was installed, to be removed after completion. The construction area was to be continuously accessible on foot from the site facilities areas through the southeast entrance, which was provided with a temporary bridge.

Phases 3a und 3b. After the main part of the structure had been completed, individual traffic was diverted back in lanes over the cleared central reservation. Finally, the temporary bridges could be dismantled, the starting blocks finally completed and the surface demolition started.

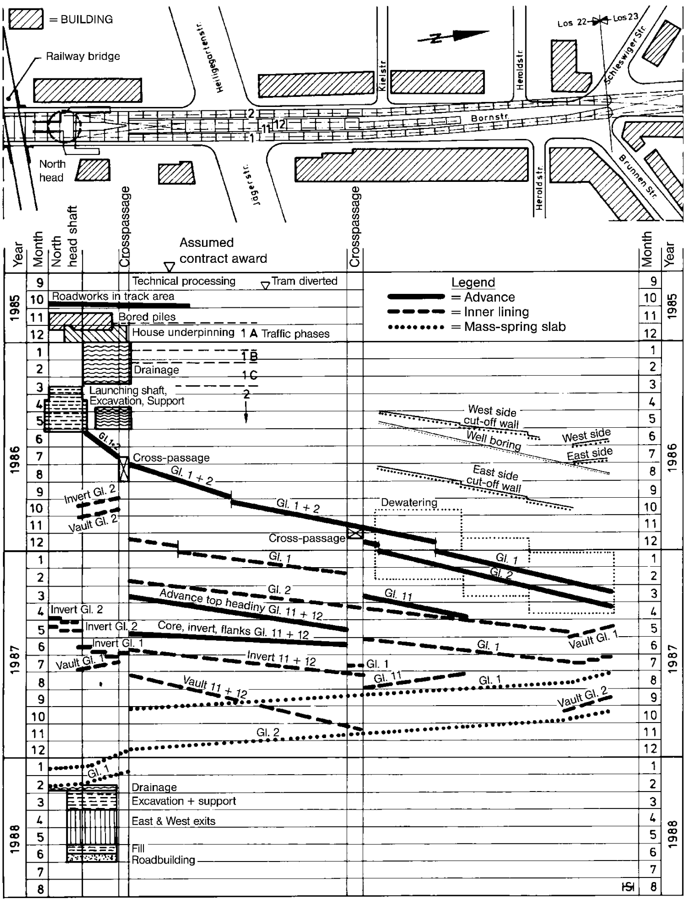


Figure 7-15 Construction schedule after one year of work.

The scheduling was undertaken using a time-distance diagram. Fig. 7-14 shows in simplified form the projected schedule after the contract award. It can be seen that the already mentioned traffic phases are integrated into this schedule. The continuous updating of this schedule to record the actual progress and the conditions encountered, which sometimes led faster progress than expected but sometimes slower, resulted in routine checking. On this contract, the state of the works after about one year was as shown in Fig. 7-15. The good agreement between the specified and the achieved advance rates can be recognised, except sewer works delayed the construction of the northern launching shaft by about one month. Nonetheless, the illustrated state of progress does not endanger the completion date of the complete structure.

7.6.5 Scheduling of a shield tunnel through the example of Stadtbahn Essen

General. Shield tunnelling has to be considered as a special case with regard to the scheduling of a site. Whichever working principle is used for the operation of the shield machine, the tunnelling method is much less flexible than shotcrete tunnelling. Any faults in the rock mass or breakdown of the machinery can lead to severe and sustained interruption of progress. Precisely the further development of automation has made shield tunnelling machines into highly sensitive devices. In addition, each new application has innovative character, so it is not always correct to rely on experience with the relevant type of machine. This all leads to a typical progress of tunnelling, which tends to be characterised by initial difficulties followed by rapid progress of the tunnelling works.

Firstly the delivery and assembly times for the shield and the excavation and mucking equipment have to be considered. In addition, it is essential in every schedule to assume a learning phase with slower advance rate for the first 30 to 50 m of any shield tunnel drive. Rapid advance rates are then required for the remainder of the tunnel distance to make up for initial interruptions.

At the moment, shield machines are used in inner city tunnelling to drive underground railways under particularly difficult geological conditions.

Stadtbahn Essen. In Essen, a shield drive was planned in 1987 as part of the northern route. The shield used for this tunnel has an external diameter of 7.12 m and works on the earth-supported face principle. The section to be driven is 2.3 km long with three stations. The tunnels are lined in two passes, the outer support is extruded and the inner lining is cast reinforced concrete using a mobile formwork unit. The driving of the station areas at track level is intended from a shaft using the shotcrete method. The schedule proposed by the contractors for the shield-driven section is shown in Fig. 7-16. What can clearly be seen are the assumed rapid advance rate during the shield drive, while the station tunnels are assumed to progress much more slowly.

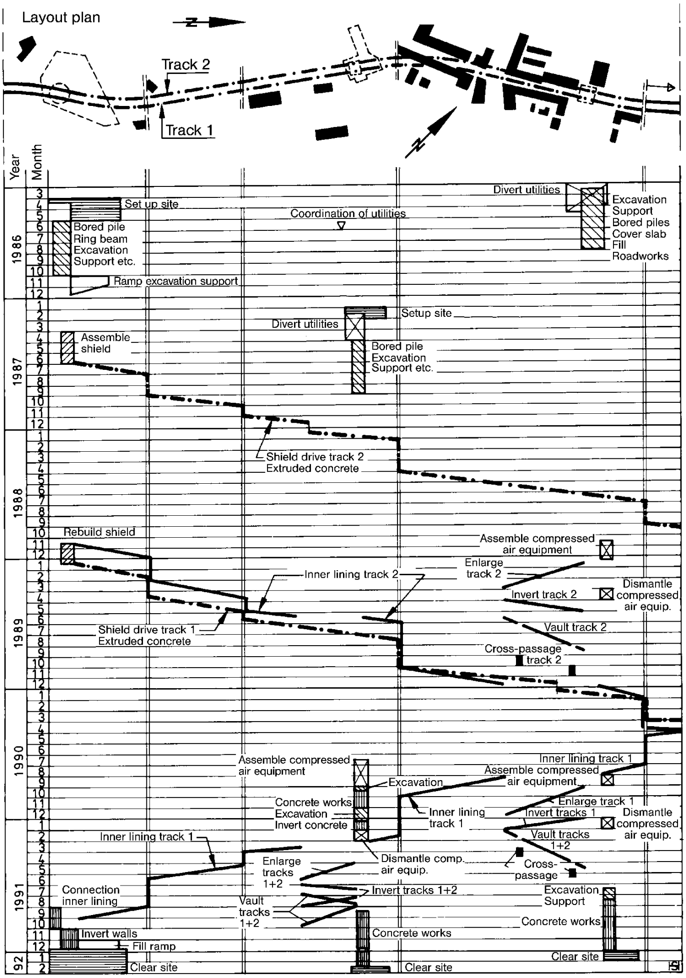


Figure 7-16 Construction schedule (bid) for the shield drive. Contract 32/33, Stadtbahn Essen.

8 Safety and safety planning

8.1 General

The term safety is defined as the state of being without threat, which arises objectively in the presence of protection measures or the absence of danger and is subjectively experienced by individuals or social groups as confidence in the reliability of safety and protection measures. Safety considerations thus presume the recognition of dangers and the resulting risk.

Safety cannot be related to the structure alone. Integrated safety (Fig. 8-1) includes three essential components, which have to be included in considerations.

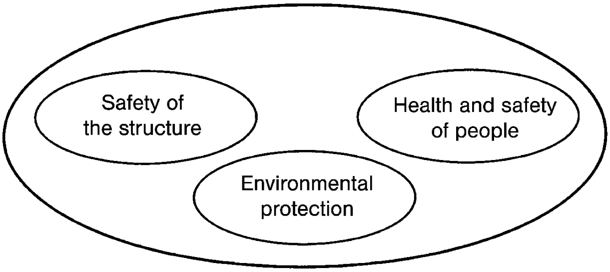


Figure 8-1 Integrated safety.

In general, people on a construction site in a constitutional state are responsible for safety as a part of their duties. The safety of the structure itself is the classic responsibility of the engineer. The inclusion of concerns of occupational safety in the design process is also justifiably required, whether in the selection of a profile or decisions about construction methods or working procedures. Only this enables the contractor to implement the proper protection measures for accident avoidance and health provision [217].

8.2 International guidelines and national regulations

The European Union deals with the areas of "safety at work" and "health protection" in diverse directives, which also provide basic rules for tunnelling.

The member states of the EU are obliged implement all directives into national law.

Directives of the council of the European community can be based on either

- Article 100a or
- Article 118a of the European Union Treaty.

Of the directives based on Article 100a, which are intended to remove technical restrictions to trade, directive 93/15/EEC of 5 April 1993 on the harmonisation of the provisions relating to the placing on the market and supervision of explosives for civil uses is applicable to the marketing and control of explosives for civil purposes [190]. The member states of the EU are – as with all directives according to Article 100a – forbidden the introduction of more stringent regulations than the directive, because this would be equivalent to a restriction of trade.

The directives based on Article 118a of the European Union Treaty are concerned with employee protection. These are minimum regulations, which may not be undercut by any member state. More stringent regulations are, however, possible and existing regulations that are more favourable for employees are not affected.

The following are essentially applicable for safety at work in tunnelling:

- Framework directive 89/391 on the introduction of measures to encourage improvements in the safety and health of workers at work [191]; all the general basics of employee protection are laid down in this framework directive. And
- the eighth individual directive 92/57 – minimum regulations applicable to temporary or mobile construction sites (construction site directive) [189].

There is not yet any specific directive for the improvement of the health and safety of employees engaged in the manufacture, storage and use of explosives, but one is being prepared.

One thing is common to all the directives mentioned: they have indeed been implemented into national law by some member states, but by no means by all.

8.2.1 Directive 89/391/EEC

– on the introduction of measures to encourage improvements in the safety and health of workers at work –

The framework safety directive is not satisfied with passive compliance with regulations, but demands active handling to improves existing working conditions, not only by employers but also by employees.

Employer's duties. The directive particularly demands that the following basic rules of danger prevention are implemented:

The avoidance of risks, the estimation of unavoidable risks, danger prevention at source, the consideration of the state of technology and the avoidance or reduction of dangerous moments.

Danger determination. Applied to tunnelling, the individual equipment, working activities and working methods are to be investigated for possible danger to health and safety – taking into account the state of training and instruction of the affected employee – by every company engaged in the construction of a tunnel according to the directive. Then the dangers are to be evaluated and protection measures against them decided. The evaluation and the protection measures decided on are to be documented.

In principle, this determination of dangers and the associated decision about protection measures does naturally not represent anything new, as it is no less than that part of construction planning, which is seen from the point of view of the employee.

In practice, this means that the contractors already determine the dangers to health and safety resulting from the individual working steps in the project preparation phase as part of work preparation and quality management, decide the necessary protection measures and document the process as an aid for the responsible managers on site and as a basis for the instruction of the miners who will work in the tunnel.

Further tasks for contractors are to coordinate their works and also their protection measures with the other contractors working on the site, make sure that dangers to the health and safety of their employees are avoided and follow the proposals of the coordinators appointed for the project.

8.2.2 Directive 92/57/EEC

– (construction site directive) minimum regulations for health and safety applicable to temporary or mobile construction sites –

The construction directive, which is applicable to the performance of construction works of any type, is based on an analysis of previous experience, that more than two thirds of all construction accidents – and parallel to this, also of all quality defects – do not have their cause in the construction phase of a structure, but could have been avoided through appropriate measures during the design phase and in the management.

This directive was passed by the Council of the European Union on 24 June 1992 and became obligatory for all member states. After the expiry of a transitional period, the directive had to be implemented into national law. The successful implementation is checked by the EU Commission.

Since it had been shown that the consideration of safety equipment and occupational safety measures in the design phase were often

- not appropriately thought through or their function not fully planned,
- tendered without the necessary technical expertise,
- not appropriately regulated in the contract and
- not available at the correct point in time and in the necessary quantity, or with the specified quality for the construction works.

This directive introduces a completely new approach to reducing the high accident rates in the construction industry through the introduction of

- health and safety coordinators,
- the prior notice,
- the health and safety plan and
- the file.

The intention is to mitigate danger points, which arise due to the simultaneous or parallel working of different trades, and which arise from the inherently dangerous nature of the relevant works, through careful planning and communally usable safety facilities.

Coordination is of course not just limited to the design phase. The health and safety plan is to be updated with the progress of the works in order to reflect circumstances, which could not have been foreseen during the design phase.

According to the principle that the responsible party is liable for the consequences, it is primarily the client, the appointed "project supervisor" and thus indirectly also the appointed consultant who are involved in the responsibility for the safety of the workers on the site [177].

The client or the appointed project manager appoints health and safety coordinators for the construction preparation phase (project coordinators) and for the construction phase (site coordinators) [178]. The client or the project manager also has to ensure that a health and safety plan is produced (normally by the project coordinator) before the start of works. The health and safety plan includes details of the essential protection measures that are applicable to the tunnel project.

The client or the project manager, and thus also the appointed consultant, also take into consideration all the principles for the avoidance of danger to health and safety during the course of the technical and organisational planning and in the estimation of the forecast duration of the works.

This means that the necessary protection measures, both for workers engaged in construction and for workers later engaged in maintenance work, are already considered during the planning and design of the tunnel project.

The Project Coordinators. The task of the project coordinators is to coordinate the application of the measures described above during the planning phase.

One other task is to produce a health and safety plan (or have it produced), in which the characteristic provisions for the site are included as well as specific measures for particularly dangerous works, above all dealing with explosives but also support measures for the tunnel. This health and safety plan is undoubtedly to be understood as a continuation of and supplement to the construction schedule and construction logistics, but with special consideration of the dangers from spatially and temporally overlapping works by the various contractors. Further items in the health and safety plan will be the layout of transport routes, access for rescue and fire service vehicles and a catastrophe emergency response plan (see also Section 8.3).

This health and safety plan should in every case be made known to all contracting companies and should ideally be part of each tender.

The project coordinators also produce a file concerning health and safety protection, which can be referred to for the implementation of later works such as maintenance or repair and also tunnel enlargement.

The Site Coordinators. One of the tasks of the site coordinator is to coordinate the general principles of health and safety with the contracting companies, the measures intended by each company as a result of their determination of dangers, and thus to have a coordinating role in the production of the detailed schedule.

Further tasks are the practical implementation of the health and safety plan and the file for later works – including any updating that may be necessary – and the organisation of collaboration and coordination of the activities between the contracting companies.

The site coordinators also supervise the works of the contractors regarding health and safety and give the employers (or their managers employed on site) the appropriate instructions.

Of course this activity of the site coordinators should not and must not be seen and performed separately from the actual construction works, particularly in tunnelling. Parallel to these safety tasks, it also seems appropriate for the site coordinators on a tunnelling project to look after quality assurance and environmental protection. This has been implemented in the German construction site regulations (BaustellV).

8.2.3 Directive 93/15/EEC

– on the harmonization of the provisions relating to the placing on the market and supervision of explosives for civil uses –

In every state of the European Union, explosives for civil purposes are subject to extensive regulations, particularly that explosives may only be placed on the market after an approval has been issued, which is based on a series of tests.

In order to harmonise the different regulations in the individual states, while still maintaining a high level of protection, basic provisions are laid down in the directive that have to be complied with in the conformity testing of explosives. The regulation of these basic requirements has not yet taken place, but is described as desirable and necessary and demanded. The appropriate standards are currently being produced by CEN, the European Committee for Standardization.

The directive describes as explosive those substances and objects, which are considered to be explosives according to the "Recommendations of the United Nations for the transport of hazardous goods" and are classified in class 1 in these recommendations (so munitions also belong to the scope of application of the directive).

The basic requirement is that no state may forbid, limit or hinder the placing on the market of explosive substances, which comply with the directive.

The explosives must since the 1^{st} January 2003 be marked with a CE mark and be subjected to a conformity evaluation, and must comply with basic requirements for operational safety.

The CE mark is to be legibly and permanently applied, if this is possible, directly to the explosive and otherwise on the packaging.

8.2.4 Directive 98/37/EC

– on the approximation of the laws of the Member States relating to machinery –

Particularly in mechanised tunnelling, the demand for safe, ergonomic and humane working places can be realised better than in conventional tunnelling. The demand to base decisions on cost-effectiveness has to be balanced against the needs of occupational and health protection, the protection of the environment and quality assurance.

Construction and equipment. The EU machinery directive 98/37/EC applies to the concerns of occupational safety in mechanised tunnelling, and this has been implemented 1:1

into German law by the Federal Republic of Germany. This was implemented in the 9th regulation for device safety protection and has thus been valid since the 1 January 1993.

If the safety features of a tunnel boring machine are constructed in accordance with the relevant harmonised EN standard, then it can be presumed that the provisions of the machinery directive are fulfilled. Therefore several EN standards are being produced by working group 4 of the technical committee TC 151 in CEN.

In detail, these standards are:

EN 815: Safety of unshielded tunnel boring machines and rodless shaft boring machines for rock.

EN 12336: Tunnelling machines – Shield machines, thrust boring machines, auger boring machines, lining erection equipment – Safety requirements.

EN 12110: Tunnelling machines – Air locks – Safety requirements.

EN 815 is currently being revised with the aim of "increasing fire protection".

EN 12336 still has the status of a provisional standard, but has been presented for formal voting; the German ministry of trade and employment has applied for a revision of EN 12110.

Manufacturers of tunnel boring machines can subject their products to a voluntary safety test, which is offered by the testing institute accredited by the body responsible for accident insurance in civil engineering. This can investigate the conformity of the machine with the machinery directive and issue an independent confirmation.

Design and operation. Clients are also assigned safety-related tasks, which are laid down in the "health and safety protection on construction sites" regulations (Baustellenverordnung – BaustellV, see Chapter 8.6). In combination with the law concerning health and safety at work ArbSchG, this implements the EU directive 92/57 EEC (see also Section 8.2.2).

The health and safety protection requirements for the handling and operation of these machines are essentially determined by the EU directives 89/391/EEC (see also Section 8.2.1) and 96/63/EC, which are based on Article 137 of the EU treaty and specify minimum regulations for all employees in the EU area. In German law, these directives are implemented in the law concerning health and safety at work (ArbSchG) from 07/08/1996 and the working equipment regulations (BetrSichV) from 27/09/2002. These laws emphasise an extensive evaluation of dangers and documentation by the employer, which must be in keeping with the state of technology.

If substances that could endanger health could be liberated by the operation of a tunnel boring machine, the provisions of the hazardous substances regulations (GefStoffV) are applicable. Measures are to be provided (extraction, filtering, etc.) in order that the limit concentrations laid down in the Technical rules for hazardous substances 900 (TRGS 900) are complied with.

If tunnel boring machines are used with a pressurised excavation chamber and this has to be accessible for checking, maintenance and repair work, then the compressed air regulations are applicable up to an excess chamber pressure of 3.6 bar. Higher pressures are only permissible with an exceptional permit from the responsible authority.

The regulations of the accident insurance body for each trade also have to be observed, particularly

BGV A 1	Basics of prevention.
BGV C 22	Construction works.
BGR 160	Safety rules for underground construction works.

In addition, information leaflets, special bulletins and brochures from the various authorities (Federal Institute for Occupational Safety and Health (BauA), Federal Ministry of Economics and Technology (BMWA)) and other bodies like the civil engineering accident insurer (TBG) and the Swiss accident insurance institute (SUVA) should be mentioned; the current situation can be found on their websites.

Further safety matters. If tunnel boring machines are used on a construction project, then the client, contractor and machine manufacturer have to concern themselves in advance with the necessary measures for fire protection, minimisation of smoke propagation and the optimisation of the rescue of persons. These measures must be carried out for each project and range from the provision of a safe retreat for the employees (rescue container) to protected laying of supply lines (compressed air, extinguishing water, communications etc.) and the provision of suitable escape and rescue equipment.

The complexity of such construction projects also makes the instruction of the employees very important. Emergency services (fire service, ambulance) also have to be equipped and trained for the special features of a tunnel project.

8.2.5 Implementation into national regulations for blasting

Germany. The basic law applicable to blasting in Germany is the law concerning potentially explosive substances (SprengG) [225], which includes all regulations about responsible persons, storage, approval, labelling and packaging of explosive substances.

Further national regulations concerning the transport of explosives can be found in the regulations concerning the transport of hazardous goods by road, which are applicable in Germany in combination with the European Agreement concerning the International Carriage of Dangerous Goods by Road ADR [3].

For the safe execution of blasting works in Germany, the accident prevention regulations for blasting VBG 46 [247] are applicable, together with the instructions for implementation. These contain general provisions about blasting and additional provisions for underground blasting in Section VIII. In addition, the safety rules for underground construction works [239] are applicable to tunnelling in general.

Switzerland. In Switzerland, blasting is regulated by the law concerning potentially explosive substances (Sprengstoffgesetz) [225] and the associated explosives regulations [226]. These contain the basic rules about responsible persons, the handling of explosives, storage, transport, use and destruction of explosives. More detailed protection provisions are given by the Swiss accident insurance institute (SUVA) in their guidelines and “occupational safety bulletins”.

Austria. In Austria, the production, storage and transport of explosives is regulated in the munitions and explosives law [198] and in the associated munitions and explosives monopoly regulations [199] and in the blasting caps regulations. The approval of explosives for use in Austria is generally regulated in the explosives approval regulations for mining [224].

Addition regulations to the ADR in Austria for the transport of explosives by road are included in the law concerning the transport of hazardous goods (GGSt) [81].

The necessary protection measures for the performance of blasting work are laid down in the blasting works regulations, and special regulations for tunnelling are provided by the 13th section of the construction worker protection regulations [15].

8.3 Integrated safety plan

8.3.1 The safety plan as a management plan

Safety as a comprehensive subject is already one of the objectives of a client in the design phase. The designer should therefore include consideration of safety requirements in the general project design, at the latest in the preliminary design. This makes safety a part of quality management, together with the serviceability, cost-effectiveness and environmental acceptability of the works.

The integrated safety plan is therefore embedded in the utilisation plan and the quality management plan for the tunnel. It should give information about the following areas:

- Safety objectives,
- Danger scenarios with risk analyses,
- Plan of measures with the measures to reduce or limit damage and injury and
- Rescue plan with the alarm organisation.

8.3.2 Safety objectives

In general, safety objectives are considered to have been achieved when risks of any type have been reduced to a acceptable degree.

The safety plan should deal with safety objectives for safety at work, health protection and environmental safety. Temporary excavation support is of particular significance because on the one hand, it is described and evaluated as part of the structure in the utilisation plan, and on the other hand early effectiveness of the temporary support is of great significance for occupational safety, for example the early strength of shotcrete in the first hours.

8.3.3 Danger scenarios and risk analyses

The recognition of hazards is the most important part of a risk analysis because unrecognised hazards are the most dangerous.

In order to describe a hazard rationally, the appropriate danger scenarios are first selected, which can come from practical experience, communications capability, intuition or the capability of recognising discrepancies. One rather analytical and systematic method of proceeding is through error trees, event trees or cause and effect diagrams [208].

A risk is a hazard evaluated according to its consequences and probability of occurrence (Fig. 8-2). According to this definition, any hazards can be recorded as a risk through the product of its consequences in the event of occurrence and its probability of occurrence.

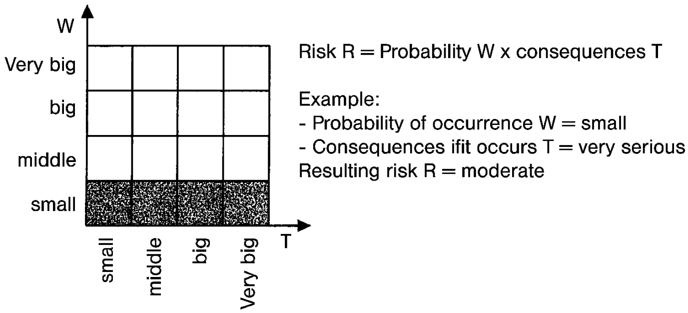


Figure 8-2 Definition of the risk *R*.

Targeted risk-minimising measures can reduce the initially evaluated risk to a bearable, mostly small residual risk. In the listing of hazards and the risk evaluation, it can be assumed that the client appoints an expert company whose staff understands the recognised rules for underground work.

In the safety plan, hazard scenarios should be included from the following fields [203]:

- Construction site facilities:
  - Traffic and transport facilities below ground.
  - Natural hazards like rockfall, flooding, avalanches.
  - Noise, as it affects the site itself and the surroundings.
- Ventilation:
  - Ventilation interruptions due to defective fans, power failure.
  - Progress of blasting fumes through working areas (possibly requiring a fume container).
  - Insufficient quantity of fresh air to thin dangerous fumes.
  - Increase of natural gas concentration due to increased ingress or interruption of ventilation.
  - Damage to air ducts in the completed part of the tunnel.
  - Concentration of hazardous substances and ventilation after breakthrough.
- Fire in the tunnel:
  - Fire affecting a single vehicle or due to the collision of two vehicles resulting in smoke build-up in the tunnel / with fire damage to the air ducts.
  - Fire due to construction works (construction chemicals) also in the completed part of the tunnel.
  - Fire after breakthrough.
- Transport facilities for material and passenger transport:
  - Persons on site being hit or run over at the face or in the completed tunnel (wheeled vehicles reversing, impaired view of a locomotive driver).
  - Collision with working scaffolds.
  - Runaway cars with rail transport.
  - Falls into shafts due to insufficiently secured transport routes or working platforms.

- Lighting:
  - Injury due to insufficient lighting for the workplace, transport routes or persons outside the workplaces.
- Site electricity supply:
  - Injury due to incorrectly installed high-voltage facilities (for example cables being driven over).
  - Fire or explosion.
  - Consequences of a power failure (drainage, ventilation, measurement and monitoring instrumentation).
- Excavation and support:
  - Injury due to cave-in or rock burst (early strength of the shotcrete in the first few hours).
  - Injuries from falling rock (protective roofs on machines).
  - Water or mud inflow.
  - High dust concentration from shotcrete.
  - Injuries from accidents due to inexpert handling of explosives.
- Gas escape:
  - Explosion due to unacceptably high natural gas concentration.
  - Contamination due to unacceptably high radon concentration.

These fields should be completely investigated for hazard scenarios for each new project.

8.3.4 Measures plan

Safety measures of an organisational or material nature provide complete or at least partial defence against the danger, they reduce the risk to a acceptable degree. In order to cope with injuries, which occur despite this, the measures plan should also include a rescue plan with the associated alarm organisation.

The national organisations SUVA, TBG, AUVA have published aids, instructions and guidelines, which should help every consultant to produce the measures plan for hazard scenarios (for example "safe working" for tunnelling from the TBG) [235, 240].

For the rescue plan, a three-part structure is suitable, divided into structural, material and personal measures. There follows an explanation of these measures with examples.

Structural measures serve to provide communication and fire protection measures:

- Communication.
- Alarm organisation.
- Measures in the event of fire with alarm and extinguishing systems.
- Rescue plan with escape routes, internal site transport facilities, helicopter landing pad etc.

Material measures include measuring instruments, rescue material, fire extinguishing material:

- Measuring instruments, mobile and permanent, for the expected gas concentrations including oxygen.
- Rescue material; breathing apparatus, resuscitation equipment, first aid material.
- Fire extinguishing materials like hoses, hand fire extinguishers etc.

Personal measures provide training for first aiders, rescue services:

– General first aid training.
– Training in the use of emergency breathing apparatus.
– Training of rescue personnel to use breathing apparatus and gas detectors.

8.4 Transport, storage and handling of explosives

8.4.1 Transport to the site

The transport of explosives is a problem affecting the safety of the general public and is regulated by a series of international regulations for various forms of transport, and there are also national regulations.

For road transport, the "European Agreement concerning the International Carriage of Dangerous Goods by Road" (ADR) [3] applies, for rail transport, the "Regulations concerning the International Carriage of Dangerous Goods by Rail" (RID) [192] apply. There are similar applicable regulations for shipping and air transport.

All these international regulations include

– a classification of hazardous goods,
– a system for labelling hazardous goods,
– regulations concerning packaging,
– regulations concerning the transport documents,
– regulations concerning the means of transport,
– regulations concerning the necessary training for hazardous goods transport and
– regulations concerning load combinations.

As a result of the extensive requirements applicable to the transport of explosives to construction sites, the delivery of explosives to tunnel sites is not normally undertaken by the construction or blasting contractor but by the company that markets the explosives. For the stated reasons, no further details are given of the transport of explosives to the construction site.

The tunnelling contractor thus only has to take care of the safekeeping and storage of explosives on site and the transport from the site magazine to the location where they will be used at the tunnel face.

The internationally regulated packaging regulations for explosives and accessories are, however, of interest to the end-user, since the labelling that has already been applied to the packaging makes clear the content and the danger of explosion.

Packaging regulations. In all international regulations, explosives belong to Class 1. The next section repeats the essential provisions, as can be found listed in the Appendix A of the ADR [3].

Hazard divisions. Explosives are divided into sub-classes according to their dangerousness, starting with Division 1.1. (Substances and articles which have a mass explosion hazard, meaning that the entire charge explodes almost simultaneously) down to Division 1.6. (Extremely insensitive articles).

This classification of substances and articles is undertaken through practical tests according to the UN testing handbook.

