B. Maidl, M. Thewes, U. Maidl

Handbook of Tunnel Engineering
Volume I: Structures and Methods

Ernst & Sohn
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First Edition

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Bernhard Maidl
Markus Thewes
Ulrich Maidl

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Prof. Dr.-Ing. Bernhard Maidl
mtc – Maidl Tunnelconsultants GmbH & Co. KG
Fuldastr. 11
47051 Duisburg
Germany

Prof. Dr.-Ing. Markus Thewes
Lehrstuhl für Tunnelbau, Leitungsbau und Baubetrieb
Ruhr Universität Bochum
Universitätsstr. 150
44801 Bochum
Germany

Dr.-Ing. Ulrich Maidl
mtc – Maidl Tunnelconsultants GmbH & Co. KG
Rupprechtstr. 25
80636 München
Germany

Translated by David Sturge, Kirchbach, Germany
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Dedicated to My **C**hildren

Julia

Ulrich

Nadine

The authors

o. Professor em. Dr.-Ing. Dr. h.c. mult. Bernhard Maidl

Former holder of the Chair of Construction Technology, Tunnelling and Construction Management at the Institute for Structural Engineering of the Ruhr University, Bochum

Prof. Dr.-Ing. Markus Thewes

Holder of the Chair of Tunnelling and Construction Management, Ruhr University Bochum

Dr.-Ing. Ulrich Maidl

Managing director of Maidl Tunnelconsultants GmbH & Co. KG

With the collaboration of:

Dipl.-Ing. Michael Griese (overall coordination), Maidl Tunnelconsultants GmbH & Co. KG

Dr.-Ing. Götz Vollmann, Ruhr University, Bochum

Dipl.-Ing. Anna-Lena Hammer, Ruhr University, Bochum

Foreword to the English edition

The “black book of tunnelling” has become a standard work in German-speaking countries since its first 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 publisher 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 first 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 MTC, have joined me in the team of authors. Dipl.-Ing. *Michael Griese* from MTC is the overall coordinator, assisted by Dr.-Ing. *Götz Vollmann* and Dipl.-Ing. *Anna-Lena Hammer* from the chair of Prof. *Thewes*. I thank all those involved, also the translator *David Sturge* and the employees of the publisher Ernst & Sohn in Berlin.

Bochum, in March 2013 *Bernhard Maidl*

Foreword to the 3rd German edition

Almost 20 years after the first appearance of the Handbook of Tunnel Engineering and about 10 years after the 2nd German edition, a complete revision was necessary for the 3rd German edition. Detailed investigation only made clear what enormous developments had taken place in tunnelling.

In conventional tunnelling, progress was predominantly in the field of advance support methods such as pipe screens and jet grouting, which enabled the scope of application to be extended to include larger cross-sections and better mechanisation. Further mechanisation, particularly of muck clearance, enables parallel operation of excavation, support, muck clearance and transport in conventional tunnelling, which has also improved advance rates.

In mechanised tunnelling, an even greater leap of progress has taken place as a result of the wider experience that has come with increasing application. The traditionally established limits to the application of EPB and hydroshield machines are no longer so clear in practice. Mixshields and combishields can be used in various modes.

Particularly worth mentioning are developments of support methods installed directly behind the machine. The requirements connected with the construction of the large Alpine tunnels under the Lötschberg and the St. Gotthard have given a spur to this development.

Since the extent of the text in the chapters concerning "support methods" and "construction processes" has also increased, Chapter X "Implementation of construction projects" has been omitted. This has been integrated under other headings in Volume II. The chapter "Waterproofing and drainage" has also been included in Volume II.

As has already been the case with other books, I once again required intensive assistance from my employees for this book. Even when we could refer to the other books about shotcrete, steel fibre concrete, shield and TBM tunnelling, this still involved an enormous amount of work, for which I wish to thank all, and also the authors who worked on the former books.

Many thanks also to the many other helpers and also to those involved at the publishers.

Bochum, in January 2004 *Bernhard Maidl*

Foreword to the 2nd German edition

Many letters and comments from Germany and abroad have confirmed that the Handbook of Tunnel Engineering from 1984 has been well accepted as a textbook and also in design offices and on tunnel sites. Positive developments in tunnelling are also good news, and this has led to the call for specific planning regulations for underground structures already becoming a reality in some countries.

When the new edition was being investigated, it became clear that a great variety of technical innovations have been introduced in the last ten years, sufficient to justify a new edition. But time is limited and cannot be multiplied, so only the most important revisions have been undertaken. This includes revisions and extensions of the standards listed in the references. The sections concerning shotcrete and steel fibre shotcrete in Chapter II "Support methods" have been revised, as well as Chapters VI "Mechanised tunnelling" and VII "The driving of small cross-sections". Chapter IV "Shotcrete tunnelling using the New Austrian Tunnelling Method" has been rewritten and renamed and also part of Chapter X "Quality assurance in tunnelling".

For the production of the new edition, I was once again dependent on the intensive support and experience of my colleagues at the University Chair and in the consultancy. In particular, I wish to thank Dipl.-Ing. *Feyerabend* for the overall coordination, ably assisted by Dipl.-Ing. *Gipperich* and Dipl.-Ing. *Berger*.

I also have to thank *Helmut Schmidt* for the production of drawings and naturally also the publisher, for whom Dr. *Jackisch* has supported us at all times.

Bochum, in June 1994 *Bernhard Maidl*

The art of the Engineer is to avoid high ground pressure, that is not to permit it to occur, a much more difficult task than to overcome ground pressure after it has occurred.
And let us dare to resist the former with intellectual, the latter with raw material work.

Franz RŽIHA, 1874

Foreword to the 1st German edition

Leopold Müller, my teacher, said with slight resignation at the 28th Geomechanics Colloquium in 1979 in Salzburg: "Experience on construction sites and at congresses gives cause to reflect on the state of development in geomechanical research related to construction in rock, but experience shows that this development is largely uncontrolled and not always coordinated with the needs of practitioners and theoretical progress. Many results of scientific research remain unknown in practice or do not become accepted, while in the other direction, the research needs of practicing engineers are not recognised satisfactorily or indeed not at all". It remains for us as his former students to consider the reasons for this development and to attempt to divert progress to a sensible path.

On tunnel projects in recent years, the "new Austrian tunnelling method" has become very successful in construction practice. But it has still today been verified by astonishingly few calculations, and that despite many Geomechanics Colloquia. However, the opinion is becoming ever more common that tunnels should be designed and built using refined calculations and with a lot of work and paper. Refined calculations demand particularly from the consultant the mastery of model formation and the application of the results of calculations in practice on site; the responsible site manager on the other hand should be able to judge how the parameters of rock mass, support and construction process affect the results of the calculation, particularly under varying geological conditions. But how is he meant to do that when the calculation has been performed externally and the given results are scarcely understandable for him?

If we investigate the failures, losses, schedule and cost overruns in tunnelling in recent years, then the calculations do not normally turn out to have been performed too little or too roughly, and there have been plenty of expert reports. The causes are rather more inadequate studies of the ground parameters, insufficient care in the selection of a construction process, inadequate adaptability to changing geological conditions, gross construction errors in important details, lack of skilled personnel, insufficient measurements; the list could be continued. Perhaps we feel too safe and become careless having put too much trust in too much paper. In addition, the structural design of tunnels cannot be compared to the structural verification of other engineered structures. The construction of a tunnel has more to it and thankfully most tunnellers appreciate this.

On every tunnel construction project, the correct selection of a construction process is a precondition for technical and commercial success. The factor time has not yet been considered in calculations in tunnelling, but it can only be influenced through the construction process with the various excavation processes on agreed schedules and the effect of the support. The surrounding ground belongs to the structure; the sequence of operations influences the loading on the support and the load-bearing behaviour of the rock mass. The literature of tunnelling is today aligned into specialist areas, which are often aligned with the specific activities of the chairs of university lecturers, like rock mechanics, foundation engineering, structural engineering, construction management and transport. Construction process technology should in this case be understood as a structural subject, which includes the influences of construction on the design, including the consideration of construction states. Such a systematic way of thinking has only been taught for a few years as an individual scientific subject in civil engineering; the number of publications is not yet too large.

Volume I "Details and construction processes" puts the emphasis on construction process technology as a constructive area of tunnelling. Support materials and construction, excavation and advance processes and their directions of development are also dealt with. Some sections have been dealt with in more detail though the research work carried out at the Chair of Construction Technology, Tunnelling and Construction Management at the Institute for Structural Engineering at the Ruhr University, Bochum in the fields of shotcrete, steel fibre shotcrete an the driving of small-diameter tunnels, through my experience on the tunnels of new lines for German Railways and also through my former work into the drilling of blast holes. An extensive provision of illustrations shows numerous practical examples and the tables contain much technical data.

This handbook is based on my lectures for the specialised course "Construction process technology and operations". I would like to thank the managing director of the publisher Verlag Glückauf GmbH, Dr.-Ing. *Rolf Helge Bachstroem*, for the encouragement to develop this handbook from my lecture notes on Underground Construction. In the course of long discussions, he advised me about the final version of the text, assisted in the selection of illustrations and tables and made constructive suggestions for many improvements as publisher and expert. I have also received valuable support from my employees at the Construction Technology, Tunnelling and Construction Management and in the consultancy. I wish to thank Oberingenieur Dr.-Ing. *Dietrich Stein*, my brother Dipl.-Ing. *Reinhold Maidl* and particularly Dipl.-Ing. *Harald Brühl* for their intensive collaboration. I thank *Agatha Eschner-Wellenkamp* for her inexhaustible industry with the writing work, and *Helmut Schmidt* and *Walter Zamiara* (publisher) for the preparation of many drawings.

Volume II will have the subtitle "Basics and auxiliary works in design and construction"; the volume should include geotechnical aspects, rock classification, stress states in the rock mass, structural verifications, monitoring instrumentation, dewatering, surveying and scheduling.

Bochum, in January 1984 *Bernhard Maidl*

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1 Introduction

1.1 General

Tunnelling is one of the most interesting, but also the most difficult engineering disciplines. It unites theory and practice into its own construction art. For the weighting of the many influential factors, practice is sometimes more important, and at other times theory, according to ones own state of knowledge. Tunnel engineering is normally performed by civil engineers. Everyone, however, should be aware that knowledge about structural analysis and concrete engineering alone is not sufficient. Geology, geomechanics, mechanical engineering and particularly construction process technology are equally important.

1.2 Historical development

Tunnels and caverns already existed in nature before mankind started to create them artificially to meet vital interests.

Tunnel engineering in the 20th century could also make use of existing specialised knowledge from mining. One of the founding fathers was Georg Agricola, whose 1556 work De Re Metallica, Libri XII covered mining and metallurgy.

Drill and blast

The building of significant tunnels in the Alps had already led to a first heyday of tunnelling before 1900, which explains why the railway engineer Franz Ržiha, mining superintendent of the duchy of Braunschweig, considered tunnel engineering as a separate discipline from mining in his 1867 textbook of tunnelling. This heyday continued to the start of the 20th century, after which there were only a few spectacular tunnel projects (Table 1-1) until 1960. The building of the Mont Blanc Tunnel was the start of a new phase in Europe, which continued with the construction of the Tauern Autobahn Tunnels, the Arlberg Tunnel and the new Gotthard Tunnel. The construction of more than a hundred tunnels by the German Railways (Deutsche Bundesbahn, later: Deutsche Bahn AG) continued the development. A new phase opened with the Seikan Tunnel, the Channel Tunnel and the base tunnels through the Alps.

The extent of the enormous development in tunnelling enabled by currently available support materials and machinery is illustrated in Fig. 1-1 and Fig. 1-2 for conventional tunnelling. The introduction of shotcrete as a means of support introduced a new phase of development, which made much greater use of machinery. Only years later did mechanisation take off again, permitting simultaneous working at the face and removal of the excavated material. The development of tunnel boring machines was even more impressive, and this is dealt with in detail in Chapter 6.

Table 1-1 Historical overview of some notable tunnels

	Year	Length m	Excavated quantity m^3	Notes
Tunnel, water supply to Jerusalem (Palestine)	700 BC	540	20,000	Broken out with hammer and chisel
Eupalinos Tunnel (Samos)	500 BC	1,052	3,409	Broken out with hammer and chisel
Malpas Tunnel, Languedoc Canal (France)	1679 to 1681	175	9,000	Tuff partially loosened with fire
Galerie de Tronquoi canal, St. Quentin (France)	1803	500	30,000	Squeezing rock, partial-face excavation
Mont Cenis rail tunnel West Alps (France, Italy)	1857 to 1870	12,200	700,000	Drill and blast
St. Gotthard rail tunnel, central Alps (Switzerland)	1872 to 1878	14,990	1,100,000	Impact machine, drill and blast
Mont Blanc road tunnel, west Alps (France, Italy)	1959 to 1964	11,600	930,000	Drill and blast
Niagara Falls power station (Canada)	1950 to 1958		3,350,000	Drill and blast
Grande-Dixance power station group tunnel system (Switzerland)	1955 to 1964	150,000	1,500,000	Drill and blast
Gotthard road tunnel	1969 to 1980	16,322	1,300,000	Drill and blast
Arlberg Tunnel (Austria)	1974 to 1978	13,972	1,450,000	Drill and blast
Seikan Tunnel (Japan) Pilot tunnel	1964 to 1984	22,292	404,000	Initially TBM, then blast
Landrücken Tunnel for the new railway line Würzburg – Hannover	1983 to 1986	10,710	1,400,000	shotcrete lining, drill and blast
Channel Tunnel	1986 to 1993	50,450	10,028,749	Shield machine Ø = 8.72 m
Elbe Tunnel 4th bore	1997 to 2002	2,561	400,000	Shield machine Ø = 14.20 m
New railway line Cologne – Frankfurt Schulwald Tunnel	1997 to 2001	4,500	50,000	shotcrete lining
Rennsteig Tunnel A77	1998 to 2003	7,900	150,000	shotcrete lining
Lötschberg Tunnel SBB	1999 to approx. 2007	34,600	8,900,000	TBM, shotcrete lining
Gotthard Base Tunnel SBB	1999 to approx. 2011	57,000	13,300,000	TBM, shotcrete lining
Madrid M-30	2004 to 2006	8,344	1,506,092	Shield machine Ø = 15.16 m

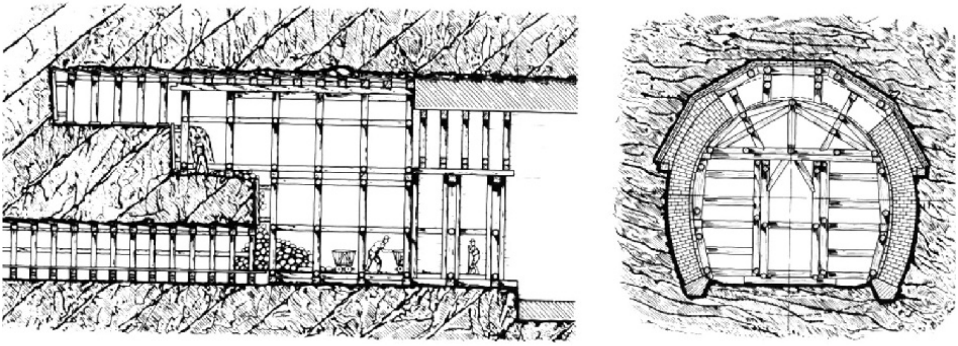


Figure 1-1 Construction of the Semmering Tunnel in 1848 using the old Austrian method [302]

Table 1-2 The data for the various Gotthard Tunnels

Project	Dates	Length km	Cross-section m²	Advance rate m/d	Cost, million sFr	Number of workers	Serious injuries	Accidental deaths
Rail tunnel	1872/81	14.9	45	3.5 to 4[a]	55.5	2500 to 4000	260 av. 8%	177 av. 5.4%
Road tunnel	1969/80	16.3	69 to 83 82 to 96	6[b]	560	up to 700	25 av. 3.5%	12 av. 1.7%
Base tunnel	1999/ 2011	57.0	41 standard, 250 MFS	approx. 1 to 20	About 10,000	Up to 1800	still being built	still being built

[a] In 2 x 12-h shifts and 7-day week. [b] In 2 x 10-h shifts (peak operation 3 x 8 h) and 5-day week.

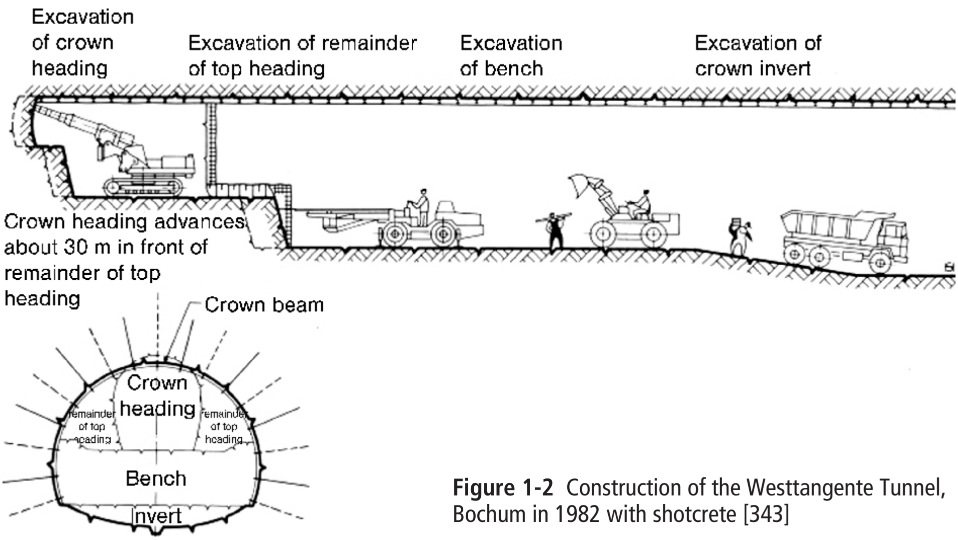


Figure 1-2 Construction of the Westtangente Tunnel, Bochum in 1982 with shotcrete [343]

Comparison of the data from the various Gotthard Tunnels in Table 1-2 shows that extremely short construction times were already possible many years ago, but the number of accidents has been greatly reduced by the modernisation of tunnelling technology.

Large underground structures have also been built for hydropower stations outside Europe, for example at Tarbela in Pakistan and Cabora Bassa in Mozambique. Many projects are still urgently needed, even if these have not yet been implemented due to financing problems. Underground and urban rapid transit lines, but also road tunnels will have to be extended to solve environmental and traffic problems in the cities. New developments in tunnel boring machines and in microtunnelling will assist the requirement in cities for environmentally friendly construction of extensive transport works, water supply and drainage, district heating, post and other utilities.

In German coal mining, development headings and drifts have amounted to annual totals of more about 100 km in rock and 400 km of coal roads in the past. As coal mining in Germany is discontinued, this will no longer be needed, although mining will maintain its significance outside Germany.

Tunnel boring machines

The development history of the first tunnel boring machine (TBM) has featured many tests that failed due to various problems, with the exception of the successful work of the Beaumont Machines in the Channel Tunnel. Sometimes the limitations of the available materials had not been considered, or else the ground to be driven through was simply not suitable for a TBM. Early applications proved successful where the ground offered ideal conditions for a TBM drive.

As early as 1851, the American Charles Wilson developed and built a tunnel boring machine, although he didn’t patent it until 1856. This machine already showed all the features of a modern TBM and can thus be described as the first tunnel boring machine; see also Chapter 3.4.

Shield machines

Tunnel builders learnt long ago to support unstable rock or loose soil with timbering followed by a masonry lining. This was also successful in rock with seepage or joint water, but working below the water table in permeable soil or particularly under open water remained impossible well into the 19th century. The situation changed in 1806, as the ingenious engineer Sir Marc Isambard Brunel in London discovered the principle of the shield machine and later obtained a patent. The purpose of the invention was the building of a link across the Neva in St. Petersburg that could remain open in winter. As this project was not built, the machine was only developed on paper for the patent. Brunel was only able to try out his ideas in practice on the Thames Tunnel in London (see Chapter 3.4 “The Classic Shield Machines”).

1.3 Terms and descriptions

In order to describe and understand underground structures, knowledge of the most important specialist terms and descriptions is essential. As there is not always a generally accepted term from the variety of terms derived from mining, preference has been given to those that have become most widely used and most precisely describe the subject (Table 1-3).

Table 1-3 Categories of underground structures [50, 51]

Structure	Examples	Purposes
Tunnels	Rail tunnels, Underground rail tunnels, road tunnels, canal tunnels	to provide transport routes
Small tunnels	*Main structures*	
	Unpressurised tunnels, pressure tunnels, siphons	transport of drainage water, drinking water and service water
	Access tunnels	all-year access to caverns with avalanche protection
	Ventilation tunnels	supply of fresh air to underground cavities
	Auxiliary structures	
	Grouting tunnels	access for grouting works
	Pilot tunnels	investigation of geological conditions
	Viewing tunnels	surveying of underground structures
	Adits	to provide an additional starting point
Shafts	*Main structures*	
	Inclined and vertical shafts	transport of drinking water and drainage pressure relief (surge) in hydropower stations supply of fresh air to underground cavities transport of personnel and material
	Auxiliary structures	
	Mucking shaft	transport of excavated material
	Surveying shaft	surveying
Pipelines	Sewers and drains Water supply pipes District heating pipes Gas supply pipes Oil pipelines Cable ducts	transport of goods, energy or news
Caverns	Industrial caverns	to house power station turbines or assembly halls
	Storage caverns	storage of goods
	Protection caverns	provision of underground shelters for the population in case of air raids or military bunkers
Chambers	Storage chambers	storage of goods
	Explosive chambers	storage of explosives during the construction of a tunnel

Underground structures can be categorised according to the purpose of the completed structure:

Tunnels are extended, flat or only slightly sloping underground cavities with excavated cross-sections of over 20 m^2. They are mostly intended for road or rail transport. Each tunnel has two openings to the surface.

Adits, drifts or galleries are extended underground cavities, horizontal or sloping at less than 25° to the horizontal, with small diameters. They house pipes or cables or provide access and serve as auxiliary structures during the construction phase or for permanent use. They often only have one opening to the surface.

Shafts are extended, underground, vertical or inclined (more than 25° to the horizontal) cavities to overcome level differences. They serve similar purpose to adits, drifts or galleries.

Underground **pipes** mostly have inaccessible cross-sections. They serve to transport liquids, heat or gases and to house cables (ducts).

Caverns are underground cavities with large cross-sections and relatively short lengths. They serve for the storage of solid, liquid or gaseous goods, to house machinery and vehicles, underground generation plant, assembly halls and military facilities. They are normally connected to the surface through tunnels, adits or shafts.

Chambers are small compact underground cavities. They serve for the storage of goods during construction work or permanently.

The terms for individual parts of a structure that are normally used in underground construction are shown in Figs. 1.3 and 1.4 in cross-section and Fig. 1-5 along the tunnel. The basic terms are explained in more detail below.

Tunnel driving denotes the entirety of excavation works to advance underground cavities.

Temporary support denotes the temporary support of the excavated cavity until the complete installation of the permanent lining. The temporary support can also provide part of the structural function of the permanent lining if it is integrated. The temporary support may also be described as:

- excavation support.
- temporary lining.
- outer lining.
- primary lining.
- shoring, timbering or lagging.

The **lining** provides the structural support of the cavity and waterproofing measures. It may also be described as the secondary lining, inner lining or permanent support.

Installations are structures and fittings required for the operation of the completed tunnel. This can include dividing slabs and partitions, wall linings, cable ducts and channels and technical equipment.

The **construction process** denotes the entirety of the technical and organisational measures used for the implementation of the tunnel drive, the temporary support and the lining. The construction process is characterised by the *construction method* and the *operational method*.

The **construction method** is the sequence of construction activities in the excavated cross-section and refers to the division of the excavation cross-section into partial sections for excavation, temporary support and lining (see Fig. 1-1 to 1.3).

The **operational method** is the sequence of construction activities along the tunnel and refers to excavation and support activities, but also to supply and disposal activities along the entire length of the tunnel (see also Fig. 1-1 and Fig. 1-2).