Compatibility. Explosives are also classified into compatibility groups. Compatibility group B normally denotes primary explosive detonators, group D black powder, brisant explosives and detonating cord, and group S includes non-explosive electrical detonators and fuse igniters.

Substances in compatibility group D must never be loaded together with substances in compatibility group D.

Classification. The combination of sub-class and compatibility group is described as the classification code.

Some examples of classification:

All brisant explosives have the classification code 1.1.D, Emulsion blasting agents and ANFO explosives can also be included in 1.5.D.

Detonating cords are classified in 1.1.D or also 1.4.D, blasting caps, detonators and delay connectors are classified in 1.1.B or, if they are not mass-explosive, 1.4.B.

Labelling. The transport package must be labelled with the UN number (from the UN recommendations), the name of the substance (emphasised in italics) according to the ADR, the trade name of the explosives and the hazard label, with the classification code being found on the hazard label.

The hazard label is an orange diamond shape with a black border, which shows:

– For sub-classes 1.1., 1.2., 1.3., the upper half shows a bomb, the lower half the classification code and in the bottom corner is a small number 1.
– For the other sub-classes, the upper half shows the sub-class, the lower half the compatibility group and the bottom corner shows a small number 1.

8.4.2 Storage on the site

Since considerable quantities of explosives every day in drill and blast tunnelling, setting up of a magazine to store the explosives on site is normally essential.

Magazines can be above ground or below ground.

Health risks. The health risks from the storage of explosives are normally confined to gelatin explosives.

In case of fire, it should be borne in mind that explosives burn with the formation of particularly poisonous fumes.

Accident risks. Due to the good handling safety of the explosives used in tunnelling, unintended detonation due to external effects is indeed unlikely but can naturally not be ruled out absolutely.

Explosive detonators are much more dangerous, as these contain highly brisant primary explosives, which can be detonated by a flame or an impact.

In case of fire, it must be borne in mind that the stored explosives will indeed first burn, but that the burning could in a catastrophe result in a detonation.

Protection measures. Appropriate artificial and/or natural ventilation of a magazine can lower the concentration of nitroglycerine vapour to the point where there is no health risk to the employees.

The primary objective must be to ensure that no dangerous mechanical or thermal actions can affect the explosives in the site magazine. The secondary objective should be to reduce the consequences for persons and the effects on the surroundings in the case of a catastrophe with unintended detonation.

The first principle should therefore be to place the site magazine in such a location that mechanical actions like rockfall, rock pieces thrown by blasting etc., do not pose any danger and the second principle should be that in case of a catastrophe involving unintended detonation, the effects on the surroundings of air blast, vibration, thrown debris and poisonous fumes should be kept to an acceptable minimum.

In every case, the explosives magazine must be placed at an appropriate distance from transport routes, working and accommodation areas and other facilities. The door to the magazine must be lockable and should point in the direction that would lead to the least danger and the least destruction in case of a detonation.

In addition, magazines are built to provide sufficient resistance to unauthorised entry and to resist mechanical (thrown rocks, rockfall) and thermal actions (fire). This can be achieved by positioning the magazine in rock or stable ground with the use of appropriately resistant construction materials. A suitable lightning conductor system should be provided and the electric lighting installation should be explosion-safe.

From the organisational point of view, there are no fireplaces or ovens etc. in the magazine and no other materials are stored together with the explosives. In order to avoid any risk of mechanical actions, handling of explosives (opening the cases, packages and packaging, issue of explosives etc.) is undertaken outside the actual magazine. Fire, open flame and smoking are forbidden in and around the magazine.

Brisant explosives and detonating cords are kept separate from primary explosive detonators and from each other, so that – in case of a catastrophe – detonation of detonators cannot detonate the stored explosives or detonating cords.

In addition, explosives are not stored in their entire quantity, but rather separated into smaller quantities and stored so far apart from each other that – in case of a catastrophe – detonations cannot carry over.

Explosives magazine above ground. In order to store the large quantities of explosives required, magazines are placed in natural rock or stable ground, or covered with appropriately thick earth fill. An earth barricade is constructed in front of the entry, and detonators are kept under lock and key in their own chamber (Fig. 8-3).

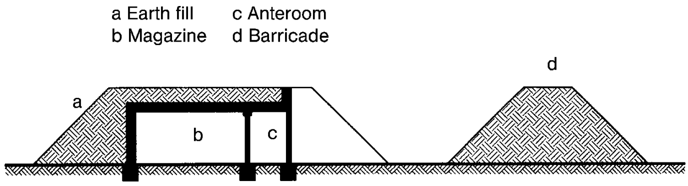


Figure 8-3 Explosives magazine on the surface.

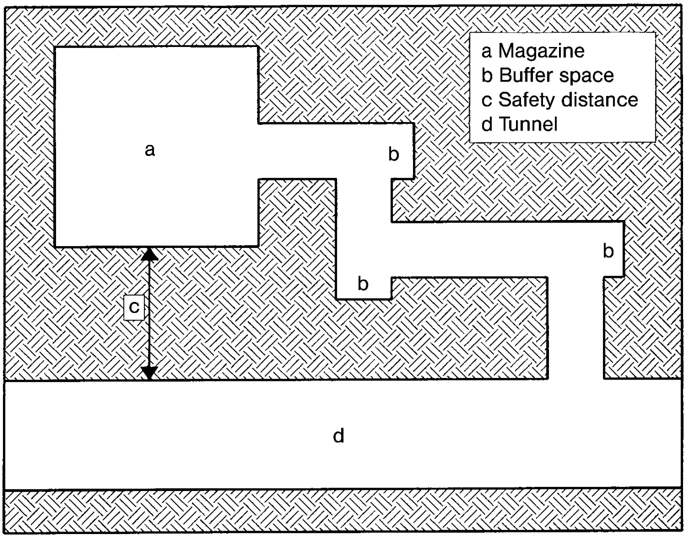


Figure 8-4 Explosives magazine below ground.

Explosives magazine below ground. Access to the magazine is provided with many dog-legs with buffer spaces at the corners in order to detonate the consequences of any detonation. There must be an adequate safety distance from the magazine to the tunnel or to other protected spaces (Fig. 8-4).

Intermediate store. In addition to the actual magazine on site, it can be appropriate for very long tunnels not to deliver the necessary explosives from the site magazine immediately before use but to transport the quantity required for a shift before the shift from the magazine to a store near the face for intermediate storage.

Such intermediate stores are laid out to hold one day's requirements. The explosives can be stored in rooms (containers, magazine) or smaller quantities in containers.

The same basic principles also apply for intermediate stores: they must be placed at sufficient distance from the face to be safe from external mechanical or thermal actions, the store must be kept locked, explosives and detonators are kept as far as possible from each other and no other objects or materials are stored together with explosives.

8.4.3 Transport on site

Transport from the explosives magazine to the place of use at the tunnel face is undertaken by carrying or in a vehicle.

Health risks in the transport of explosives concern gelatin explosives; nitroglycerine could be ingested through the skin or inhaled through the windpipe.

Accident risks in the transport of explosives are the result of the unintended detonation of explosives, above all of primary explosives, which detonate more easily.

Danger is acute when electrical detonators are transported loose in vehicles without their delivery packaging and without alternative packaging. In this case, contact of the ignition wires with the chassis could lead to ignition.

Protection measures. All vehicles used for the transport of explosives are appropriately labelled so that it is clear to all parties that explosives are being transported.

When explosives with nitroglycerine content are used, contact with the bare hands is to be avoided, so protective gloves have to be used. The risk of inhaling nitroglycerine vapour can be considered as low since these explosives are transported in the original packaging or in suitable containers.

In order to minimise accident risks, the safety principle also applies in this case that explosives and detonating cords are transported separately from detonators. Explosives and detonating cords on the one hand and detonators on the other hand are always transported in separate vehicles or by different people. There can only be an exception to this basic rule for small quantities, which is provided in all national regulations. Small quantities of explosive and detonators can be transported in a suitable container of non-sparking material, but in separate compartments of the container.

All explosives should generally to be transported in unopened original packaging. If this is not possible, the explosives should be transported in suitable containers of non-sparking material.

When transported in vehicles, explosives are stowed so that no dangerous movement of the load is possible, with the load being secured against sliding, shock and impact and also against falling out.

Smoking is of course always forbidden for safety reasons when explosives are transported.

8.4.4 Handling

Health dangers

Nitroglycerine. When gelatin explosives are handled, there is a risk that the explosive oil used, which could by nitroglycol or a mixture of nitroglycol and nitroglycerine, could enter the body, either by ingestion through the skin or inhalation of the vapours.

Nitroglycol and also nitroglycerine can penetrate into the body through the skin or by inhalation through the windpipe and can widen blood vessels (vasodilatory) and attack the blood vessels of the heart. This can reduce blood pressure and result in headaches, dizziness, feeling sick or even loss of consciousness.

Fumes. In tunnelling, miners are exposed to numerous substances, which could impair their health. These include – apart from possible ingestion of nitroglycerine – high fine dust concentration (primarily due to spraying of shotcrete using the wet or particularly the dry process), enclosed emissions from diesel motors (nitrous gases and particles) and also dangerous fumes produced by the detonation of explosives.

Accident risks

Drilling into the remains of holes from the previous round. The most serious accident risk is that when the blasting holes for a round are drilled, the drill enters the remains of a hole from the previous round, which still contains explosive residues. The energy imparted by a hammer drill could then detonate these residues. Tests have shown that gelatin explosives will with great probability detonate when drilled into, while emulsion blasting agents will probably not react.

Explosive residues in the muck pile. There is also a risk that explosives are not detonated but are ejected from the hole and end up in the muck pile. This risk can be regarded as low in the case of cut and stoping blasts, even if some holes are not loaded with charges. The risk becomes real in the case of contour holes, in which the small-calibre cartridges cannot be tamped unless the holes are provided with stemming.

Particularly dangerous is the extremely rare case that a detonator does not detonate despite having received the correct impulse (defect in the delay connector) and the priming cartridge is thrown out into the muck. In this case, mechanical damage to the priming cartridge when the muck is loaded can result in a detonation.

Loading charges before the completion of drilling. Above all when large-diameter tunnels are excavated and when blasters are waiting and not engaged in drilling, the temptation can be great to save time by starting to load the charges before the completion of drilling work. In this case, there is a danger that a drill enters a hole that has already been charged and the charge detonates.

Firedamp. Another specific danger in tunnelling is the occurrence of firedamp, which is flammable air with methane content. Since methane mixed at concentrations of 5 to 15 % in air forms an explosive mixture, there is a danger that a spark could ignite the explosive air or that the firedamp could detonate together with the explosives and produce an unintended reinforcement of the blasting effect.

Stray currents. Particularly when explosives are transported by rail with an electric locomotive, there is a danger of stray currents, which in some circumstances could have enough energy to cause an unintended early detonation of an electric detonator.

Lightning. A thunder strike could also lead to unintended early detonation of an electric detonator, unless the tunnel is already sufficiently deep underground.

Working from both ends. When the two tunnel drives are sufficiently close, a blast on one side could lead to a danger situation on the other (falling rock pieces etc.).

Protection measures. In order to expose the miners to the least possible extent to the health dangers from nitroglycerine, carbon monoxide and nitrous gases – according to the basic principle of the EU guideline to combat dangers at their source and reduce dangerous moments – explosives should be used, which lead to the lowest possible concentrations of harmful agents. This is a very clear argument for the use of emulsion blasting agents.

Nitroglycerine. When emulsion blasting agents are used, which do not contain any explosive oils such as nitroglycerine, there can be no ingestion of nitroglycol or nitroglycerine through the skin or inhalation. Headaches are often suffered by miners when loading gelatin explosives, but cannot occur when emulsion blasting agents are used. If nonetheless gelatin explosives are used, any contact of the bare hands with the explosive should be avoided (protective gloves should be worn).

Fumes. If emulsion blasting agents are used, the explosion only produces a fraction of the hazardous content in the fumes, specifically about half the carbon monoxide and a third of the nitrous gases compared to gelatin explosives.

In order to further minimise the formation of hazardous substances in the fumes, cares should be taken that the explosion is initiated by the appropriate detonators (danger of excessively weak detonation). Suitable, fresh, unimpaired explosives with the least possible formation of hazardous materials should be used and care should also be taken that there is adequate excess oxygen. If explosives in powder form are used, care has to be taken that these do not become damp or become compacted during loading.

Another reduction of harmful substances can be achieved through meticulous filling of the shots, particularly with the use of water-stemming ampoules, and adequate spraying of the muck pile, where the nitrous gases collect.

Adequate ventilation must always be provided to ensure adequate thinning of the hazardous gases and dust (see also Volume I Section 8.2).

Drilling into the remains of holes from the previous round. The risk of an accident caused by the detonation of explosive residues remaining in a hole from the previous round can primarily be countered by always loading the priming cartridge at the end of the hole. If this is done, it can be practically ruled out that subsequent drilling encounters explosive residues.

Explosive residues in the muck pile. The provision of stemming, particularly with stemming screens for contour holes, can practically rule out the ejection of cartridges. The muck pile should nonetheless be inspected for any explosive residues before loading and carting away.

Loading charges before the completion of drilling. Unintended drilling into an already charged hole can be ruled out by always maintaining a safety margin between the drill and the nearest charged hole, and this should be at least one hole depth.

Firedamp. In the presence of methane, unintended detonation of an explosive mixture of air and methane – it only takes a spark to ignite it – must be avoided. All tunnels, in which the geological conditions do not permit the occurrence of firedamp to be ruled out in advance, should be provided with continuously functioning monitoring instruments to detect any potentially explosive concentrations. When 50% of the lower explosion limit for methane is detected (corresponding to 2.5% concentration in the air), not only blasting work but also all works should be stopped until appropriate ventilation has reduced the reading to below this limit.

In addition, in cases where the occurrence of firedamp has to be expected, only safety blasting explosives (and also the corresponding detonators with copper casings) should be used. These do indeed have the disadvantage of less explosive effect, but cannot detonate firedamp along with the blast. As an additional measure, the priming cartridge should be loaded last and all holes should be carefully sealed by stemming.

Stray currents. The danger of early detonation due to stray current is primarily countered by using highly unsusceptible detonators.

Rails, pipes and electrical conductors running parallel to them should be electrically connected at a defined spacing, normally every 50 m, and earthed. The firing circuit and the firing cable should be separated from rails, pipes etc. by an adequate distance, normally at least 50 cm.

Lightning. When electric detonators are used near the surface, all blasting work should be stopped when a thunderstorm is approaching and already loaded charges should be disconnected.

Working from both ends. When the two tunnel drives are near enough to represent a risk, normally within 25 m, the miners in the opposing drive should be informed before blasting so they can leave their workplace and find a safe location. When the two drives are still nearer, normally within 10 m, excavation should only take place from one side.

8.5 Training of skilled workers

The EU framework directive [191] makes no reference to special training for blasting work. In general, the employer is only obliged to consider the suitability of employees with regard to health and safety before entrusting them with such tasks.

Due to the level of danger, most national regulations demand evidence of personal reliability and special training and appropriate qualification of all employees who are engaged in blasting work, especially the blasters who are responsible for carrying out the blasting safely. The duration and type of training to achieve a certificate of competence to carry out blasting work are very variable in the various national regulations.

The national regulations in Germany, Switzerland and Austria also require that if there are any grounds to presume that blasting work cannot be carried out safely, for example when a mistake has led to an accident (in Switzerland, only after a binding judgement), a certificate of competence can be withdrawn.

Germany. In order to be permitted to carry out blasting work in tunnelling, a worker in Germany must first have taken part in 16 appropriate blasts as an assistant before being permitted to participate in the actual one-week basic training course for underground blasting work and become entitled to undertake blasting. This basic training course for underground blasting work is equivalent to successful participation in a basic training course for general blasting (with the precondition of having taken part in 50 blasts as an assistant) with a subsequent special course for underground construction. In addition, the approved blaster must attend a repeater course every 5 years, at which new knowledge is imparted.

Approved blasters are responsible for the safe storage and safe transport of explosives on site as well as the successful performance of blasting works. They can also be assisted by blasting assistants, who however can only undertake less dangerous work under the requirements of the accident prevention regulations.

Switzerland. In Switzerland, a blaster is responsible for the performance of blasting works in tunnelling. The blaster gains a permit to perform blasting by passing an exam (after completing a course) to gain the so-called C-pass. The blaster is also responsible for the appropriate instruction of the relevant workers.

Austria. Similarly to the German regulations, an authorised blaster in Austria is responsible for the safe storage and safe transport of explosives on site and the safe performance of blasting works. The authorised blaster must successfully complete a 90-hour general basic training course in order to become entitled to perform any blasting works – also including blasting above ground. Before being approved to take part in such a course, applicants must first provide confirmation of their reliability from the security agency [179].

For certain simple and less dangerous work as defined in the blasting regulations, the authorised blaster can employ blasting assistants.

8.6 The construction site regulations (BaustellV)

8.6.1 General

The German Bundestag and Bundesrat pass national laws and thus implement European law into national law. The construction site regulations (BaustellV) were passed on 10 June 1998 and serve in combination with the German health and safety law (ArbSchG) to implement the already mentioned EU directive into German law.

The duties of the employer and employees according to the ArbSchG and other regulations remain unaffected by these regulations. In addition, the BaustellV has introduced no new technical regulations for the health and safety protection of workers. Only the duty of documentation has been clearly regulated to create safe conditions without impairment of health on each site.

As initiators of a construction site, clients bear the responsibility. They therefore have a duty to introduce and implement the occupational safety measures laid down in the BaustellV, both during the design phase and through the coordination of construction works.

The BaustellV supplements German health and safety law with the following duties for clients:

- Consideration of the basic principles according to §4 ArbSchG in the planning of the construction of the project.
  - The work is to be organised so that danger to health and safety is avoided as far as possible and the residual risk is kept as low as possible.
  - Dangers are to be countered at source.
  - The measures have to consider the state of technology, occupational medicine and hygiene and proven discoveries of ergonomics.
  - Measures are to be planned with the objective of linking technology, work organisation, other working conditions, social relationships and impact on the environment in an appropriate manner.
  - Individual protection measures are subsidiary to other measures and specific dangers to groups of employees who are particularly in need of protection are to be considered.
  - The employees are to be given appropriate instruction.
- Prior notice of larger construction projects to the responsible authority two weeks before setting up the site.
- Appointment of one or more health and safety coordinators, when employees of more than one employer are active on the site.
- Production of a health and safety plan, if either a prior notice is required and employees of more than one employer are active on the site or particularly dangerous works are to be performed and employees of more than one employer are active on the site.
- Particularly dangerous works as defined by the BaustellV are:
  - Works, in the course of which employees are exposed to the danger of sinking or being buried in a construction excavation or in a trench with a depth of more than 5 m or falling from a height of more than 7 m.
  - Works, in the course of which employees are exposed to substances or compounds, which are potentially explosive, highly flammable, carcinogenic (category 1 or 2), genetically altering, hazardous to reproduction, very poisonous according to the hazardous substance regulations or biological agents in risk groups 3 and 4 according to the directive 90/679/EEC of the Council from 26 November 1990 on the protection of workers from risks related to exposure to biological agents at work (ABI. EG Nr. L 374 S. 1).
  - Works with ionising radiation, which requires the provision of control and monitoring areas according to the radiation and X-ray regulations.

  – Works within 5 m of high-voltage cables.
  – Works with an immediate danger or drowning.
  – Well-sinking, underground earthworks and tunnelling.
  – Working with diving equipment.
  – Working under compressed air.
  – Works, in which explosives of detonating cords are used.
  – Erection or demolition of massive construction elements with an individual weight of more than 10 t.
- Production of a file for later works about the constructed facilities, when more than one employer is employed.

The already mentioned duties of clients according to BaustellV are summarised in Table 8-1:

Table 8-1 Overview of the measures to be undertaken according to the BaustellV depending on the conditions on site [240a].

Site conditions					Measures to be undertaken according to BaustellV				
Employer	Working days	Employed	Person-days	Dangerous works	ArbSchG §4	Prior notice	Coordinator	Health and safety plan	File
1	< 31	< 21	< 501	no	yes	no	no	no	no
1	< 31	< 21	< 501	yes	yes	no	no	no	no
1	> 30	> 20	> 500	no	yes	yes	no	no	no
1	> 30	> 20	> 500	yes	yes	yes	no	no	no
> 1	< 31	< 21	< 501	no	yes	no	yes	no	yes
> 1	< 31	< 21	< 501	yes	yes	no	yes	yes	yes
> 1	> 30	> 20	> 500	no	yes	yes	yes	yes	yes
> 1	> 30	> 20	> 500	yes	yes	yes	yes	yes	yes

The introduction of the BaustellV, in addition to the main intention of improving the health and safety protection of employees, is intended to have the following positive effects:

- Improved cost transparency, since all necessary facilities to be used communally and whose subsequent consideration would make the project more expensive are already included in the tender.
- Optimisation of the construction schedule since interruptions are avoided, the deadline risk is reduced and the quality of the work performed is improved.
- Reduction of later maintenance and repair work since the necessary advance measures for later works to the structure have already been looked after in the design phase and documented in a file.

8.6.2 The tools of the construction site regulations

The prior notice provides the basis for the production of the health and safety plan on every construction site. It is a binding statement and recognition by the client of the BaustellV and a declaration of the intention of complying with its provisions.

Clients have to send it correctly, completely and on time to the best of their knowledge in order to avoid an offence under §7 of the BaustellV.

In the BaustellV, it is assumed that variations in construction operations will lead to the content of the prior notice having to be revised. In this case it is decisive for the client to judge whether new additional or different health and safety coordination tasks arise for him on the site.

"Significant variations" according to §2 (2) BaustellV are defined as

- a change of the client or third parties appointed by him according to §4 BaustellV,
- a shortening of the duration of the construction works, if this means increased simultaneous or shift working that was not originally intended,
- a significant increase of the maximum number of employees working at the same time or the number of employers or contractors on the site,
- a splitting of the contract from only one contractor to more than one company.

The flow diagram in Fig. 8-5 illustrates when a dependency on the extent of the works or the number of simultaneously employed workers makes a new prior notice necessary.

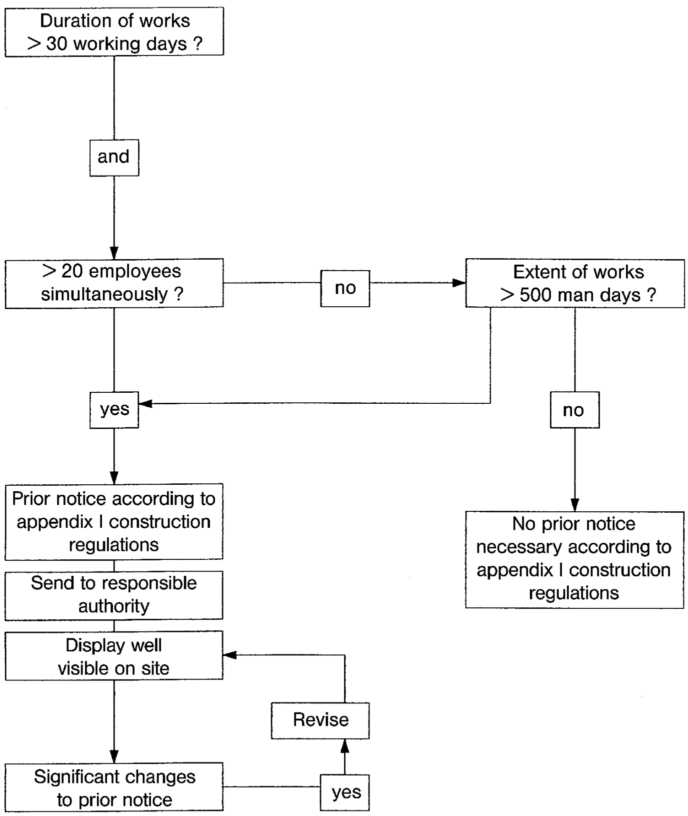


Figure 8-5 Flow chart for the determination of the necessity of a new prior notice according to the BaustellV

Fig. 8-5a shows a form for a prior notice with all details that are relevant and required by the Federal Ministry of Labour and Social Affairs.

To (responsible construction authority)

Prior notice
according to § 2 of the construction site regulations

1. Description and location of construction site: ....
Road/No.: ....
Postcode/town: ....

2. Name and address of client:
....
....
....

3. Name and address of responsible third party representing client
....
....
....

4. Type of construction project:
....

5. Coordinator(s) (if required) with address and telephone, fax or email
- for the planning of construction ....
- for the construction ....

6. Expected start of construction work
from .... to ....

7. Expected max. number of employees simultaneously on the site: ....

8. Expected number of employers:
....

9. Expected number of companies without employees: ....

10. Already selected employers and companies without employees:
1. ....
2. ....
3. ....
4. ....
5. ....
6. ....
7. ....
8. ....
9. ....
10. ....
(use a separate sheet if required for further entries)

....	....	....
(Place/date)	(Name)	(Signature)

(Client or responsible third party representing client)

Distribution: 1x responsible authority / 1x construction site poster / 1x client

Prior notice

Figure 8-5a Form for a prior notice

The health and safety coordinator (SiGeKo). Fig. 8-6 illustrates under which conditions a coordinator has to be appointed and what duties the coordinator has to perform in the design and construction phases. Finally, the responsibilities of a coordinator are examined in detail.

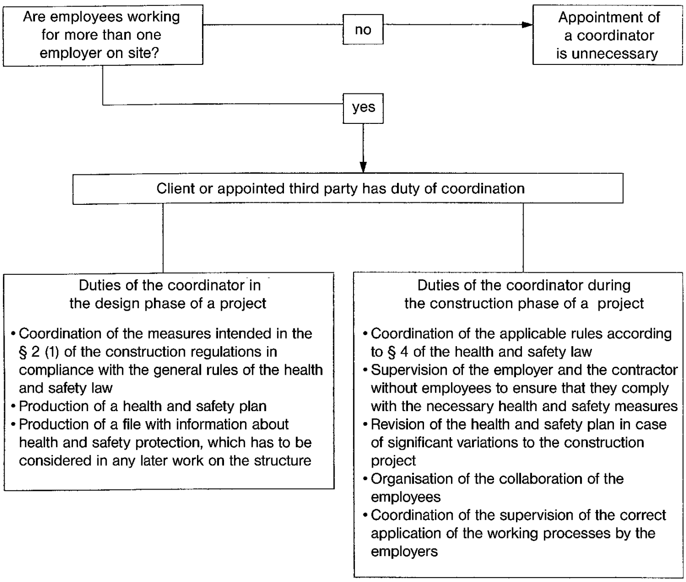


Figure 8-6 Construction site preconditions for the appointment of health and safety coordinators and their duties

When must a coordinator be appointed? The client has, according to the type and extent of construction works, to appoint one or perhaps more coordinators for planning of the construction and for the construction itself, if it can be expected that employees of more than one employer will be working on the site. The size of the project is actually insignificant.

The employment of sub-contractors, that is companies which independently undertake partial works such as electrical installation as part of the overall project, denotes the presence of more than one employer.

The coordinator must be appointed in good time so that the duties can be fulfilled during the design phase and also during the construction phase.

Another reason clients may have to appoint a coordinator to look after their duties under the BaustellV is given when the knowledge of the clients about the coordination of their site and the creation of healthy and safe conditions is not sufficient.

What are the duties of the coordinator? As experts in the subject, health and safety coordinators have the duty of supporting and advising the client, and the consultants, architects and contractors in their collaboration to integrate health and safety concerns during the various phases of a project. They also have to contribute with their expert knowledge to ensuring the safe running of the project, the construction schedule and later works to the structure at any time.

Tasks of coordinators during the planning of the construction of the project

– Integration of health and safety protection into the organisational and management concept for construction.
– Development of measures to protect against dangers due to and during the collaboration of more than one employer.

- Development of measures for the communal use of safety facilities and health protection facilities.
- Inclusion of health and safety concerns into a concept for later works on the structure.

These general tasks lead to the following individual tasks:

- Consideration of the general principles of §4 ArbSchG into the design, identification of ways of avoiding health and safety risks.
- Production (or organising the production) of the health and safety plan, discussion of health and safety measures with the client or with the appointed consultants.
- Determination of interactions between works on the site and other operational activities or influences on or near the site, which affect health and safety.
- Collaboration in the design of site facilities.
- Advice about the design of permanent safety facilities for later maintenance and repair and production of the file with the necessary information for safe and healthy implementation of later works.
- Working toward the consideration of health and safety concerns in the tender and award documents, collaboration in the checking of the bids.
- Advice about scheduling, particularly concerning the planning of time periods for construction works in order to avoid dangers, which could result from simultaneous parallel working.
- Collaboration in the production of the prior notice and its sending to the responsible authority (normally the factory inspectorate or the safety office).

In case more than one coordinator is appointed, intensive discussions are necessary, particularly when coordination is undertaken by different coordinators during the design phase and during the construction phase.

Tasks of coordinators during the construction of the project

- Organisation and coordination of collaboration between contractors regarding health and safety as the works proceed.
- Checking compliance with the health and safety measures in the collaboration between contractors.
- Updating and adaptation of the health and safety plan and if appropriate the file for later works to the structure.

These general tasks lead to the following individual tasks:

- Posting and if necessary adapting the prior notice.
- Announcement, adaptation and updating of the health and safety plan and working toward its compliance and the implementation of any necessary health and safety measures by the contractors.
- Information and possibly discussion with all contractors including sub-contractors with detailed explanation of the health and safety measures.
- Coordination of the collaboration between the contracting companies regarding health and safety and the application of the general principles according to §4 ArbSchG.
- Working toward compliance with site rules and site facility plan, if these have been produced, to avoid reciprocal hazards.

– Consideration of health and safety interactions between works on the site and other operational activities or influences on or near the site; taking care of securing the site to avoid reciprocal hazards.
– Updating and completion of the file with the necessary details for the safe and healthy implementation of later works.
– Organisation and implementation of safety meetings and walk-arounds and evaluation of the results.

Who can be appointed as a coordinator? Clients and their appointed third parties have the following possibilities: They can undertake the function of coordinator themselves, when they have the suitable capabilities, or they can appoint a coordinator (for example an external service provider or a consultant) who possesses the necessary capabilities.

According to the type and complexity of the construction measure, architects, engineers, technicians or masters can be regarded as suitable. This group of people can either be entrusted with the coordination duties according to BaustellV exclusively or in addition to their already existing tasks in design or construction.

In every case, coordinators must ensure in their personal behaviour that they devote adequate attention and influence to the duties.

Independent of the type and extent of the construction project, it may also be necessary to appoint more than one coordinator. In this case the duties and authority should be clearly delineated.

What is not permissible is the blanket transfer of the duties of coordinator by the client to one of the contractors as part of the usual tender process, since part of the tasks of the coordinator should already have been performed at the time of the award.

What qualifications does a coordinator need? Coordinators must possess knowledge and experience in construction and health and safety protection. They may have gained this knowledge and experience in the course of a professional training (as architect, engineer, technician or master) or through a training in health and safety protection (for example as a health and safety officer with experience in construction) or in the course of many years of activity in the construction industry (for example in preliminary design, design or detailed design, or in construction) and in health and safety responsibilities on site. They must also have knowledge of the specific activities, tasks and duties of a coordinator. This knowledge for a coordinator can be gained on training courses recognised by the accident insurance body.

For the specific needs of tunnelling, the requirements demanded of a health and safety coordinator in this specialist field could include:

– Required knowledge and capabilities in construction: architect or engineer.
– Required knowledge and capabilities in health and safety protection: health and safety officer or able to demonstrate extensive knowledge and experience in the application of health and safety regulations on large sites.
– Experience: extensive project-specific experience (about five years) in design and/or construction according to the coordinator duties.
– Coordinator duties: construction project-related knowledge and experience of the special tasks, activities and duties under BaustellV.

What should be considered in the appointment and assignment of coordinators? It is in the interest of clients to note the following aspects and consider the following duties in connection with the selection and appointment of coordinators:

- Careful selection and timely appointment of suitable persons.
- Transfer of duties and the associated authority.
- Creation of the preconditions for the fulfilment of the tasks.
- Making sure that coordinators undertake the duties transferred to them.

Coordinators should in every case be appointed in writing, including in particular details of the extent of their work, the necessary working time, and if necessary their authority and insurance cover.

8.6.3 The health and safety plan (health and safety plan)

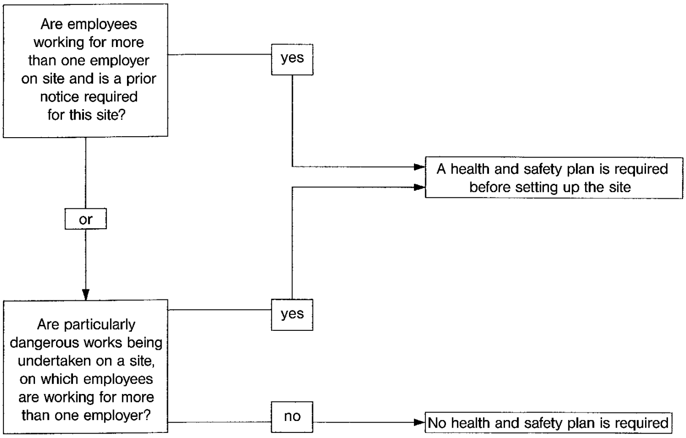


Figure 8-7 Flowchart to test whether a health and safety plan is required.

Requirements and objectives for a practical health and safety plan for a construction site.

A health and safety plan (Fig. 8-7) must fulfil the following minimum requirements according to BaustellV:

- The health and safety plan should be produced as part of the preparation of the project (§3 (2) 2.) and adapted during the construction phase to the progress of works and any variations that have been introduced (§3 (3) 3.).
- The health and safety plan should list the provisions applicable to the specific site including business activities on the lot if appropriate (§3 (3) 3.).
- The health and safety plan should also include specific measures related to any works, which fall under one or more of the categories of dangerous work according to Appendix 11 (§2 (3)).

The objectives of the health and safety plan are primarily to improve information for clients about working safety and health protection on their site and to create a qualified foundation for discussion about the contents with the responsible parties on the site. In addition, workers on the site including the self-employed (company or person) should be made aware of questions of health and safety. In addition to the requirement for the function of a coordinator as an expert person covering all areas, another objective is to improve the acceptance of the agreed measures as an aid to preventing interruptions of any kind. In order to comply with this, the construction accident insurance bodies in collaboration with other interested parties promptly looked into ways of efficiently organising the health and safety plan. The result is that practical and brief solution guidelines are now available, which are described in the next section.

Content and structure of the health and safety plan. The sample of a health and safety plan developed by the construction accident insurance bodies is in the large format usual in construction as a design and decision-making aid, which shows the user at a glance the main points and special features of health and safety protection on each construction site. The health and safety plan thus fits neatly into the series of necessary construction documents, like for example construction schedule, layout plan and site facilities plan, and ensures the desired good acceptance among the parties to a contract.

The health and safety plan is structured into four main subjects (Fig. 8-8):

- In the left-hand part, the hazards expected during the operation of the construction site are listed with the associated measures, categorised according to trade.
- The middle part is dominated by the construction schedule with the hazards, which result from the time dependencies of the various works.
- The lower part shows the results of coordination, the necessary safety facilities and their period of use.
- In the right-hand part of the plan are references to tender texts concerning health and safety and the relevant bill items as well as references to plans and instructions that have to be followed and references to the applicable provisions for the particular site.

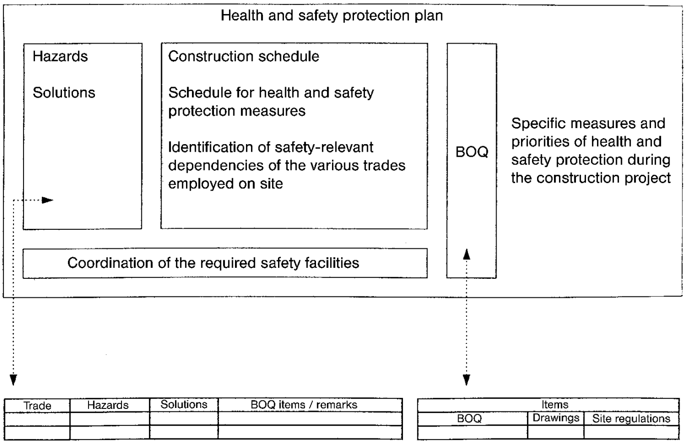


Figure 8-8 Layout and content of a health and safety plan.

8.6.4 Working steps in the production of a health and safety plan

Appraisal and trade-specific evaluation of hazards with the aid of the guideline. The coordinator first studies the available documents about site organisation, like building permit, project description, reports and drawings.

The intended trades are extracted from these documents and entered in the health and safety plan according to the structuring (see above). With the hazard catalogues to prepare the site, the health and safety coordinator has a tool available to

- determine the hazards, which could occur around the project and enter them into the health and safety plan,
- propose occupational safety measures and enter them into the health and safety plan,
- taking the figures from:
  - BM – Blaue Mappe: sample bill items for tenders
  - StL – Standard book of bill items
  - GM – Gelbe Mappe: "building blocks" from the construction insurance bodies.

The checklists for setting up the site provide assistance to the coordinator in order to

- propose the site facilities from the point of view of health and safety, and enter this into the health and safety plan and
- take the appropriate figures from the BM / StL / GM into the health and safety plan.

With the catalogue of hazards by trade, the health and safety coordinator has an aid available in order, to

- determine the hazards and the associated health and safety measures individually for each trade and enter these into the health and safety plan and
- adopt the appropriate figures from BM 1 StL GM into the health and safety plan.

Transfer of the intended construction schedule into the health and safety plan. The construction schedule is drawn in the form of a bar chart in the central part of the health and safety plan. At this stage the appraisal and the collection of data with the aid of the guideline is complete and the actual coordination can begin.

Evaluation of reciprocal hazards resulting from spatial and temporal proximity. When reciprocal hazards can be expected, which affect more than one trade and are due to temporal and spatial dependencies, the coordinator should propose an alteration of the construction schedule. If this is not possible, protection measures should be provided to counter the problem. The result is then adopted into the health and safety plan. The health and safety plan must therefore be available before the construction schedule can be produced.

Coordination of the necessary safety facilities taking into consideration the construction schedule. After the technical safety measures (scaffolding, support etc.) have been determined separately for each trade, these can now be synchronised (duration of use, scaffolding class, type of support etc.). The items of communally used safety equipment determined in this way are entered in the lower part of the health and safety plan with their duration of use. The coordination work is now complete.

Proposals for bill and specification items for the tender. Now the numbered items in the bill of quantities for the intended health and safety works can be assigned according to trade and entered into the right-hand side of the health and safety plan. In addition, the required drawings and instructions are decided with their intended time of production (design phase, construction phase). In order to create a legal basis, reference is made in the last column of the health and safety plan to selected regulations, which are taken from the hazard catalogues (Fig. 8-9).

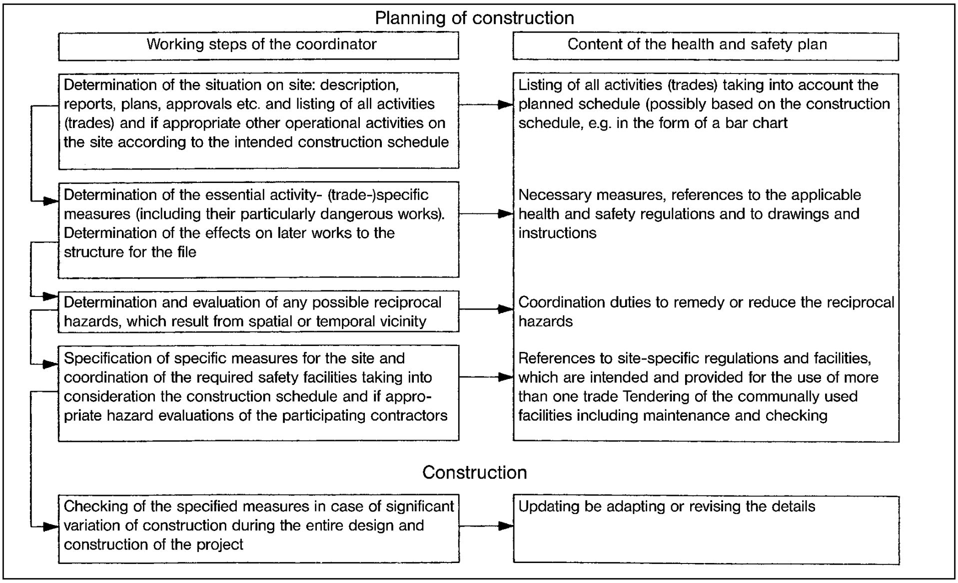


Figure 8-9 Summarised (minimum) procedure diagram for the production of a health and safety plan.

8.7 Example of a tender for health and safety protection

8.7.1 General

The tender text for health and safety is part of the additional technical conditions (ZTV) in the tender.

The coordinator undertakes the rights and duties of the client according to the German construction site regulations of 10 June 1998.

Coordinators have to ensure that their collaboration leads to safe construction and completion of this challenging construction project. They contribute to the organisational and safety-related questions related to site facilities, demolition works, support measures, danger-relevant construction processes, fire protection measures and site security.

The German construction site regulations and the agreements between the coordinator and the client result in the following services to be provided by the contractor:

The contractor undertakes to provide the appointed coordinator with all documents that are necessary in this matter or present them for approval/preparation.

For the production of health and safety plans, the planned schedule is to be provided to the coordinator separately for each construction section.

Contractors have to name their safety officers in writing before the start of construction to act as contact partners for the coordinator. The safety officer is given the task by the contractor of immediately remedying defects. This applies to their own company and to sub-contractors and suppliers.

At the start of construction, initial and instruction meetings take place, at which the concerns of the coordinator and the health and safety plans are presented and explained.

During the construction phase, coordination meetings and walk-arounds are held regularly. The nature and the extent of these are agreed with the contractor's safety officer depending on the particular project, construction progress, risk analyses etc.

The coordinator chairs and documents each meeting.

After the award of the contract, the client produces a set of site rules based on the concepts of the contractor that have been handed in and discussed.

8.7.2 Health and safety concept

In order to enable the production of an overall health and safety concept, the contractor has to produce and hand in the following documents:

– Hazard analyses and measures plans.
– Fire, escape and rescue concept.
– Health protection concept.
– Site facilities plans.
– Concept for traffic control measures inside the site area.

Contractors enclose the documents listed above, in accordance with their intended construction process and sequences, in the form of concepts with their bids.

Contractors provide detailed documents for the coordinator to check eight weeks after receiving written notification of the award, but at the latest three weeks before starting work and in a form agreed with all involved parties.

These documents are to be updated and if necessary supplemented during the progress of the construction works, and necessary alterations are to be implemented.

The fire, escape and rescue concept and the site facilities plans are to be discussed with the fire services and the appropriate authorities to enable the appropriate reaction by the fire service.

Documents that have to be discussed and agreed are handed in five weeks early for checking. They are to be provided as ten copies in colour if necessary.

The contractor delivers a monthly report about any essential foreseeable alterations to the construction process and construction sequences, which results in proactive updating of the health and safety concept.

All costs, which accrue for the contractor from the production of the concept or the updating of the required documents and from the measures to be undertaken as construction progresses, are to be included in the unit prices and are not paid separately.

The contractor has provide when required all the equipment, machines, plant, materials and tools that are necessary for the implementation of the concept for the duration of construction until acceptance without defects, and remove them after completion of construction.

The contractor also has to provide these to third parties involved in construction without charge and permit their use. The required space is to be considered in the planning of site facilities.

8.7.2.1 Hazard analyses

The contractor produces and hands over hazard analyses for all potentially dangerous construction measures and processes, updates them and produces concepts to reduce risk.

8.7.2.2 Fire protection, escape and rescue concept

Contractors undertake hazard analyses in accordance with this concept, and produce an overall concept and a plan of measures including an evacuation programme for all persons who could be affected on site. This applies to underground and overground works. Measures are also given for precautionary fire protection.

Contractors have to consider the following points in their concepts:

- An escape and rescue plan is to be produced for each construction section and displayed at suitable locations. These plans must be adapted to suit the state of construction progress.
- The escape and rescue routes are to be kept in proper condition at all times.
- Precautions for the vertical transport of patients on stretchers also have to be considered; in particular, all construction excavations with a depth of more than 7.50 m are to be provided with a passenger hoist (l_{min} = 2.10 m).
- Access for rescue vehicles must be ensured at all times. This applies both to the rescue of construction workers and to the rescue of inhabitants in the area affected by construction.
- In order to ensure access for rescue vehicles, the size of open construction excavations is to be limited to a maximum length of 12.50 m at locations where there is no adequate vehicle access at the side.
- The necessary sickness containers are to be determined and their locations are to be planned considering any nearby hospitals.
- The possibility of rescue by helicopter is to be investigated and included.
- An emergency call system for all construction states, and an alarm plan including consideration of any working times (shift operation) are to be produced.
- Rescue gathering places and their signage are to be identified and included.
- It is a requirement that all employed trades personnel are trained in first aid, independent of the “A2 Gelbe Seiten”.
- In the course of the tunnel advance, the following additional measures are to be considered:

Fixed extinguishing equipment. Fixed extinguishers are to be provided in shield tunnelling machines and their backups at potentially dangerous locations like transformers or grease injection points. An automatic activation system is to be provided. The backup is to be fitted with a water mist system toward the tunnel mouth designed to prevent smoke penetrating the tunnel and thus into the rescue and fire fighting access. Extinguishing systems are to be provided with a supply pipe from outside if water is a component of the extinguishing agent. A water tank of 10 m³ must be installed on the backup.

Fire alarm system. A warning (optical and acoustic) must be given in the shield tunnelling machine. A backup alarm button is to be installed in the construction supervision location outside the tunnel.

Extinguishing water supply. A wet pipeline is to be installed to supply extinguishing water. This is continued to the shield machine and extended with construction progress. Connection points are to be provided every 50 m (fixed connection for 50 mm fire hose, ball valve).

In the tunnel, 800 l/min is supplied at 5 bar flow pressure. The internal diameter of the pipeline is at least 80 mm. In addition, a water connection is provided for a small-diameter hose, l = 50 m, every 100 m for people to help themselves. Employees are to be trained to use extinguishing systems.

Fire extinguishers. Sufficient suitable fire extinguishers are to be placed in the shield tunnelling machine and other mobile machines (trains, cranes). Employees are to be trained to use extinguishers.

Railway. Railways must be provided with a passing place every 500 m in the tunnel. The railway must be suitable for transporting injured persons on stretchers and a mobile pressure chamber. A spare locomotive and a material wagon are to be provided for use by the fire service.

Escape. In the event of fire, it must be possible for persons working in the shield tunnelling machine to escape. In order to assist this, a sufficient number of suitable emergency escape breathing apparatuses is to be provided. The employed workers are to be trained to use the devices.

Rescue container. For a tunnel of the length of 900 to 1,000 m, a rescue container is to be provided to offer escaping persons protection against smoke, poisonous gases and vapours. The rescue container consists of two rooms to store equipment for the fire service and to accommodate up to 10 people (> 5 m^2) and store escape equipment. In addition, the container must be equipped with external ventilation with an excess pressure of 50 Pa and a telephone. There can be no thermal insulation, or only non-flammable insulation, and no windows. The container is also provided with safety lighting.

The safety container may not be positioned at the location of a railway switch.

Escape and rescue route signage. Escape routes are to be marked with signs every 50 m (reflecting and persistent – in case the tunnel is provided with emergency lighting during construction, persistence is not necessary). The distance to the exit is also to be shown.

Fire services exercise. Measures for fire service exercises are to be included.

8.7.2.3 Health protection concept

Contractors are to present concepts with measures to prevent the impairment of health.

Working time. Working times have to comply with §10 of the German employment law. Exception permits of any type are to be applied for in good time from the “ Staatliches Amt für Arbeitsschutz Köln” (STAFA). Only permitted working times can be considered in the concepts.

Dust, diesel emissions. For conventional tunnelling, a special ventilation and extraction concept is to be produced:

- Conventional shotcrete tunnelling produces dust from rock cutting and from dry shotcrete spraying. Measures are to be described to ensure compliance with the new dust thresholds according to TRGS 900.
- In order to maintain the thresholds mentioned above, it may be necessary to provide extraction ventilation. The contractor should plan filter systems in order to avoid dust nuisance in inner-city areas. This also applies to silos (bentonite, shotcrete, cement silos).
- Diesel-powered vehicles produce diesel emissions. In addition to the ventilation and extraction concept, the contractor is to demonstrate how monitoring and evidence of the pollutant emissions from the vehicles in use can ensure compliance with the permissible occupational exposure limits (TRK) according to TRGS 554 of 0.3 mg/m^3. The measures intended to prevent this threshold being reached are to be explained.
- When shotcrete is used, the production process is to be selected with regard to health protection and compliance with dust thresholds, independent of the ventilation and extraction concept. Only alkali-free shotcrete is to be used. If dry-mix spraying is intended, the mix is to be wetted to reduce dust development. Appropriate eye protection is compulsory.
- When welding work is undertaken – above all in underground works – ventilation and extraction are to be carefully planned. In particular, the BGV C 20 is to be complied with.

Compressed air. For compressed air working (shield tunnelling, pipe jacking), reference is made to the regulations for working in compressed air in the version of 19 June 1997. RAB 25 (Rules for health protection on construction sites) is also applicable.

In order to treat patients, who are injured in the pressurised area, a pressure chamber is to be provided. This applies to all areas where compressed air working takes place. A suitable lifting device is to be provided to recover this pressure chamber.

In tunnelling, a compressed air cabin is to be provided in the shield machine when compressed air is used. The fire services are to be notified about compressed air working (include in alarm plan).

Noise and vibration

The following are to be described for underground and for overground working:

- Measures to prevent health impairment due to noise and vibration.

The valid regulations are to be complied with, particularly:

- BGV B3 administrative regulation for protection against construction noise, 19 August 1970
- Decree of the German ministry for environment and nature conservation, 31 July 2000
- Measurement, evaluation and reduction of vibration emissions”

Contaminated ground. It is possible that contaminated ground is encountered during excavation works.

In this case, the following should be noted:

- Work in this case is to be interrupted immediately and measures undertaken to reduce damage. The site management of the client is to be informed immediately.

- The affected locations are to be determined by the contractor, published by posting a notice, and notified if required.
- The necessary technology is to be used.

The current regulations are to be complied with, particularly:

- LAGA 1995/1996.
- The guideline of the construction accident insurer “working in contaminated areas” (ZH1/183).
- TRGS 524 “Clean-up and working in contaminated ground”.
- BGV A4 “Medical care”.
- GM Component, D150.

8.7.2.4 Site facilities plans

The contractor presents site facilities plans to the coordinator for checking. The scope of checking includes the use of the planned machines and devices, parking of devices and machines, maintenance of workplace regulations, maintenance of regulations for the layout of the workplace and the requirements of the fire service and other rescue services.

8.7.2.5 Concept for traffic control measures inside the site area

When a construction site is in a densely built-up inner city area, construction activities may take place immediately adjacent to existing buildings. Access to residences and businesses is always to be maintained at all phases of construction, as well as access for the fire service. In some sections, however, pedestrian and delivery traffic may have to pass through the construction area.

Time limits and other measures to make access possible (back entrances etc.) or temporary accommodation elsewhere are to be discussed with the affected parties and the client at an early stage.

The concept is to include all planned measures for the control of traffic and secure the construction area, in order to rule out danger to third parties. This can include temporary access routes, pedestrian bridges, walkways, if necessary closures, fall protection, signage, manual direction of traffic, fixed contact partners for local inhabitants and more of the same.

All measures are to be discussed and agreed with the coordinator. ZTVSA and RSA are to be complied with.

8.7.2.6 Documents with information for later works to the structure

The contractor delivers all documents with the required information about the health and safety concept, which are relevant for later works.

The contractor also delivers corresponding information according to RAB 32.

8.7.2.7 Measures to prevent danger to third parties resulting from the duty to maintain road safety

The contractor has to comply with the provisions of the ZTV SA and RSA during all measures on public streets. Apart from the permit application process with the office for road and traffic technology and the police, all road safety measures are to be notified to the coordinator.

The contractor ensures compliance with the specified measures through checks to be discussed and agreed with the coordinator.

All alterations of access to the construction works and the traffic closures are to be presented to the coordinator for approval.

9 Special features in tendering, award and contract

9.1 General

In the GATT-WTO agreement concerning public procurement, the signatory states undertake to award public works above defined values to the most advantageous bidder overall. The wording clearly states the most advantageous, not the cheapest tender. This provision is completely in the interest of clients, because they are more certain to gain works of the appropriate quality, also in relation to the cost and time frameworks.

For clients, it is therefore advantageous to formulate criteria for the suitability of companies in a suitable manner in the tender and award criteria, which they intend to apply for the evaluation of tenders. When companies receive such requirements, they hand in well thought-out tenders that are specific for the project.

In contrast to drill and blast, there is no standard procedure for contract conditions, tendering and award in mechanised tunnelling. Therefore the following explanations and examples concentrate on the special features of mechanised tunnelling and the current procedures in various countries, which are continuously adapted in the light of experience.

The implementation of tunnelling projects, which permit the use of mechanised tunnelling, demand particular care in addition to the usual circumstances in tunnelling. The leading operation in mechanised tunnelling is the tunnel boring machine, and all other parameters like construction time, costs, susceptibility to interruptions, qualification of the workers etc. depend on the machine. All other operations have to be adapted to suit the machine. Not only the technical and personnel aspects but also contractual aspects demand particular consideration and discussion. Above all the introduction of technical machine influences into the delineation of risk between the clients overall planning, the contractor's overall planning, which in turn orders the machine, and the ordered machine, plus all the suppliers of mechanical accessories, has to be clarified and contractually formulated. The risk distribution should include consideration of not only questions of stability, but also the quality of the process (accident risk) and the risk of settlement in the risk of cost overruns and construction delays.

9.2 Examples of forms of contract

9.2.1 Procedure in Switzerland

General. The tendering of public works in Switzerland is mainly regulated by the BoeB (BoeB = federal law concerning public procurement) and the VoeB (VoeB = public procurement regulations). Both law and regulations became necessary with the introduction of the GATT agreement. Swiss Railways SBB are not subject to this law but normally comply with it voluntarily.

The legal regulation provides clear rules for tendering, checking of the tenders and award. Aptitude criteria represent the general first hurdle. Suitable tenders are evaluated according to award criteria. The type of tender evaluation described below has proved successful many times.

Tender evaluation. The evaluation of the tenders assumes behaviour in good faith according to the basic principle "one's right, another's duty":

– Contractors have a duty to comply with the submission conditions. They can, however, rely on the client rejecting tenders that do not comply.
– Clients do not level the tenders; on the contrary, they create decision-making basics to enable an optimal selection within the legal framework.

The following tender evaluation system has proved highly suitable for large tunnels, also those including bridges, works above ground and shafts in the scope of the project.

Starting from the basic principle that the most economically advantageous tender should win the contract, in a first phase the technical criteria are evaluated in six sections (Table 9-1) are weighted with the marking made known in the tender (Table 9-2).

After the technical evaluation has been completed, the overall assessment is undertaken with the tender price, with this being set in a previously determined ratio to the technical criteria. This cost/use consideration then enables the most advantageous tender to be determined.

Table 9-1 Award criteria with weighting [205].

Criterion	Weight [%]
Personnel qualifications (with references) Site manager Foremen Technical experts	10
Organisation of the bidder Leadership and technical management Sub-contractors (with references) Extent %	10
Construction sequence Planned construction programme with consideration of all dependencies	25
Machinery use Intended machinery with performance details, special machines, age of machines Construction ventilation: ventilation concept, fans, dedusting plant	15
Phasing of works / construction programme Phasing of works with performance details for the individual activities Overall programme with details of average performance and possible peak performance	20
Quality management Description of the QM of the bidder Position of the QM manager in the overall organisation diagram Description of the intended control mechanisms to ensure quality	20

Table 9-2 Marking of the partial criteria according to Table 9.1.

No statement	0 mark
Inadequate statement or not capable of evaluation	1 mark
General statements, which are not significant for implementation	2 marks
Adequate: statements to be evaluated with reservations	3 marks
Good: the statements can be evaluated as "practical" for implementation	4 marks
Particularly suitable: the bidder is particularly suitable in terms of know-how and experience	5 marks

Quality management. Clients as contract partners could also assume until now that contractors would take the necessary care demanded of them to ensure quality. When contractors tender with a quality management (QM), then they are obliged to align their duty of care according to it.

Such a system may be adequate as simple quality assurance in individual cases. Generally, however, the quality targets formulated by the client, which are only derived from the intended use in intensive processing during the design phase, are missing.

In contrast to the mechanical engineering or consumer production industries, the construction industry in effect only produces one-off products. Quality management for a construction work as a whole – which thus has to include the clients, designers and all contractors – needs to be organised according to these special features. Controlling quality assurance must differ from a handling of quality that is mainly restricted to supervision. The prime aim of project-oriented quality management (PQM) must therefore be to proactively avoid mistakes in design and construction. This is done in two steps:

- The performance of targeted analyses to produce hazard scenarios early in the design phase, derive the associated risks and plan measures in order to ensure a consistent residual risk within the bounds of cost-effectiveness. Risk is always to be regarded as a danger that is evaluated according to its consequences and probability of occurrence.
- To control quality through suitable measures in design and construction.

The usefulness of such a multi-function PQM system has been demonstrated by

- a high degree of design with good construction quality,
- consistent but not ridiculously increased construction quality,
- clear contract mechanisms,
- a considerably cost saving (in the case of the Murgenthal Tunnel, the client estimated this at 10%).

Such a multi-function PQM system is summarised in a quality control plan. Fig. 9-1 shows a flow diagram for the production of a quality control plan stating the most important documents required for the procedure. The essential stages in the overall procedure are:

- Utilisation plan with
  - formulation of the intended uses.
  - Correlations of intended use - serviceability – cost-effectiveness.
  - Risk analysis based on potential hazard scenarios.
  - Safety plan.

  - Verification of serviceability.
  - Cost-effectiveness including considerations of cost relevance.
  - Checking compliance with the intended use.
- Quality requirements for the quality-relevant work categories.
- Control plan structure-ground with the geotechnical monitoring concept.
- Working instructions for the contractor: the contractor returns an acknowledgement that the aims have been met with the correct implementation of the quality requirements. These working instructions should include at least:
  - A definition of the construction works,
  - Organisation,
  - Technical basis,
  - Working procedures,
  - Testing and checking plan,
  - Traceability,
  - Accident prevention,
  - Environmental protection.