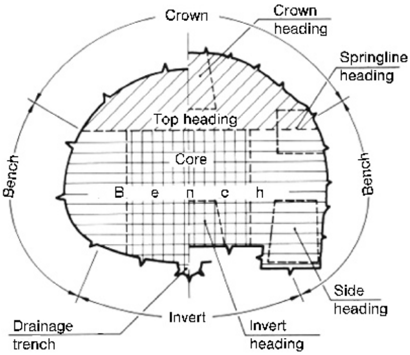


Figure 1-3 Description of the parts of a cross-section

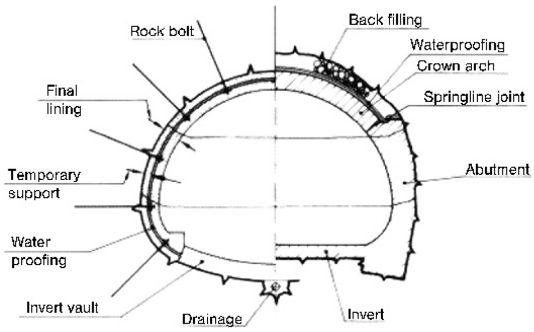


Figure 1-4 Description of the parts of the support system

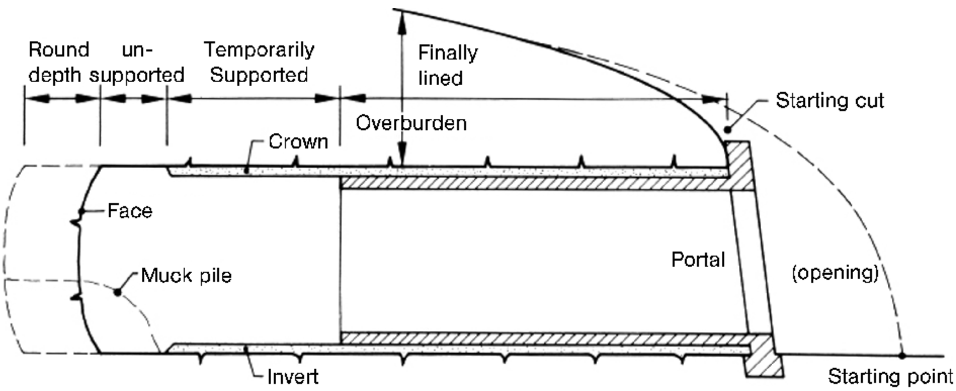


Figure 1-5 Description of the parts of a longitudinal section

Tunnel portals are the structure provided at the ends of a tunnel to support against slope sliding, lateral earth pressure and falling rock (Fig. 1-5). Tunnel portals should also fit the tunnel into the landscape.

Waterproofing is the description of measures to protect the structure against water ingress from outside and also to prevent the escape of liquids to the outside.

Isolation denotes measures to protect the structure, adjacent buildings and cables against unwanted electrical effects.

2 Support methods and materials

2.1 General

For the construction of structures in rock, it should generally be the intention to preserve the load-bearing capacity of the surrounding rock mass. Support measures, both temporary or permanent, are intended to enable, assist and favourably influence the load-bearing contribution of the surrounding rock mass, and should ideally serve to reinforce the surface of the rock mass. Support measures possess more or less support stiffness, which can be calculated. The following preconditions are necessary to preserve the load-bearing capacity of the rock mass:

- The selection of a suitable shape for the cross-section, which considers the rock mass properties encountered.
- The selection of suitable methods for construction and operation.
- The selection of a suitable support procedure.
- Consideration of the time factor for rock mass and support.
- The use of excavation methods, which are as gentle as possible to the rock mass so as to reduce its strength as little as possible.

The boundaries between temporary and final support will become blurred rather than exactly defined in the future. Future developments are trending towards single-layer construction, with the temporary support having to be integrated into the overall final support system. In order to reflect this development, support materials are considered together in this chapter, with the emphasis on a complete description of materials and stages in development history, even if some of these are no longer up-to-date. In tunnelling, which has a wealth of tradition, some old ideas have already been revived.

2.2 Action of the support materials

The structural behaviour of the support materials is determined by their deformability or stiffness, the degree of bonding between support and rock mass and the time of installation (Fig. 2-1 and Table 2-1).

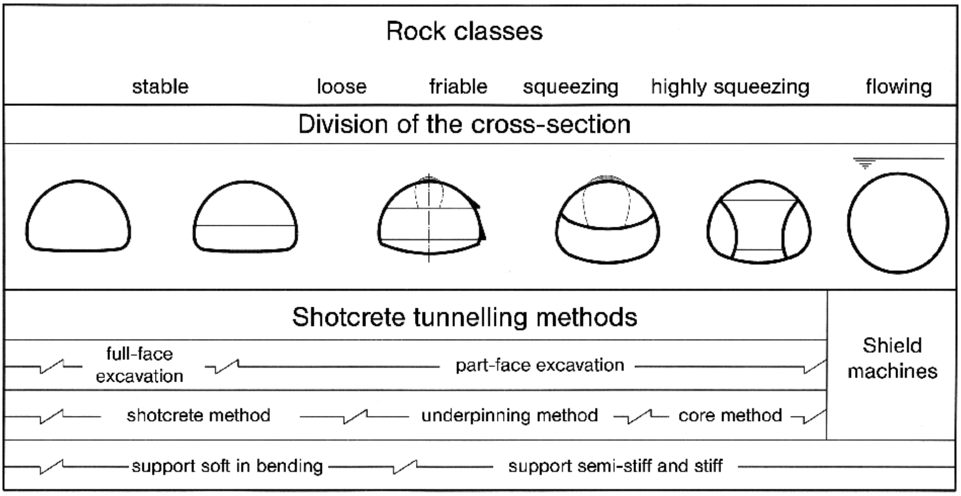


Figure 2-1 Categorisation of shotcrete processes according to rock mass classes [243]

2.2.1 Stiffness and deformability

Support systems are regarded as highly stiff in bending (rigid) when they can stand up freely without affecting the rock mass and only show negligibly small deformation under load (Table 2-1). As deformations are small when highly stiff support systems are used, restoring forces in the rock mass to reduce the bending moments in the lining can not be assumed. For the loading on the lining, the prevailing vertical and horizontal ground pressures, and particularly their relationship, are of great significance. As this is however generally not known, many limit cases of loading have to be investigated. After a highly stiff support system has been installed, subsequent stress redistribution in the rock mass is prevented. Because the loading that occurs is larger than with a deformable lining, highly stiff support systems should if possible only be installed after stress redistribution has largely taken place, in order to be cost-effective. Linings that are highly stiff in bending are today mostly made of reinforced concrete and relatively thick. Considering the cost, they should only be used for the final lining.

Table 2-1 Correlation of terms

Stiffness of the support	Deformability	Resistance to section forces	
		Bending moments	Normal forces
highly stiff in bending	rigid	high	low
stiff in bending	semi-rigid	high	low
weak in bending	highly deformable	low	high (plus shear forces)

Support systems that are stiff in bending (semi-rigid) are stable without the surrounding rock mass, but also deformable so that they can evoke structural response from the rock mass by stress redistribution. They can be used as temporary or final support, although mainly for the final support. Good bonding with the rock mass is important.

Some examples of the application of support systems that are highly stiff or stiff in bending are:

– Tunnels, which are constructed in an open excavation and subsequently backfilled (except for special linings like Armco Thyssen).
– Tunnels in heavily faulted zones.
– Tunnels with very shallow cover.
– Tunnels with weak bonding or no bonding between support and rock mass.
– All examples, where no structurally active ring can form in the surrounding rock mass.
– Low deformations and settlements.

Support systems that are weak in bending (highly deformable) are not stable under load on their own, but require the interaction with the surrounding rock mass. The support should be considered at the limit state as a reinforcement of the edge; it resists normal forces and shear forces, but no bending or only very slight bending. The normal application should be as yielding temporary support. It mostly consists today of unreinforced or reinforced concrete or shotcrete, also steel fibre shotcrete. Deformations are only permissible to such a magnitude that the assumption of restoring forces is justified. The bond with the rock mass is particularly important. The structural design should check the shear forces. The deformation capability of the lining can also be improved by deformation gaps and yielding elements (see Chap. 2.8.7) in special cases where deformation could otherwise not be controlled.

2.2.2 Bond

The bond between the support and the rock mass should ensure the transfer of radial forces over the whole area and the continuous transfer of tangential forces. Except for rare cases, the transfer of radial forces remains the most important requirement, as the shear transfer between the support measures and the rock mass is not normally assumed in calculations. If tangential forces are assumed, then the magnitude of the shear strength of rock mass and concrete has to be estimated or determined in tests. If a waterproofing membrane is installed between rock mass and support, or between temporary and final support, then no tangential forces can be transferred.

Support measures consisting of timber, steel, precast concrete elements or other assembled parts only support the rock mass at points, so there is no bonding effect. If shotcrete or pumped concrete is used with subsequent grouting (crown filling), then it can normally be assumed that there is a bond at the edge of the excavation between support and rock mass, which greatly influences the transfer of forces between rock mass and support.

2.2.3 Time of installation

If the rock mass is unstable, temporary support should be provided during or even before excavation. If the rock mass will stand up temporarily, the support is installed after excavation. In a stable rock mass, support is not generally necessary, although head protection may have to be considered. If the elastic condition of the rock mass can be largely preserved, then the support is installed early. If, however, the creation of large areas with plastic deformation in the rock mass is unavoidable, then installation after a controlled delay can lead to limited deformation of the rock mass and thus to reduced and bearable ground pressure.

Deformation of the rock mass leads to a reduction of the cavity cross-section, and this has to be taken into account in the design of the cross-section. If it is assumed that a deformable edge reinforcement increases the load-bearing behaviour of the structurally active ring in the surrounding rock mass, then it should be installed as early as possible as shotcrete or steel fibre shotcrete with high early strength. The jointed body bonding of the structurally active ring is also preserved; rock falls that could disturb the geometry of the load-bearing vault are avoided and the rock mass is protected against weathering. Immediate sealing of the exposed surface is especially important in ground susceptible to swelling.

For the time of installation, not only the support of the crown and sides is important but also the closure of the ring. This can be achieved through support measures like concrete, shotcrete, steel fibre shotcrete, precast reinforced concrete elements, steel ribs or rock bolts; or in the appropriate ground conditions by the rock mass itself. The ring closure time and the ring closure distance should be differentiated. The ring closure time is the time from the opening of the face to the installation of support measures to produce a load-bearing ring. The ring closure distance is generally the distance between the face and the location of the load-bearing ring closure. The ring closure distance is determined by operational considerations and mainly ensues from the construction method and the support materials used. For example, steel fibre shotcrete can favourably reduce the ring closure time and distance due to the reduced number of working steps and its high early strength behaviour.

2.3 Timbering

2.3.1 General

Timber is only used for temporary support and should always be removed. The original forms of longitudinal and transverse timbering are hardly ever used today. This is a result of the development of newer, more suitable support materials like steel, shotcrete, steel fibre shotcrete and rock bolts. Special cases for the application of timbering could still be: partial collapses, transition profiles and particularly as emergency support after collapses (Fig. 2-2). Timber is also suitable as a reserve material for immediate measures due to its adaptability and should be available on all tunnel sites.

Advantages and disadvantages of timbering are:

Advantages:

– Any critical increase of ground pressure is visibly and audibly indicated.
– Easy to transport.
– Easily worked and adaptable.

Disadvantages:

– No bonding with the rock mass.
– Large deformations under load.
– Seldom reusable.
– The support has to be removed, making re-bracing and underpinning necessary.
– Qualified craftsmen required.

Figure 2-2 Timbering as emergency support after a collapse in the Euerwang Nord Tunnel [136]

2.3.2 Frame set timbering

As the tunnel is advanced, poling boards are pushed over the cap pieces (Fig. 2-3) or driven into the ground. The space between boards and ground is wedged or stowed with stones. If high ground pressure is expected, then a sill piece is also installed. The normal spacing of frame sets is 1.0 to 1.5 m. In squeezing ground, the spacing can be so small that the sets are directly next to each other. This method is suitable for headings with partial face excavation and trapezoidal full-face excavations up to 9 m^2.

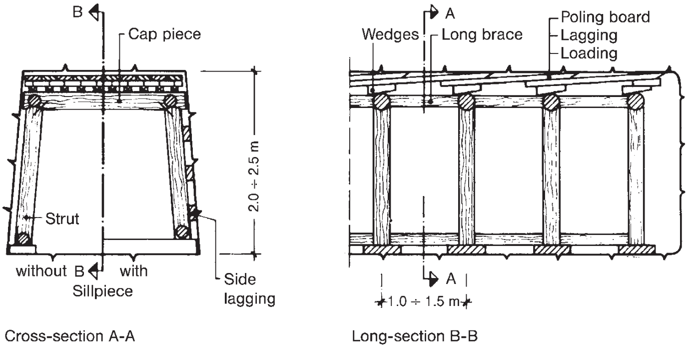


Figure 2-3 Frame set timbering. Lagging with timber poling boards

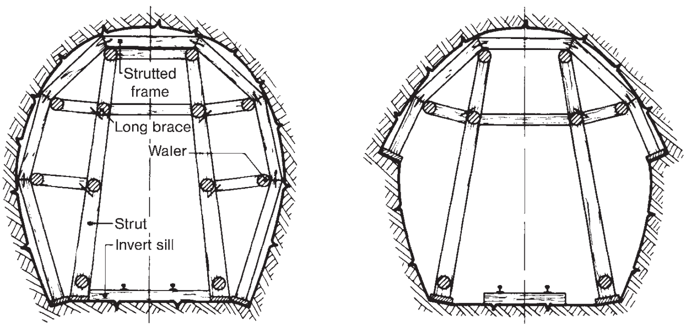


Figure 2-4 Trussed timbering. Fully braced cross-section (left) and strutted frame supported at the sides

2.3.3 Trussed timbering

Trussed timbering is a multi-part truss supported from the sills, a support construction or from the tunnel sides. As truss timbering is laterally unstable, bracing is important (Fig. 2-4).

2.3.4 Shoring and lagging

If the ground is friable, then the space between the sets or ribs has to be supported. This is done with lagging or forepoling boards, which rest on timber frames or a steel construction and are pushed forward, driven forward, pressed or simply installed in contact with the ground (Fig. 2-5). When the lagging can be pushed over an already installed frame, this is described as forepoling.

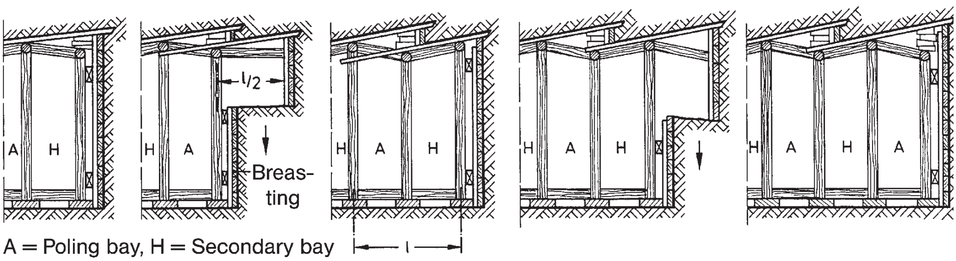


Figure 2-5 Driven lagging of forepoling boards

Timbering provides the support with driven lagging. The basic method is still in use today although with other materials as timber.

2.4 Steel ribs

2.4.1 General

Steel has been increasingly used instead of timber since the middle of the 20th century, since steel enables standardisation of support elements and prefabrication of the support. Steel support ribs are made of rolled profiles, U profiles, special mining profiles or composite sections, as closed arches or open at the bottom [240].

Advantages:

- Prefabrication is possible.
- Immediate load-bearing if in contact with the rock mass.
- Can be installed vertically or inclined according to the form of the face.

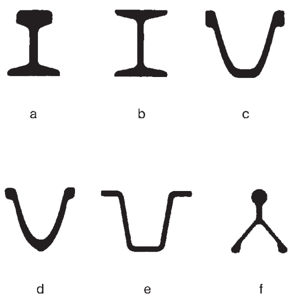
Disadvantages:

- Heavy profiles are difficult to handle.
- Poor flexibility.
- Long ordering times.

2.4.2 Profile forms

In contrast to the profiles that are commonly used in steelwork, special profiles have been developed for mining, which were than used for engineered tunnelling. Fig. 2-6 shows some types from various manufacturers; the criteria for their development were: large moment of inertia with small cross-sectional areas and weights, high strength and easy handling.

The steel quality can be DIN 21530-3 [71] type 31 Mn 4 and S235JR and S355JO according to DIN EN 10027 [76].



a Rail
b GI profile (Bochumer Eisenhütte Heintzmann GmbH & Co KG; Thyssen Industrie AG)
c TH channel profile (Bochumer Eisenhütte Heintzmann GmbH & Co KG)
d Bell profile (Bergbaustahl GmbH)
e Channel profile (Bergbaustahl GmbH)
f Star profile (Bochumer Eisenhütte Heintzmann GmbH & Co KG)

Figure 2-6 Steel profile shapes commonly used in German tunnelling and mining

2.4.3 Examples of typical arch forms for large and small tunnels.

The geometrical form of steel arches is determined by the intended cross-section of the excavation. The ribs are normally made to size (Fig. 2-7), although there are standardisations for mining.

For better or simpler handling, ribs are made in two or three parts. A variety of butt connections have been developed to meet different requirements. Fig. 2-8 shows, for example, rigid and yielding butt connections. The manufacturers will have to be contacted for special requirements like defined yielding or friction forces. Such problems occur in mining for the overcoming of large convergences or also in tunnelling with large-scale plastic deformation of the rock mass.

Steel ribs are anchored with special anchor clamps, or the anchors can pass directly through the steel profiles. Some examples are shown in Fig. 2-9.

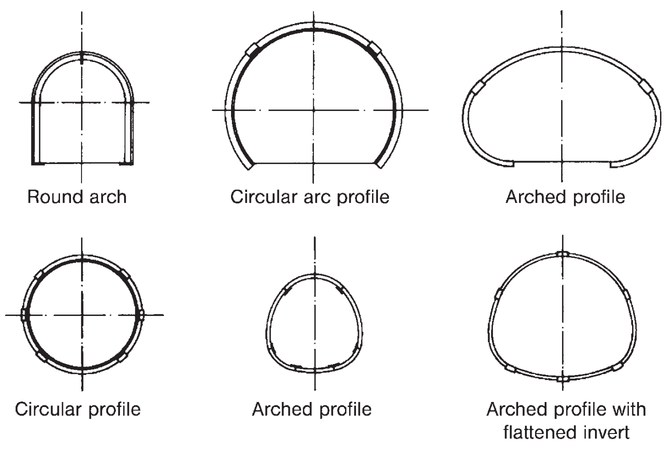


Figure 2-7 Typical arch forms with open invert (top) and with closed invert (bottom)

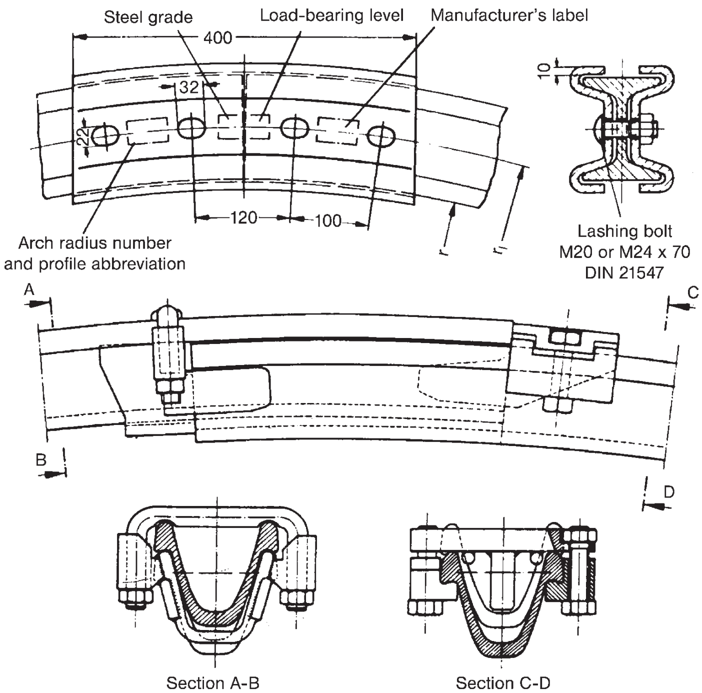


Figure 2-8 Examples of rigid (top) and yielding butted connections

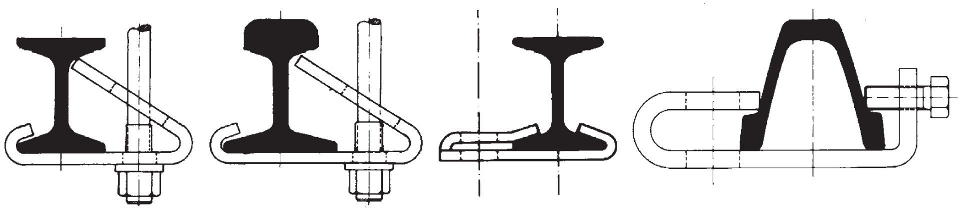


Figure 2-9 Examples of anchoring clamps for rails , GI, SI, TH, and GP profiles

2.4.4 Installation

Steel ribs can be erected immediately after excavation of a round. Special equipment can be used to simplify installation and also permit the use of larger arch segments, which reduces the number of butted connections. Erectors for ribs are used particularly with tunnel boring machines and shield machines. A simpler way of assisting installation is the use of longitudinal beams fixed to the already installed ribs.

Sufficient longitudinal stiffening and bracing is important in order to avoid deformation, sideways buckling or movement of the ribs. The feet of the arches should also be secured with rock bolts into the rock mass or stiff connections to each other. Propping against the rock must be improved by packing or grouting in the course of further work.

Longitudinal stiffening or bolting is normally provided by a sufficient number of spacing bars, which locate the ribs to prevent movement and have to be able to resist tension and compression forces, particularly with ungrouted profiles. When welding on site is intended, it should be noted that the higher permissible stresses of tempered steel can no longer be fully exploited; the supervisory structural engineer would object to such a solution on site. It is therefore recommended to order a longitudinal bracing system together with the prefabricated steel arches. If the ribs are installed with shotcrete, then a reusable longitudinal bracing system can be used.

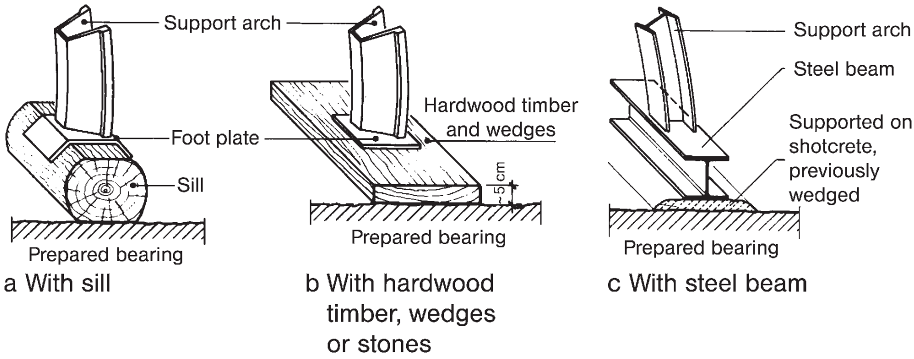


Figure 2-10 Examples of foot support

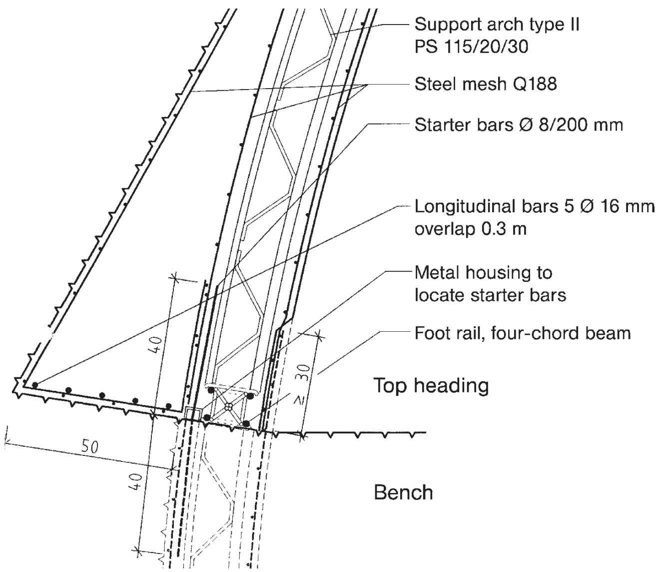


Figure 2-11 Structural detail of foot support with foot rails and starter bars

The advantages of immediate load-bearing capacity can only be exploited when the foot support is designed and constructed so that the loads can be transferred into the ground immediately in a suitable way. Details of the point of transfer of loading into the ground are shown in Fig. 2-10.

Fig. 2-11 shows a structural detail of a foot support.

Contact with the rock mass is achieved by local wedging or bracing against the excavated surface, a technique used in German mining and in American tunnelling. The steel ribs are wedged behind with timber. Under load, the arch is loaded at points and only uses the advantage of continuous and immediate load-bearing around the arch to a limited extent. There is naturally no bonding effect between support and rock mass.

If settlement has to be kept very small, closed steel arches equipped with expanding presses can also be used (see Fig. 2-12).

Good contact with the rock mass is normally achieved by immediately bedding the steel rib in shotcrete. This wall-type structural action exploits the immediate contact with the rock mass and the associated bonding effect. Fig. 2-13 shows various ways of filling steel ribs with shotcrete or steel fibre shotcrete. In contrast to mining, immediate spraying is normal in European tunnelling. Shotcrete containing steel fibre reinforcement offers a simplification by reducing the number of working steps and the thickness of the layer. The load-bearing capacity of a shotcrete vault between the ribs in mostly much greater than the ribs on their own.