Assignment of risks in the contract. The risks that are inherent in any construction project should be borne by one or other of the contract partners alone. The often vaunted performance specification tender often leads to a transfer of all risks to the contractor.

The Swiss standard SIA 198 "Underground Structures" [218] includes in Appendix 5 a possible assignment of risks in tunnelling, mostly resulting from ground conditions.

For TBM tunnelling, this results in the following recommendation:

Scope of risk of client

- Presence of gas.
- Collapse due to geological conditions.
- Larger convergences than were assumed in the contract, leading to:
  - Jamming of the tunnelling machine,
  - Already laid invert segment being laid deeper,
  - Enlargement of already bored tunnel sections,
  - Rebuilding of the machine to a larger diameter,
  - Alteration of the formwork for a smaller diameter.
- Rock quality encountered significantly outside the limits given in the tender documents:
  - Cuttability significantly less favourable,
  - Bracing of the machine proves impossible or has to be packed due to collapses,
  - The invert cannot provide the necessary bearing pressure and the TBM cannot be supported at the correct level,
  - Rock is so fractured that it corresponds to soil.

Scope of risk of contractor

- Change of rock properties inside the agreed limits.
- Tunnelling through variably hard rock layers.
- Stoppages in the tunnelling system due to stickiness of the excavated ground.
- Qualification of the employees.

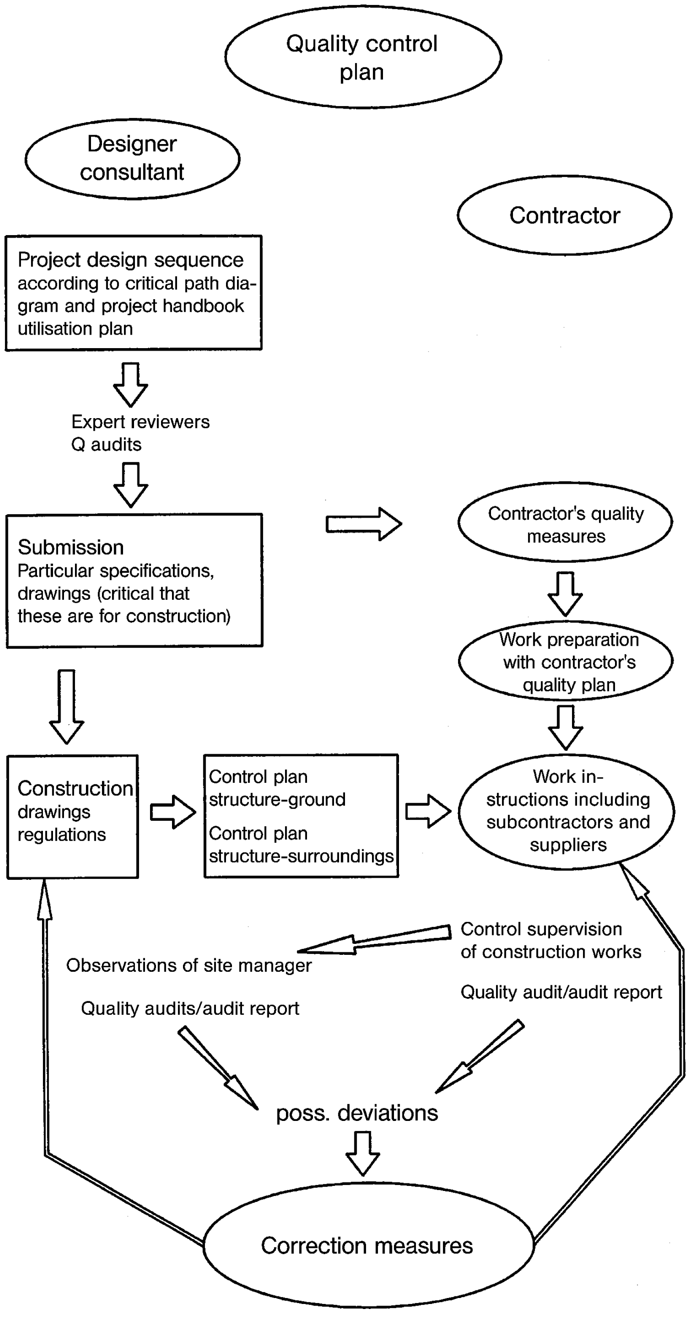


Figure 9-1 Flow diagram for the production of a quality control plan.

One special case is obstruction due to water ingress, which almost always reduces performance. Standard 198 [218] provides sample contract conditions, which have proved successful. A basic difference is made between obstruction, which is covered by items in the bill of quantities, and changed deadlines because the deadlines in the contract can no longer be achieved.

For the reduction of the specified performance, the values in Table 9-3 apply. The appropriate reductions are practically applied directly to the relevant figures in the matrix of excavation class / boring class.

Table 9-3 Reduction factors for performance figures in case of water ingress.

	Tunnel with theoretical excavation area dia. ≤ 5m		Tunnel with theoretical excavation area dia. > 5m		Reduction factor
Advance	**rising**	**falling**	**rising**	**falling**	
Water ingress	10 to 20	5 to 10	10 to 20	5 to 10	0.2
[l/s]	> 20 to 30	> 10 to 20	> 20 to 40	> 10 to 20	0.4
	> 30 to 40		> 40 to 60	> 20 to 30	0.6

Geological/geotechnically changed conditions, variation orders, deadline adaptations. With the application of excavation classes and boring classes and their contractually agreed performance matrix, the adaptation of contract deadlines is a simple deduction. There remains only to be agreed the provision times, shorter or longer, for the construction site facilities.

The regulation of changed conditions and exceptional circumstances is much more complicated.

Unusual ground conditions are present when the information given in the tender documents is faulty. It is not very helpful to bypass the special nature of the conditions by stating widely spread geological information. The decisive information in the contract remains the implementation of the geology by the client in the bill of quantities and in the geological-geotechnical longitudinal section.

The attempt to include all possible and impossible eventualities in the contract has also proved unsuccessful.

9.2.2 Procedure in the Netherlands

The following section describes the procedure in the Netherlands through the example of the award of the Botlek Tunnel, one of the largest tunnels of the Betuwe line for the client NS Railinfrabeheer (NS RIB). All the other awards of large tunnels in the Netherlands have in principle followed this procedure.

Tendering and negotiation procedure through the example of the Botlek Tunnel [36]. The European negotiating process was chosen for the Botlek Tunnel in order to optimally enable the bidders to offer their own technical solutions. This consists of a sequence of fixed steps, which NS RIB arranged in a particular form in order to achieve the intentions of the tendering process. One feature of this process is that the candidates have to produce their own technical solutions before tendering. Since the tender is based on a fixed price, this means that the bidders have to undertake considerable design work.

The negotiating process consists of the following steps:

Selection. The aim of the selection phase is the selection of candidates who are in a position to perform the required works in a correct manner. The client produces objective selection criteria for this purpose in advance.

Call for tenders. The tenders are produced on the basis of designs prepared by the candidates. The call for tenders consists of a functional performance specification. In order to be sure that the specification leads to a good solution, and in order to be able to evaluate the proposals of the candidates, NS RIB has already prepared a reference design fulfilling the issued requirements in advance.

Contract conditions. The contract is awarded on the basis of a clearly formulated design & construct (D & C) contract. Since construction contracts in the Netherlands do not provide for the design being produced by the contractor, NS RIB have developed their own D & C contract. The contractor bears complete responsibility for the suitability of the design for construction and the performance of the works.

Due to the quality assurance requirements, the bidder group has to possess an ISO 9001 certificate. An outline quality plan is produced by the candidate as part of the tender.

Liability for the quality of the completed tunnel remains with the contractor for as long as possible. For this purpose, the contract assumes a maintenance period of five years and a guarantee period regarding waterproofing lasting ten years after acceptance.

Consulting phase. The aim of the consulting phase is that the candidate reaches a technically optimal solution, which then serves as basis for the tender. Costs are not discussed in the consulting phase. The client consults with each candidate separately and guarantees the confidentiality of these discussions.

Submission of tenders. Bidders have to demonstrate with the submission of their tenders that they have been informed about all the risks that are relevant to the project and demonstrate with their own risk analysis that these risks are limited and that they can overcome them.

Each candidate submits two tenders. One is based on the NS RIB design and the other is based on the alternative proposed by the candidate. As compensation for the work involved in design preparation, NS RIB paid each bidder half a million DM (about one quarter million €).

The essential points where the proposals from the contractors differed from the reference design were:

- Volume and type of ground improvement,
- Construction of the transverse connections between the tunnels,
- Dimensioning of the segment lining,
- Construction of the sealing block at the transition shaft/tunnel,
- Location of the shafts,
- Type of shield machine,
- Diaphragm walls against combined pile/sheet pile walls,
- Pile types for the ramps,
- Riverbed protection in the Oude Maas River.

Evaluation of the tenders. The intention of the evaluation is to select one or more parties, who are then invited to the negotiation phase. These are the bidders who have submitted the economically most advantageous tenders.

Since the bidders prepare their own technical designs, the tenders are not directly comparable. This makes it impossible to issue detailed award criteria before the tendering process. These were only described in outline with qualitative factors and price being differentiated. The application of this process achieved design freedom. The tender sum is, in contrast to a traditional tender submission, only one of the award criteria.

When the tenders were evaluated, technical quality turned out to be very significant for the selection of the contractor who was invited for negotiations (Fig. 9-2). This was not the bidder with the lowest price.

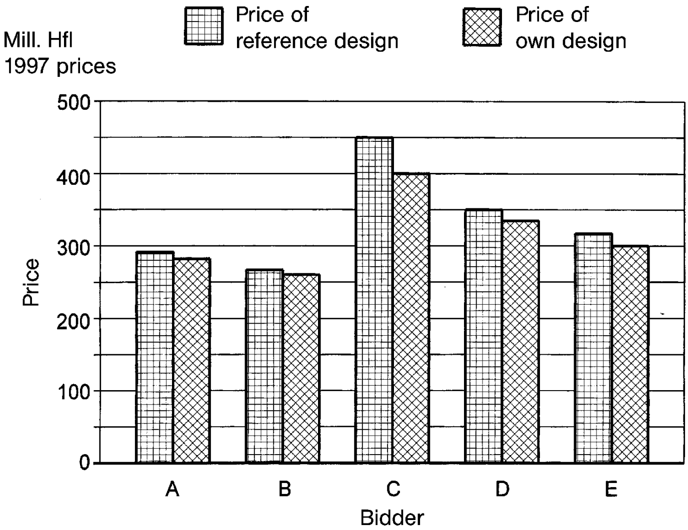


Figure 9-2 Financial evaluation of the tenders.

The technical solution selected for the Botlek Tunnel differs from the reference design above all in the higher location of the tunnel, which increases the capacity of the tunnel for goods transport. This is achieved by the use of the EPB process to bore the tunnel. The solution leads to a five to ten percent increase of the capacity of the tunnel, which will also essentially determine the future capacity of the railway to the port. In addition, the solutions for ground improvement and tunnel lining are more economical.

The general trend was that tenders were keenly estimated. The alternatives proposed by the candidates always led to a saving.

Negotiation. The intention of the negotiation phase was to reach agreement on all points related to the project.

Award. The contract was awarded to the joint venture (ARGE) “BTC Botlek”. This joint venture consists of various large Dutch construction firms with the German partner Wayss & Freytag AG.

Arbitration process. The bidder with the lowest price called for an arbitration process against the decision of the client after having been rejected on stated grounds. The arbitration tribunal ruled that

– NS RIB is not obliged to state grounds under EU regulations and
– NS RIB has to be allowed a certain planning competence in the application of the tender criteria.

The claim made by the party was rejected.

Above all the second statement is of great significance. If the client did not have this planning competence, the lowest price would always be the decisive factor. In this case, NS RIB would have had to abandon future tendering procedures based on D & C contracts, since the quality of such tenders is of great importance.

The tendering and award process that was used led to a good result.

It is clear that the candidates accepted the possibility of delivering a design contribution as a challenge and many made extreme efforts, particularly to prepare a technical proposal for the implementation of the risky boring work in a safe and economical manner.

One disadvantage of the process used is the long duration of about ten months. This could have been shortened by about three months if a traditional tendering process had been used. A second disadvantage is the large amount of work that the bidders have to undertake to compile a tender. This amounted to about 10,000 hours per candidate for a contract value of about 300 million DM (150 million €). It may be possible to reduce this somewhat in the future.

The contract award of the Botlek rail tunnel demonstrates that European negotiation processes are very applicable to the tendering of complex building and civil engineering works, and also to stimulate the construction sector to look for innovative solutions.

9.2.3 Procedure in Germany

The procedure in Germany for mechanised tunnelling was dealt with exhaustively in [141]. The recommendations of the DAUB [44, 45] normally provide the basis for the consideration of additional special features. These are extended with specific conditions; see also the publication Tunnelbau Taschenbuch 2001 [144].

9.3 Design and geotechnical requirements for the tendering of mechanised tunnelling as an alternative proposal

9.3.1 General

A tunnel project with tender documents for shotcrete tunnelling can only be imperfectly tendered for mechanised tunnelling. Even when the geotechnical conditions are appropriate, the submission of an alternative proposal is often impossible because the specified concept cannot be fulfilled by mechanised tunnelling. This is confirmed by the following examples [133].

In the end, the geotechnical, design and contractual requirements in the tender documents were formulated in a way that provided the preconditions for the submission of a tender based on mechanised tunnelling.

The costs and the people involved in the phases of design, award and construction have a decisive influence on the consideration of an alternative proposal for mechanised tunnelling.

Only when these points are considered can a practical and economic tender be submitted.

9.3.2 Examples: Adler Tunnel, Sieberg Tunnel, Stuttgart Airport Tunnel, Rennsteig Tunnel, Lainzer Tunnel

Adler Tunnel. The Adler Tunnel is part of the new line Muttenz – Liestal near Basle as part of the Bahn 2000 plan of Swiss Railways SBB. The underground part of the two-track rail tunnel has a length of 4.3 km and an excavated diameter of 12.58 m. The geology of the Adler Tunnel is characterised by the claystone, marlstone and limestone rocks of the Jura (Fig. 9-3), although a third of the tunnel is located in gypsum Keuper (anhydrite, susceptible to swelling) with clay formations. In the course of the tunnel drive, extensive clay formations had to be overcome. The hydrology of the Adler Tunnel features formation water containing sulphates and chloride, which also tends to cause severe sintering due to the calcium content. The water pressure was max. 1.0 bar and the water inflow 10 to 20 l/s.

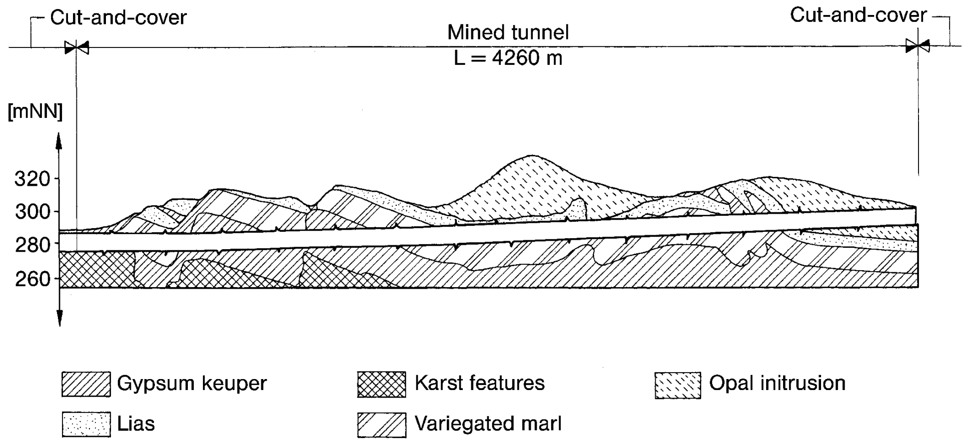


Figure 9-3 Geological profile of the Adler Tunnel.

Comparative design and tendering was carried out for two variants – TBM with shield and shotcrete tunnelling.

After a detailed analysis, the TBM option was preferred, although the initial section was to be supported with shotcrete. One essential reason for this decision was the swelling behaviour of the anhydrite.

The TBM drive of the Adler Tunnel was completed at the start of 1998. Early after the start of the temporary works for the starting area by shotcrete tunnelling, a collapse occurred. Another collapse occurred soon after the subsequent start of the TBM. During the advance of the shielded machine, there was another collapse after 1,710 m, which led to complete jamming of the cutterhead.

After the performance of additional works like drainage from the surface, grouting ahead of the machine from the shield and further improvements to the machine, the shield drive was completed without problems, with average advance rates of 15.5 m/d being achieved; 5.4 m/d was even achieved in a fault zone.

The resulting extra costs led to an arbitration process, at which all the problems that had led to interruptions and costs were worked through with the project parties. Despite the stated additional works and the alteration of the organisation structure on site, the decision to use a shielded machine proved correct.

Sieberg Tunnel. The Sieberg Tunnel is part of the high-speed rail line (HLS) Vienna–Salzburg. The two-track tunnel has a length of 6.5 km and an excavated diameter of 12.5 m. The contract was awarded at the end of 1996. The mined sections of the Sieberg Tunnel lie exclusively in the beds of the molasse zone with intercalated siltstone and fine sandstone (Fig. 9-4), called Miocene schlier and Oligocene schlier. At the four valley crossings, there were extensive weathered zones. The hydrology of the Sieberg Tunnel mostly shows two groundwater tables and a maximum water pressure of 2.9 bar. The water ingress during the drive was very low and only seldom reached quantities of 5 l/s. The overburden was max. 55 m, although this reduced at the four valleys to less than 10 m in places. The tunnel did not pass under any buildings on the surface.

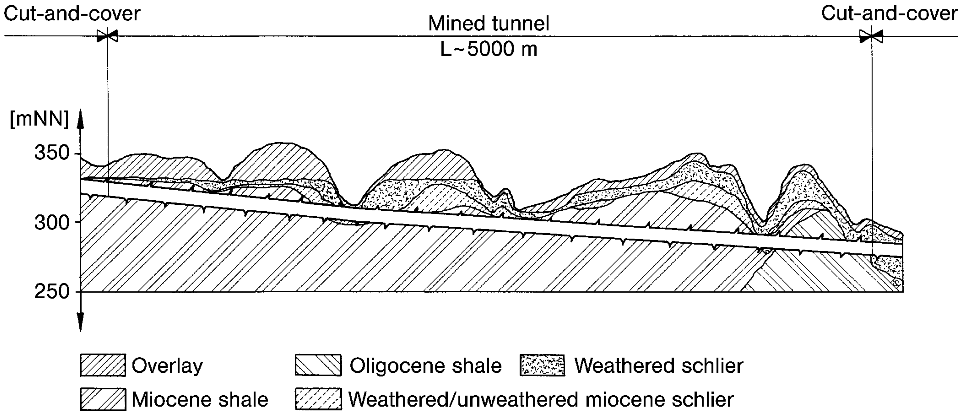


Figure 9-4 Geological profile of the Sieberg Tunnel.

The tender documents for shotcrete tunnelling included a tunnelling classification according to Austrian Ö-Norm standards into seven classes for the top heading and five classes for the invert. Classes six and seven required advance support measures (spiles, IBO anchors and in some cases pipe screens) for a length of about 3.5 km. One 1.26 km long section of the tunnel was to be constructed in cut-and-cover. The tender documents did not include any details of constraints affecting a shield tunnel drive.

Although the tender documents had intended shotcrete tunnelling, an alternative proposal for the entire length including the cut-and-cover section was submitted. This intended the reuse of the tunnelling machine from the Adler Tunnel. The flat rock cutterhead was fitted with disc cutters and picks and could be extended on all sides, although retraction of the cutterhead was not possible. The tender intended final lining of the tunnel with segments. No additional measures were mentioned in the alternative proposal for the sections with shallow overburden.

The mechanised tunnel drive was rejected because the tender was incomplete. The assurance by the bidder to accept all risks and costs was also regarded as unachievable by the client.

The difficult geology compared to other projects and the initial experience from the Adler Tunnel made all parties careful.

The experience with the tendering and award of the Sieberg Tunnel shows clearly that the client should include the criteria and requirements for mechanised tunnelling in the call for tenders. This would require at least a preliminary design to enable competition.

Airport Tunnel, Stuttgart. The airport tunnel is part of the S-Bahn rapid transit network in the metropolitan area of Stuttgart. Two single-track rail tunnels were constructed, each with a length of 2.2 km and an excavated diameter of about 8.5 m. The S-Bahn tunnels run through the Jurassic clay-siltstone with intercalated limestone and sandstone banks of variable frequency (Fig. 9-5). Swelling and increased horizontal stresses also had to be expected along the alignment.

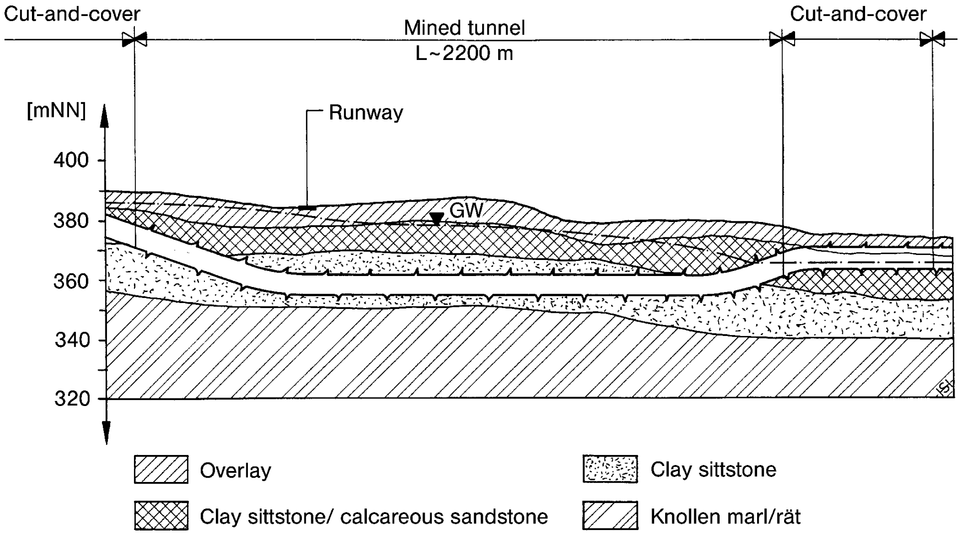


Figure 9-5 Geological profile of the airport tunnel, Stuttgart.

The clay-siltstones are generally of low permeability, so little water ingress had to be expected in these sections. The limestone and sandstone banks, on the other hand, are highly permeable and increased water ingress had to be expected in them. The maximum forecast water pressure along the tunnel was about 2.5 bar. Both the permeable limestone and sandstone banks and the clay-siltstones are suitable for grouting. The environmental requirements are influenced by the route of the tunnel passing under many roads and buildings and also the runways of the airport with stringent settlement requirements. The minimum radius for the tunnel was $R = 300$ m.

The tender design intended the use of shotcrete tunnelling. This was to be carried out without groundwater lowering as a low-settlement tunnel drive. The tunnel cross-section was circular.

Nevertheless, an alternative proposal was tendered for a shield drive equipped for compressed air working where required, to provide active face support. The closed cutting wheel of the full-face machine was equipped with discs and scrapers and an articulated joint was provided in the shield to cope with the tight radii. All the required equipment was of the latest state of technology for the reduction of settlement. The lining of the tunnel was to consist of segments, as is usual in shield tunnelling.

The consultant and specialist consultants declined the alternative proposal despite its lower cost.

The stated grounds were:

– The ability of the machine to cut the limestone banks was questioned, although positive experience was already available from the Adler Tunnel and the cutting wheel was equipped with discs and scrapers,
– The possible jamming of the shield skin was seen as a problem; this was dealt with by the provision of copy cutters.
– The waterproofing and the durability of the segment lining were doubted. Single-layer segment linings are, however, now the state of technology.
– It was feared that the application of shield tunnelling could not comply with the permissible settlements that had been specified. On the other hand, it should be mentioned that no blasting is necessary in shield tunnelling and synchronous grouting equipment with the latest control systems had been tendered.

Controlled overcutting with rapid ring closure also spoke for the use of a shield machine. These are both advantages compared to shotcrete tunnelling.

The shield tunnel drive was rejected although extensive experience was available under similar conditions (see Adler Tunnel) and the solution was cheaper and technically better.

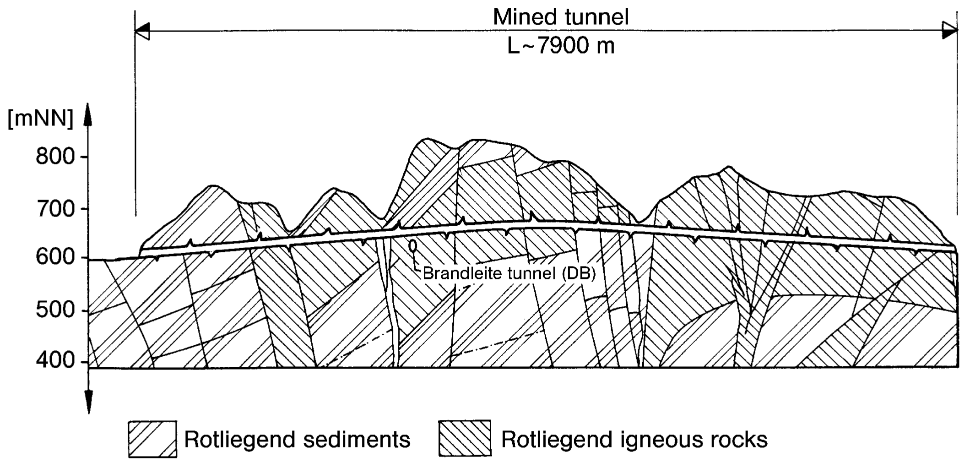


Figure 9-6 Geological profile of the Rennsteig Tunnel.

Rennsteig Tunnel. The Rennsteig Tunnel is 7.9 km long and when completed will be the longest road tunnel in Germany. The four lanes of the autobahns A71/A73 will run through two tunnel bores. The crossing of the ridge of the Thüringer Wald hills will be followed by the Alte Burg, Hochwald and Berg Block Tunnels, a total tunnel length of 12.6 km. The excavated cross-section of the tunnels is about 11 m. The geological conditions for the Rennsteig Tunnels are characterised by Permian porphyries, which are mostly very compact and hard (Fig. 9-6). Intercalated in and below this igneous rock are further strata of sandstone, clay stone and Keuper clay. The groundwater table alternates from below to above the vertical alignment. The maximum water pressure is about 8.5 bar and the water ingress is forecast up to 5 l/s per 100 m tunnel. The maximum overburden is 200 m. The tunnel also passes below the active two-track Brandleite rail tunnel. Safety requirements demand the provision of stopping bays every 700 m and the two bores are also connected every 350 m by escape tunnels.

The tender design intended the use of shotcrete tunnelling. Only shortly before the completion of the tender documents were a few conditions included for an alternative proposal using a shield machine.

Quote from the tender documents: "Under the given geotechnical conditions, the use of a shielded tunnel boring machine (TBM-S) is fundamentally possible. The construction process can be tendered as an alternative as long as bulk material handling is considered."

The tender documents contain brief details applicable to alternative proposals concerning construction process concept, machine concept, segment lining, material handling and transport concept.

The Rennsteig Tunnel was awarded as a shotcrete tunnel, since this turned out to be cheaper than mechanised tunnelling, which led to tenders about 10% more expensive.

Meanwhile the first experience of constructing this tunnel is available. The geological conditions would have been suitable for a machine along about half of the tunnel length. The rock mass classes are often more advantageous than forecast and the water ingress is much less. According to experience in Switzerland, a TBM could have achieved much faster advance rates.

Lainzer Tunnel. The Lainzer Tunnel connects the Westbahn, Südbahn and Donauländerbahn lines in the city area of Vienna with a total length of about 5.5 km. The project is divided into the Hetzendorf Tunnel with a length of about 2 km and the Lainz Tunnel about 3.5 km long, of which 0.3 km is supported with shotcrete (Fig. 9-7). The diameter of the two-track tunnel is 13.8 m.

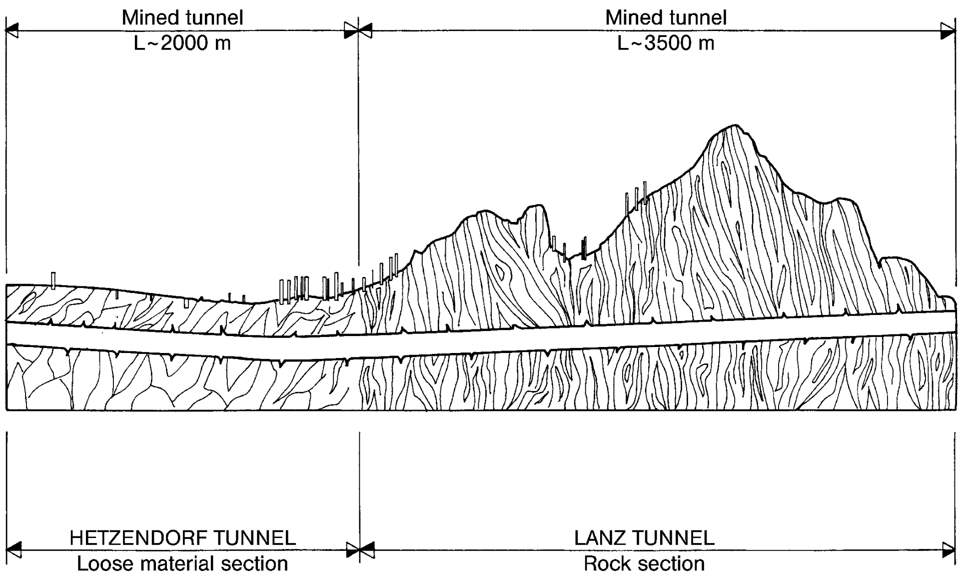


Figure 9-7 Geological profile of the Hetzendorf and Lainzer Tunnels.