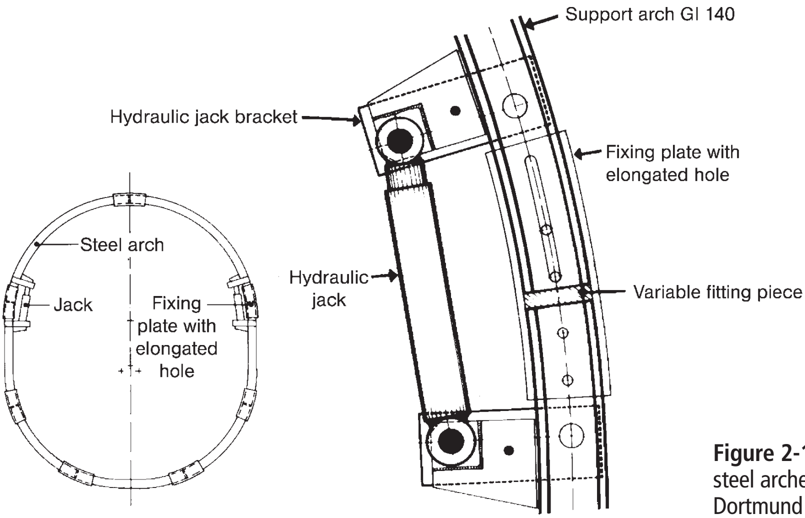


Figure 2-12 Expanding steel arches, Stadtbahn Dortmund contract 13 [82]

If unfavourable geological conditions on site mean that shotcrete alone will not provide adequate support, and an immediately load-bearing support system is required for safety reasons, steel arches are used as structural elements. The arches are erected, sprayed in and loaded by the rock mass through the shotcrete. As it hardens, the stiffness of the shotcrete increases and it resists the increasing ground pressure. Lighter ribs with low support resistance can also be used as a surveying aid and for better maintenance of the excavation profile. The concrete between the ribs then acts as an independent structural element. It serves to brace the ribs against each other and also ensures the structural connection between the ribs and the rock mass. Steel arches can be taken into account in the structural design of the outer lining (see Ril 853 [296] and ZTV-ING Part 5 [384]).

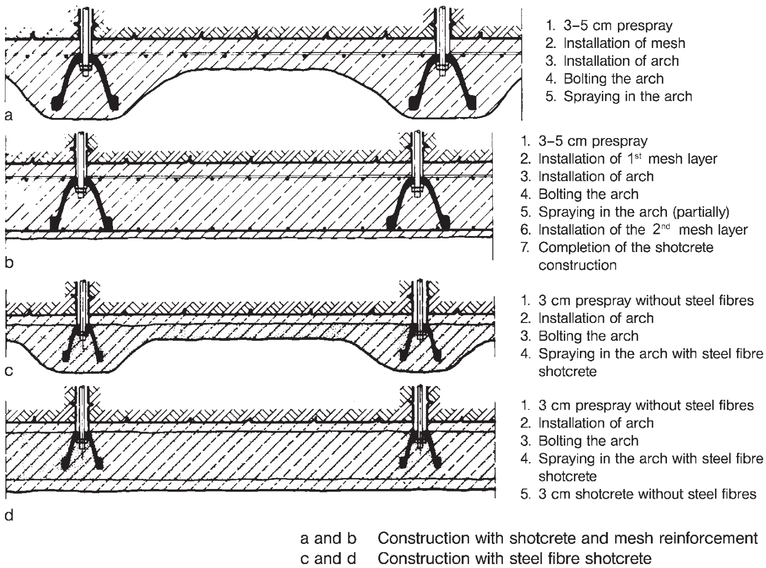


Figure 2-13 Sprayed filling of steel ribs

2.5 Lattice beam elements

Prefabricated reinforcement elements can be installed with shotcrete or cast concrete as temporary and permanent support. Like steel ribs, they also provide protection and assist profiling. For design purposes, they are considered as reinforcement, meaning that all DIN requirements apply and lapping bars are required at butted joints (see Fig. 2-14). Fig. 2-15 shows a detail of the construction of a lattice beam.

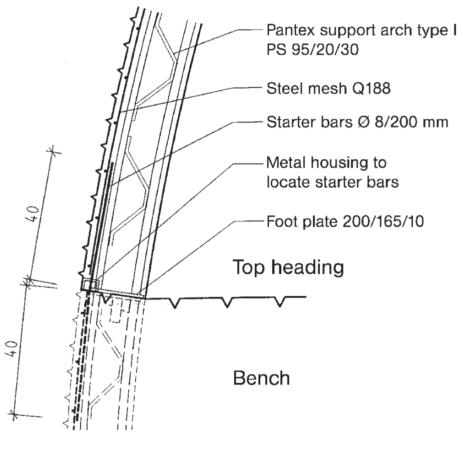


Figure 2-14 Lapping reinforcement at butted joints of between top heading and bench

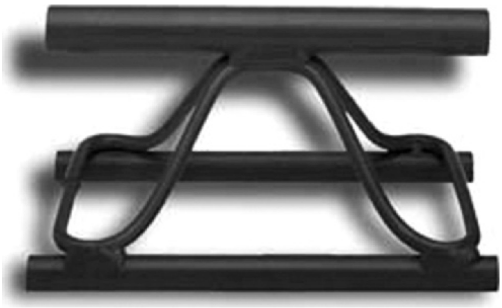


Figure 2-15 Detail of a Pantex 3-chord PS beam; TAT GmbH

Partial spraying-in produces a skeleton load-bearing element, and complete spraying produces a wall-type element (Fig. 2-16). The number of working steps and the wall thickness can be reduced if steel fibre-reinforced shotcrete is used. The solutions shown in Fig. 2-16 a and d are not recommended because the protruding part of the lattice beam is fouled by shotcrete and is extremely difficult to clean before the spraying of the next layer. Of the single-layer solutions, only c can be recommended, with reservations also b, as long as the layer is completed within one day before the surface of the shotcrete becomes dirty or dusty. If solution d is used, it is important that the lattice beams are not dirtied by the shotcrete.

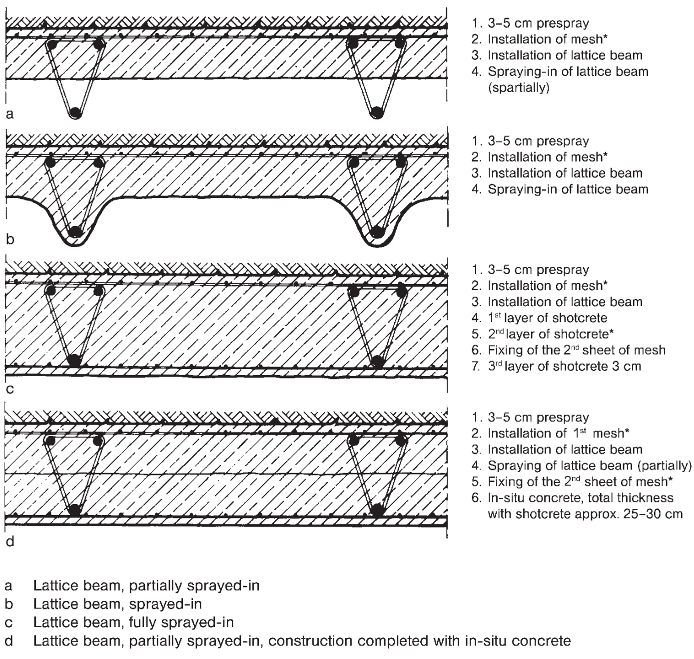


Figure 2-16 Sprayed-in lattice beams with mesh reinforcement (the working steps shown with a star can be omitted when steel fibre shotcrete is used)

The working joint between the individual rounds should be positioned outside the lattice beam (Fig. 2-17 and Fig. 2-18). For single-layer solutions in the groundwater, it can be appropriate to provide a filter layer behind the shotcrete layer to enable grouting if these locations leak water.

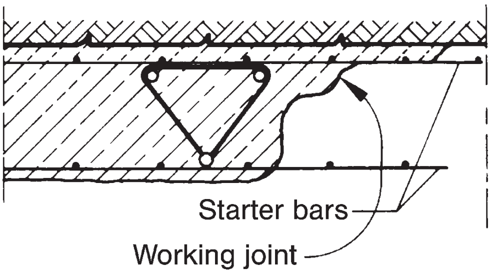


Figure 2-17 Working joint in a permanent shotcrete lining with mesh reinforcement. Care should be taken that the sheets of mesh, which form the starter reinforcement for the next section, are not dirtied by the subsequent spraying of shotcrete

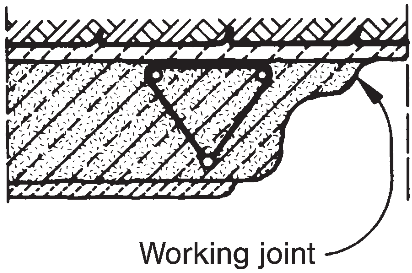


Figure 2-18 Working joint in a permanent shotcrete lining with steel fibre shotcrete. Care should be taken that the unreinforced shotcrete applied as a protective layer does not get into the area of the working joint

According to the profiles of the excavation and structure, it can be possible to prefabricate cross-sectional shapes. As with wall-type structural elements, the geometrical cross-section shapes are based on the intended excavation cross-section. The various profile shapes are easier to handle than full-wall profiles due to their lighter weight. Lattice beams, however, need similarly greater construction depths, which means that thin shotcrete layers can no longer be achieved.

2.6 Advance support measures

Local worsening of the rock mass properties, particularly the shortening of the stand-up time of the rock mass after excavation, may require special advance measures at the face. Advance support measures can also be used when low settlement is specified, for example when tunnelling below buildings or sensitive facilities. Depending on the project-specific local conditions, the following measures could be appropriate:

– Steel lagging sheets and plates.
– Spiles, tube spiles.
– Canopies, as pipe screen or jet grouting screen.
– Ground freezing.

The individual measures described here have been well known and used to a lesser or greater extent for many years and are increasingly and systematically being integrated into the tunnelling concept.

2.6.1 Steel lagging sheets and plates

Closed lagging with driven steel sheets, “Cologne support” (Fig. 2-19), is similar to classic tunnelling with frame sets and forepoling, except steel sheets are used instead of the former timber elements. The support arch corresponds to the cap piece of the frame set; it is only used for temporary support, as is the head support, which advances with the excavation. One variant of this method is the driving of the sheets over only one field, which however tends to break the spiles or sheets if they are not driven far enough into the unexcavated ground.

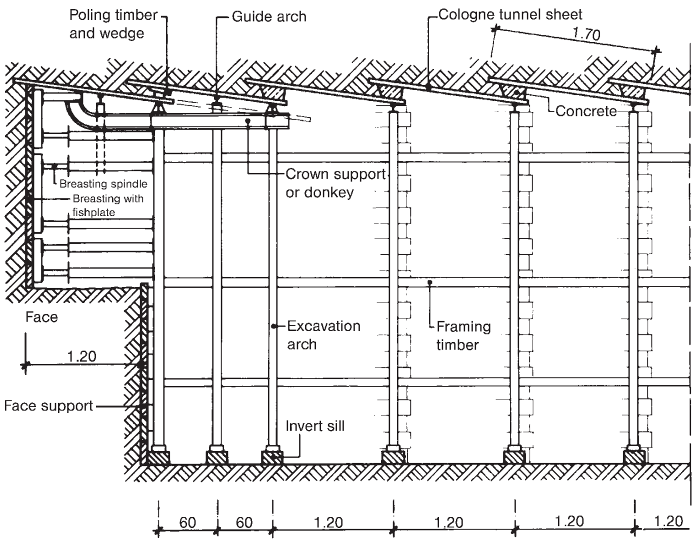


Figure 2-19 "Cologne support": driven steel lagging sheets

In non-cohesive soils, the soil tends to run out, and cavities tend to be created particularly at the sheet overlaps. Mortar filling behind the driven sheets has been found useful for the reduction of settlement (Fig. 2-20).

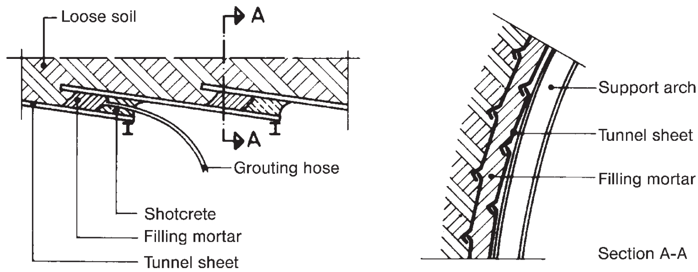


Figure 2-20 Mortar filling behind steel sheets (Stadtbahn Mülheim, contract 7, 1978) [90]

Lagging driven with gaps between in combination with shotcrete combines the advantages of traditional support with those of shotcrete. As the tunnel advances, the sheets are driven with gaps and sprayed with shotcrete in order to reduce settlement resulting from any deflection of the sheets under increasing ground pressure (Fig. 2-21).

Lagging sheets are also used together with timber frame sets and are normally driven into the soil (Fig. 2-22). Lagging sheets provide the support between the ribs. They are normally placed and the gap to the excavated ground is backfilled.

Driven lagging sheets can be rammed with or without side guides and are available in a great number of types to suit various loadings. Lagging sheets have a trapezoidal cross-section and are placed over already positioned arches.

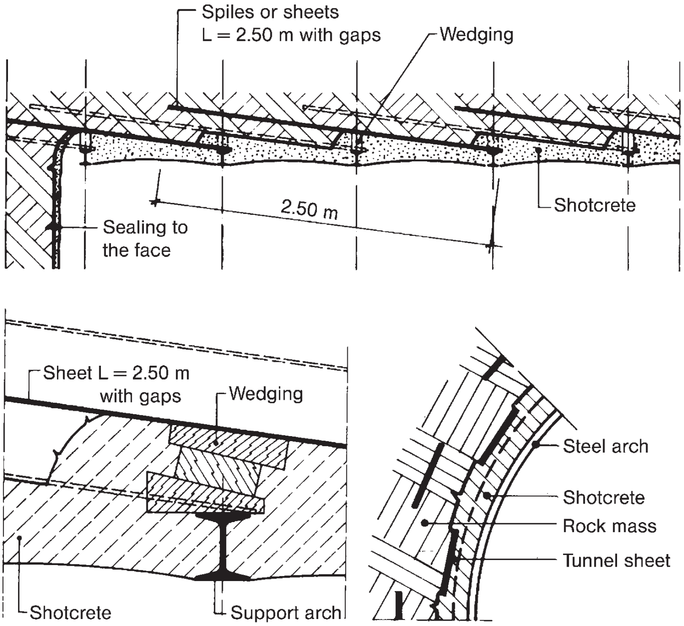


Figure 2-21 Lagging sheets driven with gaps and sprayed with shotcrete (Stadtbahn Dortmund, contract 13, 1979) [82]

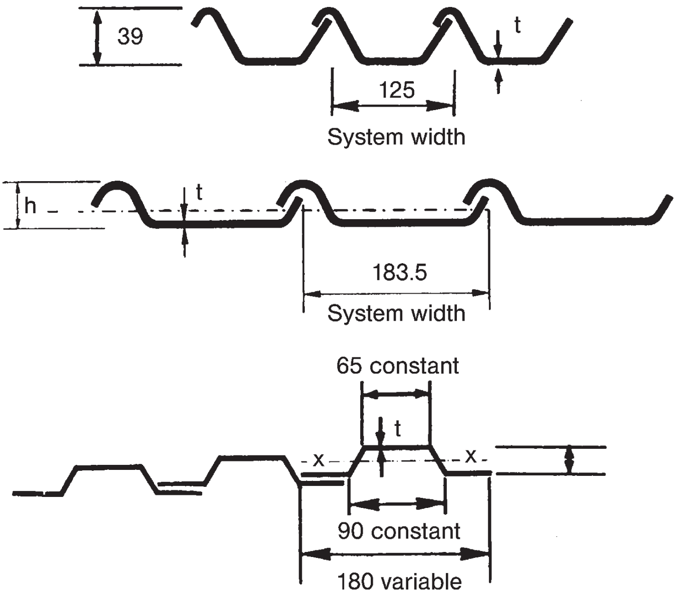


Figure 2-22 Driven lagging sheets (top and middle) and lagging plates; Thyssen Industrie AG

2.6.2 Spiles

Spiles have long been used to support the crown in brittle ground or ground susceptible to rock falls (Fig. 2-23).

Figure 2-23 Use of spiles in the Euerwang Tunnel

The action of spiles is more to preserve the geometrical shape of the excavation and thus the load-bearing ring of rock around the tunnel than to bear load themselves. The length of the spiles should extend over three rib spacings, and they are driven over each rib or every second (Fig. 2-24).

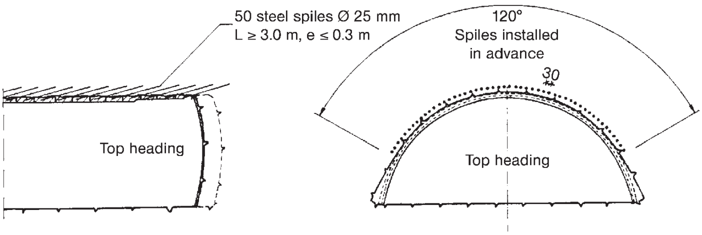


Figure 2-24 Application example: crown support with spiles

Depending on the geological conditions, rammed spiles or driven spiles can be used (Table 2-2). Rammed spiles are used without grouting in loose ground, being driven with compressed air or hydraulic hammers. Bored spiles are used in rock and installed with or without mortar into predrilled holes.

Table 2-2 Crown support with spiles

Process	Application (purpose)	Application (ground, geology)	Description	Equipment
Rammed spiles	– crown support	– heavily weathered rock (suitable for ramming) – cohesive soil	– steel bar (e. g. Ø 25 mm) rammed	– drilling rig
			– steel tube (e. g. int. Ø 38) rammed	
Bored spiles		– (weak) weathered rock (not suitable for ramming)	– steel bar (e. g. Ø 25 mm) driven into hole without grouting – steel bar (e. g. Ø 25 mm) driven into hole filled with mortar – steel tube (e. g. int. Ø 38) pushed into hole filled with mortar	– drilling rig – grout pump

2.6.3 Injection tubes

Injection tubes, as a further development of mortared spiles, are used in brittle to squeezing rock to provide advance support (Fig. 2-25). With this method, the rock is additionally improved by injecting suitable grout into the existing cavities. This increases the structural stability of the rock and simultaneously reduces water permeability.

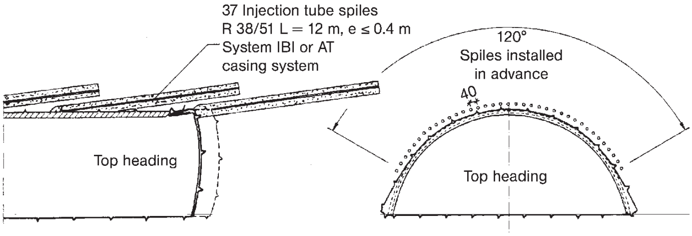


Figure 2-25 Application example: advance support with injection tubes

Injection tubes can be categorised into rammed injection spiles, self-drilling injection tubes and injection tubes (Table 2-3). With self-drilling injection tubes and rammed injection spiles, the grout can be injected once through the tip of the tube after installation. Injection tube spiles also enable secondary grouting through additional secondary grouting valves.

Table 2-3 Advance support with injection tubes

Process	Application (purpose)	Application (ground, geology)	Description	Equipment
Rammed injection spiles Self-drilling injection tubes	– advance support of the crown	– loose ground and rock suitable for grouting – non-cohesive soil where drilled holes would be unstable	– threaded tubes (e. g. dia. 25, 32, 38 mm) with ramming tip or drill bit – injection (e. g. cement grout) through the tip of the tube – drill heads etc. screwed on the external thread	– drill rig – grout pump – drill head, sleeves, spile tips
Injection tube spiles		– loose ground and rock suitable for grouting – non-cohesive soil where drilled holes would be unstable	– threaded tubes (e. g. dia. 25, 32, 38 mm) with drill bit – injection (e. g. cement grout) through the tip of the tube and additional grouting valve for secondary injection – drill heads etc. screwed on the internal thread	– drill rig – grout pump – drill head, sleeves, spile sections, spile sections with secondary valves

2.6.4 Pipe screens, grout screens, jet grout screens

In ground where the stand-up time is insufficient for the required excavation cross-section or where the settlement has to be limited due to buildings on the surface, advance support canopies can be created consisting of longitudinal tubes or grout screens. For example, many sections of new rail tunnels have been driven beneath autobahns using this method. Pipe screens without grouting, grouted pipe screens and jet grout screens can be differentiated (Table 2-4).

Table 2-4 Advance support with canopies

Process	Application (purpose)	Application (ground, geology)	Description	Equipment
Pipe screen without injection	– advance support – tunnelling beneath facilities – settlement limitation	– loose ground or rock that is difficult or impossible to inject	– fan-shaped/ horizontal predrilling and insertion of steel pipes dia. approx. 100 mm), screen length up to 30 m – concreting of the pipes	– drill rig, jacking equipment – drill head, pipe section – mortar/concrete
Grout screen		– jointed rock (very) suitable for injection – loose ground (very) suitable for injection	– fan-shaped/ horizontal predrilling and insertion of steel pipes dia. about 100 mm), sometimes with reuse of the drill head (dependent on the system); screen length up to 30 m – injection into the ground of cement-bentonite suspension through valve openings in the side of the pipes	– drill rig, jacking equipment – drill head, pipe sections – mortar pump, grouting unit
High-pressure grout screen (jet grouting)		– variably bedded loose round with differing grading distribution (feasibility should be demonstrated)	– horizontal predrilling from the tunnel, if required with simultaneous primary injection to fill the pore volume – high-pressure injection as the drill rod is withdrawn out of the hole – production of overlapping or tangential jet grouted "columns" (dia. approx. 0.50 m to 1.00 m according to type of soil); screen length up to 15 m	– drill rig, drill platform – high-pressure grouting equipment (pumps, mixer, dosing unit) – particular technical monitoring and measurement and data recording

After the production of the advance support, the planned cross-section of the tunnel can be driven under the protection of the screen canopy. One advantage for construction operations is that pipe screens no longer have to be produced by specialised sub-contractors with special equipment, but can be drilled with tunnel drilling jumbos (although new drilling equipment is required) (Fig. 2-26). Special niches for the production of the pipe screen are often no longer required.

Figure 2-26 Works for the production of a grouted pipe screen using the drilling jumbo in the Irlahüll Tunnel

Pipe screens without grouting. Examples of these are the Bodex system and the AT casing system. The pipe screen is installed using a centrally mounted overburden drilling system, with drilling and tubing being carried out in one operation. The pipe remains in the ground and is subsequently concreted. Horizontal drillings of up to 30 m length are manageable today, although deviations depend greatly on the prevailing ground conditions. The spacing of the individual pipes depends on structural requirements and is normally some decimetres.

Grout screen. Grout screens are in principle produced like pipe screens but the spacing of the individual pipes is wider and the ground is subsequently grouted. In rock, the grout is injected using packers, which seal the hole and prevent grout escaping. This method is not possible in loose ground, so manchette tubes are inserted into the holes. The grout in this case is injected into the ground through openings in the manchette tube, which are sealed with a rubber sleeve (manchette). In the Irlahüll Tunnel, a grout screen was used to tunnel beneath an Autobahn (Fig. 2-27). The 15 m long injection lances were installed at a slope of 7°, with a precision of ± 2%. The grout was then injected through valves in the tubes with a pressure of 10 to 15 bar. The tube was finally filled with a cement suspension.

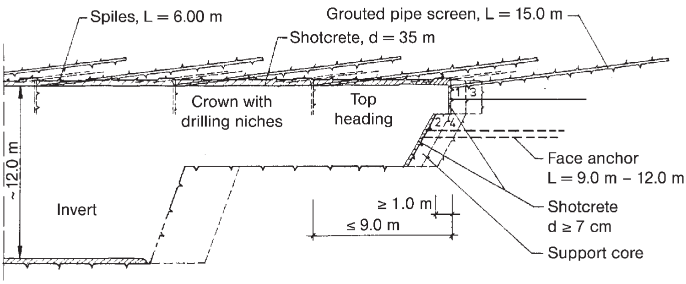


Figure 2-27 Application of a grout screen at the Irlahüll Tunnel

Jet grout screens. In the jet grouting or high-pressure grouting method, a high-energy jet of cement suspension cuts into the soil, which is cut, filled with suspension and mixed. The loosened soil thus acts as the aggregate in the resulting soil concrete. A soil body similar to a column is formed by simultaneously injecting grout and withdrawing the drilling rod. In general, jet grout columns can be overlapped or in tangential contact, although the maximum length is restricted to about 15 m. The jet grout process was used in a 282 m long underground section of the Frankfurter Kreuz Tunnel, which was excavated beneath an advance support consisting of a canopy of overlapping jet grout columns about 14.5 m long (Fig. 2-28).

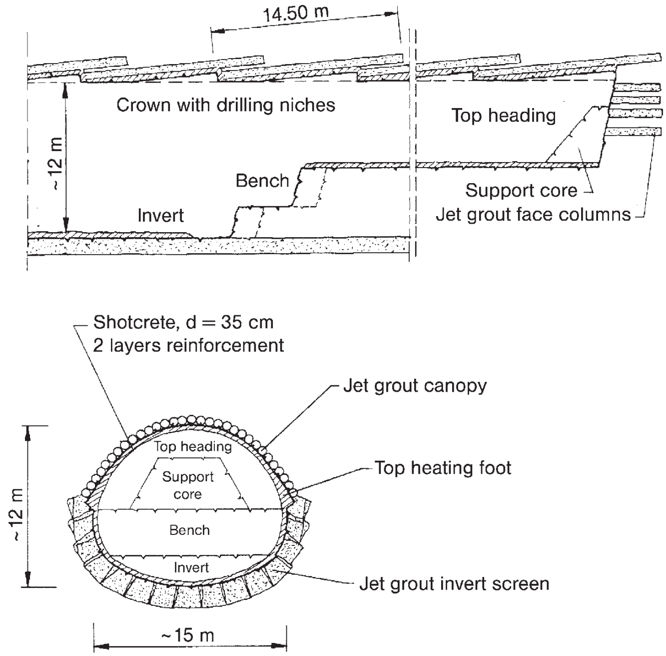


Figure 2-28 Application of a jet grout screen at the Frankfurter Kreuz Tunnel [25]

2.6.5 Ground freezing

The ground freezing process, which is only mentioned here for sake of completeness, is a well-known specialised civil engineering technique, and it can be used as advance support and ground improvement in water-bearing ground that is unsuitable for grouting (Table 2-5). In principle, this is also in the form of a screen over the tunnel, which in contrast to grouting only alters the hydrological conditions during the construction period.

Table 2-5 Advance support through ground freezing

Process	Application (purpose)	Application (ground, geology)	Description	Equipment
Ground freezing	– advance crown support – tunnelling below facilities – settlement limitation – water-proofing	– loose ground below the groundwater table	– produced from above ground, from a pilot tunnel above the tunnel section or horizontal freezing holes from the tunnel – the length of the freezing holes depends on the drilling precision; sections up to 30 m long enclosed with a transverse bulkhead are feasible	– Drilling rig, drilling platform – freezing system (freezing set, pumps etc.) including pipework, monitoring instruments and data recording – equipment for temperature monitoring and recording of data

2.7 Rock bolts

2.7.1 General

The use of rock bolts is regulated in DIN 21521 [69]. If rock bolts are anchored by grouting, then it is necessary to investigate on a case-by-case basis to what extent DIN EN 1537 [78] is applicable. The term ground anchor covers rock bolts, rock nails, strand anchors and tunnel anchors in loose ground. Rock bolts are normally used to secure the correctly profiled excavation outline until the final lining is installed, but also to support the face. Until the final lining is complete, rock bolts have to prevent rupture and falling rock in order to avoid the surrounding ground weakening and thus further deformation.

There have also been proposals to use rock bolts for permanent support. Although approved permanent anchors are already available, these are not generally used because many questions still have to be answered; particularly regarding load redistribution from temporary to permanent support, creep behaviour, the composite action of rock mass and support materials and the actual load resistance. What can be applied is a certain degree of ground improvement or homogenisation of the rock mass in the form of an improvement of the cohesion or the Young's modulus, which can be assumed in calculations.

2.7.2 Mode of action

Single rock bolts are used to secure individual blocks, wedges or slabs of rock (Fig. 2-29 left). In rock jointed with large banks, they serve to ensure the safety of the workers and preserve the required geometrical excavation shape with regard to structural action and overbreak.

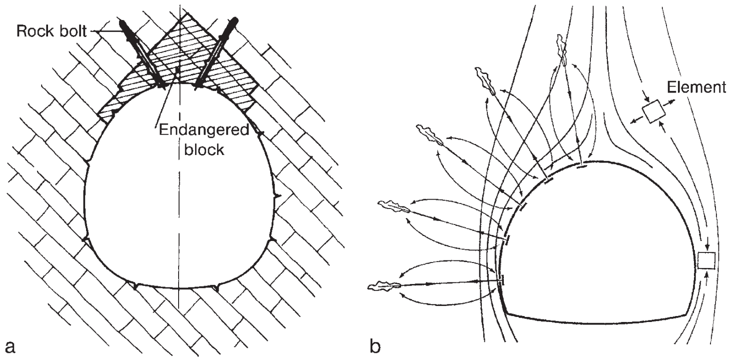


Figure 2-29 Support with single rock bolts (left) and pattern bolting (right)

Pattern bolting serves to assist the formation of a structurally active ring in the rock mass (Fig. 2-29 right). The rock bolts can be stressed with about 100 to 150 kN but their action is also due to an additional dowel effect.

After the cavity has been excavated, the stress in the rock mass is redistributed. The triaxial stress state changes to a biaxial state since the radial stress is no longer present. The tangential stresses increase and can exceed the (uniaxial) compression stress of the rock if the rim is not reinforced. Due to the effect of the formation of a structurally active ring of rock, the stresses move deeper into the rock mass, but a weakened zone is still created. The example of a shaft porch in Fig. 2-30 shows qualitatively and quantitatively the shares of the structural action of rock mass and support.

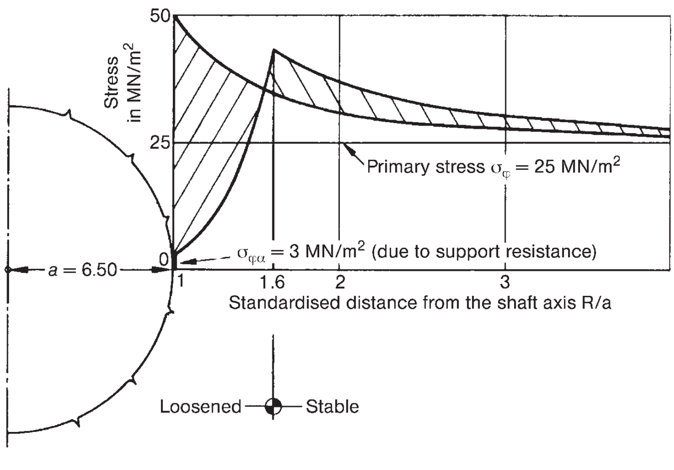


Figure 2-30 Stress distribution after excavation and after support, in this example a shaft porch at General Blumenthal [220, 241]

When pattern bolting is installed quickly in combination with spraying of shotcrete or fibre shotcrete, this acts to provide support resistance and has the result that a near-triaxial stress condition can occur at the edge of the cavity. This acts against the weakening of the rock mass, loosening is slowed down and the structurally active ring in the rock mass moves outward less. The more rapidly this equilibrium state in the composite material consisting of tunnel support and structurally active ring is established, the smaller the forces that can develop. Considering conditions in three dimensions, a vault moving forwards forms as the tunnel is excavated. A simple analogy for this was presented by L. von Rabcewicz (Fig. 2-31). If the height of the loosened zone is assumed to be 1/2 to 2/3 of the round length *t*, the anchor length is given as approximately *t*. But this may be too low, particularly for larger diameter tunnels, as round lengths do not normally exceed 4 m due to the maximum drilling depth. The proposal from L. v. Rabcewicz is not applicable either for shorter round lengths due to the rock being less competent, since such rock requires longer anchor lengths.

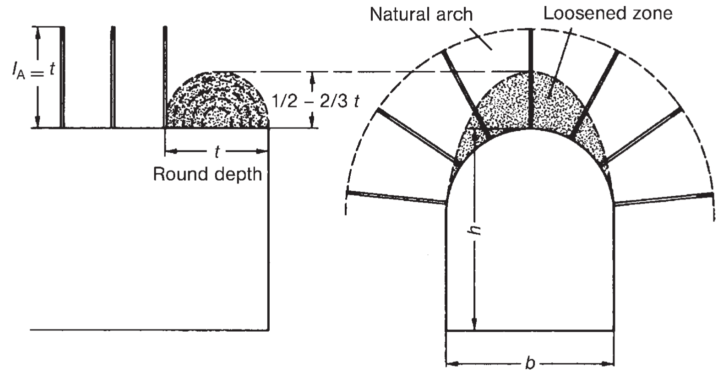


Figure 2-31 Creation of a self-load-bearing vault with rock bolts [289]

Arrangement of the rock bolts parallel to the jointing surfaces should be avoided; they should be positioned if possible at an angle of 45 to 90°, but never less than *30°,* to the bedding. Rock bolts installed parallel to the jointing surfaces can only bear little or no loading and are thus useless (Fig. 2-32). Dowelling of strata of low strength at the edge of the excavation to strata of higher strength is possible both with single rock bolts and with pattern bolting. The arrangements of rock bolts shown in Fig. 2-32 are to be found in the literature but are seldom practical.

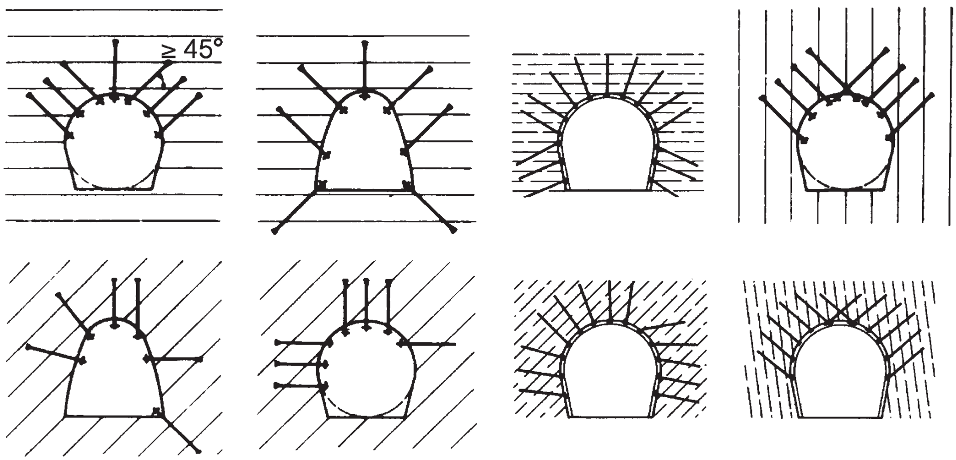


Figure 2-32 Arrangements of rock bolts as far as possible at right angles to the bedding

A list of the advantages and disadvantages of the use of anchors is given below:

Advantages:

– The desired yielding characteristics can be adapted.
– Addition of additional bolts is possible.

– It is possible to check the anchor force.
– Retightening is sometimes possible.
– Good adaptability to changing ground conditions.
– Combination with and without prestressing possible.
– Good adaptability in combination with other support measures.
– Good adaptability for all construction methods, like steel ribs, shotcrete, fibre shotcrete, lagging plates.
– Rock bolts can also be used to secure the foot of the arches.
– Can be used to support the face.
– Single rock bolts produce little disturbance to subsequent construction work (not always the case with pattern bolting).
– Drilling can often be carried out with the equipment already available for drilling shot holes.

Disadvantages:

– Not all types of ground can be anchored.
– Pattern rock bolting requires much more material and time.
– Parallel to the bedding without full action.
– Additional equipment required in mechanised tunnelling.
– Drilling depth is normally restricted by space considerations.

2.7.3 Anchor length and spacing

Fig. 2-33 shows the improvement of the load-bearing capacity of the rock mass by means of the radial and tangential stress curve and the Mohr failure criterion. The support resistance achieved by pattern bolting depends both on the magnitude of the effective anchor force and also on the thickness of the structurally active ring in the rock mass and its characteristics.

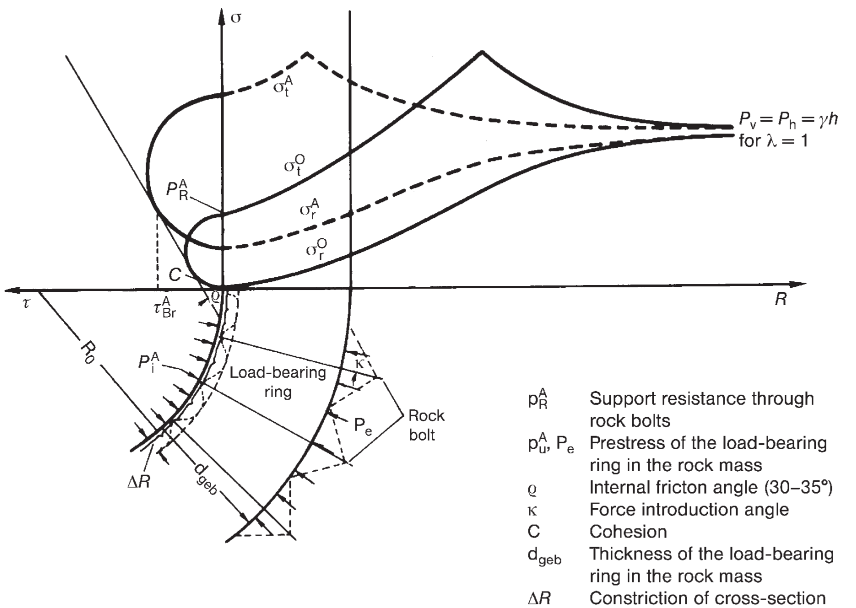


Figure 2-33 Pattern bolting. Effect of the self-load-bearing capacity of the rock mass [182]

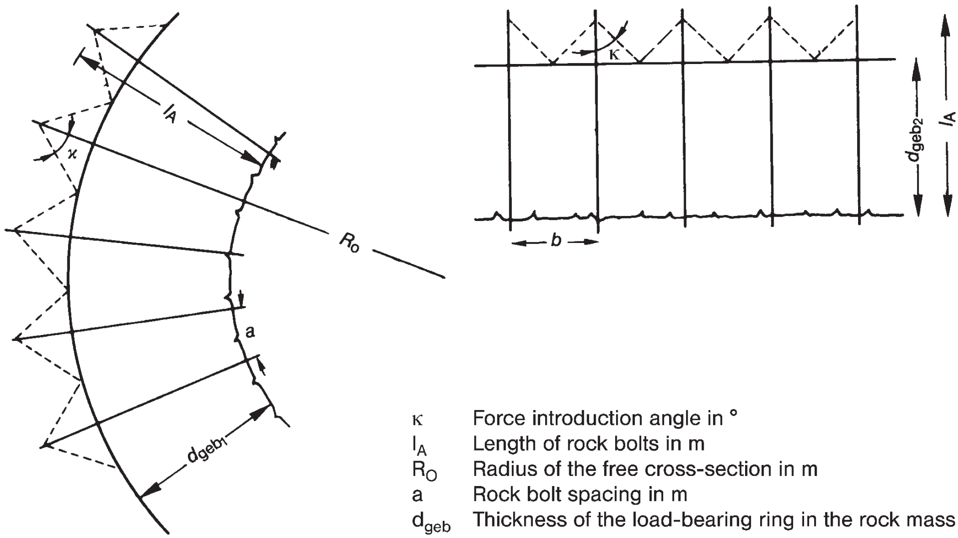


Figure 2-34 Pattern bolting. Cross- and longitudinal sections through the anchored structurally active ring in the rock mass [182]

The following calculation methods only give the "calculated" magnitude. In order to determine the anchor force and the relevant ground behaviour, it is necessary to test the rock bolt system to be used on site. Assuming that the effective anchor force is introduced into the rock mass at an angle of x and the edge of the cross-section is uniformly stressed, this gives a thickness of the structural ring for a circular profile from the geometrical relationships according to Fig. 2-34:

$$d_{geb_1} = l_A - \frac{a\,(R_0 + l_A)}{2\,R_0 \tan\varkappa + a} \quad \text{in m} \qquad [1]$$

$$d_{geb_2} = l_A - \frac{b}{2 \tan\varkappa} \quad \text{in m} \qquad [2]$$

As the shear stresses in the principal stress case are introduced into the rock mass at an angle of x = 45°, tan x =1 can be inserted. Due to the requirement $d_{geb} = d_{geb1} = d_{geb2}$ (rock mass structural ring acts longwise and crosswise), this gives the equation:

$$b = \frac{a\,(R_0 + l_A)\,2\tan\varkappa}{2\,R_0 \tan\varkappa + a} \quad \text{in m} \qquad [3]$$

There is a direct proportionality between the required anchor spacings a and b. According to Leins and Löttgen [182], the structurally active ring in the rock mass is uniform in cross-section and longitudinal section when the equation is complied with.

2.7.4 Load-bearing behaviour

Bolt shaft. The load-bearing capacity of a rock bolt depends on its shaft and its head, and furthermore on the anchor force in the rock mass. Stressed rock bolt shafts are normally made of special steel of grades St E 335, St E 460, 1 C 45 and the shafts of untensioned rock bolts of grade BSt 420/500. The load-bearing capacity of the shaft is determined by its permissible load. The permissible load is the force at the yield limit divided by a safety factor, which is normally set at 1.5 to 2. Table 2-6 shows examples of permissible loads of grouted anchors [92].

Table 2-6 Technical data of the TITAN grouted anchor, Ischebeck GmbH

Achor type/pile type	Unit	TITAN 30/16	TITAN 30/11	TITAN 40/20	TITAN 40/16	TITAN 52/26	TITAN 73/53	TITAN 103/78	TITAN 105/53
External diameter	mm	30	30	40	40	52	73	103	105
External diameter for structural calculations	mm	27.2	26.2	36.4	37.1	48.8	69.9	100.4	98.5
Internal diameter	mm	16	11	20	16	26	53	78	52
Permissible loading in tension and compression	kN	100	150	240	300	400	554	900	1500
Permissible shear force	kN	58	88	138	164	240	329	535	899
Failure load	kN	220	320	539	660	929	1160	1950	3460
Weight	kg/m	3.0	3.5	5.6	6.9	10.5	12.8	24.7	43.2
Minimum cross-section	mm^2	382	446	726	879	1337	1631	3146	5501
Force at yield limit	kN	180	260	430	525	730	970	1570	2726
Yield stress	N/mm^2	470	580	590	590	550	590	500	500
Moment of inertia	cm^4	2.37	2.24	7.82	8.98	25.6	78.5	317	425
Section modulus	cm^3	1.79	1.71	4.31	4.84	10.5	22.4	63.2	86.3
Plastic section modulus	cm^3	2.67	2.78	6.70	7.83	16.44	32.1	89.6	135

For special requirements or longer anchor lengths, strand anchors can also be used. In zones, which are later to be mechanically excavated, the use of anchors made of hardwood, plastic or glass fibre can be appropriate, since these cause less damage to the cutting tools of the machine.

Fig. 2-35 shows some values for purposes of comparison. Particularly significant is the failure strain, since a high strain represents a "safety valve" in case of overloading. Other characteristics that should not be overlooked are resistance to ageing and bending strength.

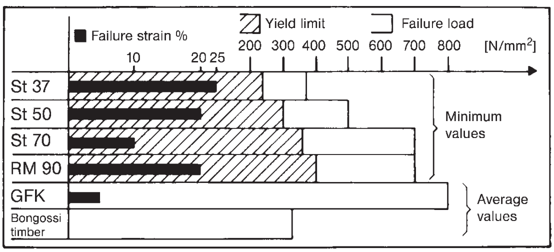


Figure 2-35 Comparison of the load-bearing capacity of various anchor shaft materials

Comparison of the materials shows the following main properties:

– Timber: light, cheap, easily worked, low shear strength.
– Glass fibre: light, high tension strength, corrosion resistant, easily worked, low strain.
– Steel: elastic, extendable, deformable, defined behaviour under load, not workable.

Bolt head, plate. The form of the head of rock bolts is regulated in DIN 21522 [70]. Many shapes are possible, as can be seen in the illustration. The plate should transfer load to the shift as axially as possible, so they are often made dished or with a slotted hole (Fig. 2-36).

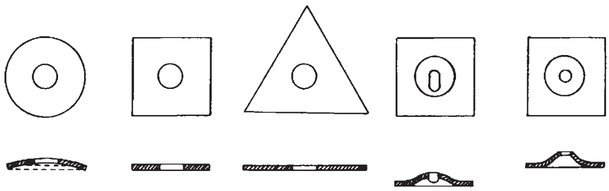


Figure 2-36 Examples of anchor plates [306]

Transfer of the anchor force into the rock mass. This is best determined through approval tests before the start of construction work. For grout and artificial resins, experience from the use of prestressed concrete grouted anchors is applicable. Anchors are also technically approved according to this action. Primarily, stressed and unstressed anchors are differentiated according to functional principle.

In stressed anchors, the force is produced either externally or naturally by the differential movement of the side of the excavation and the anchoring length in the rock mass. Externally prestressed anchors are stressed using torque wrench, impact screwdriver or tensioning press and then tested. Prestressing is only possible after the grout has hardened. Anchors with a defined prestress should first be stressed to 65 % of their failure load to test their load-bearing capacity. The stressing force is then reduced to the specified permanent load after about 15 minutes.

Stressed rock bolts can either be made as anchors with an unbonded length permitting continued retightening and testing, or as locked anchors. Anchors with an unbonded length (Fig. 2-37) need to be corrosion-protected if required to be permit regulation over a longer period and especially where the free parts without corrosion protection are exposed to aggressive water. It is normally sufficient to fill the hole with unpressurised cement or synthetic mortar. They should be used where the rock mass has not yet settled down or when further excavation could provoke renewed movement. Important points to note are not only the free unbonded length but also that there is tolerance space for lateral deformation. Anchors with an unbonded length can also be used as monitoring anchors.

Fully bonded anchors can be used where no further movement of the rock mass is expected. The bolt is fixed by filling the whole length of the anchor hole with grout and locking the head. Anchors should only be fully bonded and sprayed over with shotcrete when no further reduction of stressing force can be detected.

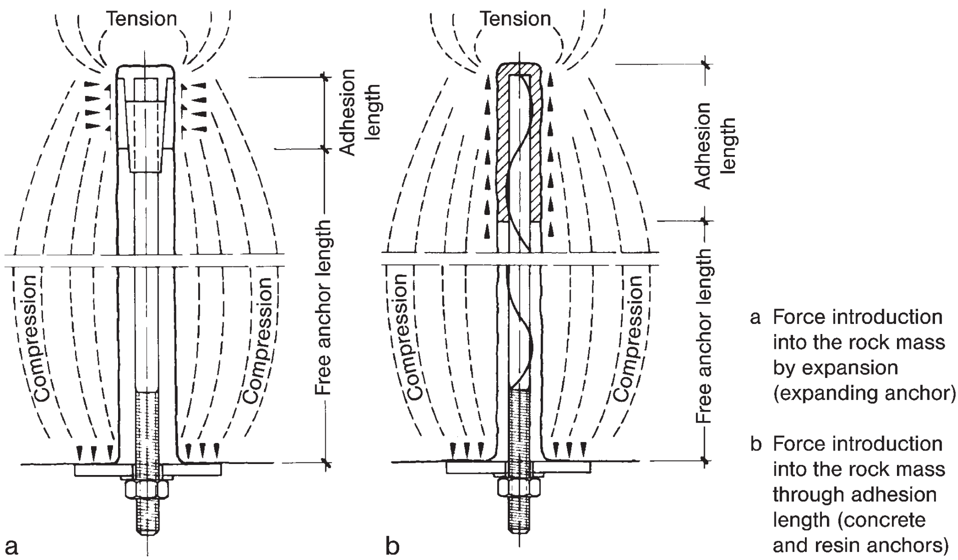


Figure 2-37 Anchor with an unbonded length. Installation principle and diagram of the force curves

Unstressed anchors are not tightened at first and act over their cross-section as dowels, particularly when being sheared. But unstressed anchors can also be stressed where movement of the rock mass applies loading to an anchor that has been locked with completely bonded grout.

The following types of anchors are mostly used in tunnelling to stabilise the rock mass:

– Expanding rock bolts.
– Concrete- or mortar-grouted anchors.
– Synthetic resin anchors.
– Glass fibre-reinforced plastic anchors (GRP anchors).
– Friction tube anchors.

2.7.5 Anchor types

a) Expanding rock bolts

Expanding anchors are only installed for temporary support. In drill and blast tunnelling, the seating of the nearby anchors should be checked after every round, and they should be retightened or replaced if necessary. The foot of the anchor is expanded against the sides of the hole using wedged elements. No resin or grout is necessary to fix the anchor. Figs. 2-38 to 2.40 show various types of expanding bolts.

The anchor sleeves can be made of steel grades ST 70 or BST 500S or plastic. While a metal sleeve becomes instable after the anchoring sleeve has slipped, plastic sleeves can still hold.