The entire length of the Hetzendorf Tunnel runs through the sediments of the Vienna basin. These are very variable and extremely heterogeneous beds of gravelly and silty clay. The maximum water pressure is 1.8 bar and high water inflow has to be expected. In the Vienna tegel (similar to marl), only pore water pressure has to be expected due to the low water permeability. The local conditions are characterised by the route of the tunnel below residential areas, which restricts the access and launching possibilities. The overburden is also shallow, so the requirements of permissible surface settlement were increased.

For the Lainzer Tunnel, most of the tunnel alignment passed through moderately hard solid rock of 80 % Tonstein (claystone) as well as siltstone and marlstone, with 20 % of the alignment in “Klippenstein”, sandstone and limestone rocks. Water only occurs locally, but can reach quantities of up to 10 l/s and the maximum water pressure is 6.5 bar.

The soil section (Hetzendorf Tunnel) was tendered for shotcrete tunnelling with side headings. The aquiferous beds were to be dewatered from the surface using deep wells. Grouting of the soil was intended to reduce settlement.

The rock section (Lainzer Tunnel) was also tendered for shotcrete tunnelling. This section was to be excavated using a roadheader or tunnel excavator in the partial sections top heading, bench and invert. In some areas, rock bolts, spiles and pipe screens were planned.

As an alternative to shotcrete tunnelling, investigations were also undertaken for both sections into the use of a shield machine as a single-track or two-track solution. The requirements for the tunnelling process in the soil section would best be fulfilled by a shield machine with slurry-supported face, with a single-pass segment lining for the permanent support.

In the rock section, the use of a shielded TBM is possible, although it should be possible to close the excavation chamber to overcome the water pressure. In accordance with the Swiss tunnelling method, a two pass lining with umbrella waterproofing can be used for the permanent support.

The example of the Lainzer Tunnel clearly shows the influence of local conditions; the launching area for the shield machine and thus the entire tunnel route cannot be optimally adapted to suit the local conditions.

The construction of switch niches after the shield drive worsened the preconditions for mechanised tunnelling, although the escape niches could be replaced by box niches inside the shield-driven cross-section according to the design handed in for approval.

The use of a tunnelling machine is possible in principle under the existing geological and hydrological conditions. The construction of the required external rescue niches and switch niches is time-consuming and expensive, so shotcrete tunnelling was estimated to cost less and was used for construction.

Figs. 9-8 to 9-10 show some examples of the different details for the construction of safety niches. Fig. 9-8 shows a cross-section with safety niches in conventional tunnelling.

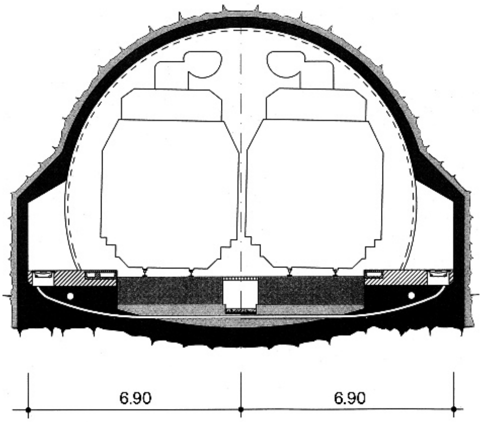


Figure 9-8 Cross-section with safety niches – conventional tunnelling.

The construction of box and safety niches for shield tunnelling is shown in Figs. 9.9 and 9.10, which demands the opening of the segment lining shown in Fig. 9-9. The box niche can be integrated into the existing structure gauge of the segment ring.

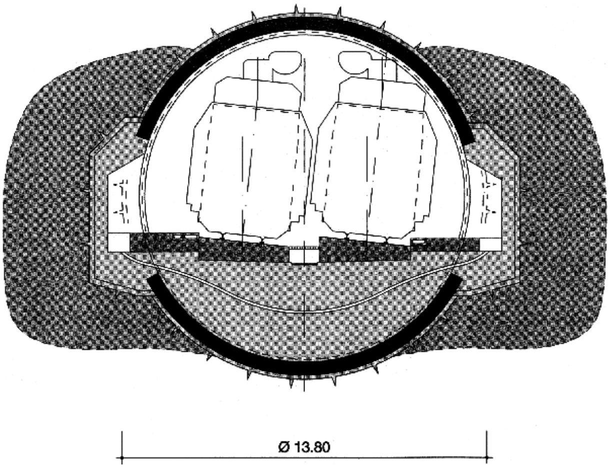


Figure 9-9 Cross-section with safety niches – shield tunnelling.

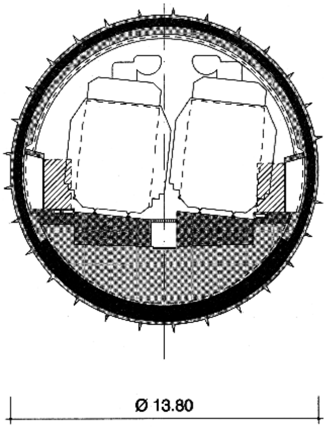


Figure 9-10 Cross-section with box niches – shield tunnelling.

9.3.3 Additional requirements for mechanised tunnelling in the tender documents

The following section describes the necessary geotechnical information, which should be included in tender documents in order to enable alternative proposals for mechanised tunnelling. The basic design concepts for mechanised tunnelling and the design of the structure are summarised. Finally, the requirements for the tender documents and contract are described. Fair competition between the two methods is only possible on the basis of these criteria.

Geology and hydrology. In order to produce an alternative proposal for mechanised tunnelling, binding statements about the type and extent of the obstructions in the ground are necessary. The geotechnical data have to be evaluated regarding the overcoming of fault zones.

The soil mechanics properties for the estimation of potential sticking and the wear parameters for excavation tools and mechanical parts (seals) also have to be determined and evaluated.

In addition, the suitability of the ground for grouting and conditioning should also be assessed. In particular, the environmental acceptability of the materials used has to be decided.

From the information available about the geotechnical conditions, the suitability of the excavated soil for tipping or recycling and for separation also needs to be assessed.

Finally, when a tunnel boring machine with grippers is to be used, the capability of the rock to resist bracing forces also has to be estimated.

Design and construction process. The design concept has to be optimised for mechanised tunnelling with regard to the layout of niches, breakdown bays and cross-passages. The number of niches has to be restricted to the necessary minimum, the tunnel cross-section enlarged to integrate escape routes (excepting niches, breakdown bays) and the segment concept adapted to suit the necessary emergency measures.

The performance of additional measures, delivery and assembly times, segment production and logistics are all requirements, which have an effect on construction operations. Further aspects of importance for the planning of construction operations are the launching and reception processes, the construction of niches and connecting tunnels, the analysis of interruptions and risk, fire and explosion protection and a geotechnical measurement programme.

The specification for mechanised tunnelling should include specific additional points concerning advance probing, advance dewatering, advance ground drainage, overcoming obstructions, checking excavation quantities, recording operational data and shield control with safety equipment, and the visualisation of geological faults. Any rebuilding or conversion of the tunnelling machine or alteration of material conveyance to another operating mode should also be included in the specification.

The design should take into account requirements affecting the lining such as fire/explosion protection, waterproofing and impact loads. Requirements resulting from geometrical criteria (curve radius, tolerances), influences resulting from the advance of the shield and grouting openings also have to be considered in the design of the lining.

Tendering and contract. The tendering of a tunnel project for mechanised tunnelling should include the determination of geological risk. It is the task of clients to describe the ground conditions in detail and produce geotechnical forecasts. Contractors are responsible for technically correct dealing with the ground and should also perform their own site investigations.

The contract should also regulate the implementation of additional measures, which could become necessary to overcome fault zones or for passing under buildings susceptible to settlement. The interruption scenarios envisaged in the contract should also regulate the method of payment for shorter and longer stoppages. The quality criteria should also be specified for the tunnel lining, and in order to enable comparability of tenders, the calculation model, the waterproofing class to be achieved, the tolerances of the lining and the required fire protection for the tunnel lining should all be specified.

The tender documents should include special requirements for the tunnelling concept with regard to settlement, calculation basis for the stability of the face, technical requirements for the shield construction and additional measures undertaken from the shield. Control and regulation programmes including quality assurance system and loading actions on the segment lining should also be specified.

9.3.4 Costs as a decision criterion

In the consideration of costs (cost security), the decision phases and the parties involved also have to be taken into account. The decision phases are structured into design phase, tendering phase and construction phase.

Design phase or tender design. Three cases should demonstrate in a simplified manner the influence that a consultant can exert on tendering:

Case 1. The consultant roughly estimates the cost of a shield drive without producing a full design basis and comes to the conclusion that the cost is higher, resulting in a recommendation to the client that there should be no provision for a shield tunnelling in the tender documents and no approval of an alternative proposal for mechanised tunnelling.

Case 2. A shield tunnel drive is indeed fundamentally permitted as an alternative proposal, but the design and geological information required for the production of a tender are not included in the tender documents. The alternative proposal that is received in this case is either incomplete, too expensive or also even too cheap, if the price has been manipulated rather than estimated (for example by omitting additional measures).

Case 3. A shield tunnel drive is planned parallel to shotcrete tunnelling and the tenders are open to free competition. This procedure is only known to have been fully implemented in Switzerland.

The consultant has a decisive influence on the acceptance and implementation of shield tunnelling.

Tendering phase. Mechanised tunnelling, which is tendered on the basis of a design intended for shotcrete tunnelling, is normally more expensive (about 10 to 15 %). This means that mechanised tunnelling often has no chance of winning the contract. When, however, shield tunnelling is tendered in fair competition with a design that considers shield tunnelling, it has good chances of competing with the shotcrete method.

As a conclusion, it should be stated that the comparison of construction methods should be subject to competition and not the opinion of the consultant.

But the question of costs is not concluded at this stage. The phases of construction and final payment also have to be considered.

Construction phase and final invoice. A tendency can be detected that tunnels constructed by shotcrete methods often lead to far higher final invoices than tender sums. In contrast to this, experience from Switzerland shows that with a few exceptions (for example the Adler Tunnel), no appreciable cost overruns occur with the final invoices of shield tunnels.

The theme of cost security is only one criterion at the moment, but further investigations could come to more substantiated conclusions.

9.3.5 Outlook

Shotcrete tunnelling and mechanised tunnelling will continue be able to define and maintain their scope of application. If, however, the project-specific local conditions like tunnel length, cross-sectional profile, geology and hydrology offer the preconditions for the use of mechanised tunnelling, the requirements of mechanised tunnelling should already be considered in the design phase.

The formulation of appropriate geotechnical, design and contractual requirements in the preparation of tender documents provides a clear and defined basis for the evaluation of tenders based on mechanised tunnelling. In Switzerland, this principle has been pursued with success, and this would not have been possible without cost advantages and the reduction of risk.

Mechanised tunnelling offers innovative and highly technical processes with a high degree of working safety and mechanisation and thus represents a decisive contribution to futuristic tunnelling methods.

10 Process controlling and data management

10.1 Introduction

Tunnels driven with shields can be regarded as construction measures with a high degree of difficulty, with pronounced interaction between the structure, materials and surroundings. They are also subject to particular risk because the principal construction material, the surrounding ground, is difficult to recognise and describe. The safety, also the effectiveness of the use of resources, of highly technical shield tunnelling can be improved considerably through the thorough analysis of the process data. The aim of process controlling is to analyse the system behaviour – in situ and if possible in real time – under consideration of all interactions between ground and construction process.

The interactions of structure, construction material and surroundings are also described as system behaviour in DIN 1054 thus following the Austrian standard ÖNORM B 2203-2 “Underground works – Works contract – Part 2: Continuous advance” [228]. This term includes the behaviour of the overall system resulting from ground and tunnelling process. The new versions of ÖNORM EN 1997-1 (Eurocode 7) [226] and DIN 1054 [74] prescribe the observation method for complex geotechnical structures. The purpose is to verify measures, which were decided before the start of construction, during the construction phase with measuring systems. Prognoses are to be checked or the calculation model is to be adapted when the behaviour of ground and structure do not settle down as expected. If the serviceability or even the structural safety is endangered, then countermeasures should be taken.

10.2 Procedure

The method of process controlling [209] described below is based on successful methods of optimising production processes in mechanical engineering and originally comes from system engineering. The production process of the shield tunnelling machine is for this purpose split into partial processes. Fig. 10-1 shows the partial processes and their interaction for the example of an earth pressure balance shield.

Controlling is a description for the summary of the target-actual comparisons of the essential process parameters and quality criteria. Only when it is clear which quality criteria a process has to fulfil can the process sequence be sensibly developed and monitored. This means that suitable methods have to be laid down for process controlling, which can ensure that the processes fulfil the expectations placed on them. Suitable methods therefore have to be laid down for process controlling, which can be used to ensure that the processes fulfil expectations. A well thought-out concept of process controlling not only serves as an instrument for evaluating whether the intended targets have been reached but also offers the longer-term advantage of delivering the necessary indicators for continuous improvement of the process. This requires the setting of objective quality criteria (Key Performance Indicators or KPIs). The objective quality criteria can be used to check whether processes are running or there is a need for improvement. Therefore the overall targets have to be fixed before the decision of suitable KPIs. With these targets in mind, suitable indicators for the evaluation of the successful execution of a process can be determined. Which KPIs are finally selected depends on factors including the tunnelling process, individual experience and the available possibilities for determining the values. In the ideal case, the indicators can be calculated automatically, for example using a numerical simulation (support pressure specification) or mathematical algorithms (tool wear). The measurement procedures defined here therefore also represent at the same time requirements for the system to be implemented [303].

In process controlling, the aim is not to specify as many KPIs as possible. In practice, it has often been found that an excessively complex structure of indicators leads to a disproportionately large amount of work, is little accepted and is therefore not used any more after a short time. It is recommended instead to define only a few but significant indicators so that the amount of work involved in determining the indicators and reporting remain within reasonable bounds.

10.3 Data management

Data management is the most important element of a process controlling system. It includes the entire reporting and thus includes all essential indicators for design and construction. Highly mechanised shield tunnelling is particularly suitable for the implementation of computer-based data management systems. Every one to ten seconds, between 200 and 400

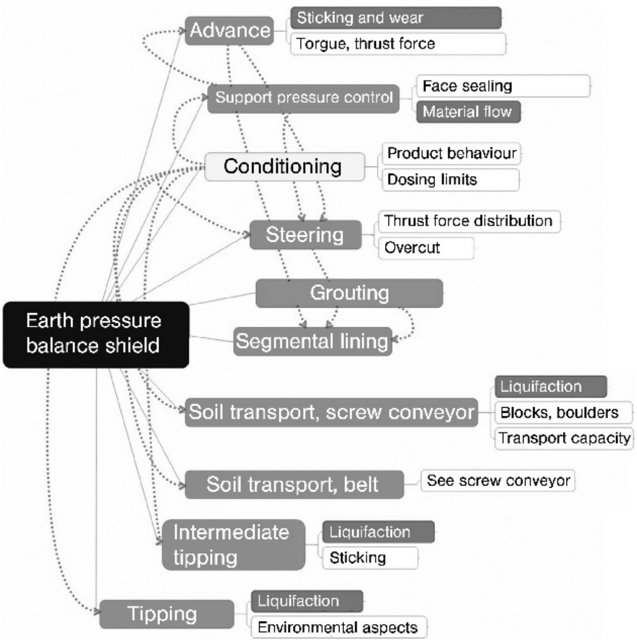


Figure 10-1 Process sequence scheme for an earth pressure balance shield

items of machine data, called momentary values, are recorded. This amounts to between 1.7 and 3.5 million items of data per day, which can be collected automatically for each advance cycle in average and final value files. Separately from this, a great amount of geodetic and geotechnical data is usually logged during the advance. While this was formerly done manually and thus at very varied intervals, measurement robots are now used, which automatically record defined survey points and transmit the data digitally.

The special challenge of the observation method is the real-time processing of the information and data as the advance progresses. Fig. 10-2 shows the database structure for the implementation of a controlling system based on construction process. The database contains process-related data related to the ground, the tunnelling machine and the results of geotechnical measurements. In addition to the machine data and the geodetic survey data, data can be recorded at other process interfaces (separating plant, segment stores etc.), which is normally only available in analogue form in shift or daily logs. The implementation of construction operations data, for example time-dependent costs, cost of materials and electricity consumption, is also possible. All information relevant to the advance is transmitted in real time over a VPN link and analysed by a programme developed specially for shield tunnelling according to Key Performance Indicators (KPIs), like for example face stability, advance rate, specific electricity consumption, foam injection and expansion rates. Operational interruptions, changes of ground conditions and their consequences for the process can be followed in real time by remote analysis or subsequently.

10.4 Target-actual comparison

The target-actual comparison considers all significant process-related and geotechnical parameters. The computer-based results are compared with the design specification (reference values) and the results are made available to the responsible geotechnical specialists (regulator-control unit) in evaluated and visualised form. The target-actual comparison is then interpreted by experienced users and instructions are issued and implemented by the site management.

The design specification (reference values) is derived from the geotechnical calculations and forecasts (Section 18.5) carried out before the start of construction. In accordance with the philosophy of the observation method, the observation data from the surveying and monitoring system is included. As shown in Fig. 10-3 the behaviour to be expected is forecast using detailed analyses, back analyses and simulations.

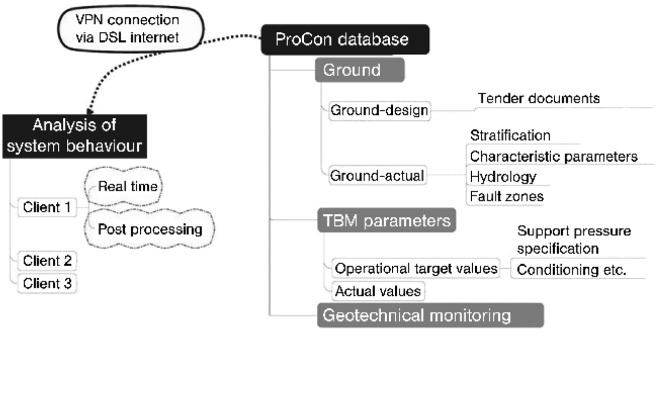


Figure 10-2 Database structure for process controlling in shield tunnelling [209]

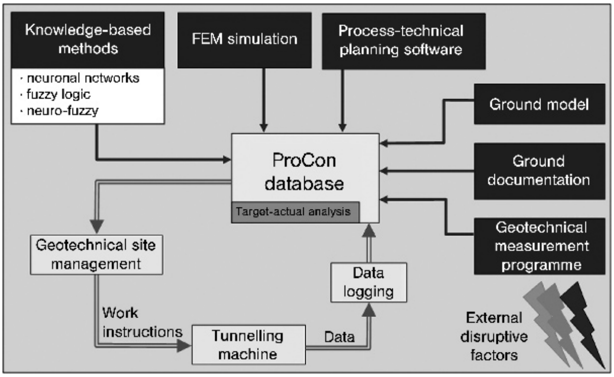


Figure 10-3 The cybernetic system as part of an expert target-actual comparison [209]

As the classic cycle of control regulation through the site management and the geotechnical specialists would not be capable of making real time decisions due to the flood of data, a future method shown in Fig. 10-3 would be to implement so-called reaction programmes or expert systems (knowledge-based methods) to aid decision making. For shield tunnelling, the following methods are particularly suitable:

– analytic mechanical, fluid mechanical and geotechnical calculations,
– expert systems,
– statistics, stochastics,
– neuronal networks,
– fuzzy logic,
– neuro-fuzzy.

Fuzzy Logic makes is possible to consider engineering know-how (expert knowledge) and intuition in data analysis and process controlling. The excellent compatibility and integration possibilities in standardised regulation and PLC systems (PLC = programmable logic control) have proved to be particularly suitable for use in shield tunnelling.

Neuronal networks (NN) are mostly used when the complexity of a problem being considered is too great or the knowledge is too unstructured. In contrast to conventional deterministic methods, closed mathematical modelling of the system is not used in this case, i.e. no statements are required about the significance of the individual variables or their interaction.

Fuzzy logic or neuro-fuzzy in particular are interesting possibilities for mechanised tunnelling as they make it possible, as shown, to implement expert knowledge gained through practice into automatic evaluation and analysis processes. As the resulting systems are based on linguistic descriptions and rules corresponding to human thinking, they can be understood and subsequently manually optimised. They also offer the advantage that they can be easily integrated into the standard hardware and software of control circuits. In addition to the analysis of data for the specification of guideline values, this makes further automation of partial processes leading to integral process control systems conceivable in the future.

Further tools, which contribute to the gaining of knowledge, can be integrated into the control circuit. The automatic advance investigation of the ground (e.g. the SSP system from Herrenknecht AG) could make an important contribution in the future. The more reliable and unambiguous the information is about the ground conditions, the easier it is to improve and calibrate the simulation model.

10.5 Target process structure

How should the decision be reached, whether a process is “running well” or not? How can the target performance be quantitatively or qualitatively defined for the individual partial processes? Different methods are used for the various partial processes as shown in Table 10-1.

FEM simulation of the shield advance is an important tool for the determination of the design specification and also serves to verify the actual system behaviour through “ back analyses”. With the assistance of FEM simulation, it is possible with the current state of the technology to display the extremely complicated interactions between ground and tunnelling machine. The FEM simulation can be structured into the following phases:

- Geometrical and process-technical modelling with realistic discretisation of all machine elements (face support, shield gap grouting, shield tail grouting etc.),
- material modelling with the selection of a suitable constitutive law (consideration of positive pore water pressures, creep effects, non-linear-elastic behaviour etc.),
- step-by-step analysis to consider the effects of different phases of construction on the stress state in the ground,
- verification of the results and plausibility checking.

Table 10-1 Procedure for the determination of the KPIs/Key Performance Indicators

Partial process	Intended process analysis	KPIs
Ground excavation (tool wear)	analytic calculation model of cuttability (balance of forces), structural calculation model based on laboratory tests	contact force, torque, penetration, skin friction, material density, cuttability index
Face support	FEM simulation, kinematic support pressure calculation, analytic calculation model for volume control	support pressure, suspension losses, foam injection pressure, density distribution
Soil conditioning	FEM simulation of the hydraulic processes, analytical calculation models	support pressure distribution, foam injection pressure, foam parameters
Annular gap grouting	FEM simulation, analytical calculation models	Grout volume under consideration of the overbreak, grouting pressure
Soil tipping	analytical calculation model (consistency development, hazardous materials calculations)	liquidity index, carbon content,-tenside/polymer concentration
Soil separation	analytical calculation models	Suspension densities, solids content, viscosity measurements, yield strength

Fig. 10-4 shows the 3D modelling of a shield tunnelling machine in the surrounding ground. The excavation chamber, shield area, annular gap grouting and segmental lining can be recognised. The model is designed so that all measured technical machine parameters, for example the support pressure at the working face, support pressure in the annular gap or the “volume loss” through overcut at the shield skin are taken into account. With this procedure, it would theoretically be possible to import measured machine data without delay directly into the model and receive selected calculation results, for example surface settlements, ground stresses, loading on the tunnel lining etc.

Optimisation of the process requires further knowledge about the fluid mechanics processes in the excavation chamber. When active face support is provided, the rheological properties of the soil in the excavation chamber are important for optimised active support pressure control, in addition to the pressure values P1, P2, P3 according to Fig. 10-5.

The material flow in the excavation chamber can also be realistically simulated using numerical models.

When active conditioning is not used, the material flow in the excavation chamber concentrates in the immediate vicinity of the screw conveyor; in the other parts of the excavation chamber, there is scarcely any flow of material. There is a danger that the soil consolidates in these areas and thus loses its flowing properties. Through process-controlled soil conditioning, the flow pattern can be reconstructed and actively manipulated or optimised. For this reason, all sensors for the control of pressure and volume are included in the analysis of the flow behaviour.

In summary, it can be stated that decisive interactions between ground and tunnelling machine can be at least qualitatively investigated through numerical simulations. The knowledge gained is used as part of the process controlling to optimise the advance. The shield tunnelling processes can thus be represented with their complex interactions between ground and machine and analysed using efficient system tools. The optimisation of the key parameters in real time and subsequently can be done through the implementation of the know-how of the experts and through knowledge-based methods like fuzzy logic and neuronal networks in the process controlling system, which makes the system capable of learning.

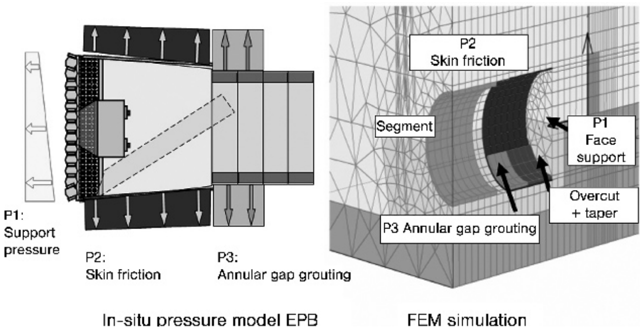


Figure 10-4 FEM model for the analysis of system behaviour [209]

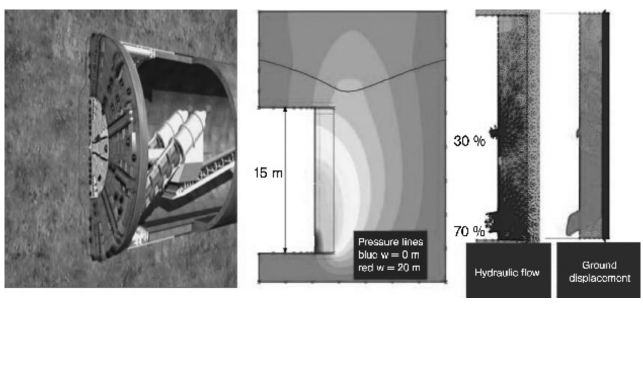


Figure 10-5 FEM model for the analysis of the hydraulic behaviour in the excavation chamber [209] (Herrenknecht EPB; Madrid M-30; D = 15.16 m)

10.6 Analysis of the actual process

The process sequence is now archived and analysed in real time by the computer according to the procedure described above. Processes with sharp criteria, for example the control of support pressure, can be clearly visualised with target curves and coloured bandwidths. Particularly advantageous is the display of the geological longitudinal section and layout plans including the display of measured settlements.

Experience shows that in case of complex interactions, the sole consideration of single KPIs (Table 10-1) does not normally lead to an unambiguous result. It is not normally the quantitative value of a single indicator that is significant but rather the relationships and the interaction of the single quality criteria. In this case, the expert knowledge of the user is indispensable.

For example, if the yield strength is too low in a shield advance with slurry-supported face, this does not necessarily represent an impairment of the stability of the face as long as the suspension has sufficient density and the suspension losses are low. The KPI yield strength must therefore always be considered in relation to the soil permeability and suspension density (Fig. 10-6). The suspension losses in turn can be verified through the development with time of the bentonite level during stoppages.

The results of knowledge-based process analyses can be provided for the responsible parties on the project in real time. As a part of the collection of evidence, the completely analysed progress of the advance is permanently archived and can be called up at any time according to time or position.

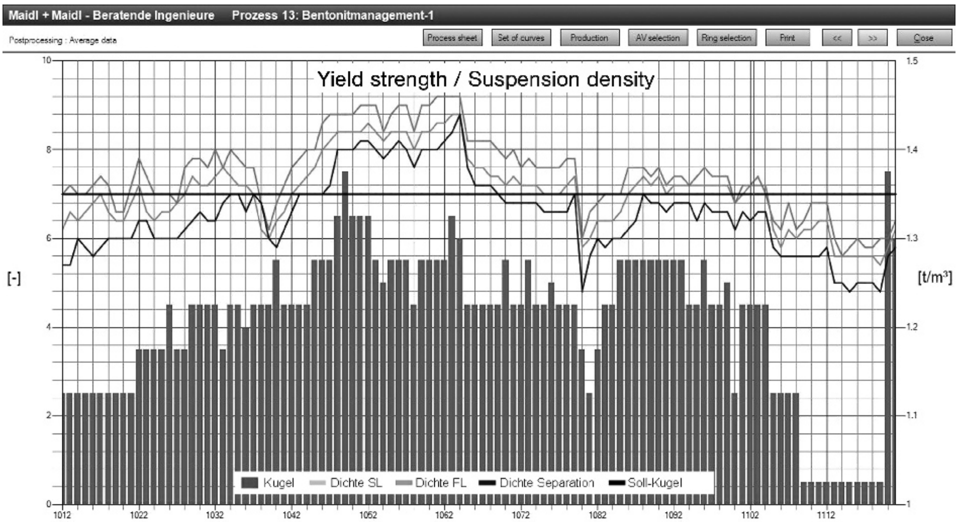


Figure 10-6 Visualisation of the KPI yield strength/suspension density [209]

11 DAUB recommendations for the selection of tunnelling machines

Chapter 11 now repeats a translation of the recommendations of the German Tunnelling Committee DAUB[1] for the selection of tunnelling machines.

11.1 Preliminary notes

The purpose of these recommendations is to provide a decision-making basis for the selection of tunnelling machines for use in rock and soft ground based on process-technical and geotechnical criteria. This takes into account the prevailing technical, local and environmental conditions, also process and machine technology. The recommendations are intended as an additional decision-making aid for the engineer, but cannot replace project-related analysis, which will remain the most important basis for the selection of a tunnelling machine.

These recommendations particularly supplement the existing recommendations of DIN 18 312 "Underground Construction Work"; VOB Part C. For the selection of tunnelling machines for pipe jacking, reference is also made to guideline DWA-A 125 of the German Association for Water, Wastewater and Waste DWA e. V.

1 The recommendations were produced by the working group „Recommendations for the selection of tunnelling machines" of the German Tunnelling Committee (DAUB).

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11.2 Regulatory works

The following documents were used for the production of these recommendations:

11.2.1 National regulations

– Supplementary Technical Conditions of Contract and Guidelines for Engineering Structures (ZTV-ING) of the BMVBS from December 2007:
  - Part 5: Tunnel Construction, section 3 “Mechanised Shield Tunnelling”.
– Guideline 853 “Design, construction and maintenance of rail tunnels” of DB Netz AG from 01/12/2008:
  - Module 853.2001 “Structural Stability Calculations” (including regulations concerning the actions from thrust cylinders of tunnelling machines),
  - Module 853.4001 “General rules for tunnelling, support and lining”,
  - Module 853.4005 “Segmental lining” (including regulations concerning annular gap grouting),
  - Module 853.6001 “Construction, construction documents and documentation” (with regulations concerning the control of shield tunnelling works),
– Worksheet DWA-A 125: Pipe jacking and associated processes,
– Regulations for working under compressed air (Compressed Air regulations),
– Regulations for health and safety on construction sites (RAB 25): further details to the Compressed Air Regulations,
– Guideline “A code of practice for risk management of tunnel works“ of the International Tunnelling Insurance Group (ITIG).

Laws and regulations concerning the use of conditioning agents

– General administrative regulations to the water supply law with the categorisation of potential water pollutants into water risk classes (VwVwS), 1999,
– General administrative regulations for the revision of the administrative regulations concerning potential water pollutants, 2005,
– Law concerning the environmental acceptability of washing and cleaning agents (WRMG), 2007.

Laws and regulations concerning landfill

– Interstate waste committee (LAGA); Note M20; Requirements for the material recycling or mineral residues/wastes – technical rules (version 6 November 2003),
– Federal Ministry for the Environment, Nature Conservation and Nuclear Safety: Regulations for the simplification of landfill law-draft, 2008,
– European Union: decision of the Council from 19/12/2002 to lay down criteria and procedures for the acceptance of waste on landfill sites according to Article 16 and Annex II of the directive 1999/31/EG.

11.2.2 International standards

- DIN EN 815/A2: Safety of unshielded tunnel boring machines and rodless shaft boring machines for rock – Safety requirements; English version EN 815: 1996/prA2: 2008,
- DIN EN 12110/A1: Tunnelling machines. Air locks – Safety requirements; English version EN 12110:2002/prA1:2008,
- DIN EN 12336:2010-03 (D): Tunnelling machines – Shield machines, thrust boring machines, screw boring machines, lining erection equipment – Safety requirements; German version EN 12336:2005,
- Code of practice for the planning and implementation of a health and safety concept for underground construction sites. Issued by: DACH; DAUB; FSV; SIA/FGU,
- SIA 198 (SN 531 198) Underground structures. Construction; Swiss Engineers and Architects Association, issue 10/2004,
- ÖNORM B 2203 Underground works – Works contract, issue 1994
- ÖNORM B 2203-2 Underground works – Works contract – Part 2: Continuous driving, issue 2005,
- Guideline for shield tunnelling from the Austrian associations B; FSV, OIAV,
- Recommendations and guidelines for tunnel boring machines (TBMs), working group No 14 Mechanized Tunnelling ITA,
- BS 6164, Code of practice for safety in tunnelling in the construction industry,
- Detergent regulations, Regulation (EC) 648/2004 of the European Parliament and Council (2004),
- Organisation for Economic Co-operation and Development (OECD), Guidelines 201–203; 301 B and 302 B: Freshwater alga and cyanobacteria, growth inhibition test, 2006; Daphnia sp. Acute Immobilisation Test, 2004; Fish, Acute Toxicity Test, 1992; Ready Biodegradability, CO_2 evolution test, 1992; Inherent Biodegradability: Zahn-Wellens/EMPA test, 1992).

11.2.3 Standards and other regulatory works

- DIN 4020: Geotechnical investigations for civil engineering purposes,
- DIN EN ISO 14688-1 (2003): Geotechnical investigation and testing – Identification and classification of soil – Part 1: Identification and description,
- DIN EN ISO 14689-2 (2004): Geotechnical investigation and testing – Identification and classification of soil – Part 2: Principles for a classification,
- DIN 18122: Soil, investigation and testing – Consistency limits,
- DIN 18130: Soil, investigation and testing – Determination of the coefficient of water permeability,
- DIN 18196: Earthworks and foundations – Soil classification for civil engineering purposes,
- DIN 1054: Subsoil – Verification of the safety of earthworks and foundations,
- DIN 18312: German construction contract procedures (VOB) Part C: General technical specifications in construction contracts (ATV) – Underground construction work
- "Minimum measures for the avoidance of injury in case of significant dangers from fire, gas ingress, water ingress and rockfall/collapse (Appendix A to the code of practice for the planning and implementation of a health and safety plan for underground construction sites, produced by the DAUB working group "Incident plans").

11.3 Definitions and abbreviations

11.3.1 Definitions

Abrasiveness The abrasiveness describes the influences resulting from the geology on the wear of tools. The abrasiveness of hard rock is often characterised with the CAI test (Cerchar Abrasivity Index) and that of soft ground often with the LCPC test (Test of the "Laboratoire Central des Ponts et Chaussées"), in addition to the mineralogical composition and strength parameters.

Active face support Measured and monitored support of the face by a suitable medium (for example slurry or remoulded earth) based on a support pressure calculation.

Air pressurisation The excavation chamber is pressurised with compressed air to hold back groundwater. Support against ground pressure is only possible in almost impermeable soil or if the face is sealed, e.g. by a filter cake.

Annular gap Cavity between the sides of the excavated cavity and the outer face of the segments.

Articulated shield Shield machine with more than one shield section, which are articulated with active steering cylinders or passive hydraulic cylinders in order to improve steering around curves.

Blowout Uncontrolled escape of compressed air to the ground surface or riverbed associated with loss of support effect.

Breasting (plates) Additional mechanical support to the face with extendable plates.

CAI (Cerchar Abrasivity Index) Value from the test of the same name for the characterisation of the abrasiveness of solid/hard rock.

Clamping units Side-mounted bracing apparatus in double shield machines intended to transfer the thrust forces radially into the surrounding rock mass, resist rolling and stabilise the tunnelling machinery.

Closed mode In closed mode, the excavation chamber of a tunnelling machine is held under a measured and monitored positive pressure. The pressure is applied by slurry, remoulded earth or also compressed air.

Cuttability The facility of excavating rock with a tunnelling machine depending on the rock properties. The most important process technology parameters for the quantitative description of the ease of boring are the cutterhead penetration and the contact pressure.

Cutterhead A tool carrier in hard rock tunnel boring machines fitted with disc cutters for full-face excavation. In soft ground, the term cutting wheel is normally used.

Cutting wheel Mechanism for the full-face excavation of a tunnel cross-section in soft ground. The ground is excavated as the wheel rotates and the design and tool equipment of the wheel are suited to the relevant ground properties. In hard rock, the term cutterhead is normally used.

Disc cutter (disc) Hard rock tool with a rotating hardened cutter ring, which can destroy the structure of the rock at the face.

Geotechnical report Description of the site investigation with characteristic values for the rock and rock mass parameters according to DIN 4020 Number 10.

Gripper Side-mounted, radially acting bracing apparatus in hard rock tunnel boring machines, intended to transfer the thrust forces into the surrounding rock mass, resist rolling and stabilise the TBM.

Ground behaviour Behaviour of the unsupported ground at the face and also at the sides of a tunnel without consideration of the construction process.

Ground profile Geometrical assumptions about the profile of natural formations or strata (DIN 4020 Appendix C2) or of homogeneous zones.

LCPC abrasiveness coefficient (ABR) Value from the test of the same name for the characterisation of the abrasiveness of soft ground or broken rock, named after the "Laboratoire Central des Ponts et Chaussées".

Liquefaction Loss of the shear strength of a soil due to positive pore water pressure.

Mechanical face support Support of the face with breasting plates.

Open mode In open mode, the excavation chamber is not under pressure.

Overcut Differential dimension between the bored radius and the shield radius measured at the shield blade. The overcut serves e.g. to improve driving round curves, to reduce the skin friction and relieve stress on the ground.

Primary wear Wear on the excavation tools solely due the excavation of the face; influenced by the strength, jointing and abrasiveness of the rock mass.

RMR Rock Mass Rating: Value for the classification of rock mass based on 6 rock mass parameters.

Rock mass behaviour Behaviour of the unsupported cavity in hard rock (referred to as ground behaviour in soft ground). The rock mass behaviour is determined by the properties of the rock and the jointing structure, the stress and formation water situation and the shape and size of the excavated cavity.

RQD index Rock Quality Designation index: value for the characterisation of rock quality based on the sum of the lengths of drill core pieces larger than 10 cm out of the total length of core taken according to ASTM D6032-02.

Surcharging Provision of an additional loading on the ground through e.g. filling above sections of tunnel with shallow overburden.

Secondary wear Secondary wear results from the rubbing and grinding action of the already excavated ground. Poor material flow and sticking increase the secondary wear.

Separation Description for the separation of fluid and solid in hydraulic material transport.

Stability Stability describes the stability of the ground including consideration of thee effect of the construction process. The stability is verified with calculations.

Standup time of the ground The length of time the ground can stand up without support. The verification through calculation of the stability of the face and sides of the excavation remain, however, decisive for the final evaluation and selection of the tunnelling machine.

Sticking Adhesion of excavated material to excavation tools and blocking of material removal passages and equipment in clay soil through adhesion, bridging, cohesion and insufficient dispersion capability.

System behaviour Behaviour of the overall system of ground/rock mass and tunnelling machine.

Temporary stability In the construction state, the temporary stability can be verified with reduced factors of safety on the actions.

11.3.2 Abbreviations

ABR Abrasiveness coefficient according to LCPC

BR Breakability according to LCPC

CAI Cerchar Abrasivity Index

DSM Double shield machine

EPB Earth Pressure Balance (support)

ETBM Enlargement tunnel boring machine, reamer

GV Rock mass behaviour

LCPC Laboratoire Central des Ponts et Chaussées abrasiveness test

RMR Rock Mass Rating

RQD Rock Quality Designation

SM Shield machine

SM-V Shield machine for full-face excavation

SM-T Shield machine for partial excavation

SV System behaviour

TVM Tunnelling machine

TBM Tunnel boring machine (for hard rock)

11.4 Application and structure of the recommendations

A procedure in seven steps is recommended for the selection of a tunnelling machine, as shown in Fig. 11-1.

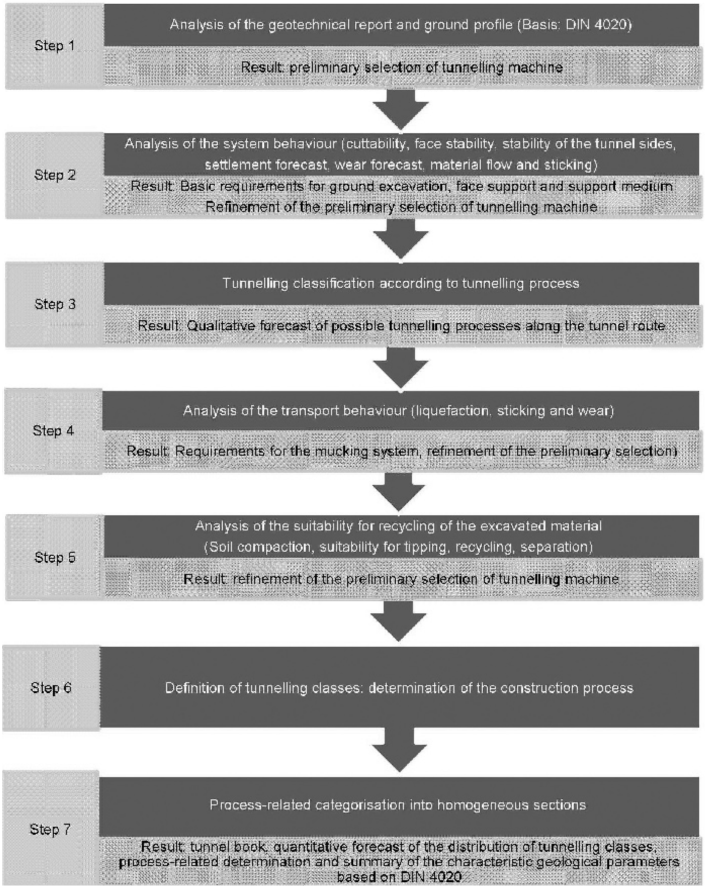


Figure 11-1 General diagram of the tunnelling machine selection process

The analysis in the geotechnical report, the ground profile (Step 1) and the system behaviour (Step 2) provide the basis for the selection of a suitable tunnelling machine. In Step 3, an exclusion procedure is used to find excavation processes, which are suitable in principle for individual tunnel sections. Taking into consideration the transport behaviour of the excavated material (Step 4) and the analysis of the recycling potential of the excavated material (Step 5), there then follows a further limitation to the optimal process for the specific technical and economic constraints of the project (Step 6). In the last step (Step 7), it is recommended to divide the tunnel into homogeneous sections for the selected process. A longitudinal section showing the tunnelling technologies is a particularly useful form of diagram, which can also be used as a contract document for construction planning and estimating and should thus also include all contract-relevant geotechnical parameters.

These recommendations define the key process-relevant geotechnical parameters for the analysis in the geotechnical report (Step 1) and are then restricted to the analysis of system behaviour (Step 2) and the preliminary selection of the tunnelling machine (Step 3). The subsequent steps should be carried out for the specific project, also considering the economic aspects.

The core of the recommendations can be found in Sections 11.5 and 11.6. In Section 11.5, the technical features of the tunnelling machines are explained and the various processes are categorised into types (Appendix 1). Section 11.6 explains the system behaviour of each machine type and includes details of the interaction between the machine and the surrounding ground. Then the significant ground/soil parameters used in the production and analysis of the geotechnical report (Step 1) are defined in relation to the processes (Appendix 2). Environmental aspects are dealt with in Section 11.7 and other significant constraints in Section 11.8. Finally, recommended applications are formulated in Section 11.9 for each type of machine based on the key parameters in the form of an application matrix (Appendix 3) for the selection of the tunnelling machine (Step 2).

11.5 Categorisation of tunnelling machines

11.5.1 Types of tunnelling machine (TVM)

Tunnelling machines either excavate the entire tunnel cross-section with a cutterhead or cutting wheel or excavate partial sections using appropriate excavation equipment.

Tunnelling machines can be differentiated into tunnel boring machines (TBM), double shield machines (DSM), shield machines (SM) and combination machines (KSM).

The machine is either continuously or intermittently driven forward as it excavates.

A systematic categorisation of tunnelling machines is shown in Fig. 11-2 (see also Appendix 1 “Overview of tunnelling machines”).

11.5.2 Tunnel boring machines (TBM)

Tunnel boring machines are used for the excavation of tunnels in stable hard rock. Active support of the face is not necessary and is not technically feasible. These machines can generally only bore circular cross-sections.

TBMs are differentiated into machines without shield skin (gripper TBM), enlargement tunnel boring machines or reamers (ETBM) and tunnel boring machines with shield skin (TBM-S).

11.5.2.1 Tunnel boring machines without shield (Gripper TBM)

Tunnel boring machines without shield are used in hard rock with medium to long standup time. They have no completely surrounding shield skin. The scope of economic application can be greatly influenced and limited by elaborate conventional support measures and the cost of wear to the excavation tools.

In order to be able to apply the contact force to the cutterhead, the machine is braced radially by plates (grippers) hydraulically driven against the sides of the tunnel. Rock is excavated, with little damage to the surrounding rock and to an exact profile, by disc cutters mounted on a cutterhead. The machine fills most of the cross-section. Systematic support of the inner surface of the tunnel is normally only carried out behind the machine (10 to 15 m and more behind the face). In less stable rock, particularly when there is a danger of rockfall, it must be possible to install steel arches, poling plates and rock bolts as close as possible behind the cutterhead.

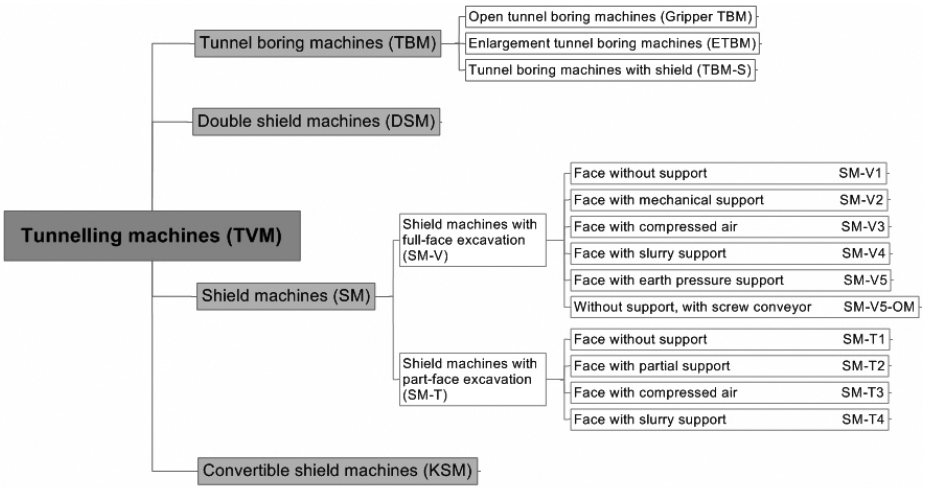


Figure 11-2 Categorisation of tunnelling machines

In case a shotcrete lining is required in the tunnel, this should first be applied in the back part of the backup in order to keep the contamination of the drive and steering units in the front part of the machine as small as possible. In exceptional cases, however, spraying of shotcrete must be possible as close behind the cutterhead as possible.

If poor rock or non-heterogeneous conditions (high degree of jointing, fault zones) are forecast, it is recommended to fit the machine with equipment for advance investigation and perhaps also rock improvement ahead of the machine.

The excavation of the face produces material in small pieces with associated dust development. These machines therefore require equipment for reducing the production of dust and for dedusting. This can be:

– spraying water at the cutterhead,
– dust shield behind the cutterhead,
– dust extraction with dedusting on the backup.

The material handling and supply of the machine require following backup facilities. These sometimes extend to a considerable length.

11.5.2.2 Enlargement tunnel boring machines (ETBM)

Enlargement tunnel boring machines (reamers) are used in hard rock to enlarge an already bored pilot tunnel to the planned final diameter. The enlargement to the full diameter is performed in one or two working stages by an appropriately designed cutterhead.

The main components of the machines are the cutterhead, the bracing and the thrust mechanism. The bracing in this special machine is ahead of the cutterhead and is supported by grippers in the pilot tunnel. The cutterhead of the machine is drawn towards the bracing as it bores. In disturbed rock formations, measures can be undertaken to stabilise fault zones from the previously bored pilot tunnel and the risks to the progress of the main tunnel can be minimised.

11.5.2.3 Tunnel boring machine with shield (TBM-S)

For rock with a short standup time or in rock liable to rockfall, tunnel boring machines are fitted with a shield skin. In this case, installation of the lining in the protection of the shield skin is appropriate (segments, pipes etc). As it advances, the machine can support itself off the lining, so the gripper equipment is usually omitted. Otherwise, the statements already made about tunnel boring machines apply correspondingly.

11.5.3 Double shield machines (DSM)

Double shield machines (DSM) consist of two parts one behind the other. The front part is equipped with the cutterhead and the main thrust cylinders, and in the back part are the auxiliary cylinders and the grippers. The front part of the machine can be extended forwards from the back part by a complete ring length with a telescoping mechanism.

In a stable rock mass, the grippers resist the torque and the thrust forces during advance. The back part of the machine is securely fixed by the grippers and the assembly of the segment ring can continue as the machine advances. In stable rock, the installation of segments may not be necessary.

In an instable rock mass, in which the grippers cannot grip adequately, the advance can be supported from the last ring of segments. The front and back parts of the machine in this case are telescoped together and the thrust forces are transferred to the segment ring by the auxiliary cylinders.

It is usually not possible to provide active support to the face and the excavated sides of the tunnel.

The rapid advance of the back part of the machine to reposition the grippers after the completion of a boring stroke means that the rock mass must be able stand up without support until the annular gap has been fully grouted or filled with pea gravel.

11.5.4 Shield machines (SM)

These are differentiated into shield machines for full-face excavation (with a cutting wheel; SM-V) and shield machines for partial excavation (with a cutting head, excavator; SM-T). Shield machines are used in soft ground above and below the groundwater table and the ground around the excavated cavity and the face normally have to be supported. Shield machines can be further categorised according to the type of face support (Fig. 2).

11.5.4.1 Shield machines for full-face excavation (SM-V)

Face without support (SM-V1)

If the face is stable, e.g. in clay with stiff consistency and sufficient cohesion or in solid rock, open shield machines can be used. A cutting wheel fitted with excavation tools excavates the ground and the excavated material is cleared on a conveyor belt.

In rock liable to rockfall, shields are mostly used, which are fitted with a largely closed cutterhead fitted with disc cutters and fully protected from instable ground by a shield skin. The thrust forces and the cutterhead torque are transferred to the last ring of segment by the thrust cylinders.

Face with mechanical support (SM-V2)

With tunnelling machines with mechanical support, the face is supported during excavation by elastically supported support plates arranged in the openings of the cutting wheel (between the spokes). In practice, however, experience shows that that no appreciable mechanical support of the face can be provided by the rotating cutting wheel. For this reason, this type of cutting wheel did not prove successful in unstable ground and is no longer in use today. Mechanical face support by the cutting wheel or the support plates should only be considered a supplementary safety measure and the supporting effect should not be taken into account in calculations to verify the stability of the face.

Face with compressed air support (SM-V3)

Shield machines of type SM-V3 can be used below the groundwater table even if it cannot be lowered or groundwater lowering is not allowed. In this case, the water at the face must be held back by compressed air. A precondition for the displacement of groundwater is the formation of an air flow to the surface. Impermeable strata above the tunnelling machine can retain the applied air and prevent effective displacement of the water (and thus the formation of an air flow). The permeability limit of the surrounding ground is therefore significant.

As no pressure difference can be built up at the face, compressed air cannot generally provide support against earth pressure, which applies particularly in permeable soil. The loss of apparent cohesion in non-saturated soil is also possible.

For the duration of tunnelling work, either the entire tunnel is pressurised or the machine is provided with a pressure bulkhead to maintain the excavation chamber under pressure. In both cases, air locks are required. Particular attention needs to be paid to compressed air bypassing the shield tail seal and lining. The recommendations and requirements for working under compressed air should complied with.

Any additional support of the face provided by the cutting wheel or support plates should be regarded solely as an additional security. It is not permissible to consider the supporting effect in calculations to verify the stability of the face.

Face with slurry support (SM-V4)

Tunnelling machines with slurry support provide support to the face through a pressurised fluid, which is specified depending on the permeability of the surrounding ground. It must be possible to vary the density and viscosity of the fluid, and bentonite suspensions have proved particularly successful for this purpose. In order to support the face, the working chamber is closed from the tunnel by a pressure bulkhead.

The required support pressure can be regulated very precisely with an air bubble behind a submerged wall and by adjusting the output of the feed and slurry pumps. The required and the maximum support pressures over the entire length to be bored should be calculated before the start of tunnelling (support pressure calculation).

The soil is excavated from the full face by a cutting wheel fitted with tools (open-mode or rimmed wheel) and removed hydraulically. Subsequent separation of the removed suspension is essential.

If it is necessary to enter the excavation chamber, for example to change tools, carry out repair work or to remove obstructions, the support slurry has to be replaced by compressed air. The support slurry then forms a low-permeability membrane on the face, which however is of limited durability (risk of drying out). The membrane permits the support of the face by compressed air and may need to be renewed regularly. The support slurry can be completely (empty) or only partially (lowering) replaced by compressed air. The maximum partial lowering is limited particularly by the requirement for sufficient working space. This should be chosen so large that safe working is possible at all times and an adequately large space is available for the workers to retreat.

If an open cutting wheel is used, it should be possible to mechanically close the face with shutter segments in the cutting wheel or with plates, which can be extended from behind, in order to protect the personnel working in the excavation chamber while the machine is stopped, which is also sensible due to the limited duration of the membrane effect.

Stones or rock benches can be reduced to a size that can be removed by discs in the cutting wheel and/or crushers in the working chamber.

In stable ground, the slurry shield can also be operated in open mode without pressurisation, with water being used for muck removal.

Any additional mechanical support of the face provided by the cutting wheel or support plates should be regarded solely as additional security and it is not permissible to consider the supporting effect in calculations to verify the stability of the face.

Face with earth pressure support (SM-V5)

Tunnelling machines with earth pressure balance support provide support to the face through remoulded excavated soil. The excavation chamber of the shield is closed from the tunnel by a pressure bulkhead. A cutting wheel, fitted with tools and more or less closed, excavates the soil. Mixing vanes on the back of the cutting wheel (rotors/back buckets) and on the pressure bulkhead (stators) assist the remoulding of the soil to a workable consistency. The pressure is checked with earth pressure cells, which are distributed on the front of the pressure bulkhead. A pressure-tight screw conveyor removes the soil from the working chamber.

The support pressure is regulated by varying the revolution speed of the screw conveyor or through the injection of a suitable conditioning agent controlled according to pressure and volume. The pressure gradient between excavation chamber and tunnel is provided by friction in the screw conveyor. If the soil material in the screw conveyor cannot ensure the sealing of the discharge device, an additional mechanical device has to be installed. Complete support of the face, in particular the upper part of it, is only successful if the soil acting as a support medium can be remoulded to a soft or stiff-plastic mass. This is particularly influenced by the percentage content of fine grains smaller than 0.06 mm. The scope of application of earth pressure shields can be extended using soil conditioners such as bentonite, polymers or foam, but attention needs to be paid to the environmentally acceptable disposal of the material.

In stable ground, the earth pressure shield also can be operated in open mode without pressurisation with a partially filled excavation chamber (SM-V5-OM). In stable ground with water ingress, operation is also possible with partially filled excavation chamber and compressed air.

If the groundwater pressure is high and in ground liable to liquefaction, the critical location of the transfer of material from the screw conveyor to the conveyor belt can be replaced by a closed system (pumped material transport).

Any additional mechanical support of the face provided by the cutting wheel or support plates should be regarded solely as additional security and it is not permissible to consider the supporting effect in calculations to verify the stability of the face.

11.5.4.2 Shield machines with partial face excavation (SM-T)

Face without support (SM-T1)

This type of shield can be used with a vertically or steeply sloping and stable face. The machine consists of a shield skin and the excavation tools (excavator, milling head or ripper), the spoil removal equipment and the thrust cylinders. The excavated material is removed on a conveyor belt or scraper conveyor.

Face with partial mechanical support (SM-T2)

For partial support of the face, working platforms and/or breasting plates can be used. In platform shields, the tunnelling machine is divided into one or more platforms at the face. Natural slopes form on these, which support the face. The ground is excavated manually or mechanically. Platform shields have a low degree of mechanisation.

A disadvantage is the danger of large settlement resulting from uncontrolled face support. In shield machines with face support, the face is supported by breasting plates supported on hydraulic cylinders. In order to excavate the soil, the breasting plates are partially withdrawn. A combination of breasting plates and platforms is also possible. If support of the crown alone is sufficient, hinged breasting plates can be fixed at the crown.

Face with compressed air support (SM-T3)

The use of this type of machine (Fig. 1-12) is appropriate when types SM-T1 and SM-T2 are used in groundwater. Either the entire working area is pressurised, including the already excavated tunnel, or the machine is fitted with a pressure bulkhead (comparable to type SM-V3). The excavated material is transported hydraulically or through the lock in dry form.

Face with slurry support (SM-T4)

Many attempts have been made in the past to achieve face support using a support medium with partial face machines (e.g. thixshield). The excavation chamber in this case has to be completely filled. The soil can be excavated mechanically or by high-pressure jets.

As the excavation of the soil cannot be controlled sufficiently, this method of tunnelling did not prove successful and is no longer used.

11.5.5 Adaptable shield machines with convertible process technology (KSM)

Numerous tunnels run through very changeable ground conditions, which can range from rock to loosely consolidated soil. The process technology therefore has to be adapted to suit the geological conditions and appropriate adaptable shield machines have to be used. The various types are:

a) Shield machines, which can be operated with a different process without rebuilding:

– earth pressure balance shield SM-V5 ↔ compressed air shield SM-V3

b) Shield machines, which can be operated with a different process but have to be rebuilt. The following combinations have been tried:

– slurry shield SM-V4 ↔ shield without support SM-V1
– slurry shield SM-V4 ↔ earth pressure balance shield SM-V5
– earth pressure balance shield SM-V5 ↔ shield without support SM-V1

The rebuilding work normally lasts several shifts.