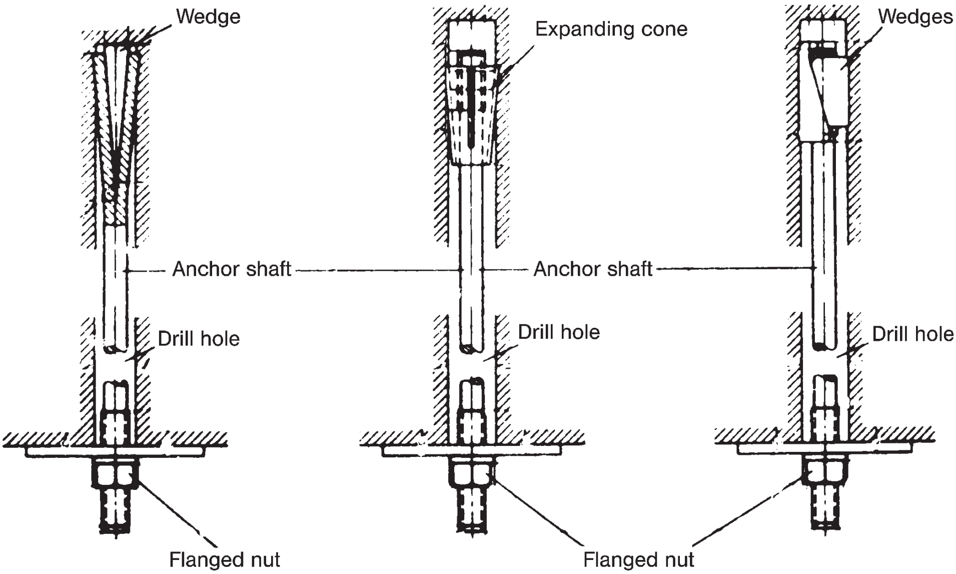


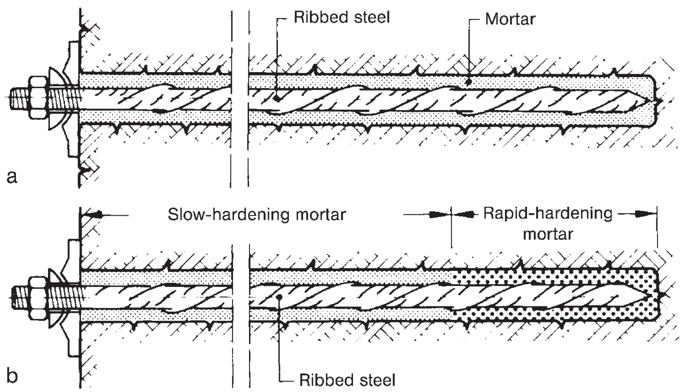
Figure 2-38 Slotted wedge anchor [217]

Figure 2-39 Expanding wedge anchor [217]

Figure 2-40 Double wedge anchor [217]

b) Concrete or mortar anchors

Concrete anchors can either be filled or the anchor simply pushed into the mortar. They are installed as unstressed anchors. If concrete anchors are to be stressed, then they will initially only be fixed deep in the hole (Fig. 2-41). This is produced by using rapid-hardening mortar in the adhesion length and slower-working and less strong filler in the unbonded length.



a Unstressed (filled with uniform mortar)
b Prestressed (with different mortars)

Figure 2-41 Concrete anchors

Mortar-filled anchors, also called FM or SN anchors (SN after the site at Store-Norfors where they were first used), are installed by first pumping mortar with a pump (compressed air or hydraulic) into the drilled hole (Fig. 2-42), with the hoses being pushed fully into the hole and withdrawn as the material is filled. The anchor shaft is inserted by hand or driven with a compressed air or hydraulic hammer. This process requires great experience and holes in the crown can prove particularly difficult.

Various makes of mortar can be used for filling, and hydraulic ready-mixed mortar for rock anchorage and grouting is suitable. The mortar must be easily workable, capable of being pumped and injected and should not be susceptible to separation. In order to ensure successful overhead working, only thixotropic mortars should be used. The associated hardening time of about 8 hours before stressing should be observed.

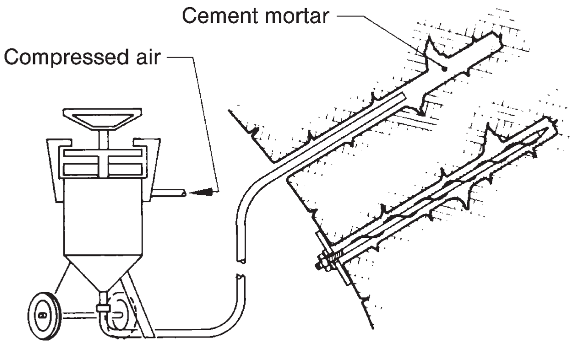
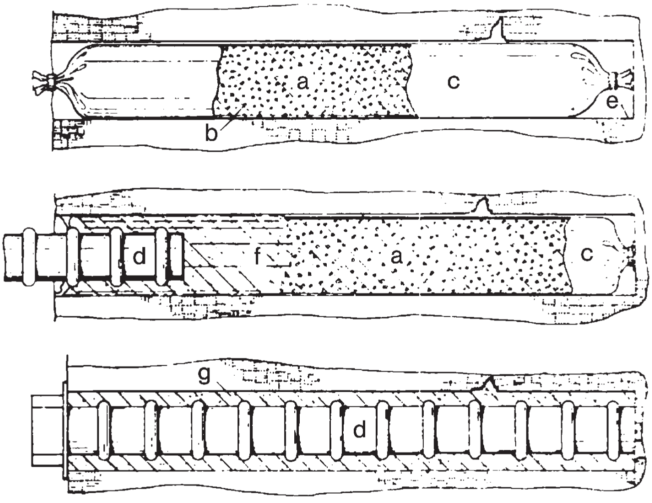


Figure 2-42 Installation of a mortar-filled anchor

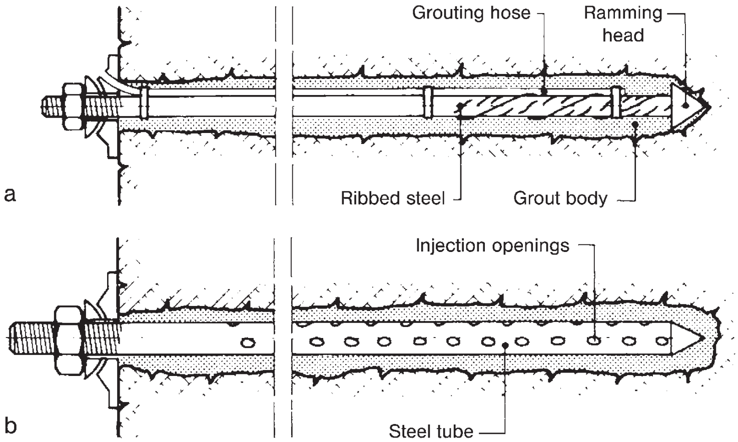
The hardening time can be considerably shortened through the use of high-early-strength cement or the use of a cement cartridge (Fig. 2-43). The insertion of an anchor into a hole containing a cement cartridge breaks the cartridge and the microcapsules filled with water (b in Fig. 2-43), so that the cement is wetted by the insertion of the anchor. The wet cement mortar lubricates the drilled hole and permits full insertion of the anchor, which is followed within seconds by the hardening of the material so that the mix can no longer flow out of the hole. Adequate anchoring strength is reached within 5 minutes.



a Cement
b Micro-water capsules
c Envelope
d Anchor
e Drill hole wall
f Mortar
g Hardened cement mix

Figure 2-43 Construction and mode of functioning of a cement cartridge [334]

The rammed and grouted anchor is a type of unstressed fully grouted concrete anchor. This type of anchor was developed for ground, which can only be drilled with difficulty or not at all, like slope debris, moraine, alluvial, conglomerate, non-cohesive but groutable loose ground. There are two basic types; one consists of a profiled steel bar with a warm-fitted, cone-shaped ramming head (Fig. 2-44 a) with one or two grouting hoses, and the other consists of a thick steel tube with one end pointed and openings in the sides at appropriate spacings (Fig. 2-44 b).



a Profiled steel bar with ramming head and grout hose
b Pointed steel bar with injection openings

Figure 2-44 Available types of rammed grouted anchors

A new development has taken place in anchor technology in the last few years. While the drilled holes formerly had to be lined with tubes, self-drilling grouted anchors can be used. The use of these is even more advantageous, the greater the danger is that the drilled hole may fall in or run off course. The pulling out of the drill stem is no longer necessary, the hole is smaller and smaller drills can be used. Self-drilling grouted anchors consist of a steel tube of approved fine-grained construction steel St E 460 and St E 355 with a coarse thread, which can act both as lost drill dowel tube and grouting tube (Fig. 2-45). Self-drilling grouted anchors enable drilling, grouting, anchoring and stabilising in one working step [200].

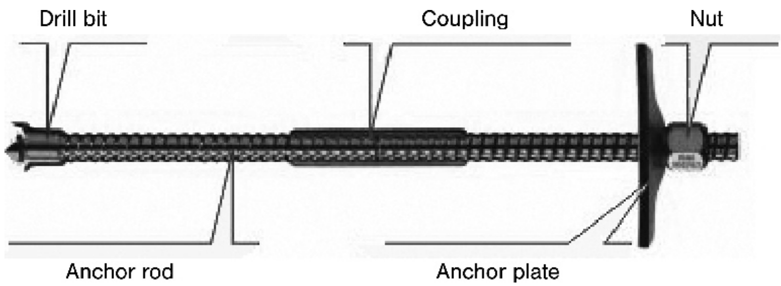


Figure 2-45 Construction of a self-drilling grouted anchor; Mai International GmbH

c) Synthetic resin anchors

In artificial anchors, a synthetic resin mortar in a filled cartridge mostly containing components is pushed to the end of the hole. Fig. 2-46 shows the construction of a modern synthetic resin adhesive cartridge, which consists of a resin mortar paste (A) and a hardener (B) [53]. Both components are packed, in separate chambers, into a foil cartridge. The locating element at the end of the cartridge (C) has fins to prevent the cartridge slipping out of the hole. The cartridges can be inserted into the hole by hand, using a loading tube or pneumatically. After the cartridge has been inserted, the anchor is drilled into the hole (Fig. 2-47). This destroys the cartridge and mixes the components. After reaching the end of the hole, the anchor setting device is left in position until the resin hardens. The reaction time depends on the temperature of the cartridge and the rock mass and also on the heat developed during the setting process; according to the particular case, this can range from 15 s to 5 min.

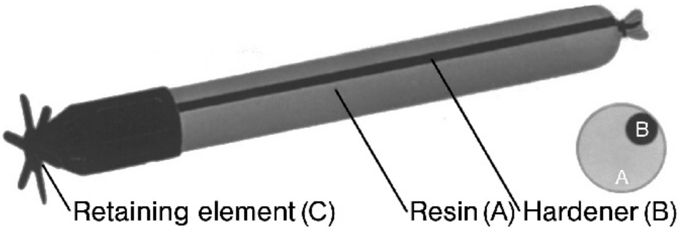


Figure 2-46 Construction of an adhesive cartridge; CarboTech Fosroc GmbH

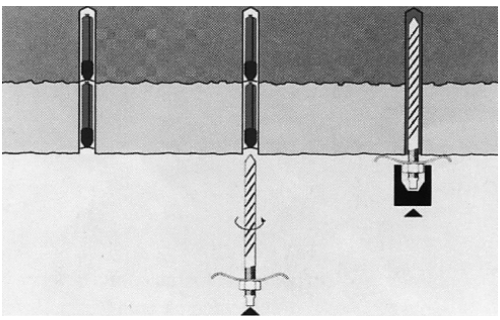


Figure 2-47 Setting an adhesive anchor; CarboTech Fosroc GmbH

Resin anchors normally have shorter hardening times and earlier, but also higher, load-bearing capacity than expanding or grouted anchors. The advantages of the earlier load-bearing of synthetic resin are exploited; the dowel action and corrosion protection are provided by the mortar. In damp rock, advance tests are necessary, as wet holes and low rock mass temperatures influence the time required for the development of load-bearing and also the load-bearing capacity. Initial tests are always necessary to determine the adhesion and the permissible anchor force. Resin anchors are also used in combination with grouted anchors.

The installation of resin anchors requires particular experience, as the system reacts very sensitively to factors like rock and rock mass characteristics, formation water, resin and hardening properties, temperature, geometry of the anchor shafts and drilled holes. When correctly installed in a hole, the synthetic resin mortar guarantees a mechanical bond between the tension member (shaft of steel or GRP) and the sides of the hole.

d) Friction tube anchors, Swellex anchors

As an example of friction tube or also steel tube anchors, Fig. 2-48 shows a Swellex anchor from Atlas Copco. It consists of a steel tube of diameter 41 mm, which is reduced to a diameter of 28 mm by double folding.

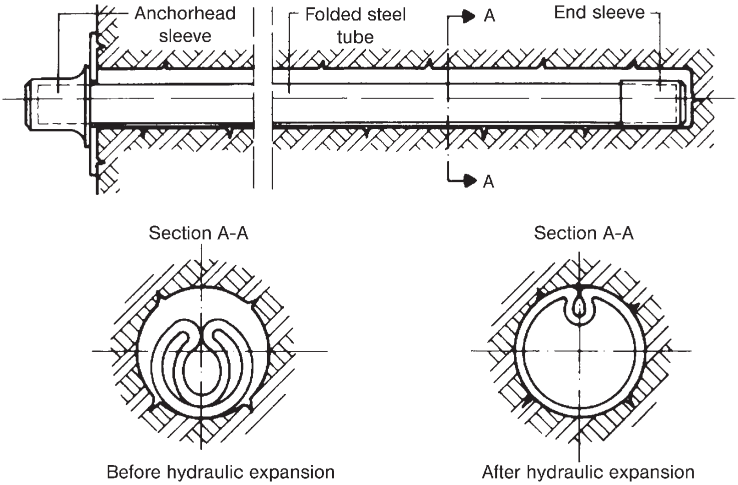


Figure 2-48 Swellex anchor; Atlas Copco Deutschland GmbH

The hollow folded tube, which is closed at both ends with a bush, is installed in the drilled hole. Fig. 2-49 shows the sequence of setting a friction tube anchor. First, the hole is drilled and the bolt is inserted. Then the anchor is hydraulically expanded by water pressure of up to 300 bar into the borehole to provide the anchoring effect [12].

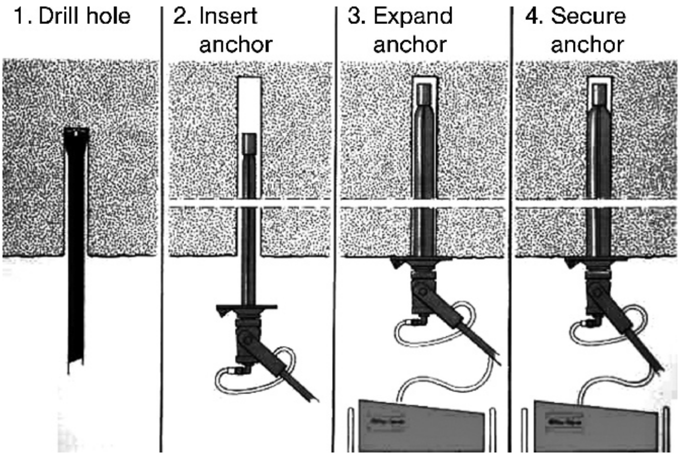


Figure 2-49 Setting a Swellex anchor; Atlas Copco Deutschland GmbH

The bonding to the rock mass is provided by friction between the hydraulically expanded anchor and the rock mass. Friction tube anchors are fully load-bearing and can resist movement of the rock mass immediately after being set. The load-bearing capacity is developed over the entire length of the installed anchor.

The essential advantages are

- Immediate full load-bearing capacity over the entire installed length.
- Not sensitive to vibration, e. g. from blasting.
- Safe and simple installation.
- Good adaptability to variable hole diameters.

This results in good safety for tunnelling and cost optimisation for pattern bolting.

e) GRP anchors

GRP anchors consist of one or more bars of glass-fibre-reinforced plastic, consisting of textile glass and polyester resin (Fig. 2-50). This results in a weight reduction of 60 to 70 % compared with chrome steel, aluminium or galvanised steel for the same permissible load [52].

Figure 2-50 Examples of GRP anchors; SAH-Ankertechnik GmbH

Glass reinforced plastic (GRP) anchors also have the following advantages over steel or aluminium, which result from the properties of the composite material:

- High tension strength.
- High elasticity.
- Low weight.
- Good corrosion resistance.
- Good electrical insulation properties.

In the tunnels for the new railway line from Bologna – Florence, the face is supported in advance using 15 m long glass fibre anchors with an overlap of at least 5 m. Glass fibre anchors are being used to stabilise the core of the advance and thus secure the face with a large excavation cross-section, shallow overburden below a built-up area and friable rock mass (Fig. 2-51).

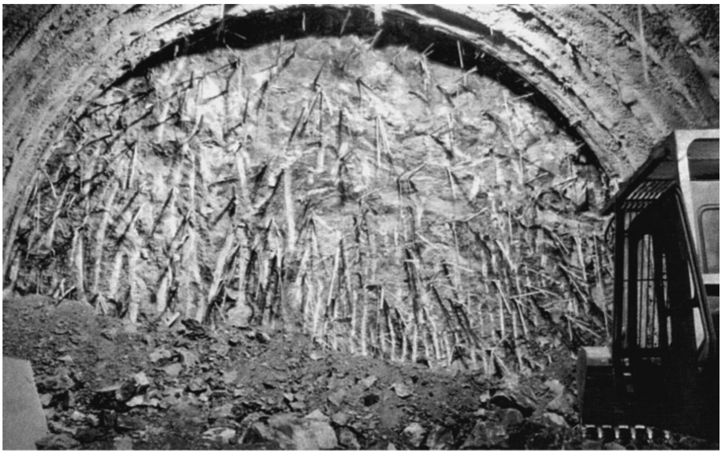


Figure 2-51 Use of glass fibre anchors to support the face in a tunnel for the new line Bologna – Florence

Used for face support, the drilled hole can be filled from the far end hole with a suitable grout pumped through a central hole (Fig. 2-52). The properties of glass fibre permit subsequent tunnel excavation with normal cutting tools.

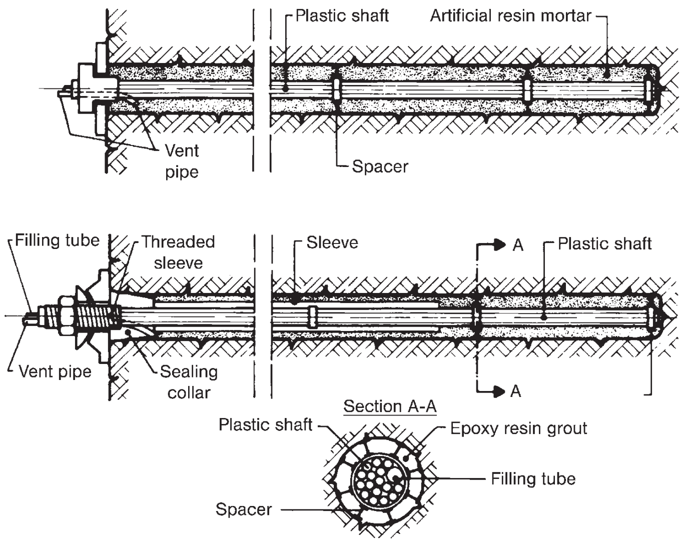
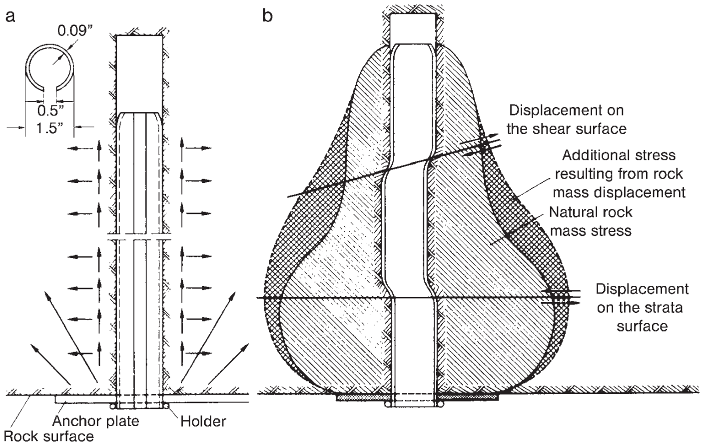


Figure 2-52 Unstressed (top) and stressed (bottom) synthetic resin mortar anchors of plastic bars

f) Special types

Timber anchor bars. In addition to the steel anchors already described, timber can also be used, for example bamboo. This material offers almost no obstruction to subsequent mechanical excavation.

Steel tube anchors. The split-set anchor represents a further development in controlling the rock mass according to J. Scott [270] (Fig. 2-53). Split set anchors are made with an external diameter of 38 mm (1.5”), but are normally driven into a hole of only 35 mm diameter. An anchoring force of 25 to 50 kN/m is achieved by the friction of the compressed steel tube with the sides of the hole. In contrast to the Swellex anchor, which is widely used in practice, the split set anchor has not got past the development phase.



a Force directions around anchor and anchor plate
b Stress distribution around the anchor

Figure 2-53 Split set anchor; Ingersoll Rand Company

These anchors produce an internal reinforcement and homogenisation of the rock mass and lead to the formation of a structurally active ring:

- The design is simple and capable of extension.
- They adapt to the degree of restraint, which is required for the rock properties encountered.
- They provide yielding support.
- They represent a compression preloading of the rock mass by increasing its stability.
- They increase safety for the miners.
- They are cheap.

As these are scarcely used, they are only mentioned here for sake of completeness.

Yielding anchors. When convergences are large, yielding construction of the anchors is necessary, as otherwise the exceeding of the maximum load would lead to the anchor failing or tearing out, making it completely ineffective. Development of yielding anchors is being particularly driven in mining, where dynamic loading can be heavy.

a) Sliding anchors: with a shear head and sliding construction, relatively large convergences can be resisted with a defined anchoring force; this is a development of the former Bergbau-Forschung GmbH (Fig. 2-54).

b) Yielding anchor head construction: defined yielding is easier and cheaper to achieve at the anchor head, but this construction cannot provide such large yielding travel as sliding anchors. Designs with strain sections in the anchor shaft or with spring and deformation zones inside the construction of the anchor head have already been tested.

Long-term anchors. Solutions are available for corrosion protection to make rock bolts suitable for long-term use with a cement-grouted, mostly with double-walled PVC pipe being used in the free anchor length and grouted with cement.

Strand anchors. Strand anchors are often used in mining to secure large excavated surfaces [258].

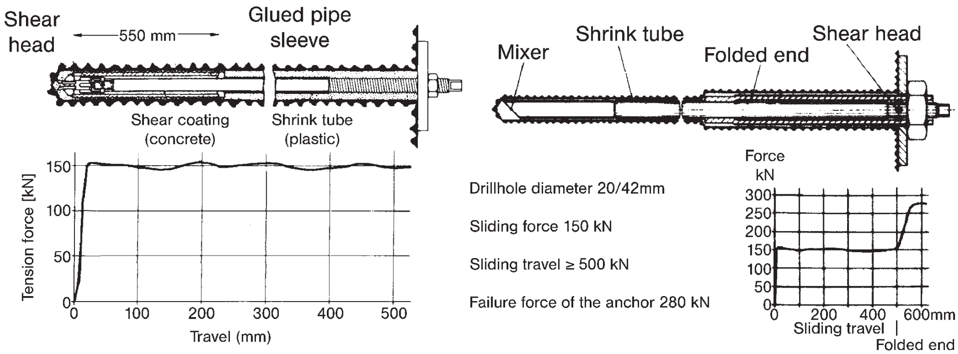


Figure 2-54 Force-deflection diagram for yielding anchors used in coal mining [344]

2.8 Concrete in tunnelling

2.8.1 General

Concrete in tunnelling is generally produced and handled according to the following standards and guidelines:

– DIN 1045 Parts 1 to 4 07.01 "Concrete, reinforced and prestressed concrete structures" [65].
– DIN EN 206 "Concrete – Part 1: Specification, performance, production and conformity" [80]
– DIN 1048 Parts l to 5 6.91 "Testing concrete" [65].

Due to lack of space, the production of concrete normally takes place outside the tunnel, at least the production of the dry mix. Table 2-7 gives an overview of the uses of concrete and mortar.

Table 2-7 Concrete and mortar use in tunnelling and mining

	Conveyance of components and concrete	Concrete delivery and placing	Concrete compaction
Shotcrete Steel fibre shotcrete Sprayed mortar	Special vehicle Rail car Falling pipe Skip (shaft)	Dry-mix and wet-mix machine Pump	Compaction through the impact of the mix
In-situ concrete, reinforced In-situ concrete, unreinforced Steel fibre concrete	Special vehicle Rail car Pumped pipeline Falling pipe Skip (shaft)	Manual, Conveyor belt, Flexible pipe, Slide, Pump, Pipe, Hose, Concreting lance	Tamping Internal vibrator External vibrator Vibrating floor (consistency, pumping pressure)
Cement mortaring and grouting	Special vehicle pumped pipeline Rail car Skip (shaft)	Pumps for mortaring and grouting (low- and high-pressure pumps)	Consistency Pumping pressure

The demand for special standards for the use of concrete and especially shotcrete in tunnelling is not new and also not completely unjustified, but the works listed above apply to the delicate components manufactured in precast works, sometimes as in a factory, just as in the tunnel.

The "rough and tumble" of tunnelling, with constantly changing temporary states, therefore requires careful application using expert experience and interpretation to comply with the valid regulations.

2.8.2 Construction variants

A differentiation is normally made between:

– Outer lining (temporary support).
– Inner lining (permanent support).

In order to increase the cost-effectiveness of tunnel drives, the intention is to integrate the outer lining into the final lining, in order to achieve the ideal of a single-layer or one-pass lining. The construction variants are described in more detail below.

2.8.2.1 Two-layer construction

Two-layer construction represents the conventional variant and features a strict separation between the temporary and permanent support. The outer layer is installed as the tunnel advances and serves, as a composite with the load-bearing capacity of the rock mass on its own, to secure the cavity until the final lining is installed. The two layers are separated either by a waterproofing, for example a plastic membrane with appropriate protection layers or applied by spraying, or else when the inner lining consists of waterproof concrete, a separation layer of plastic foil such as polyethylene to prevent interlocking between the outer and inner layers.

Two-layer construction without waterproofing or separation foil, with a waterproof concrete inner lining cast directly against the shotcrete surface, is also possible for tunnelling in the groundwater [312].

A summary of the standard variants of in-situ concrete inner linings for two-layer constructions is shown in Fig. 2-55.

The outer layer of support is regarded as a rotting support with only a short lifetime. The inner lining is then designed to fulfil all requirements of structural stability, durability and serviceability. It is assumed in this case that the outer lining will fail and thus no longer functions. There are, however, examples, in which the outer lining is also assumed to undertake a load-bearing function in the completed state for design purposes.