11.5.6 Special types

11.5.6.1 Blade shields

In blade shields, the shield skin is split into blades, which can be advanced independently. The ground is excavated by partial face machinery, cutting wheel or excavator. An advantage of blade shields is that they do not have to be circular and can, for example, drive a horseshoe-shaped section, in which case the invert is normally open. This is described as blade tunnelling. Because of various negative experiences in the past, however, blade shields are seldom used today.

11.5.6.2 Shields with multiple circular cross-sections

These shields are characterised by overlapping and non-concentric cutting wheels. The type of machine is currently only offered by Japanese manufacturers and mostly used to drive underground station cross-sections. The machines are difficult to steer and have not yet been used in Europe.

11.5.6.3 Articulated shields

Practically all types of shields can be equipped with an articulated joint to divide their length. This is provided particularly when the length of the shield skin is longer than the shield diameter in order to make the tunnelling machine easier to steer. The layout can also be necessary to drive very tight radius curves.

The description of the tunnelling machines is then according to one of the categories described above. A separate category of “articulated shields” is no longer usual.

11.5.7 Support and lining

With the tunnelling methods described here, the tunnelling machine and the support or lining are a combined process. For this reason, the most important support and lining methods are now described.

More detailed information about the various types of support and lining measures can be found in the appropriate standards, guidelines and recommendations (see Chapter 11.2).

11.5.7.1 Tunnel boring machines (TBM)

Due to the excavation process being relatively gentle to the surrounding rock mass and the favourable circular form, the extent of support measures is normally less than, for example, drilling and blasting. In less stable rock, the exposed surfaces have to be supported in good time in order to limit the loosening of the rock mass and thus mostly preserve the quality of the rock mass. If rupture occurs at the cutterhead, the extent of the support measures required can increase greatly.

Rock bolts

Rock bolts are normally installed radially in the cross-sectional plane of the cavity, and a layout oriented on the jointing can increase the effect of shear dowelling. If installed locally, they can hinder the spalling or breaking out of rock slabs, and if installed in a pattern they can reduce the loosening of the exposed sides of the tunnel. Rock bolts are particularly suitable for subsequent increasing of the support resistance, as they can also be installed later. The rock bolts are normally installed from the working platform behind the machine, or in special cases also directly behind the cutterhead shield.

Shotcrete

Shotcrete support is normally applied from a working platform at the back of the backup. The shotcrete serves to partially or completely seal the exposed surface of the rock mass (thickness 3 to 5 cm) or provide a load-bearing layer (thickness 10 to 30 cm). In order to increase the load-bearing action of the shotcrete layer, this is reinforced with one layer (rock side) or two layers (rock and air sides) of mesh reinforcement. Alternatively, steel fibre shotcrete can be used. The use of shotcrete robots enables high rates of spraying and is particularly beneficial from the point of view of health and safety.

Support arches

Support arches provide effective support to the rock mass and protection for the working space immediately after excavation and the installation of the arches. They are therefore mostly used in rock, which is liable to rockfall, instable or has squeezing characteristics. Either rolled steel profiles or lattice beams sections can be used for support arches. They are normally installed immediately behind the cutterhead in partial sections in the crown or as a closed ring.

11.5.7.2 Tunnel boring machines with shield (TBM-S), Shield machines (SM, DSM, KSM)

With tunnel boring machines with shield, or shield machines, the final support is installed in the protection of the shield skin or else the shield machine is operated at the front of a jacked pipe.

Precast elements installed in the shield tail (segments) serve to support the surrounding ground and as the abutment for the thrust force. The structural bond between the lining and the ground is created by grouting of the shield track as continuously as possible.

Segments and pipes are often used as a single-layer lining.

Concrete and reinforced concrete segments

The prefabricated elements used today are mostly precast concrete or reinforced concrete segments. The loading on the segment during transport and installation is often sufficient to require the installation of steel bars as reinforcement. Alternatively, segments with steel fibre reinforcement or a combination of rebars and fibres can be used. Steel fibres are particularly useful for the strengthening of the edges and corners, which are difficult to reinforce sufficiently with rebars.

SGI lining and steel segments

Spheroidal Graphite Iron (SGI) lining are now scarcely ever used because of the cost and fire protection problems. When the ground conditions are especially difficult, particularly if the bedding is poor, there is a danger of high deformation (convergence) or ring offsets. Welded steel segment rings, which are stiff in bending, are also often used to cope with the unusually high and asymmetrical loading at crosscuts, niches and other openings.

Hybrid segments

The hybrid segment is a combination of reinforced concrete and steel segments and offers an economic alternative to the use of full steel segments. These can be welded steel compartmented constructions filled with concrete or conventional reinforced concrete segments with integrated steel boxes bolted into the longitudinal or ring joint. This increases the stiffness of the system and the deformations are reduced.

Extruded concrete

Extruded concrete is a concrete tunnel lining, which is placed in a continuous process as unreinforced or fibre-reinforced concrete behind the tunnelling machine between the shield tail and a travelling inner formwork. The extruded concrete thus already supports the surrounding rock mass in the wet state. The use of extruded concrete is also possible below the groundwater table. Elastically supported face formwork, which is pushed forwards by the wet concrete pressure, ensures constant support pressure in the wet concrete.

Timber lagging

In ground without water, the primary support can consist of timber or steel lagging, which is installed between steel profiles in the protection of the shield tail (ribs and lagging).

After the steel profiles have left the shield tail, they are braced against the ground by hydraulic cylinders, thus providing support. The tunnelling machine can be pushed forward against this braced support.

This method of support is not used in Europe due to the lack of fire protection during the construction period.

Pipes

Pipe jacking is a special process, in which reinforced concrete or steel pipes are jacked forward from a shaft and serve as support and final lining. These are normally circular but the use of rectangular sections is also possible.

Reinforced concrete

As with shotcrete, in-situ reinforced concrete can also be used with tunnelling machines to support the sides of the tunnel. As no thrust force can be transferred to the support, this type of support is only used with blade shields. The reinforced concrete is placed conventionally with a travelling formwork unit in 2.50 to 4.50 m wide sections in the protection of the following blades, which are still in contact with the last section to be concreted.

This process is no longer used in Central Europe on grounds of cost.

11.5.7.3 Advance support

The use of advance support measures, which are installed in the ground from a shield tunnelling machine, should only be used for short sections in emergency, since the implementation is technically laborious due to the poor accessibility and uneconomic due to the interruption of tunnelling advance. All possibilities of providing ground improvement from the surface should be exhausted first.

It is generally possible with current technology to implement rock bolts, pipe screens, drilled grouting and inclined or horizontal high-pressure grouting.

In order to make this possible, the tunnelling machine should be supplied with the necessary equipment, since later installation of drilling equipment is very expensive. Drilling booms are usually mounted on the segment erector and can drill through inclined pipes passing forwards through the shield skin (minimum angle to the shield centreline: about 8°).

Holes can also be drilled into the face through sealed openings in the pressure bulkhead. It should, however, be noted that broken drilling rods, which cannot be recovered, will lead to a severe obstruction of further advance.

The production of closed grouting bodies from the machine should not be provided, as this process is not practical for geometrical reasons. There is a basic risk in grouting that the grout can penetrate uncontrolled into the annular gap or the excavation chamber and thus lead to a failure of the face. When consolidation of the face is required, the excavation chamber should therefore be filled with a soil substitute first.

11.5.7.4 Support next to the tunnelling machine

If the ground is insufficiently stable, a tunnelling machine with active support and a shield should be selected. Next to the tunnelling machine, the ground is then supported by the shield skin, which however also completely obstructs access to the ground. Better passive support is provided by a short cylindrical shield than by a longer tapered shield.

11.6 Ground and system behaviour

11.6.1 Preliminary remarks

The system behaviour is of essential importance for the selection of a tunnelling machine, i.e. the behaviour of the overall system consisting of ground and selected tunnelling process [Austrian standard ÖNORM B 2203-2]. When a tunnelling machine is used, the ground behaviour criteria are fundamentally different from those in conventional tunnelling.

The geotechnical investigations are generally carried out based on DIN 4020. The determination of the characteristic values, the display and evaluation of the results of the geotechnical investigation and the conclusions, recommendations and notes should already be matched to the (probable) later tunnelling process early in the design phase.

More extensive and meaningful the preliminary investigations provide better preconditions for the selection of a process and a tunnelling machine. In this regard, it is recommended to include consideration of the entire process chain from excavation of the face, clearance of the muck and the final tipping or recycling of the excavated material in the planning of the geotechnical investigation.

The essential geotechnical parameters are summarised in Appendix 2 for each process and can serve as a rule of thumb for the selection of a tunnelling machine. They should be determined on each project for the relevant ground conditions. It should be noted that deviations of these ground parameters from the assumed values can result in complex and inconsistent consequences for the process chain. It is therefore recommended to provide appropriate provisions in the contract.

It is helpful and practical to display the expected ground conditions in a geotechnical longitudinal section and assign sections to relevant tunnelling classes.

There now follow basic remarks about the process-oriented analysis of system behaviour. A summary of the required characteristic values – split into soft ground and hard rock – is given in Appendices 2.1 and 2.2.

11.6.2 Ground stability and face support

The stability of the ground is the primary criterion for the selection of a type of tunnelling machine. The basis is the global and local stability of the face.

For an initial approximate evaluation of the stability (Austrian standard ÖNORM B2203, 1994, Tab 1), the following assignments can be defined according to the RMR classification:

A1 “stable”:	RMR 81–100
A2 “liable to rockfall”:	RMR 61–80
B1 “brittle”:	RMR 51–60
B2 “very brittle”:	RMR 41–50
B3 “non-cohesive”:	RMR 21–40
C squeezing rock”:	RMR < 20

For tunnels under built-up areas, a statement should also be made about the expected ground deformation or surface settlement with appropriate verification through calculations.

11.6.3 Excavation

The advance rate depends not only on the characteristic values of the ground but also on the selection of excavation tools, the geometry and design of the cutterhead/cutting wheel and the operating parameters of the machine. Changes in the geotechnical parameters can be unfavourable but also favourable for the advance. Because of the extremely complicated interactions between ground and tunnelling process, detailed analyses should be performed to clarify the causes.

Sticking in the excavation chamber and increased wear on the excavation tools in particular are the most frequent causes of disappointing progress and increased costs. These are now described in detail.

11.6.3.1 Sticking

The inclination of the soil to stick can have a decisive effect on the advance rate in mechanised tunnelling. Sticking reduces the advance rate because, for example, the excavation chamber of slurry-support machines has to be flushed or time-consuming manual cleaning leads to unplanned stoppages. In addition, sticking in combination with a high content of minerals liable to cause wear can lead to heavy wear on the cutting wheel and excavation chamber. Any propensity of the ground to stick should therefore always be described in geotechnical reports.

Soft ground with clay content, but also solid rock containing clay minerals, can result in considerable delays through sticking. Clays with pronounced plasticity and sedimentary rocks containing clay, like for example conglomerates/breccias with clay mineral content, siltstones and particularly claystones have proved particularly susceptible to sticking. Sticking often occurs in combination with water, which can come from natural formation water with open and earth pressure balance machines or process water (support suspension, soil conditioning, cutterhead jetting in hard rock).

The hindrance of progress through sticking can best be countered by recognising a potential sticking problem before the start of construction and appropriately adapting the equipment of the machine and the planned advance rate to take the problem into consideration. Geotechnical reports should provide the following information in this regard:

– Determination of the Atterberg limits and the consistency of the sol as an indication of the sticking potential according to DIN 18 122 for soft ground,
– Clay mineralogical analyses for the determination of the content of the most significant minerals (montmorillonite, kaolinite, illite, smectite, quartz etc.),
– Closer pattern of investigation in areas containing clay minerals for the more precise determination of the affected sections and the content of clay constituents at the face.

11.6.3.2 Wear

The wear on excavation and mucking components depends on the abrasiveness of the ground, the type of mechanical loading, the selection of tool materials and the operating parameters of the machine.

In soft ground, the mineralogical composition and the strength are relevant for tool wear but also the grading distribution, the grain shape and particularly the content of boulders and blocks. The test of the Laboratoire Central des Ponts et Caussées (LCPC test) offers one method of evaluating the abrasiveness of samples of soft ground with various mineralogical compositions and also takes the breakability of the grains into account. The verbal classification based on the ABR value used in the tables in Appendix 3 was not intended specifically for mechanised tunnelling and is currently being checked and revised in research programmes. Wear forecasts should therefore not be based on a verbal description of the abrasiveness shown by the values in the tables in Appendix 3, but use the index value (ABR value). In addition, the mineralogical composition, cutting wheel design, type of tool and process-related aspects of the excavation process should be taken into account.

In rock, wear can vary widely depending on rock strength, mineralogical composition, jointing and tunnel orientation to the texture of the rock mass. The Cerchar Abrasiveness Index (CAI) classifies the abrasiveness of rock. The most important parameters are the equivalent quartz content and the rock strength. High rock strength and correspondingly high CAI values lead to high primary wear in compact rock. In case abrasive, hard to break rocks are loosened out of the rock mass in an uncontrolled manner, the wear can increase over-proportionately due to impermissible shock loading. If the material flow is poor due to sticking or the design of the cutting wheel is unfavourable to material flow, a further increase of wear is likely (secondary wear). Further factors, which determine wear, are: brittleness, ductility, grain size, texture, porosity, mineral hardness, any foliation, the design of tools /cutting disc spacing, disc cutter diameter etc.), the materials used for tools, mode of operation and tool management (checking and replacement cycle).

In coarse- and mixed-grained soils, the primary wear is mainly determined by the breakability and strength of the coarse-grained fraction, boulders and blocks. The secondary wear increases with increasing equivalent quartz content and deterioration of the material flow and ease of excavation, particularly in wide-graded grain mixtures. Depending on the type of tunnelling machine used and the tools fitted, it is necessary to investigate whether breaking and grinding processes will occur in order to estimate wear rates.

11.6.3.3 Soil conditioning

The addition of additives in liquid or powder form, suspensions or water can be used to modify the properties of the excavated material. The concentration of the conditioning agent used can be estimated from experience and the characteristic values of the ground. The design of the cutting wheel, technical machine parameters and the required support pressure also have to be considered. Products should be chosen so that they do not flow uncontrolled into the surrounding ground but enable a homogeneous soil mixture.

For shield machines with slurry support to the face, conditioning can be in the form of liquid additives. For shield machines with earth pressure-supported face, conditioning in non-cohesive soft ground is normally provided by tenside foams with the possible addition of polymers, while in cohesive soft ground, polymer, bentonite or clay suspensions or even water can be used.

The purpose of any conditioning agent is the alteration of the properties of the excavated material to ensure the most trouble-free and economic tunnelling possible. For shields with slurry-supported face, this can mean that sticking and separation in the excavation chamber are reduced or prevented. For shield machines with earth pressure-supported face, non-cohesive soil can be processed into a plastic material by conditioning, sandy clays can be conditioned for less abrasive properties and in clay, conditioning is often used to reduce sticking and adhesion problems.

The additives added to soft ground should comply with the following minimum criteria:

- Simple and controllable dosage (ensured by the use of liquid additives),
- Avoidance of blockages in the additive feed and in the pipeline pumping the conditioned material out of the excavation chamber,
- Rapid development of effectiveness, in order to be able to react to geological alterations,
- Avoidance of environmental hazards.

11.6.3.4 Soil separation

On a tunnel project with slurry-supported face, the soil is separated from the transport medium (typically bentonite suspension) in a separation plant. Boulders and gravely and sandy soil contents are mechanically removed from the suspension on screens (coarse stage), cyclones and oscillating dewaterers (medium stage). Grain sizes below the sand fraction are separated from the suspension by chamber filter presses, centrifuges or high-performance cyclones (fine stage). Separation in centrifuges is improved by the previous addition of flocculants.

The configuration and dimensioning of the separation plant is mainly based on the grading distribution and the suspension circulation quantity. It should be borne in mind that ground improvement measures and breaking and grinding processes during excavation of the soil can increase the fines content and can worsen the properties of the suspension. High suspension densities and abrasive minerals increase wear to the excavation tools and the hydraulic mucking equipment.

11.6.3.5 Soil transport and tipping

In order to fully investigate cost-effectiveness for the selection of a tunnelling machine, muck transport and tipping also have to be considered. The characteristic parameters of the ground can be significantly altered by excavation, any soil conditioning and the individual control of the tunnelling machinery.

Uncontrolled ingress of formation water in shields without active face support can lead to liquefaction of the muck, which should be taken into account in the planning of transport and tipping.

Further information about tipping and conditioning is contained in Section 11.7.

11.7 Environmental aspects

Outside factors, which do not derive from the system behaviour (ground/tunnelling process, see Section 11.6), can sometimes also influence the selection of a tunnelling machine. Particularly when two different processes of equal technical value are possible, the factor "environmental impact" can be decisive. Particularly the suitability of the excavated material for recycling or landfill can be of great significance. The soil conditioning used with EPB shields, such as the addition of foams or polymers, can rule out the filling of the material in certain landfill sites.

Conditioning

The purpose of conditioning agents is described in Section 11.6.3.3. They only penetrate slightly into the subsoil, or not at all, but are transported out of the tunnel with the muck and thus have a significant effect on the suitability of the material for recycling or tipping.

Conditioning agents can be classified into various categories. These include water pollution classes (WGK 0, WGK 1, WGK 2, WGK 3), degradability (min. 60 % primary biological degradability and min. 80 % biological degradability) and the toxological threshold values for mammals (LD50) and water organisms (EC50).

Because of the wide range of conditioning agents, the composition of soft ground and its properties, no general classification of conditioned soil is possible. It is necessary to investigate on a case-by-case basis which threshold values are complied with and how the conditioned material should be processed. Information is given in the regulations in Section 11.2. According to the threshold, applicability of a conditioning agent can be so severely limited that the result can affect the tunnelling process (see also suitability for tipping).

Separation

In the separation plant, the excavated soil is separated from the transport medium, as described in Section 11.6.3.4.

When bentonite suspension is used as a transport medium, some residual bentonite content will always remain in the separated soil. This bentonite does not, however, alter the LAGA class (see below) of the soil. Recycling of separated soil is therefore possible, or not possible, according to the LAGA class of the excavated soil.

The separation of the fines in centrifuges is assisted by the previous addition of flocculants. Because of the number of flocculants available on the market, no general statements about the environmental acceptability of these products are possible. Information on this point can be found in the safety data sheet supplied by the manufacturer.

The material produced by centrifuges, filter presses and high-performance cyclones is very fine-grained and mostly of a pasty consistency. Recycling is therefore impractical, so the material has to be tipped. The same applies to used bentonite suspension, which should be passed to appropriate plants as liquid waste.

Working in groundwater

Bentonite suspension and additives for soil conditioning have both been used for many years in tunnels below the groundwater table. The same applies to the grout used for grouting the annular gap and the biologically degradable grease used for the sealing of the annular gap.

Tipping

The material removed from the tunnel should be processed and recycled if at all possible. If this is not possible, the soil will have to be tipped. When conditioning agents are used, attention should be paid to whether the excavated and conditioned soil complies with the chemical and physical requirements for tipping.

The tipping of conditioned soil is regulated in Germany by the guidelines of the Länderarbeitsgemeinschaft Abfall (LAGA) (States working collaboration on waste) and particularly by Guideline 20 "Requirements for the material recycling of mineral residues/wastes – Technical rules". This guideline governs the recycling of excavated soil and thus the tipping of excavated material from tunnelling. Only when the analytically determined value of chemical content rules out open tipping according to LAGA Guideline Nr. 20 (tipping classes Z0 to Z2), does the material have to be tipped in a regulated landfill site or even a tip for special waste (tipping classes Z3 to Z5). This is regulated in the Technical instructions for recycling, treatment and other disposal of municipal waste (TA Municipal waste). For ecological reasons, unrestricted or restricted open tipping is preferable.

Material from the coarse and medium stages of separating plants can normally be recycled. The fines content is mostly less than 5 %. However, these soils have special soil mechanics properties. The bentonite residue can swell again on contact with water and result in material with similar properties to cohesive soil. The material should therefore only be tipped in locations protected from water. For example, it can be used for backfilling a road tunnel beneath the carriageway. Alternatively, a further stage can be provided in the separating plant to wash the material. The simplest method is to spray the soil with water on the oscillating dewaterer. This can significantly reduce the residual bentonite content in the soil, which increases the quality of the soil and the possibilities for recycling.

Muck transported as sludge and the product of band filter presses and centrifuges cannot generally be tipped without further processing, as it tends to plastic flow. Conceivable ways of improving the suitability for tipping are consolidation through the addition of lime or storing on an intermediate stockpile until the material has dried and thus gained strength.

Concerning the suitability for tipping of soils, which have been treated with additives, the information about biological degradability supplied by the manufacturer is not sufficient on its own. The relevant regulations concerning pollutant content for each tipping class should be complied with to check the permissibility of tipping, particularly with regard to the residual content of hydrocarbons. Not least for economic reasons, the use of additives with slurry shields and earth pressure balance shields should be reduced to a minimum.

The soil to be tipped should already be classified into various categories during the design phase. This could be assignment to the classes according to LAGA (Z0, Z1, Z2, Z3, Z4, Z5) and the tipping classes (DK 0, DK I, DK II, DK III).

11.8 Other project conditions

In addition to the requirements resulting from the ground conditions and the location of the project in the surroundings, questions concerned with legal approvals and health and safety can also influence the selection of a tunnelling machine, and some of these are discussed below. This is not an exclusive list but examples are given, which could be of importance for the selection of a tunnelling machine in practice.

Planning decisions, requirements under water protection laws

The requirements and conditions of official approval, as can be attached to a planning decision, often restrict the selection of a tunnelling machine. For example, the temporary pumping of groundwater and the resulting lowering of the groundwater table may be limited or even forbidden, so that a tunnelling machine capable of operation under water pressure has to be used instead of an open machine. Another aspect could be conditions regarding the discharge of water into sewers or rivers.

Settlement and tunnelling beneath buildings

In urban areas, particularly when tunnelling beneath buildings and infrastructure, the permissible deformation of the ground at the surface is normally limited. In addition to the maximum absolute value of deformation, the extent and gradient of the settlement trough are to be included as criteria. Considering these "threshold values", which have to be calculated during the design phase, a suitable tunnelling machine should be selected to comply with the limits.

Material transport, restriction of construction traffic

The material can be transported in the tunnel by rail, truck, hydraulic slurry transport, sludge transport or on conveyor belts. In addition to the available space in the cross-section of the tunnel, the selection of a method is mostly based on the tunnel length, the possibilities of vertical transport in shafts and follow-up transport on the surface.

For transport above ground, the varying degrees of nuisance for local inhabitants are often significant (particularly in inner-city areas). The permissible limits for emissions, construction traffic restrictions (e.g. a night transport ban), duration of traffic disruption and vibration are the essential factors for transport, and these normally have to be considered in the approval process.

Occupational health and safety

The regulations concerning the protection of health and safety on construction sites (Baustellenverordnung – BaustellV) in addition to the workplace regulations serve to implement the Council Directive 92/57/EEC concerning the minimum health and safety regulations for construction sites, which are of limited duration or mobile. These regulations apply to all construction sites and thus apply to underground construction. Their application means that health and safety has to be considered in the design phase, which can well affect the selection of a tunnelling process.

The "Code of practice for the planning and implementation of a health and safety plan for underground construction sites" from DAUB and the national tunnelling associations of Austria and Switzerland (D-A-CH) is based on the regulations mentioned above among others and includes detailed requirements for the operation of tunnelling machines. In order to evaluate safety at work, a risk analysis is to be produced including consideration of the construction process and local conditions. The results of this risk analysis are then included in the decision process to select a tunnelling machine with a heavy weighting.

If, for example, the occurrence of gas like methane or argon in the ground is to be expected, the construction ventilation must be designed to cope with it or firedamp-safe machinery will have to be used. The presence of asbestos in the rock also demands special attention; appropriate continuous monitoring measuring devices should be installed permanently in the machine and in the tunnel and combined with an optical and acoustic alarm, which is activated automatically in case a critical value is measured. Closed machine types with active face support (SM-V 5, SM-V4) with closed material transport systems are advantageous, and the requirements for the sealing of segment gaskets should be defined. A two-layer lining system should be considered for the completed tunnel.

11.9 Scope of application and selection criteria

The recommendations for the scope of application and selection criteria are summarised for each type of machine in Tables 1 to 11 (Appendix 3).

11.9.1 General notes about the use of the tables

The feasibility of a system is first evaluated based on the key geotechnical parameters and processes, and economic evaluation criteria remain largely ignored. The tables are suitable for a preliminary selection on the exclusion principle. In case more than one type of machine would be possible, the final overall evaluation of suitability is then undertaken after an analysis of all project-specific parameters and processes, including consideration of economic and environmental aspects.

11.9.1.1 Core area of application

The fields marked black (Symbol "+") denote ranges, in which the type of machine has already been successfully used without many supplementary measures being required. The technical performance of the machine can vary among manufacturers, and the experience of the contracting company is also significant. The main areas of application shown for one parameter may be extended or restricted by the inclusion of other parameters.

11.9.1.2 Possible areas of application

The use of a tunnelling machine in the fields marked dark grey (Symbol "0") may require special technical measures, but the technical feasibility has been demonstrated. The achievable advance rates and cost-effectiveness may be reduced in comparison to the core area.

11.9.1.3 Critical areas of application

The use of a tunnelling machine in the fields marked light grey (Symbol "–") will probably require considerable additional measures or modification of the ground, otherwise difficulties should be expected. The achievable advance rates and cost-effectiveness will be considerably reduced in comparison to the core area. A founded analysis of the technical, economic and contractual risks and a comparison of variants with other tunnelling processes are strongly recommended.

11.9.1.4 Classification in soft ground

The grading distribution represents the direct and indirect evaluation criteria for the stability and permeability of the ground. Based on the shear strength parameters and the water pressure, and including consideration of the grading distribution, the stability of the ground is evaluated and the required support pressure is determined. The technical requirements placed on the machine increase with increasing ground and groundwater pressure.

11.9.1.5 Classification in rock

The table recommendations serve primarily to select the tunnelling machine and not to assess the cuttability. The rock mass classification and evaluation of stability are undertaken based on the RMR system. It is recommended to analyse the tunnelling machine system and all six project-specific parameters of the RMR system. Calculations to verify the stability and determine the support pressure are also recommended.

11.9.2 Notes about each type of tunnelling machine

11.9.2.1 TBM (Tunnel boring machine)

The main area of application is rock classed as stable to liable to rockfall, and water ingress from strata and joints can be overcome. The uniaxial compressive strength σ_D should be between 25 and 250 MN/m^2. Higher strengths, toughness of the rock and a higher content of mineral resistant to wear represent economic limits to application. A restricted ability of the machine to brace itself may also make the use of a machine impractical. For the assessment of the rock, the tensile strength and the RQD value are used. With a degree of fracturing of the rock mass with RQD from 50 to 100 % and a joint spacing of > 0.6 m, the use of a TBM seems assured. If the fracturing is worse, the stability should be checked. In soft ground or solid rock with similar properties to soil, the use of a TBM is impossible.

11.9.2.2 DSM (Double shield machines)

Double shield machines are mainly used for tunnel projects with long stretches through stable rock but also short stretches of rock classified as liable to rockfall to brittle. In a stable rock mass (see the requirements for the use of a TBM), the machine can work in continuous mode using the grippers for bracing. In fault zones or areas of lower rock strength, where the grippers cannot be used, the shield joint is retracted and the machine pushes itself using the auxiliary thrust cylinders against the last completed ring of segments.

11.9.2.3 SM-V1 (full-face excavation, face without support)

This type of machine can only be used in stable, predominantly water-impermeable, cohesive soft ground with high fines content. The stability of the face should be verified by calculation. It should also be verified that the sides of the excavation are temporarily stable until the final tunnel lining has been installed. Loosening of the ground, which could reduce the bedding, should be ruled out. If there is building on the surface susceptible to settlement, deformation of the subsoil and loosening should be verified using the usual damage classes (e.g. gradient of the settlement trough).

In rock, this type of machine can be used in rock classed as liable to rockfall to brittle, also with water in strata or joints. The strength of the rock mass can be greatly reduced even if the rock strength is good. This corresponds to a joint spacing of ≈ 0.6 to 0.06 m and a RQD value between approx. 10 and 50 %. In general, however, this type can be used in rock with compression strengths less than 5 MN/m^2, for example strongly weathered rock.

The stability of the face and the sides of the excavated cavity should be verified with calculations. In case of high water ingress, appropriate measures should be planned.

11.9.2.4 SM-V2 (full-face excavation, face with mechanical support)

Due to its failure on numerous projects, this type of machine is no longer recommended.

11.9.2.5 SM-V3 (Full-face excavation, face with compressed air application)

The application of compressed air enables machine type SM-V1 to be used in stable ground even under the groundwater table. The air permeability of the ground or the air consumption, the verification of the formation of an air flow and safety against blowouts are the essential criteria for the use of this type of machine. The groundwater table should be above the tunnel crown with an adequate margin of safety.