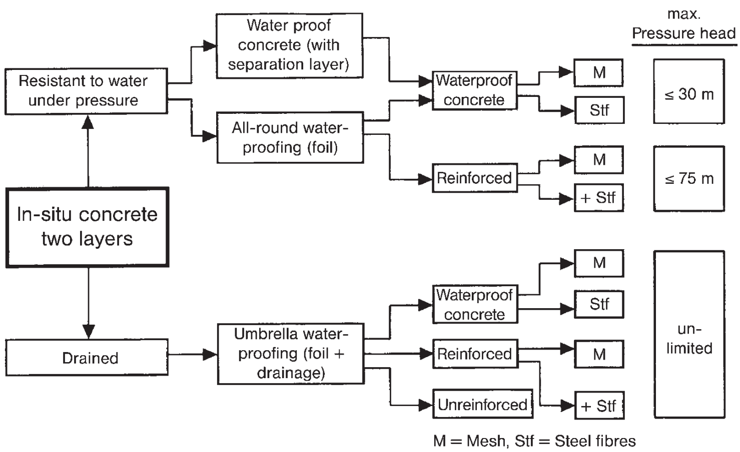


Figure 2-55
Variants of in-situ concrete inner linings in two-layer construction: drained and undrained [213]

2.8.2.2 Single-layer construction

The development of single-layer lining has been mainly initiated by three factors:

Firstly, the outer layer does not fail to the extent that is theoretically assumed for the purposes of structural design calculations. The remaining stability of the outer layer prevents the inner lining being loaded for a long time.

Secondly, it is possible today to apply shotcrete as structural concrete. If an outer layer has already been intentionally constructed as part of the final lining, only an addition to the outer lining has to be superimposed in the subsequent working step to comply with the serviceability requirements of the final support. This can save a certain proportion of the excavation quantities and also construction quantities.

Thirdly, round times can be considerably shortened through the use of steel fibre reinforced concrete or shotcrete. The overall economic benefits are considerable.

The precondition for the integration of the outer layer or also first layer into the final support is an adequate bond in the contact joint between first and second layers.

In contrast to two-layer construction, the outer layer undertakes the requirements for structural stability and has to guarantee durability (Fig. 2-57). The second layer also has to ensure serviceability. Possible construction variants are shown in Fig. 2-57.

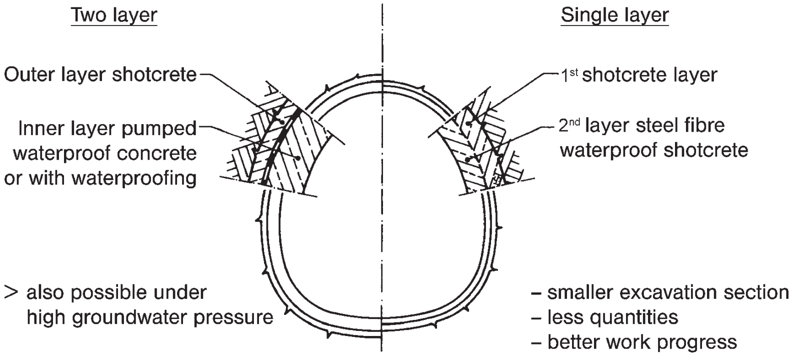


Figure 2-56 Comparison of single-layer and two-layer construction [218]

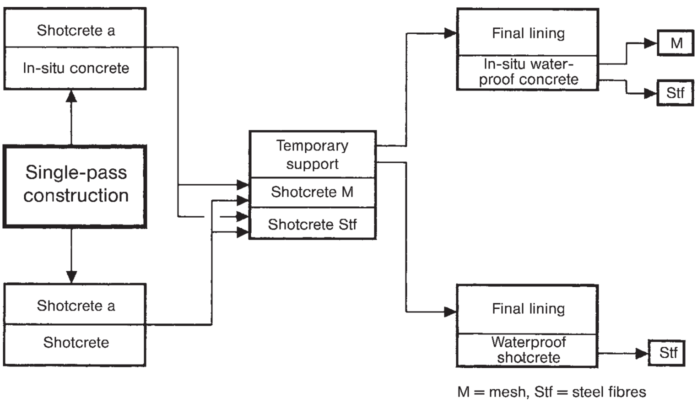


Figure 2-57 Variants of lining construction for single-layer construction [213]

Single-layer construction with a temporary support of shotcrete and an in-situ concrete inner lining of steel fibre concrete is already nearly the state of the technology. Examples of this are the underground railway contract K6a in Dortmund and the S-Bahn Tunnel in Lütgendortmund [112].

A successful example of single-layer construction in shotcrete is the test section as part of the contract 2312 of the Stadtbahn Bielefeld, where a shell structure of 15 cm conventional shotcrete support and 10 cm steel fibre shotcrete layer was applied as the final load-bearing and serviceable lining. The application of the second layer of this 104 m test section could be completed in four working days using the wet spraying process [218]. Fig. 2-58 shows the cross-section and a detail of the shell of the test section.

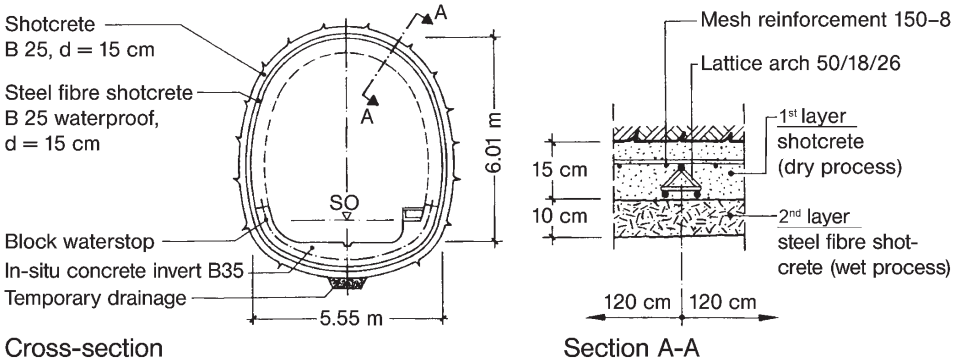


Figure 2-58 Cross-section and detail of the test section in the Bielefeld underground [226]

2.8.3 Shotcrete

2.8.3.1 General

Alternative terms are torcrete concrete, sprayed concrete or gunite. Due to the smaller grain size, the latter should rather be categorised as having the properties of mortar. Shotcrete is the most important support material in modern tunnelling. Shotcrete is applied and compacted with compressed air using the wet or dry process, hydrates on the substrate and hardens. Admixtures can be used to adapt the properties with regard to strength and adhesion. Accelerators are often used to increase the early strength. Shotcrete provides support immediately after an advance with optimal bonding to the rock mass.

Shotcrete is subject to very different quality requirements. The quality is determined by the:

– Parameters related to materials technology.
– Parameters related to the process technology.

Thanks to extensive research work in the years between 1985 and 1995, it is now possible to produce shotcrete as construction concrete in accordance with current DIN standards.

The essential regulations concerning the use of shotcrete in tunnelling are

– DIN 18 551, 03.92 "Shotcrete; production and testing" [68].
– Ril 853, "Design, construction and maintenance of rail tunnels" [296].
– ZTV- ING, Part 5: Tunnel construction [384].

At the Ruhr-University Bochum, not only the effects of materials technology but in particular the effects of process technology have been researched for the dry- and wet-mix processes. This was possible here, in contrast to other institutes, due to the availability of a dedicated test rig (see chapter 2.7.3.6). It is not practical to describe the extensive results in detail here, but reference is made to the "Shotcrete handbook" [214] with its literature references.

Shotcrete is very flexible in use and can be adapted easily to changed requirements such as part-face excavation of any size. Since it is normally applied in thin layers, shotcrete support initially has a low bending strength and due to the high creep capacity of the young concrete, it can adapt to larger deformations without fracturing. Shotcrete is normally used for temporary support, sealing, reinforcing the perimeter of the rock mass and smoothing the excavated sides and for waterproofing, although it can also be used as a permanent lining. The question, to what extent does shotcrete contribute to load-bearing, can be answered by stating that even a thin sealing layer (thickness < 5 cm) shows a greater load-bearing effect than expected, as the perimeter strengthening simultaneously closes the joints and prevents minor rockfalls, activating an effective vault in the rock mass.

Shotcrete can be unreinforced, reinforced with mesh and anchored or with steel fibres in the mix. When it is considered as constructional concrete providing a certain support stiffness, thicknesses of more than 5 cm, or more normally 15 to 40 cm are to be used (and even more in special cases).

Experience in drilling and blasting shows that the dynamic effects of blasting such as air blast and suction action, and vibration do not usually cause any significant destruction of the young concrete as long as its strength is greater than 2.5 N/mm^2.

2.8.3.2 Process technology, equipment and handling

Shotcrete can be produced by various processes, which can be categorised according to the mixing type (wet- or dry-mix process) and the means of delivery. Fig. 2-59 shows an overview of the various processes. Further variations are possible regarding the addition of the water and the addition of admixtures like accelerators or adhesives.

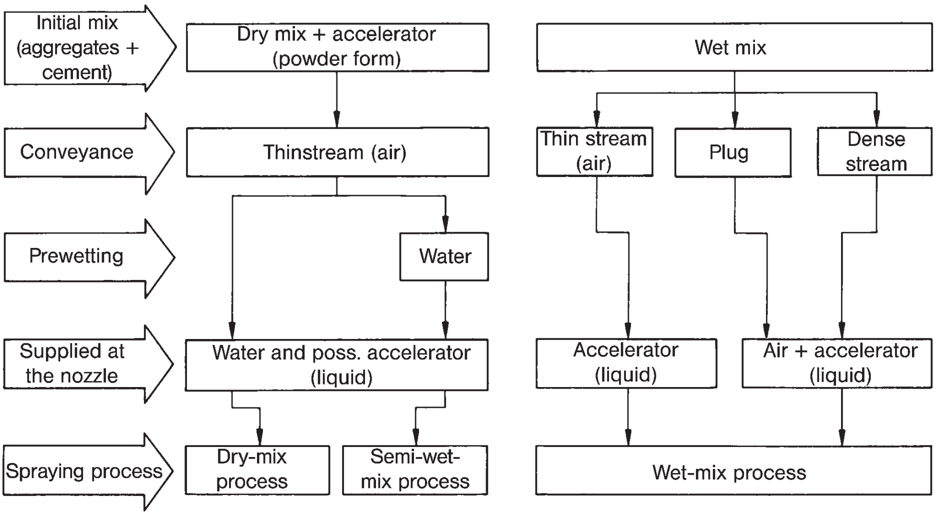


Figure 2-59 Overview of concrete spraying processes

a) Dry-mix process

In the dry-mix process, a dry mixture of cement, aggregate and if required also accelerator admixture is pumped from the mixer to the spraying nozzle with compressed air (Fig. 2-60), where the water is added through jets. The resulting mixture is applied with speeds of 20 to 30 m/s and compacted. A more recent development is the addition of liquid accelerator added before the nozzle, and preliminary wetting with water has also been used successfully.

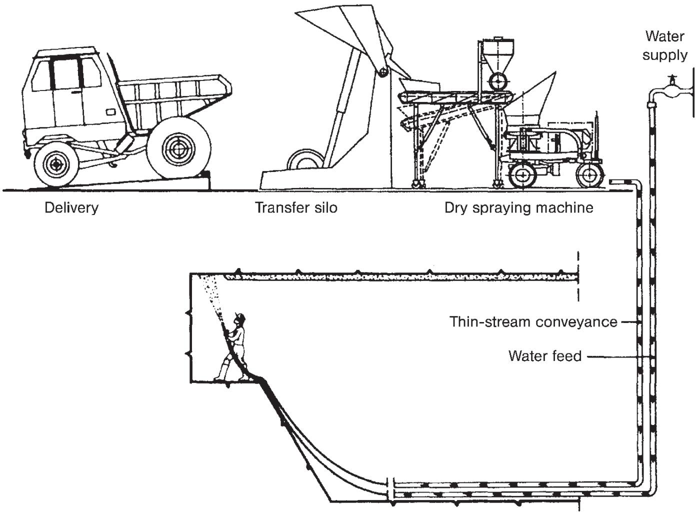


Figure 2-60 Dry-mix process in tunnelling

Traditionally, the dry-mix process was preferred in tunnelling because it was easier to handle, the equipment is not so heavy and the addition of accelerator to achieve higher early strengths was simpler. The disadvantages are higher rebound, more dust development and the health problems associated with some accelerators. The quality of the concrete depends on the qualifications of the operator holding the nozzle due to the manual addition of water, which means the homogeneity required by quality specifications is seldom met. As a result, the wet-mix process has overtaken the dry process in the last 10 years.

Dry-mix machines. Basically, there are three types of dry-mix concrete spraying machines [214]:

– Rotor machines.
– Worm pump machines.
– Pressure chamber machines.

Today rotor machines are used in most cases (Fig. 2-61). These machines consist of a flat steel cylinder with various openings arranged parallel to the axis. A feed hopper is mounted on this cylinder similar to a drum of a revolver placed horizontally.

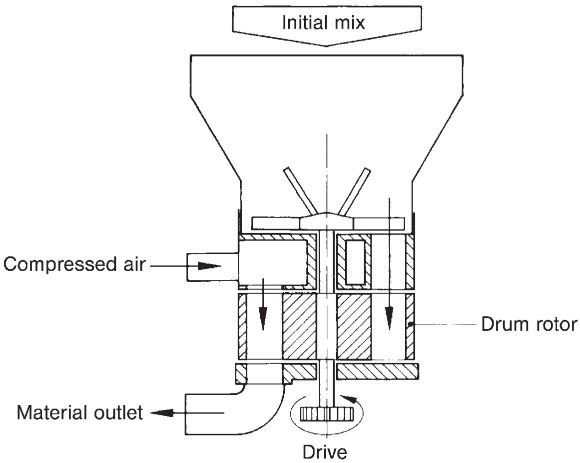


Figure 2-61 Function of a rotor machine

The spraying material fills the chambers of the cylinder and falls with the rotation of the rotor into the outlet connection below, being blown into the hose by the applied compressed air. In order to seal the pressurised outlet connection against the open holes in the rotor, a thick rubber disc is mounted below the rotor, which has to be replaced regularly due to the wear from the rotating cylinder.

The Rombold Spritzmobil (Fig. 2-62) is based on a worm pump. The machine essentially consists of a pressure vessel with a metering screw beneath it, to which the spraying hose is connected without any further pump.

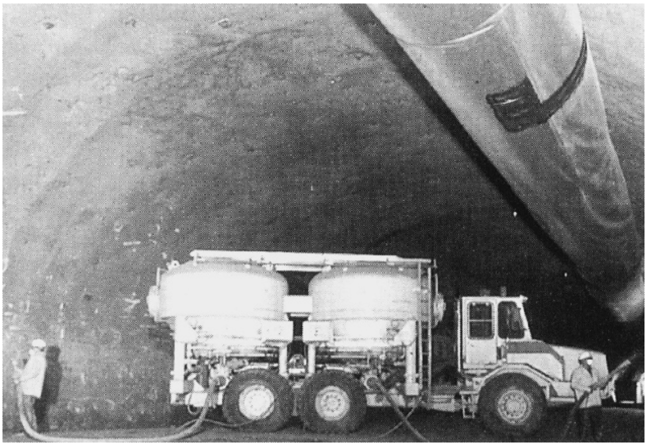


Figure 2-62 Spritzmobil with two pressure vessels and a metering screw mounted beneath; Rombold & Gfröhrer GmbH & Co KG

The metering screw, a helical worm in a tubular casing with electric drive and continuously adjustable gearing, draws the dry mix out of the floor of the pressure vessel through an opening, which can be closed with a pressure gate valve (Fig. 2-63). The pressurisation of the machine with dry compressed air pumps the dry mix in a thin stream to the spraying nozzle. In this process, a premixed dry mix with rapid-hardening cement is used, which requires oven-dried aggregates and a closed system without any possibility of moisture entering from the works until delivered in the tunnel. Since there are no movable or driven parts except for the metering screw, and moisture only at the nozzle, the system can be operated with little maintenance and wear.

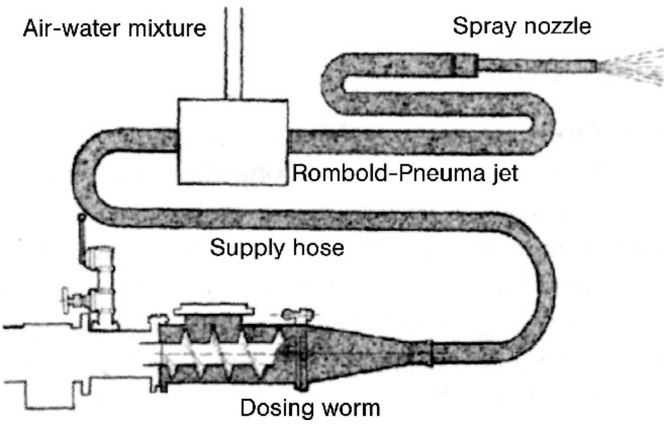


Figure 2-63 Rombold spraying system; Rombold & Gfröhrer GmbH & Co KG

Another innovative process technology from Mobil-Crete for the production of dry shotcrete is shown in Fig. 2-64. In this two-component process, shotcrete can be produced from sprayed concrete cement and aggregates with a moisture content of up to 4 %. The aggregates are delivered directly to the machine by truck or wheeled loader and loaded into a tipping aggregate container. The cement is blown into the machine with compressed air from a silo vehicle. Loading is also possible while spraying continues.

Figure 2-64 Tunnel concrete spraying machinery on electro-hydraulically driven tracks; Mobil-Crete-Spritzbeton GmbH

The use of moist aggregates can help to reduce dust at the loading point and at the nozzle. Fig. 2-65 shows a pressure chamber machine used for the dry-mix spraying process. This machine is a further development of the classic two-chamber machine. The machine only has a working and pressure chamber, from which the material is drawn.

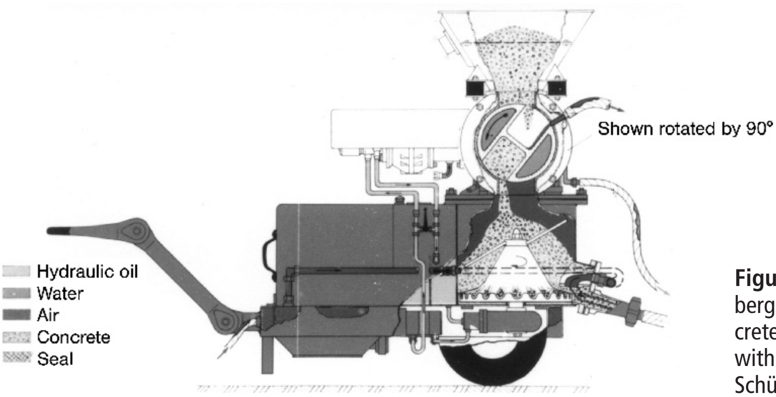


Figure 2-65 Schürenberg SBS Type B concrete spraying machine with pressure chamber; Schürenberg GmbH

Equipped with two Schürenberg concrete spraying machines, production rates of up to 2 to 12 m³ (or 2 · 6 m³ for smaller tunnel sections) of shotcrete per hour can be achieved.

b) Wet-mix process

In the wet-mix process ready concrete, a mix of cement, aggregates and mixing water and possibly admixtures, is transported to the spraying nozzle and sprayed propelled by pumps or compressed air (Fig. 2-66). This process leads to more uniform concrete quality and produces less dust, but is harder to handle and difficult to use with a wet surface, as the appropriate dosage of accelerator for such special cases is difficult. The application of thick layers and smaller quantities is less simple than with the dry-mix process, and the mix cannot be pumped such long distances.

The various methods of pumping wet-mix shotcrete are illustrated in Fig. 2-67.

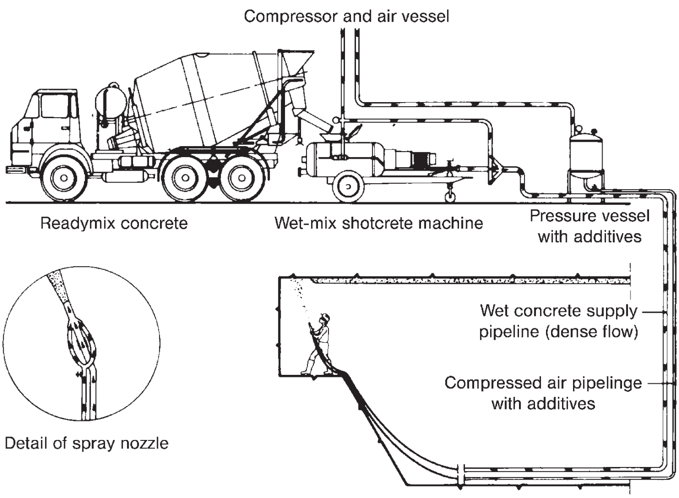


Figure 2-66 Wet-mix spraying process in tunnelling

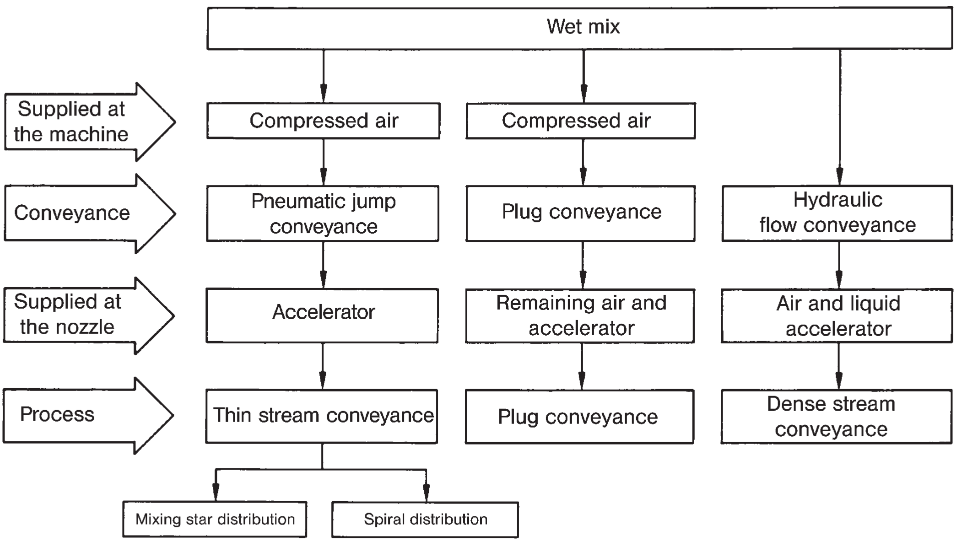


Figure 2-67 Overview of pumping methods for the wet-mix process [214]

Wet-mix machines. There are two different machine types for the wet-mix process according to the different pumping methods (thin flow or dense flow process).

Thin flow conveyance. Some spraying machines for thin flow conveyance have cylinders with pockets or holes or have a rotor, and are not very different from dry-mix machines. Another machine type is the pressure chamber machine with screw conveyor.

Dense flow conveyance. Dense flow conveyance is mostly used for the wet-mix process. The machines used are not special spraying machines but are essentially conventional concrete pumps, in some cases slightly modified for working in tunnels:

– Piston pumps (Fig. 2-68).
– Pumps with worm conveyance (Fig. 2-69).
– Rotor hose pumps (squeeze pumps) (Fig. 2-70).

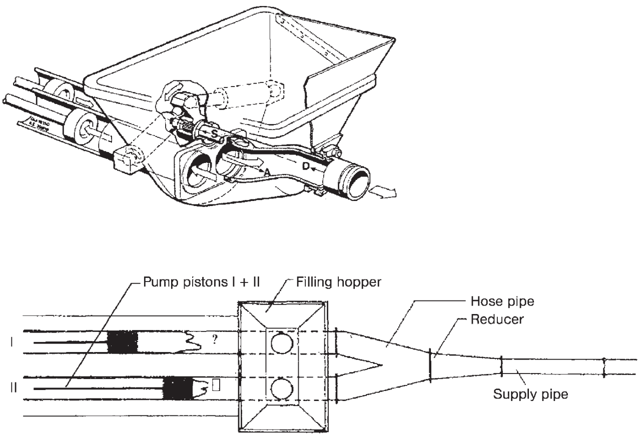


Figure 2-68 Piston pump (dense flow conveyance); Meyco Equipment MBT AG

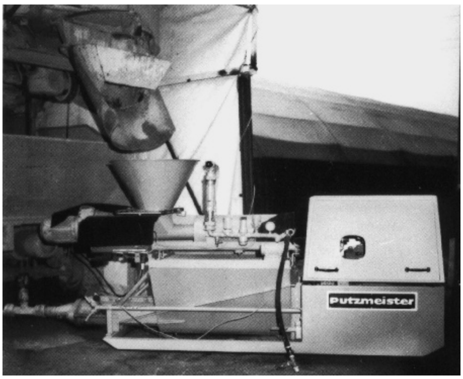


Figure 2-69 Concrete pump with worm Betojet S8 with continuous mixer system; Putzmeister AG

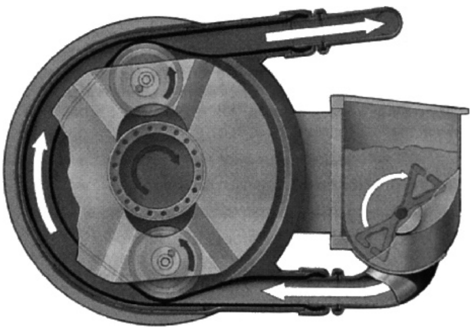


Figure 2-70 Rotor hose pump for dense flow conveyance; Putzmeister AG

Piston pumps are mostly used for the wet-mix process. In these machines, the material is pushed into the hose by a piston or two alternating pistons and uniformly conveyed to the spraying location. The development of double piston pumps, with which the changeover phase of the hose junction is significantly shorter, has enabled modern machines to achieve an almost continuous stream of concrete without pulses. Fig. 2-71 shows an example of a typical wet-mix pump [221], [214].

With plug conveyance, the wet mix in the pressure chamber is fed to the outlet by mixer arms, loosened by compressed air and accelerated to the spraying nozzle.

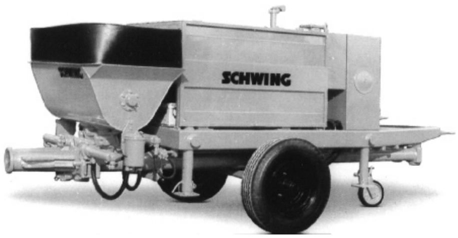


Figure 2-71 Concrete pump BP 350; Schwing GmbH

c) Spraying nozzles

Investigations at the Ruhr-University Bochum have shown that the design of the spraying nozzles has a decisive effect on dust development, material quality and cost-effectiveness, both for the dry-mix and the wet-mix process [209], [318]. But there is still room for further development. Two assemblies have to be distinguished in spraying nozzles according to their different purposes. One is the actual nozzle, which is at the end of the conveyance hose and thus forms the spray jet. These are normally straight or tapered pipes, made of plastic to save weight. The second and more important assembly of a spraying nozzle is the mixing chamber, which is where water or air, depending on the spraying process, or admixture is added in the correct quantity to the sprayed mixture.

Dry-mix process. In the dry-mix process, water is added to the stream in the mixing chamber (Fig. 2-72). As the uniform wetting of the dry mix can positively influence the mixing of the shotcrete, the detailing of the water outlet openings is an essential quality feature for dry-mix nozzles. If the water is injected with increased speed into the mixing chamber, this can significantly reduce the creation of dust although no effect on the rebound behaviour and material quality can be observed, so it is recommended to design the mixing chamber with outlet openings with the smallest possible cross-sectional area.

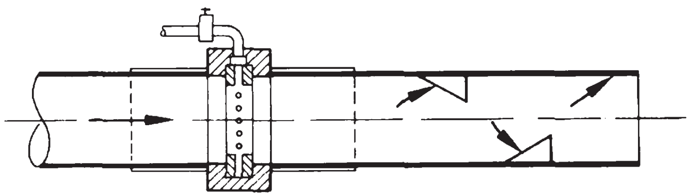


Figure 2-72
Spraying nozzle with flow obstructions

Wet-mix process. In the wet-mix process with dense stream conveyance, the mixing chamber has to break up the concrete mix pumped in a compact stream and accelerate it to the necessary speed to ensure compaction. This changeover from dense to thin flow is achieved by blowing compressed air into the mixing chamber (Fig. 2-73). The length of the thin flow section between mixing chamber and jet is decisive for the separation and pulverisation of the wet mix and thus also for heavy dust development. The shortest possible spacing creates the least dust, but with the great disadvantage that nozzles of this type are very heavy due to the high loading from the pulsating material stream and can thus only be used with manipulators or robots. For manual spraying, therefore, the use of a spraying nozzle system with the propellant air being injected at two locations is recommended. Part of the propellant air is injected into a mixing chamber, which is located in an acceleration section away from the end of the pipe. The rest of the propellant air is injected into a second mixing chamber at the nozzle. This reduces the speed difference between the propellant air and the concrete mix and enables a considerable improvement of the quality of the material produced.

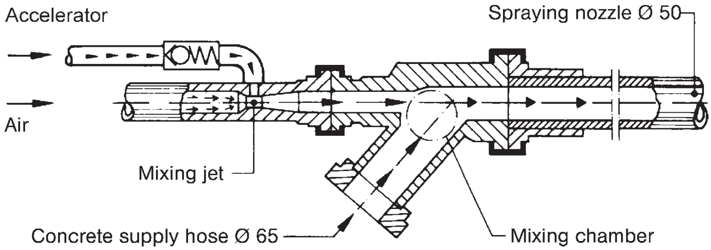


Figure 2-73 Top-Shot turbo-injector nozzle; Schwing GmbH

d) Comparison of the processes

Both processes are suitable for particular applications due to their differing process technology, material conveyance, performance figures and working conditions. Some selected performance figures of concrete spraying machines are given in Table 2-8. Further developments in recent years have shifted the advantages and disadvantages, first towards the wet-mix process and then back to the dry-mix process. The dry-mix process is generally advantageous for spraying small quantities, small sections with interrupted working, for long conveyance distances and frequent cleaning cycles. Wet-mix spraying, in contrast, is more advantageous for high quantities and larger sections, with the use of spraying manipulators almost always being a precondition due to the high weight of the hose and nozzle.

Table 2-9 shows a comparison of the two processes.

Table 2-8 Guideline technical data for spraying machines [103]

Process	Machine		Pumping output [m^3/h]	Supply hose Ø [mm]	Air quantity [m^3/min]	Air pressure [m^3/min]	Water pressure [bar]
Dense stream	Piston pump		2–20	50/65/100	4–12	6–7	
Thin stream	Rotor machine 1	Dry	4.5–6.0 6–9	50 60	8–13 11–15	2–5 2–5	3–6 3–6
		Wet	4.5–6.0 6–9	50 60	8–13 11–15	4–7 4–7	
	Rotor machine 2	Dry	8–12 13–18	60 65	11–13 13–17	2–5 2–5	3–6 3–6
		Wet	8–12 13–18	60 65	11–15 13–17	4–7 4–7	

Supply hose diameter:
- ≤ 50 mm for manual application
- > 65 mm for manipulator use
- Adapter before the nozzle with > Ø 50 mm
- hose diameter varies with manufacturer

Air volume:
- intake air volume at 1 bar [Nm^3/min]
- dense flow quantity 0.6–1.0 m^3/min – pumped concrete

Air pressure:
- 0.6–1.0 m^3/min per m^3 pumped concrete
- at the compressor outlet

Table 2-9 Comparison of dry-mix and wet-mix processes [214]

	Dry-mix process	**Wet-mix process**	
		Thin stream	Dense stream
A. Operational considerations			
Spraying performance [131], [213]	up to about 8 m^3/h		up to about 20 m^3/h
Power demand of the machine [214]	low electric power only for the insertion into the air flow, not for the actual conveyance		large hydraulic drive to provide pumping
Compressed air demand [241]	high due to the pneumatic conveyance		low due to the hydraulic conveyance
Wear [214]	slightly increased by dry pumping and high air quantity (speed)	somewhat lower due to lubrication of the wet mix (friction disc)	low due to the lubrication of the wet mix and small quantity of air
Conveyance distances [18]	up to 340 m	up to 80 m	up to 100 m
Conveyance problems [213]	danger of stoppages increases with increasing conveyance distance		danger of stoppa-ges depends on the concrete consistency, particularly if the main-tenance and cleaning is inadequate
Additive dosing [213], [58]	addition in powder form and liquid. Dosing problems above all with agents in powder form	addition of liquid, particularly high demand for thick layers	
			some dosing problems in the switchover phase of double piston pumps
Mix standing time [213]	long, when the moistu-re content is small	short due to the danger of early hardening of the readymix	
Flexibility [213], [58]	very high due to the low space required and ope-rating weight, rapid equipment times, low cleaning time and long conveyance distances	moderate due to the low space required and operating weight, rapid equip-ment times, but high cleaning time and short conveyance distances	greatly limited due to the large space required and ope-rating weight, long equipment times, high cleaning time, limited conveyance distance
Disturbance of other operational processes [58]	low due to the high flexibility		increased due to the limited flexibility
Rebound	relatively high due to the high material speed (high air quantity)		relatively low due to the low material speed (small air quantity)

Table 2-9 continued

	Dry-mix process	Wet-mix process	
		Thin stream	Dense stream
B. Concrete technology aspects			
W/C ratio [213]	not defined most variable due to the regulation by the nozzleman, low W/C ratios are archievable	clearly defined constant, mostly high W/C ratios are required (when no plasticizer is used)	
Uniformity of the concrete quality [213], [58], [186]	moderate fluctuating W/C ratio values and possible separation in the hydraulic conveyance	high constant W/C ratio but continuous addition of accelerator is essential	
Cement content [18]	300–400 kg/m^3	270–350 kg/m^3	330–450 kg/m^3
Additives	predominantly accelerator with the usual dosage	predominantly accelerator, strongly increased dosage	Accelerator, mostly with strongly increased dosage, plasticizer frequently necessary, plasticizer and accelerator are not unproblematic
Early strength	required early strength can be achieved with small dosage of accelerator	required early strength can only be achieved with high dosage of accelerator	
Final strength [58]	mostly higher due to low W/C ratios and lower accelerator dosage, but wide range of variation	mostly lower due to the higher W/C ratios and higher accelerator dosage, narrow range of variation	
C. Occupational health and safety aspects			
Dust development at the spraying machine [86]	heavy development of dry dusts	lower	
Dust development at the nozzle [86]	heavy development of dry dusts	relatively heavy development of damp dusts due to the high air demand	less heavy development of damp dusts, visible large dust mostly prevented
Weight of the nozzle	practical for manual spraying		only practical for manual spraying with low shotcrete outputs
Noise and vibration	very differentiated according to machine, earmuffs are sometimes recommended, dependent on the process		

Table 2-9 continued

	Dry-mix process	Wet-mix process	
		Thin stream	Dense stream
D. Economic aspects			
Machinery costs	relatively low due to the lesser technical complication		considerably reduced due to the lower rebound, but higher cement requirement
Energy costs	considerably higher due to the energy cost of the production of the conveyance air		low due to the hydraulic conveyance
Material costs	increased due to the high rebound quantity		considerably reduced due to the law rebound quantity, but higher cement requirement
Personnel costs /m^3 of shotcrete [119]	reduction of personnel costs /m^3 of shotcrete with increasing application quantity		
			cost reductions are possible due to the high spraying output of up to 20 m^3/h
Economical application criteria	Application for smaller spraying quantities, frequent retooling and restricted space	Application with very low W/C ratios (micro-silica- and plasticmodi-fied concrete)	Application for high spraying quantities with the use of manipu-lators

e) Special features of the spraying of steel fibre shotcrete

The process technology for the spraying of steel fibre shotcrete has now reached a level that permits the spraying of this material with conventional machines without problems, although greater wear has to be expected due to the greater abrasiveness of the steel. For dry-mix spraying, special metering equipment is no longer necessary as long as fibres are used, which do not tend to form steel fibre lumps, and when the dry mix is delivered to the site as a ready mix including the steel fibres. This is to be recommended in any case due to the more uniform quality of the steel fibre shotcrete that can be expected. The production of the ready mix is also possible on site, given the appropriate planning and supervision.

The diameter of the conveyance hose should be suitable for the fibre length, with the fibre length not exceeding 2/3 of the clear opening in the hose to avoid blockages. It should also be noted in the planning of the use of steel fibre shotcrete that it tends to rebound more. Feasibility tests should be carried out to discover what fibre content in the dry mix to be used will result in the required fibre content in samples taken from the wall. The loss of steel fibre content between the dry mix and samples from the wall depends greatly on the type of fibres and can be between 10 and 30 %. With appropriate use, it can be demonstrated that steel fibre shotcrete has led to improved quality and cost-effectiveness in completed structures (see [218]).

2.8.3.3 Mixing and recipes

In the relevant standards and additional technical regulations, details are given of the materials to be used for the production of shotcrete. The relevant documents are

– DIN 18 551 “Sprayed concrete; production and testing” [68].
– Ril 853, “Design, construction and maintenance of rail tunnels” [296].
– ZTV-SIB90, “Protection and repair of building elements” [385].
– ZTV-ING, Part 5 “Tunnelling” [384].
– DIN 1164, “Special cement” [67].
– DIN 4226, Aggregates for concrete and mortar” [74].

The following section gives detailed information about the ingredients for the production of shotcrete with examples of mixes, taking into consideration the requirements and recommendations in the standards.

1. Aggregate

In general, the use of natural round gravel is recommended [35], [45], although the required strengths can also be achieved with crushed aggregate. Compared to tetrahedral-shaped crushed grains, round grains require less cement paste to surround them and thus lower the cement content required. In addition, long, slab-shaped or chipped grains increase the danger of blockages and lead to higher rebound quantities. Also increase clearly wear on machinery and transport equipment. A certain content of crushed grains, on the other hand, has a cleaning effect on the hoses and pipes with the effect of reducing the danger of blockages. The use of long, slab-shaped or chipped grains also requires a higher water content, resulting in the formation of pores and thus a loss of strength and waterproofing qualities, also to a layered structure. Naturally round-grained gravel is therefore particularly to be preferred when the waterproofing requirements for the shotcrete are stringent.

While DIN 18551 [68] has no requirements regarding grain shape, the guideline Ril 853 [296] of German Railways DB requires additional verification of the suitability of the aggregates if crushed stone is used. This standard also includes special requirements in the presence of groundwater with chemistry that can be expected to increase the tendency of sintering of the tunnel drainage, a problem which has been encountered increasingly in the tunnels on German Railways lines, which are mostly constructed using shotcrete. In this case, only acid-resistant aggregates are to be used, whereas limestone aggregates or with a content of limestone cannot be used without additional verification.

Regarding the grading, the aggregate should consist of mixed grain sizes; a high content of coarse grains would have particularly unfavourable effects, as larger grains are mostly lost as rebound [35]. It is generally recommended to work with gradings corresponding to the B grading curves in Figs. 1 to 4 of DIN 1045 [65]. The maximum grain size should not exceed 16 mm and should preferably be restricted to 8 mm in the presence of closely spaced reinforcement. The maximum aggregate size should always be less than a third of the hose diameter and less than a third of the thickness of the construction element [45].

In contrast to DIN 18551 [68], which includes no special requirements about grading, the Ril 853 [296] requires grading distributions according to the B grading curve of DIN 1045 [65]; ‘gap-graded’ and oversized aggregates are not allowed and have to be sieved out.

The aggregate used should always have a certain moisture content, particularly with the dry-mix process. Dry material is more expensive and tends to separate during pumping due to the different conveyance behaviour of dry dust-fine and coarse-grained materials. The water contained as moisture content of the aggregate produces adhesion forces, which bind the dust-fine grains to the larger grains and to each other, due to which the dry mix takes on the flying characteristics of almost coarse-grained material [35]. Too much moisture content in the aggregate, in contrast, increases the danger of blockages. Moisture content values of 2 to 6 % by weight are practical.

When admixtures are used, it should be considered that for example accelerators start reacting with the cement immediately after being added because of the moisture content in the aggregate and thus, according to the feed location, a large part of the adhesion capacity of the mix can be lost before spraying or the reaction times may even be nearly completed.

The relevant regulations do not contain any definite requirements concerning the moisture content. The DIN 18551 [68] only specifies in this regard that mixing and conveyance must be guaranteed without problems and that the specified moisture content of the aggregates must be complied with a deviation limit of 1 %.

2. Cement

For the production of shotcrete, only

- normal cements according to DIN EN 197-1 [79], Special cements according to DIN 1164 [67] or
- special sprayed concrete cements with type approval from the Deutsches Institut für Bautechnik (DIBt)

may be used [68], [385]. Some additional requirements specific to sprayed concrete are included in Ril 853 [296]. For example, the hardening time is specified differently from DIN 1164 [67]; the onset of hardening of cements according to DIN EN 197-1 [79] and DIN 1164 [67] at a time of 1.5 h to 4 h is compulsorily specified. The specified alteration of the onset of hardening to 1.5 h at the earliest takes into consideration the conditions on the site concerning transport into the tunnel, the spraying process and the waiting time that is often necessary until the shotcrete is actually applied. The upper limit of the onset of hardening is derived from the requirements for early strength, relating to the spraying and adhesion of the shotcrete.

Regarding the fineness of the cement, a specific surface area of 3,500 cm^2/g is required in this standard, which takes into consideration the early load-bearing task of the shotcrete and the associated requirements for early strength and final strength.

In addition, the Ril 853 [296] specifies a minimum compression strength of the hardened cement paste of 10 N/mm^2 after 2 days and 35 N/mm^2 after 28 days, which is to be verified by the delivery works as part of quality testing in accordance with DIN 1164 Part 2 [67].

When accelerators are used, particular attention should be paid to the compatibility of cement and accelerator. This requires additional quality tests to determine the favourable combinations and doses [68], [296].

The selection criteria for the cement used for the production of shotcrete are not normally decided by the spraying characteristics but much more by the specified characteristics of the hardened concrete. This does not consider the effect of process technology on the final quality. Portland cements are generally used, but blast furnace cement with various slag contents and particularly sulphate-resistant cements have also proved suitable in practice.

Rapid-hardening cements are recommended for shotcrete support in mining and tunnelling, for spraying onto cold substrates, to stem water ingress to the substrate and in case of risk of the effects of vibration or frost, heat, wind and air draughts (drying out).

Practice has shown in numerous cases that the temperature of delivered cement varies widely, leading to differing rates of reaction with the accelerator and thus to scattering of the strength properties, particularly the early strength. This is a matter that should not be overlooked in the processing of shotcrete and should be verified through quality testing.

For some years, low-sulphate or sulphate-free Portland cements have also been available as shotcrete binder or shotcrete cement. With these specially adapted binders, shotcrete can also be produced without accelerator. This is considerably less harmful for the miners and also for the environment than shotcrete with the usual accelerators. The omission of alkaline hardening accelerator also considerably reduces elution and thus ensures uniformly strong and dense concrete.

The hydration of the binder begins a few seconds to minutes after contact with water, depending on the type. The hardening behaviour and the strength development of shotcrete binders approximately corresponds o those of Portland cement with added accelerator.

Two types of binder are differentiated in [190]:

Binder type SBM-T starts to hydrate after only a few seconds to about one minute and can therefore only be used together with oven-dried aggregates.

The second binder type SBM-FT starts to hydrate after more than 90 seconds and can thus be used with moist aggregates (FM). Due to the short permissible pre-hydration times, special spraying equipment is needed for spraying.

3. Admixtures

In shotcrete production, it is often necessary to intentionally alter various properties for various reasons. Admixtures can be added to alter the properties either of the wet concrete or the hardened concrete, or the process-related properties of the production process.

The compatibility of the cement and the admixture used is very important, and this should be verified with separate qualification tests according to DIN 18551 [68] and Ril 853 [296].

According to DIN 1045 [65], the volume of concrete admixtures can be neglected and does not have to be considered in concrete mix design by volume. Only admixtures with a valid technical approval are allowed. This technical approval is issued by the Institut für Bautechnik (IfBt) in Berlin, which has also specified limits to the quantities to be added (Table 2-10) in their guidelines for the issue of technical approvals for concrete admixtures [146].

The specification of a lower limit is intended to avoid the use of admixtures with very high concentrations of effective agents, which might not mix uniformly into the concrete.

The maximum permissible quantity is 50 ml (g)/kg cement when only one admixture is used. If more than one admixture is added, then 60 ml (g)/kg cement is the maximum permissible quantity.

Admixtures are listed in Table 2-11 according to their various effects on the material properties. The admixtures used in shotcrete mixes are essentially confined to concrete plasticizers/superplasticizers, accelerators and stabilisers, although it should be noted that all the concrete admixtures tested and used as dust binders have the effect of stabilisers.

Table 2-10 Limit quantities for the addition of admixtures for shotcrete application, from [146]

Minimum addition quantity	> 2 ml (g) / kg cement
Maximum addition quantity	> 50 ml (g) / kg cement > 60 ml (g) / kg cement

Table 2-11 Admixture groups according to their effect and their German codes [146]

Action group	Abbreviation	Colour code
Plasticizer	BV	yellow
Superplasticizer	FM	grey
Air-entraining agent	LP	blue
Waterproofing agent	DM	brown
Retarder	VZ	red
Accelerator	BE	green
Grouting aid	EH	white
Stabiliser	ST	violet

The admixtures listed above are used in shotcrete production and are now described in detail.

Accelerators. A shotcrete mixed without accelerator hardens like normal concrete after about two to three hours. In many applications, like for example sealing work or repairs, and particularly spraying of shotcrete as support in tunnelling or mining, which demand the application of a layer 15 to 30 cm thick often immediately followed by blasting, much more rapid hardening is required. This is normally achieved by adding accelerators, which considerably increase the strength development in the first few hours.

The most important raw materials for accelerators are [59]:

- Silicates.
- Aluminates.
- Carbonates.
- Formiates.
- Amorphous aluminium hydroxide.
- Aluminium sulphate.

These raw materials are used alone or as mixtures, as powder or in aqueous solution, as dispersion or suspension.

Silicates, carbonates and aluminates as calcium salts are poorly soluble in water and are bonded into the cement matrix. In the dosages that are usual in shotcrete, elution tests have shown that part of the stated alkali is leached out by water.

The effect of this accelerator is based on increasing the hydration speed of the cement. All the substances have in common a strong alkaline reaction in aqueous solution. This increase of the pH value leads to accelerated solution of the tricalcium-aluminates or -silicates present in the cement [262]. The acceleration effect can be achieved in three different ways [261]:

- The solution of the cement ingredients is increased.
- The formation of a retarding layer around the cement particles is hindered.
- Crystallisation nuclei are provided, which simplify the formation of hydrate crystals.

Accelerators based on amorphous aluminium hydroxide and aluminium sulphate are alkali-free and normally bear the additional description " alkali-free accelerator" or "alkali-free shotcrete additive". For alkali-free accelerators, there is a limit to the overall alkali content (total Na_2O equivalent) in the accelerator of 1.0 percent by weight of the accelerator [177]. The aluminium is fixed in the cement paste matrix as calcium aluminate phase. As this type of accelerator does not contain any alkali ions, it does not further increase the alkali content and the elutable content of the shotcrete [59].

For a more detailed description of the chemical and physical processes, reference can be made to the following further sources [56], [177], [186], [261], [262], [374].

Accelerators based on aluminates or silicates are the most common type worldwide, although a trend to use alkali-free accelerators is discernible, particularly as the Ril 853 [296] and the ZTV-ING, Part 5 [384], have recently permitted the use of alkali-free accelerators. Accelerators based on chloride compounds and water glass are no longer approved.

In principle, any increase of early strength with the use of accelerators containing alkalis is always balanced by a loss of final strength of the concrete. This disadvantage is no longer observed with environmentally neutral accelerators based on amorphous aluminium hydroxide and in combination with reactive sulphates [177].

It is particularly important that the accelerator and the cement are suited to each other, and this is a compulsory requirement of some regulations [296], [384] in the form of preliminary tests to investigate the compatibility of the cement and the accelerator and determine the optimal dosage. The gaining of information about the curves of hydration and hardening under site conditions is specially significant, with the appropriate consideration of conditions on the specific site like for example temperature of the substrate and the ambient air, air draughts, wind and application thickness.

Regarding the dosage of accelerators, increased quantities are not directly associated with continuously shortening hydration times and accelerated strength development. Fig. 2-74 shows a characteristic curve of the hydration time of shotcrete against the dose of accelerator. It is clear that the hydration time falls sharply with an increased dose but then rises again at higher dosages, so that the effect of the accelerator becomes negative.

In addition to the actual purpose of ensuring early hydration and hardening of the concrete, accelerators also improve adhesion, increase the waterproofing effect and the resistance against chemical attack [35].

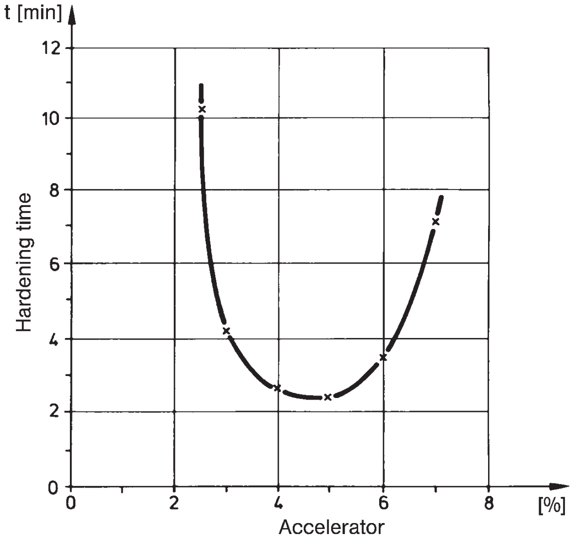


Figure 2-74 Hydration time dependent on the quantity of accelerator

In addition to the reduced final strength, further negative side effects of the use of accelerators are the reduction of the Young's modulus, creep and shrinkage as well as a strong tendency to precipitate.

Hydration accelerators can be obtained both in powder and liquid form. While only liquid accelerator is used with the wet-mix process, both types can be used with the dry-mix process.

When accelerator in powder form is added during the preparation of the dry mix, an initial hydration takes place due to the moisture content of the aggregate, which causes a part of the effect to be lost and leads to reduced strength.