11.9.2.6 SM-V4 (full-face excavation, face with slurry support)

The main area of application of slurry shields is in coarse- and mixed-graded soil types. The groundwater table should also be above the tunnel crown with an adequate safety margin. As the ground is excavated, a fluid under pressure, e.g. bentonite suspension, supports the face. Highly permeable soils impede the formation of a membrane. At a permeability of over $5 \cdot 10^{-3}$ m/s, there is a danger that the bentonite flows uncontrolled into the ground. The scope of application can be extended by adding fine-grained material and filler or additives for the improvement of the rheological properties. Alternatively, additional measures to reduce the permeability of the soil (for example filling the pores) can be necessary. Boulders and blocks too large to be pumped can be broken by a crusher in front of the inlet. A high fines content can lead to difficulties with the separation. It should be borne in mind that the rheological properties of the support fluid are worsened by fine-grained material, as the separation of clay fractions and bentonite is not possible.

11.9.2.7 SM-V5 (full-face excavation, face with earth pressure balance support)

Machine types with earth pressure balance support are particularly suitable in soils with fines content (< 0.06 mm) of over 30 %. In coarse- and mixed-grained soils and rock, the contact force and the cutting wheel torque increase over-proportionately with increasing support pressure. The flow behaviour of the excavated muck can be improved with suitable conditioning agents like e.g. bentonite, polymers or foam. Soil conditioning with foam is recommended for active support pressure control and to ensure low settlement outside the predestined area of application.

Earth pressure balance shields have the advantage that operation is possible without modifying the process technology with partially filled and unpressurised excavation chamber in open mode (SM-V5-OM) without active face support. It should be noted that in this case the cutting wheel and screw conveyor combination will grind the excavated soil/rock considerably more than with a conveyor belt through the centre (SM-V1). If the soil tends to sticking, hindrance and increased wear have to be reckoned with. In order to improve the material flow and reduce the tendency to stick, conditioning agents should be used. Particularly unfavourable for earth pressure balance shields, both in soft ground and in rock, is a combination of high support pressure, high permeability, high abrasiveness and difficulty in breaking the grain structure.

11.9.2.8 SM-T1 (partial excavation, face without support)

This type of machine can be used above the groundwater table if the face is sufficiently stable, see here SM-V1.

Partial machines always offer good access to the face, so the process can be very advantageous if obstructions are to be expected.

11.9.2.9 SM-T2 (partial excavation, face with mechanical support)

This type of machine can be used when the support provided by material piling on the platforms at its natural angle of repose is sufficient for tunnelling with a limited degree of settlement control. Breasting plates can be installed for additional support in the crown and on the platforms. The main area of application is weakly to non-cohesive gravel-sand soils above the groundwater table with the corresponding angle of friction.

11.9.2.10 SM-T3 (partial excavation, face with compressed air application)

The use of this type of machine is appropriate when types SM-T1 and -T2 are to be used in the groundwater. The entire working area, including the already completed tunnel or just the working chamber, is pressurised.

11.9.2.11 SM-T4 (Partial excavation, face with slurry support)

Partial excavation machines with slurry-filled excavation chamber are no longer used.

11.9.2.12 KSM (Convertible shield machines)

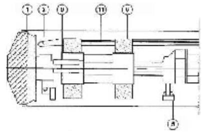
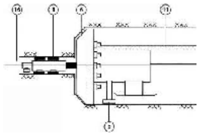
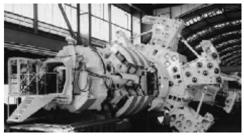
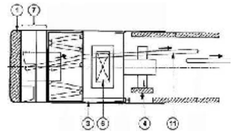
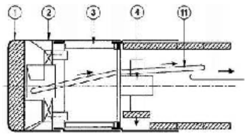
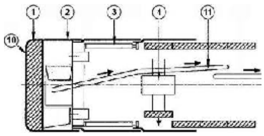
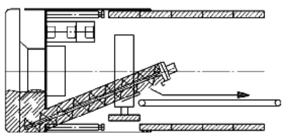
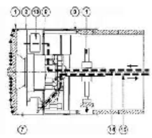
Convertible machines combine the areas of application of each machine type in changeable ground conditions. Their area of application is therefore extended to both sets of criteria.

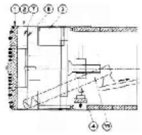
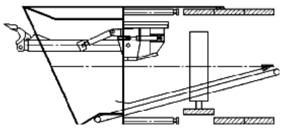
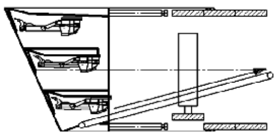
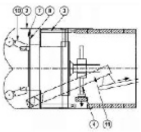
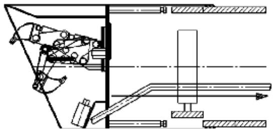
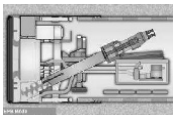
The number of conversions from one tunnelling process to another should be kept as low as possible, as rebuilding takes a long time and is expensive.

11.10 Appendices

Appendix 1 Overview of tunnelling machines

Information about the tunnelling machines and their areas of application can be found in Section 11.9.2 of these recommendations.

Brief description	Illustration (example)	
Tunnel boring machines (TBM)		
TBM Tunnelbohrmaschine ohne Schild Tunnel boring machine		
ETBM Erweiterungstunnelbohrmaschine Enlargement tunnel boring machine		
Double shield machines (DSM)		
DSM Doppelschildmaschine Double shield machine		
Shield machines (SM)		
SM-V1 ohne Stützung Without support		
SM-V2 mechanische Stützung Mechanical support		
SM-V3 Druckluftbeaufschlagung Full-face and compressed air application		
SM-V4 Flüssigkeitsstützung Full-face and slurry support		

SM-V5 Erddruckstützung Full-face and earth pressure balance support		
SM-T1 ohne Stützung Partial excavation and without support		
SM-T2 Teilstützung Partial excavation and partial support		
SM-T3 Druckluftbeaufschlagung Partial excavation and compressed air application		
SM-T4 Flüssigkeitsstützung Partial excavation and slurry support		
Convertible shield machines (KSM)		
KSM Kombinationsschildmaschinen Convertible shield machines		 

Legend:

1 cutting wheel
2 shield skin
3 cylinders
4 erector
5 rear support
6 gripper
7 excavation chamber
8 pressure bulkhead
9 openings
10 excavation tool
11 muck clearance
12 carriage
13 air bubble
14 feed line
15 slurry line
16 pilot tunnel

Appendix 2.1 Process-related geotechnical parameters for soft ground

Process-related geotechnical parameters for soft ground	Brief description	Unit	TBM*	DSM*	SM-V1	SM-V2	SM-V3	SM-V4	SM-V5	SM-T1	SM-T2	SM-T3	SM-T4
Ortsbruststützung + Senkungsanalyse **Face support + settlement analysis**													
Kornverteilung Grain size distribution		%			x	not recommended	x	x	x	x	x	x	not recommended
Dichte / Dichte unter Auftrieb Soil density wet / submerged density	γ / γ'	kN/m³						x	x		x	x	
Lagerungsdichte Compactness of the packing	D	–			x		x	x	x	x	x	x	
Reibungswinkel Friction angle	φ'	°			x		x	x	x	x	x	x	
Kohäsion Cohesion	c'	kN/m²			x		x	x	x	x	x	x	
E-Modul Elasticity modulus	E	MN/m²			x		x	x	x	x	x	x	
Dilatationswinkel Dilatancy angle	Ψ	°			x		x	x	x	x	x	x	
Porenanteil Pore content	n	–			x		x	x	x	x	x	x	
Porenzahl Void ratio	e	–			x		x	x	x	x	x	x	
Durchlässigkeit Permeability	k	m/s			x		x	x	x	x	x	x	
Erddruckbeiwert (horizontal) Coefficient of lateral earth pressure	k_h	–			x		x	x	x	x	x	x	
Grundwasserdruck Water pressure	p_{GW}	kN/m²			x		x	x	x	x	x	x	
Bodenabbau **Soil removal**													
Verklebung Sticking													
Plastizitätszahl ($I_P = w_L - w_P$) Plasticity index	I_P	%			x	not recommended	x	x	x	x	x	x	not recommended
Konsistenzzahl Consistancy index	I_c	–			x		x	x	x	x	x	x	
Fließgrenze Liquid limit	w_L	%			x		x	x	x	x	x	x	
Ausrollgrenze Rolling limit	w_P	%			x		x	x	x	x	x	x	
Wassergehalt Water content	w	%			x		x	x	x	x	x	x	

Process-related geotechnical parameters for soft ground	Brief description	Unit	TBM*	DSM*	SM-V1	SM-V2	SM-V3	SM-V4	SM-V5	SM-T1	SM-T2	SM-T3	SM-T4
Mineralogie Mineral composition					x		x	x	x	x	x	x	
Tonanteil (Siebrückstand < 0,002 mm) Percentage of clay		%			x		x	x	x	x	x	x	
Verschleiß Wear													
Abrasivität LCPC-Index Abrasiveness LCPC-Index	ABR	g/t			x	not recommended	x	x	x	x	x	x	not recommended
Brechbarkeit LCPC-Index Breakability LCPC-Index	BR	%			x		x	x	x	x	x	x	
Quarzanteil Equivalent quarz index	äQu	%			x		x	x	x	x	x	x	
Steinanteil Stone proportion		%			x		x	x	x	x	x	x	
Blockanteil Boulder proportion		%			x		x	x	x	x	x	x	
Druckfestigkeit Uniaxial compressive strength	σ_c	kN/m²			x		x	x	x	x	x	x	
Scherfestigkeit Shear strength					x		x	x	x	x	x	x	
Lagerungsdichte Compactness of the packing	D	–			x		x	x	x	x	x	x	
Bodenkonditionierung Soil conditioning													
Kornverteilung Grain size distribution		%				not recommended		x	x				not recommended
Tonanteil (Siebrückstand < 0,002 mm) Percentage of clay		%						x	x				
Schluffanteil (Siebrückstand < 0,06 mm) Percentage of silt		%						x	x				
Plastizitätszahl ($I_P = w_L - w_P$) Plasticity index	I_P	%						x	x				
Konsistenzzahl Consistancy index	I_c	–						x	x				
Stützdruck Confinement pressure	p_s	bar						x	x				
Porenanteil Pore content	n	–						x	x				
Durchlässigkeit Permeability	k	m/s						x	x				
chemische Grundwasseranalyse Chemical analysis of groundwater								x	x				
Anteil an organischen Substanzen (Kationen) Portion of organic substances (cations)		%						x	x				

Process-related geotechnical parameters for soft ground	Brief description	Unit	TBM*	DSM*	SM-V1	SM-V2	SM-V3	SM-V4	SM-V5	SM-T1	SM-T2	SM-T3	SM-T4
Bodenseparierung **Soil separation**													
Restbentonitgehalt Residual content of bentonite		%				not recommended		x	x				not recommended
Restgehalt an chemischen Additiven Residual content of chemical additives		%						x	x				
Tonanteil (Siebrückstand < 0,002 mm) Percentage of clay		%						x	x				
Schluffanteil (Siebrückstand < 0,06 mm) Percentage of silt		%						x	x				
Konsistenzzahl Consistancy index	I_c	–						x	x				
undrainierte Kohäsion Undrained cohesion	c_u	kN/m²						x	x				
Bodentransport und -deponierung **Soil transport and landfill**													
Kornverteilung Grain size distribution		%				not recommended		x	x				not recommended
Schluffanteil (Siebrückstand < 0,06 mm) Percentage of silt		%						x	x				
Tonanteil (Siebrückstand < 0,002 mm) Percentage of clay		%						x	x				
Reibungswinkel Friction angle	φ'	°			x		x	x	x	x	x	x	
Kohäsion Cohesion	c'	kN/m²			x		x	x	x	x	x	x	
Plastizitätszahl ($I_P = w_L - w_P$) Plasticity index	I_P	%						x	x				
Konsistenzzahl Consistancy index	I_c	–						x	x				
E-Modul Elasticity modulus	E	kN/m²			x		x	x	x	x	x	x	
Restbentonitgehalt Residual content of bentonite		%						x	x				
Restgehalt an chemischen Additiven Residual content of chemical additives		%						x	x				
Wassergehalt Water content	w	%						x	x				
Druckfestigkeit Uniaxial compressive strength	σ_c	kN/m²			x		x	x	x	x	x	x	
max. Kantenlänge Max. block size		mm						x	x				

* TBM and DSM are only used in hard rock.

Appendix 2.2 Process-related geotechnical parameters for rock

Process-related geotechnical parameters for rock	Brief description	Unit	TBM	DSM	SM-V1	SM-V2	SM-V3	SM-V4	SM-V5	SM-T1	SM-T2	SM-T3	SM-T4
Ortsbruststützung + Senkungsanalyse **Face support + settlement analysis**													
Gefüge Texture			x	x	x	not recommended	x	x	x	x	x	x	not recommended
Verwitterungsgrad Weathering	W	–	x	x	x		x	x	x	x	x	x	
Zerlegung Ratio matrix / fragmentation			x	x	x		x	x	x	x	x	x	
Anisotropie Anisotropy			x	x	x		x	x	x	x	x	x	
Porosität Porosity	Φ	–	x	x	x		x	x	x	x	x	x	
Quellverhalten Swelling capacity			x	x	x		x	x	x	x	x	x	
Diskontinuitäten Discontinuities													
Einlagerungen Infilling			x	x	x	not recommended	x	x	x	x	x	x	not recommended
Orientierung Discontinuity orientation			x	x	x		x	x	x	x	x	x	
Kluftabstand Normal spacing of discontinuity sets			x	x	x		x	x	x	x	x	x	
Zerlegungsgrad Degree of fracturing – Discontinuity frequency			x	x	x		x	x	x	x	x	x	
Felsdruckbeiwert Coefficient of lateral rock pressure	k_h	–	x	x	x		x	x	x	x	x	x	
Gebirgswasserzufluss Water inflow	Q_W	l/s	x	x	x		x	x	x	x	x	x	
Gebirgswasserdruck Water pressure	p_{GW}	kN/m^2	x	x	x		x	x	x	x	x	x	
Bodenabbau **Performance**													
Druckfestigkeit Uniaxial compressive strength	σ_c	kN/m^2	x	x	x	not recommended	x	x	x	x	x	x	not recommended
Spaltzugfestigkeit Tensile strength (SPZ)	σ_z	MN/m^2	x	x	x		x	x	x	x	x	x	
RQD Rock Quality Designation	RQD	–	x	x	x		x	x	x	x	x	x	
Verwitterungsgrad Weathering	W	–	x	x	x		x	x	x	x	x	x	

Process-related geotechnical parameters for rock	Brief description	Unit	TBM	DSM	SM-V1	SM-V2	SM-V3	SM-V4	SM-V5	SM-T1	SM-T2	SM-T3	SM-T4
RSR Rock Structure Rating	RSR	–	x	x	x	not recommended	x	x	x	x	x	x	not recommended
RMR Rock Mass Rating	RMR	–	x	x	x		x	x	x	x	x	x	
GSI Geological Strength Index (Hoek Brown)	GSI	–	x	x	x		x	x	x	x	x	x	
RMI Rock Mass Index	RMI	–	x	x	x		x	x	x	x	x	x	
Q-Index Q-value	Q-value	–	x	x	x		x	x	x	x	x	x	
Mineralogie Mineral composition			x	x	x		x	x	x	x	x	x	
Karbonat-Anteil Carbonate portion			x	x	x		x	x	x	x	x	x	
Verfestigung Cementation			x	x	x		x	x	x	x	x	x	
Diskontinuitäten Discontinuities													
Einlagerungen Infilling			x	x	x	not recommended	x	x	x	x	x	x	not recommended
Blockgröße Block size			x	x	x		x	x	x	x	x	x	
Orientierung Discontinuity orientation			x	x	x		x	x	x	x	x	x	
Kluftabstand Normal spacing of discontinuity sets			x	x	x		x	x	x	x	x	x	
Zerlegungsgrad Degree of fracturing – discontinuity frequency			x	x	x		x	x	x	x	x	x	
Verklebung **Sticking**													
Wassergehalt Water content	w	%	x	x	x	not recommended	x	x	x	x	x	x	not recommended
Gebirgswasserzufluss Water inflow	Q_W	l/s	x	x	x		x	x	x	x	x	x	
Mineralogie Mineral composition			x	x	x		x	x	x	x	x	x	

Process-related geotechnical parameters for rock	Brief description	Unit	TBM	DSM	SM-V1	SM-V2	SM-V3	SM-V4	SM-V5	SM-T1	SM-T2	SM-T3	SM-T4
Verschleiß Wear													
Abrasivität (Cerchar Abrasivity Index) Abrasiveness	CAI	–	x	x	x	not recommended	x	x	x	x	x	x	not recommended
Quarzanteil Equivalent quartz content	äQu	%	x	x	x		x	x	x	x	x	x	
Abrasivität RAI Rock Abrasivity Index (RAI = Equ · UCS)	RAI	–	x	x	x		x	x	x	x	x	x	
Druckfestigkeit Uniaxial rock compressive strength (UCS)	σ	MN/m²	x	x	x		x	x	x	x	x	x	
Spaltzugfestigkeit Tensile strength (SPZ)	σ_z	MN/m²	x	x	x		x	x	x	x	x	x	
Scherfestigkeit Shear strength			x	x	x		x	x	x	x	x	x	
tonmineralische Zusammensetzung Clay mineral composition			x	x	x		x	x	x	x	x	x	
Verwitterungsgrad Weathering	W	–	x	x	x		x	x	x	x	x	x	
Verfestigung Cementation			x	x	x		x	x	x	x	x	x	
Bodentransport und -deponierung **Soil transport and landfill**													
max. Kantenlänge Max. block size		mm	x	x	x	not recommended	x	x	x	x	x	x	not recommended
Druckfestigkeit Uniaxial compression	σ_c	kN/m²	x	x	x		x	x	x	x	x	x	
Gebirgswasserzufluss Water inflow	Q_W	l/s	x	x	x			x	x	x	x		
Wassergehalt Water content	w	%	x	x	x		x	x	x	x	x	x	

Anlage 3.1 Areas of application and selection criteria TBM

Geotechnische Kennwerte Geotechnical values	**TUNNELBOHRMASCHINE (TBM)** Tunnel Boring Machine (TBM)				
	Lockergestein (Soft soil)				
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	Festgestein (Hard rock)				
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 | 5 – 25 – | o	25 – 50 +	50 – 100 +	100 – 250 +	> 250 o
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25 –	gering poor 25 – 50 o	mittel fair 50 – 75 +	gut good 75 – 90 +	ausgezeichnet excellent 90 – 100 +
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20 –	schlecht poor 21 – 40 –	mäßig fair 41 – 60 o	gut good 61 – 80 +	sehr gut very good 81 – 100 +
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0 +	0 – 10 +	10 – 25 +	25 – 125 o	> 125 –
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5 +	schwach abrasiv slightly abrasive 0,5 – 1 +	abrasiv abrasive 1 – 2 +	stark abrasiv very abrasive 2 – 4 o	extrem abrasiv extremely abrasive 4 – 6 o
Quellverhalten Swelling behaviour	kein none +	gering poor +	mittel fair o	hoch high o	
Stützdruck [bar] Supporting pressure [bar]	0 +	0 – 1 –	1 – 2 –	2 – 3 –	3 – 4 –

Einsatz nicht empfohlen
Application not recommended

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

Anlage 3.2 Areas of application and selection criteria DSM

Geotechnische Kennwerte Geotechnical values	DOPPELSCHILDMASCHINE (DSM) Double Shield Machine (DSM)				
Lockergestein (Soft soil)					
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
Festgestein (Hard rock)					
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 | 5 – 25 o | o	25 – 50 +	50 – 100 +	100 – 250 o	> 250 o
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25 o	gering poor 25 – 50 +	mittel fair 50 – 75 +	gut good 75 – 90 o	ausgezeichnet excellent 90 – 100 o
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20 o	schlecht poor 21 – 40 +	mäßig fair 41 – 60 +	gut good 61 – 80 o	sehr gut very good 81 – 100 o
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0 +	0 – 10 +	10 – 25 +	25 – 125 o	> 125 –
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5 +	schwach abrasiv slightly abrasive 0,5 – 1 +	abrasiv abrasive 1 – 2 +	stark abrasiv very abrasive 2 – 4 o	extrem abrasiv extremely abrasive 4 – 6 o
Quellverhalten Swelling behaviour	kein none +	gering poor +	mittel fair o	hoch high o	
Stützdruck [bar] Supporting pressure [bar]	0 +	0 – 1 –	1 – 2 –	2 – 3 –	3 – 4 –

Einsatz nicht empfohlen
Application not recommended

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

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Anlage 3.3 Areas of application and selection criteria SM-V1

Geotechnische Kennwerte Geotechnical values	**SCHILDMASCHINE mit Vollschnittabbau ohne Stützung (SM-V1)** Shield Machine with full-face and without support (SM-V1)				
Lockergestein (Soft soil)					
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
	–	–	o	+	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	–	–	o	+	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	–	–	o	+	+
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	o	–		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	–	–	–	–
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	o	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
	+	+	+	+	o
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	+	+	+	+	o
Festgestein (Hard rock)					
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 / 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	o / o	+	+	o	o
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	o	+	+	o	o
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	o	+	+	o	o
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
	+	+	+	o	–
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	+	o	o
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	o	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	–	–	–	–

+	Haupteinsatzbereich / Main field of application
o	Einsatz möglich / Application possible
–	Einsatz kritisch / Application critical

Anlage 3.4 Areas of application and selection criteria SM-V2

Geotechnische Kennwerte Geotechnical values	SCHILDMASCHINE mit Vollschnittabbau und mechanischer Stützung (SM-V2) Shield Machine with full-face and with mechanical support (SM-V2)				
	Lockergestein (Soft soil)				
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	Festgestein (Hard rock)				
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 5 – 25	25 – 50	50 – 100	100 – 250	> 250
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4

Einsatz nicht empfohlen
Application not recommended

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

Anlage 3.5 Areas of application and selection criteria SM-V3

Geotechnische Kennwerte Geotechnical values	**SCHILDMASCHINE mit Vollschnittabbau und Druckluftstützung (SM-V3)** Shield Machine with full-face and compressed air application (SM-V3)				
			Lockergestein (Soft soil)		
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
	–	o	+	+	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} - 10^{-4}$	durchlässig permeable $10^{-4} - 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	–	–	o	+	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	–	o	+	+	+
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	o	–		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	o	+	+	o	–
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	o	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
	+	+	o	–	–
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	+	+	o	–	–
			Festgestein (Hard rock)		
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 / 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	o / o	o	o	o	o
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	o	o	o	o	o
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	o	o	o	o	o
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
		+	+	o	–
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	o	o	o	–	–
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	o	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	o	+	+	o	–

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

Anlage 3.6 Areas of application and selection criteria SM-V4

Geotechnische Kennwerte Geotechnical values	**SCHILDMASCHINE mit Vollschnittabbau und Flüssigkeitsstützung (SM-V4)** Shield Machine with full-face and fluid support (SM-V4)				
	Lockergestein (Soft soil)				
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
	+	+	+	o	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	–	o	+	o	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	– / o	o	o	o	o
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	+	o		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	o	+	+	+	+
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	o	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
	+	+	+	o	o
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	o	+	+	+	o
	Festgestein (Hard rock)				
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 / 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	o / o	o	o	o	o
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	o	o	o	o	o
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	o	o	o	o	o
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
	o	o	o	o	o
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	o	o	o
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	o	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	o	+	+	+	+

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

Anlage 3.7 Areas of application and selection criteria SM-V5

Geotechnische Kennwerte Geotechnical values	**SCHILDMASCHINE mit Vollschnittabbau und Erddruckstützung (SM-V5)** Shield Machine with full-face and earth pressure balance support (SM-V5)				
			Lockergestein (Soft soil)		
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
	–	o	o / +	+	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	–	–	o	+	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	o	+	+	o	o
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	+	+		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	+	+	o	–
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	o	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
	+	+	o	o	–
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	+	+	o	o	–
			Festgestein (Hard rock)		
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 / 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	o / o	o	–	–	–
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	+	o	o	–	–
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	+	o	o	–	–
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
	o	o	o	o	o
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	o	o	–
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	o	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	o	+	o	–	–

+	**Haupteinsatzbereich** / Main field of application
o	**Einsatz möglich** / Application possible
–	**Einsatz kritisch** / Application critical

Anlage 3.8 Areas of application and selection criteria SM-T1

Geotechnische Kennwerte Geotechnical values	SCHILDMASCHINE mit Teilschnittabbau ohne Stützung (SM-T1) Shield Machine with part heading and without support (SM-T1)				
Lockergestein (Soft soil)					
Feinkornanteil ($<$ 0,06 mm) DIN 18196 Fine grain fraction ($<$ 0,06 mm)	$<$5 %	5 – 15 %	15 – 40 %	$>$40 %	
	–	– / o	o / +	+	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	–	o	o	+	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	–	–	o	+	+
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	o	–		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	–	–	–	–
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	o	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive $>$ 2000
	+	+	+	+	o
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high $>$ 100
	+	+	+	+	o
Festgestein (Hard rock)					
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 / 5 – 25	25 – 50	50 – 100	100 – 250	$>$ 250
	+ / +	+	o	–	–
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	o	+	+	o	o
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor $<$ 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	o	+	+	o	o
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	$>$ 125
	+	+	+	o	–
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	+	o	o
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	o	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	–	–	–	–

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

Anlage 3.9 Areas of application and selection criteria SM-T2

Geotechnische Kennwerte Geotechnical values	**SCHILDMASCHINE mit Teilschnittabbau und Teilstützung (SM-T2)** Shield Machine with part heading and partial support (SM-T2)				
	Lockergestein (Soft soil)				
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
	+	+	+	o	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	o	o	+	+	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	–	o	o	o	o
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	+	o		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	–	–	–	–
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	o	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
	+	+	+	+	o
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	+	+	+	+	o
	Festgestein (Hard rock)				
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 | 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	+ | +	+	o	–	–
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	+	+	+	o	o
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	+	+	+	o	o
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
	+	+	+	o	–
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	+	o	o
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	o	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	+	–	–	–	–

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

Anlage 3.10 Areas of application and selection criteria SM-T3

Geotechnische Kennwerte Geotechnical values	SCHILDMASCHINE mit Teilschnittabbau und Druckluftstützung (SM-T3) Shield Machine with part heading and compressed air application (SM-T3)				
			Lockergestein (Soft soil)		
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
	–	O	+	+	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
	–	–	O	+	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
	–	O	O	O	O
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
	+	O	–		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	O	+	+	O	–
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
	+	+	O	–	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
	+	+	+	+	O
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
	+	+	+	+	O
			Festgestein (Hard rock)		
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 / 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	+ / +	+	O	–	–
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	+	+	+	O	O
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	+	+	+	O	O
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
	kein Zufluss – Vortrieb im Grundwasser / no waterinflow – excavation below groundwater level				
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	+	O	O
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
	+	+	O	–	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
	O	+	+	O	–

+	Haupteinsatzbereich / Main field of application
O	Einsatz möglich / Application possible
–	Einsatz kritisch / Application critical

Anlage 3.11 Areas of application and selection criteria SM-T4

Geotechnische Kennwerte Geotechnical values	SCHILDMASCHINE mit Teilschnittabbau und Flüssigkeitsstützung (SM-T4) Shield Machine with part heading and fluid support (SM-T4)				
Lockergestein (Soft soil)					
Feinkornanteil (< 0,06 mm) DIN 18196 Fine grain fraction (< 0,06 mm)	< 5 %	5 – 15 %	15 – 40 %	> 40 %	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark durchlässig very highly permeable $> 10^{-2}$	stark durchlässig strongly permeable $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach durchlässig slightly permeable $< 10^{-6}$	
Konsistenz (Ic) nach DIN 18122 Consistency (Ic)	breiig pasty 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest semi-solid 1,0 – 1,25	fest hard 1,25 – 1,5
Lagerungsdichte nach DIN 18126 Storage density	dicht dense	mitteldicht fairly dense	locker loose		
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4
Quellverhalten Swelling behaviour	kein none	gering little	mittel fair	hoch high	
Abrasivität LCPC-Index ABR [g/t] Abrasiveness LCPC-index ABR [g/t]	sehr schwach abrasiv very low abrasive 0 – 500	schwach abrasiv low abrasive 500 – 1000	mittel abrasiv medium abrasive 1000 – 1500	stark abrasiv high abrasive 1500 – 2000	sehr stark abrasiv very high abrasive > 2000
Brechbarkeit LCPC-Index BR [%] Breakability LCPC-index BR [%]	sehr schwach very low 0 – 25	schwach low 25 – 50	mittel medium 50 – 75	stark high 75 – 100	sehr stark very high > 100
Festgestein (Hard rock)					
Gesteinsfestigkeit [MPa] Rock compressive strength [MPa]	0 – 5 | 5 – 25	25 – 50	50 – 100	100 – 250	> 250
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
Abrasivität (CAI) Abrasiveness (CAI)	kaum abrasiv not very abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
Quellverhalten Swelling behaviour	kein none	gering poor	mittel fair	hoch high	
Stützdruck [bar] Supporting pressure [bar]	0	0 – 1	1 – 2	2 – 3	3 – 4

Einsatz nicht empfohlen
Application not recommended

+ Haupteinsatzbereich / Main field of application
o Einsatz möglich / Application possible
– Einsatz kritisch / Application critical

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