A good but unfortunately seldom used method of dosing is feeding through a metering screw into the material stream from the spraying machine.

Due to the disadvantages associated with the use of accelerator in powder form, the tendency is increasingly to use liquid accelerator with the dry-mix process. It is mostly fed through separately adjustable metering pumps (and in the dry-mix process, also sometimes together with the mixing water) directly to the nozzle, so that the reaction takes place later in this case than with powder accelerator.

Liquid accelerators generally consist of the active agent and a carrier material (mostly water), and the concentration of the agent can be 10 to 100 % by weight [59]. Therefore when liquid accelerators are used, an additional quantity of water is added in addition to the normal mixing water. The resulting increase of the W/C ratio, which is not inconsiderable, should be taken into account in the mix design.

Plasticizer, superplasticizer. Plasticizing additives are often used in the production of shotcrete for the wet-mix process. This is intended to achieve a given consistency, which is a measure of the workability of the concrete, and enables the concrete to be pumped through the elbow, reducing piece and pumps of the spraying machinery without problems despite a low W/C ratio. The standing times associated with the ready-mixed transport commonly used with the wet-mix process should also be considered.

The admixtures suitable for this purpose are plasticizers (BV) or superplasticizers (FM). Superplasticizers are a special group of plasticizers with twice to three times the effect. These products are predominantly based on lignin, melamine or naphtalene compounds. Their effect is essentially based on the lowering of the surface tension of the water, which results in improved wetting of the solids in the concrete. They cause a complete dispersion of the cement and thus a reduction of friction between the solid contents, which results in better mobility and thus better workability of the concrete [56].

The side effects of the use of plasticizers and superplasticizers are retarding of hydration, and if overdosed, more rapid hardening and increased creep and shrinkage (BV) as well as water bleeding and separation (FM) have been observed.

One particular issue is the use of plasticizing admixtures together with accelerators, as no detailed scientific information about this combination is yet available. The problem should therefore be given particular attention in preliminary tests and the admixtures should be matched to each other as precisely as possible.

Dust binder. A major disadvantage of the process of spraying concrete is the creation of dust, above all near the nozzle, which makes the workplace of the nozzleman very unpleasant due to dangerous fine particles and coarse dust, which reduces the visibility. The reduction of dust development has been paid plenty of attention at a time when occupational health and safety is becoming ever more important [120], [232].

Dust binding agents can be used as admixtures to reduce dust. These are categorised as stabilisers.

Concrete admixtures in the group of stabilisers (ST) are a relatively recent development as approved concrete admixtures. The first tests were performed in 1977 and published [257], and technical approval certificates have been issued by the 'Deutsches Institut für Bautechnik' (DIBt) for this new group of agents since about 1981.

Stabilisers mostly consist of organic substances such as polyethylene oxide. They have a physical effect on the cement paste, wet mortar or wet concrete. The essential effects are:

– reduction of water bleeding.
– increased viscosity of the mixture.
– increased adhesion.

While the first two effects are relatively insignificant for shotcrete production (although the increased viscosity can lead to the saving of pumping energy in the wet-mix process), the increased adhesion is mainly responsible for the reduction of dust emissions. The strong adhesion increases the bonding between the particles in the shotcrete to such an extent that they can no longer separate as dust particles after leaving the spraying nozzle. The reduction of dust emissions through the use of stabilisers also leads to a reduction of very fine particle content. The danger of the particle bonding separating due to the flow of compressed air after leaving the nozzle is reduced.

Numerous manufacturers are engaged in the production of special dust binding agents or dust reducers, which have a particular property of dust reduction, but none of these has yet been awarded a technical approval by the DIBt. The use of stabilisers for dust reduction is still unusual as the higher cost and work involved in metering have been decisive constraints until now.

The side effects that have been observed with the use of approved stabilisers in shotcrete production are reduced rebound quantities on the positive side and to some extent reduced early and final strengths on the negative side.

Retarders. For some years now, some manufacturers have offered so-called long-term retarders, which enable the concrete or the ready mix to be preserved in its original form for many hours to days. Long-term retarders consist of two components, a "conservator", which stabilises the ready mix and an "activator", which neutralises the conservation and starts the normal cement reaction.

The process is based on the principle that a thin layer of the conservator envelops the cement grains and hydration is suspended. In this condition, the material can be stored and processed for many hours. The addition of the activator can restart the cement reaction at any time. This applies for ready mixes for both the wet and dry spraying processes. With the usual dosage of 1 % of the weight of cement, the mix can be kept workable for ten or twelve hours without problems. With the dry-mix process, the use of dry aggregates with a moisture content < 0.5 % can also achieve a long period of workability for the dry mix, so the real advantages of these retarders apply to wet-mix applications. The advantages are:

– The very long workability period.
– It is no longer necessary to clean the pump after short interruptions.
– The reduction of concrete waste through pump cleaning or leftover concrete.
– Independence from concrete transport and concrete mixing.
– Simplified cleaning of the mixer and equipment.

As these admixtures are relatively new on the market, there have unfortunately not been any basic investigations into their effect on concrete quality, rebound behaviour or the fine dust emissions to be expected.

Synthetic dispersions. Synthetic additives have been used for some time to improve the quality of shotcrete in thin layers [321]. These are predominantly used in the repair of tunnels or concrete structures. The most commonly stated advantages of polymer modifications for shotcrete are [21]:

– Increased water retention capacity.
– Better workability.
– Higher density.
– Higher adhesion tension strengths.
– Less shrinkage.
– Increased chemical resistance.
– Excellent thermal behaviour (Young's modulus).
– Excellent resistance to frost and frost in combination with de-icing salt.

The improvement of shotcrete with synthetic additives works through the following mechanism: when the CSH gel forms during cement hydration and the water content in the shotcrete is reduced, the dispersion particles move together until they form a film.

Synthetic dispersions are generally added through the use of appropriately modified ready-mixed mortar or concrete, which are available from various manufacturers. As the main application is in repair work, the ready mix is then sprayed using the dry-mix process. There is little experience so far with the use of synthetically modified wet-mix shotcrete.

4. Additives

The second group of materials used to alter the properties of concrete and shotcrete are additives. As these are added in larger quantities, their volume has to be considered for mix design purposes according to DIN 1045 [65], which states "concrete additives are permitted for use when they do not impair the hardening of the cement, the strength or durability of the concrete or the corrosion protection of the reinforcement."

The required technical approval has been issued for the following concrete additives:

– Natural stone dust according to DIN 4226 Part l [74].
– Trass according to DIN 51043 [75].
– Concrete additives with a technical approval from the 'Deutsches Institut für Bautechnik'.

The palette of concrete additives includes the following materials used at the moment [56]:

– Mineral materials.
  Stone dust.
  Latent hydraulic materials.
  Pozzolanic materials.

– Organic materials.
  Synthetic additives.
  Colourings.

In addition to the synthetic admixtures, which are sometimes added to shotcrete for repair work, it is mostly mineral materials with latent hydraulic or pozzolanic properties that are used as additives in shotcrete production.

The most commonly used materials are by-products from other industries such as:

– Coal fly ash or PFA.
– Silica dust.

The properties and function of these materials are described in the following section.

Fly ash. Fly ash has been successfully used as an additive in numerous shotcrete applications [348].

A particular advantage of the use of fly ash is the improvement of the sulphate resistance of the shotcrete, provided the combination of cement and fly ash is suitable. This is especially useful when the shotcrete will be exposed to water with high sulphate content. In addition, with a carefully designed mix of cement, admixture (accelerator) and fly ash, there can be a positive effect on the strength and the rebound quantity.

In Germany, coal fly ash is predominantly used. It needs a technical approval and must be continuously monitored [370]. The issue of technical approvals and the control by the manufacturer are regulated in the guidelines of the 'Deutsches Institut für Bautechnik' [144], [145].

Fly ash is the fine-grained residue of the combustion of coal dust, which is electrostatically precipitated from the flue gas of steam generators in power stations. It consists of fine mineral grains of glassy particles, mostly round, which have a minimum required specific surface area of 3,000 cm^2/g and is thus finer than cement (2200 cm^2/g). Most fly ashes have pozzolanic properties, meaning when they are in contact with calcium hydroxide in the presence of water for a sufficient time, they react to form calcium silicate hydrates, which increase the strength of concrete. This property is useful in shotcrete production in order to reduce the loss of final strength associated with the use of accelerators. The cement content and the corresponding accelerator content are reduced to a quantity sufficient to achieve the required setting time and early strength. The final strength can then be significantly increased by the pozzolanic properties of the added fly ash, which reacts very slowly and is not influenced by the accelerator. The density of the structure is also improved. It is also worth mentioning that the fly ash leads to an increased adhesion, which results in the reduction of rebound and dust emissions.

In addition, an improvement of the transport behaviour of the dry mix has often been observed, which is due to the relatively high specific surface area and hydraulic inactivity of the fly ash. The high specific surface area provides a certain lubrication effect [153].

A property of coal fly ash that is especially interesting for the wet-mix process is the improvement of the pumping characteristics of the shotcrete and the reduction of the water requirement for the same consistency.

The addition of fly ash also increases the resistance to chemical attack, particularly sulphate attack as already mentioned.

Despite the capability of fly ash to contribute to the formation of a matrix similar to cement paste, the consideration of the fly ash content as part of the cement content is only permissible up to the appropriate minimum content according to DIN 1045 [65].

Silica dust. Silica dusts (SF) are often known under the product names "microsilica" or "silica fume". They were originally used in concrete construction only to improve compression strength and waterproofing properties, but the advantages for shotcrete have recently been recognised and put to use.

Silica dust is produced as a by-product of the production of silicon metals and alloys through reduction of quartz in an electric arc. The fly ash is then separated in large filters [165].

Silica dust is also produced synthetically in the chemical industry as dust-fine silica precipitate. This product does have a higher quality, but this quality is not absolutely necessary and synthetically produced silica dust is currently considerably more expensive than silica dust from ferrosilicon production [102]. With specific surface areas between 18 and 25 m^2/g and an average grain diameter of 0.1 μm, silica dust is almost 100 times finer than cement and with it so small that it can fill the cavities between the individual cement particles [250].

This is the reason for a large part of the positive effect of silica dust. The extremely fine particle size means that silica dust can penetrate the smallest cavities and at as a crystallisation nucleus for the cement.

The silicon dioxide contained in silica dust is also predominantly amorphous, which means reactive, so it can also form reaction products with the $Ca(OH)_2$ separated by the hydration of Portland cement clinker to form calcium silicate hydrates. These are hydration products, which are otherwise produced by the hydration of the calcium silicates of the clinker. Silica dust is therefore an extremely active pozzolanic concrete additive.

The processes described mostly have the effect of improving the properties of the hardened concrete. There is a much denser concrete structure, which leads to considerably increased compression strengths as well as increased resistance to the penetration of water, atmospheric carbon dioxide and other aggressive gases or solutions.

Silica dust is currently mostly delivered as a suspension or slurry for practical reasons and added with a quantity of 5 to 20 % of the cement content of shotcrete.

In the wet-mix process, silica dust suspensions can be added either with the initial mixing or at the nozzle. When it is added to the mix, a superplasticizer is normally also necessary due to the extremely high specific surface area of the additive.

In the dry-mix process, the initial results with the addition of silica dust to the dry mix as a powder did not prove successful and it has become more usual to add it as a suspension, normally together with the mixing water [126]. Practical reports about the use of synthetic silica dust show that it has been successfully used in the wet-mix process, normally being added through a special metering device at the nozzle.

This process accelerates hydration by exploiting the suction effect that is observed when the mixing water is suddenly bound due to the high specific surface area of the concrete additive [279].

In addition to the already mentioned improvement of a significantly higher density of the internal structure resulting in improved strength properties as well as improved resistance to the penetration of water and chemical attack, the use of silica dust also has specific advantages for the production of shotcrete. The improved cohesive properties of the shotcrete results in a clear reduction of rebound quantity and the shotcrete can also be applied in thicker layers, even without the addition of accelerators. Dust emissions can also be reduced by the addition of silica dust, although this reduction will only affect the emission of coarse dust, which results in better visibility [102], [279], [250], [152].

5. Mix examples

Some mixes for shotcrete from the literature or from current tunnel sites are given in Table 2-12, both for the dry-mix and the wet-mix process.

Table 2-12 Use of shotcrete mixes

Project	Strength class	Aggregate	Cement		Water		Admixture		Additives		Reference
			type	quantity kg/m³	quantity	*w/z*	type	quantity	type	quantity	
Dry-mix process											
Landrücken Tunnel Süd	B 25	B 16 Main sand (crushed) crushed basalt	PZ 35 F	360 kg/m³	170 l/m³	0,47	accelerator (powder-form)	6–7 %	fly ash	40 kg/m³	[142]
Schönrain Tunnel	B 25	B 12 processed sand crushed basalt	HOZ 35 L	340 kg/m³	170 l/m³	0,50	accelerator	2.6 %	silica dust	27 kg/m³	[155]
Stadtbahn Bensberg	B 35	crushed sand 0/2 crushed stone 2/8	CEM I	330 kg/m³ 42.5 R-SE	165 l/m³	0,50					[103]
Wet-mix process											
Bielefeld underground inner lining, steel fibre shotcrete	B 25	A/B 8	PZ 45 F	380 kg/m³	208 l/m³	0,55	none		fly ash steel fibres silica dust	50 kg/m³ 70 kg/m³ 11 kg/m³	[236]
Vereina Tunnel final lining	B 45	sand 0/4 gravel 4/8	CEM II	425 kg/m³ 52.5/BTC800	225 l/m³	0,53	stabiliser accelerator	5.4 kg/m³ 17 kg/m³			[103]
Irlahüll Tunnel	B 25	sand 0/2 crushed stone 2/8	CEM I 52.5	380 kg/m³	200 l/m³	0,55	superplasticizer retarder accelerator	0.6 % 0.3 % 8-10 %	fly ash	50 kg/m³	[9]
Tunnel Burgholz	B 25	0/8	CEM III 42.5	410 kg/m³	190 l/m³	0,46	accelerator	6 %	none		[8]
Sprayed cement process											
Dortmund S 10	B 25	A/B 8	CEM II 32.5 R-SE	300 kg/m³	165 l/m³	0,55	none		none		[7]
Landeck Nord	B 45	0/8	Cement	350 kg/m³	150 l/m³	0,53	none		none		[103]

2.8.3.4 Influence of materials technology and process technology

1. General

The essential criteria, which a shotcrete has to meet for an exhaustive evaluation, are:

– Shotcrete quality.
– Rebound behaviour.
– Dust development.

The information is based on investigations whose results can be found in the relevant literature and in particular the research results of the Chair of Tunnelling and Construction Management at Ruhr-University Bochum, where intensive basic research has been carried out into shotcrete since the end of the 1970s.

The following section describes only the most important factors; the subject is dealt with in detail in the shotcrete handbook [214], from which quotations are taken so that the stated standards and regulations have now been supplemented. Nonetheless, this stage of development is dealt with here because it was important for the current state of technology.

The following diagrams should be considered in combination with the material technological and process technical parameters given in Table 2-13 and Table 2-14. The absolute values can however vary from machine to machine.

Table 2-13 Material technological starting parameters

Cement type	PZ 35 F
Cement content	350 kg/m^3
Aggregate	quartzitic grain, oven-dried
Grading curve	B 8
Moisture content	3 % by weight
Water/cement ratio W/C	0.5

Table 2-14 Process technical starting parameters

Dry-mix process	
Spraying machine	ALIVA 260 with rotor distribution
Nozzle	ALIVA pre-wetting nozzle with wetting length l = 2.0 m
Conveyance hose length	40 m
Conveyance hose diameter	50 mm
Air quantity	9 m^3/min
Wet-mix process with thin stream	
Spraying machine	ALIVA 280 with rotor distribution
Nozzle	Spraying nozzle l = 0.4 m
Conveyance hose length	40 m
Conveyance hose diameter	50 mm
Air quantity	9 m^3/min

Table 2-14 continued

Wet-mix process with dense stream	
Spraying machine Nozzle Conveyance hose length Conveyance hose diameter Air quantity	Schwing Concrete pump BP 750 RE Turbo injector nozzle Top-shot (Schwing GmbH) 20 m 50 mm 4 m^3/min

2. Influence of material technological factors

The essential material technological factors, which have an effect in shotcrete production, are:

- *W/C* ratio.
- Aggregate.
- Cement.
- Additives.

a) Influences affecting the quality of the shotcrete

The compression strength of the shotcrete is mainly regarded as the most important evaluation criterion, and the factors influencing this are illustrated in Fig. 2-75.

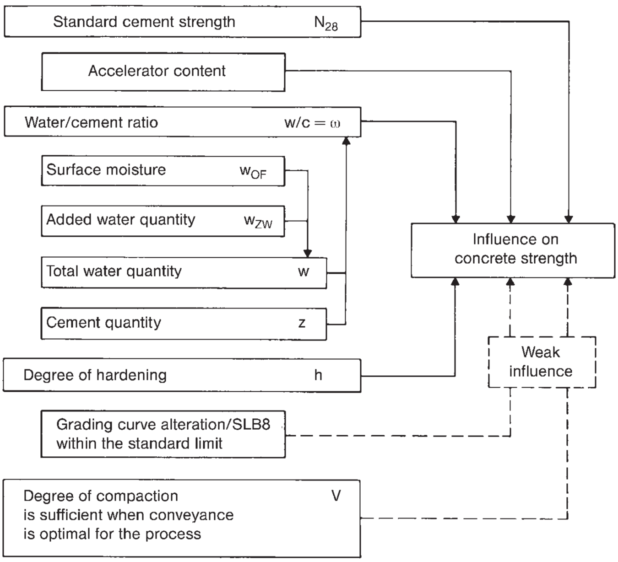


Figure 2-75 Influential factors in the production of shotcrete with regulated process technology [117]

W/C ratio. The *W/C ratio* is the most important factor influencing the compression strength of concrete. It is important to note that the water content W is the sum of the mixing water and the moisture content of the aggregate. It should be considered that the quantity of moisture is subject to variation. Continuous checking of the moisture content is therefore indispensable.

Experience shows that the range of variation of the *W/C ratio* is very much less with shotcrete processes. For the dry-mix process, *W/C ratios* of 0.45 to 0.55 are normally selected, and for the wet-mix process *W/C ratios* of 0.6 are not seldom to ensure workability, but this is surely the upper limit.

Aggregate: Has been dealt with in detail in section 2.8.3.3.

Cement: Has been dealt with in detail in section 2.8.3.3.

Additives: Has been dealt with in detail in section 2.8.3.3.

b) Influences affecting the rebound behaviour

W/C ratio. Fig. 2-76 shows the results for rebound behaviour depending on the *W/C ratio*. The cause of the reduced rebound percentage with increased *W/C ratio* is the increasingly soft consistency of the sprayed shotcrete at higher *W/C ratios*, which enables easier penetration of the aggregate grains into the fresh shotcrete layer and thus reduces the amount of rebounding grains. In addition, the rapid effect of the accelerator enables the required adhesion to be reached.

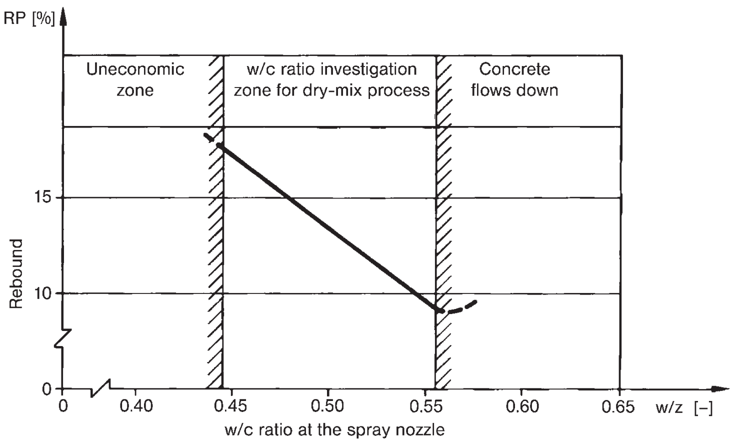


Figure 2-76 Rebound behaviour depending on the *W/C ratio* (dry-mix process) from [63]

The rebound percentages show quite different curves for wet-mix spraying depending on whether thin flow or dense flow is used (Fig. 2-77).

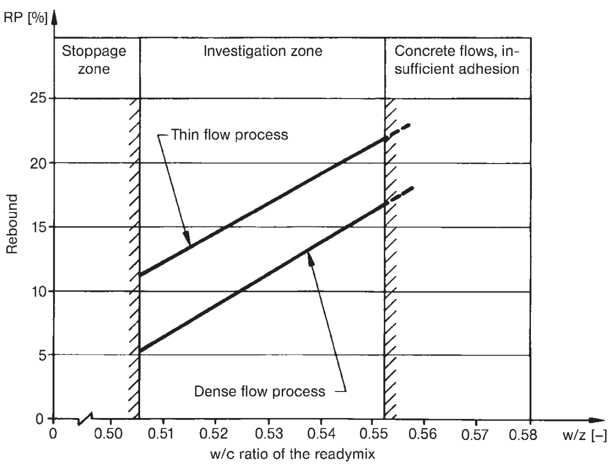


Figure 2-77 Rebound behaviour depending on the *W/C ratio* (wet-mix process) from [63]

The cause of the increased rebound with increasing *W/C ratio* is seen by von Diecken in the progressive reduction of the effect of the adhesion force due to the increased water content.

Aggregate. The influence of the type of aggregate used on the rebound behaviour with the dry-mix process can result from the type of aggregate, its grading and moisture content. Gradings with a higher content of coarse grains lead to increased rebound, as coarser aggregate grains have more tendency to be lost as rebound.

The results of the investigations show a clear reduction of rebound with increased moisture content of the aggregate. This reduction is very strong in the range from 1 to 3 % and less strong in the range from 3 to 5 %. Even at moisture contents from 5 to 7 %, there is still a slight reduction of rebound, but conveyance in this range already poses considerable problems, so that such high moisture content has to be regarded as unfavourable for conveyance.

With the wet-mix process, there is naturally no connection between the moisture content of the aggregate and the rebound behaviour, although of course the moisture content of the aggregate has to be taken into account in the calculation of the water content in the mix.

Cement. Fig. 2-78 shows the relationship of rebound percentage to the cement content of the ready mix using the dry-mix process.

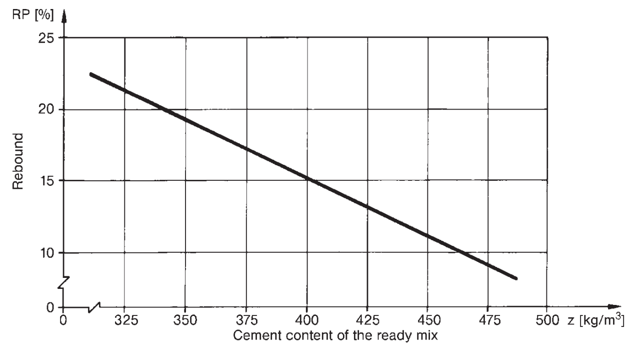


Figure 2-78 Rebound behaviour depending on the cement content (dry-mix process) from [63]

The cause of this reduction of rebound is the increase of the cement paste volume with increasing cement content. The effect is better adhesion to the substrate and easier penetration of the arriving grains of aggregate into the existing layer, without its adhesion capacity being exceeded, which is the effect of increased *W/C ratios*.

The illustrated rebound behaviour according to cement content represents an economic contradiction (falling rebound with increasing cement content). An alternative is the addition of fly ash and silica materials to provide the fines content.

The wet-mix process offers a considerably smaller range of cement contents compared to the dry-mix process, as this is primarily determined by the consideration of conveyance practicalities. If the cement content is too low, this leads to irregular conveyance and blockages. Plasticizers and fly ash are added to improve the conveyance properties.

Admixtures. Accelerator. In the tests described in [63], which were specially devoted to this subject, no influence of the accelerator on the rebound quantity was determined. The reaction time of the accelerator is clearly too long to develop an adhesion effect that could reduce rebound. Nonetheless, it should be acknowledged that it is possible that when large quantities of shotcrete are sprayed with the associated large areas and layer thicknesses, the accelerator would have an influence on the rebound quantity.

c) Influences affecting dust development

Dust development from the application of shotcrete represents a serious health risk to the miners in tunnelling, or to all those involved with the spraying of shotcrete. This applies particularly to the dry-mix process, but the problem should not be neglected when the wet-mix process is used.

W/C ratio. Test results show that using the dry-mix process, an increase of the water cement ratio, which is determined by the moisture content of the aggregate plus the water added at the nozzle, with an increasing *W/C ratio* at the nozzle, considerably reduces dust development. But the boundary conditions should still be observed, as otherwise other properties like the strength might not be reached.

Investigations [87] show that for both processes, an improvement of the fine dust concentration of up to 13 % at the nozzle can be achieved through the selection of the appropriate *W/C ratio*.

Aggregate. The influence of the aggregate on the dust nuisance is determined by the individual stages of production and thus different sources of dust. This means in particular the feed area, where the ready mix is fed into the spraying machine, and the machine itself with its various air outlets, but also the nozzle.

The most important factor for the evaluation of the dust development concerning the aggregate is still, however, the moisture content of the aggregate.

Fig. 2-79 shows the results of a quantitative investigation from Handke [120].

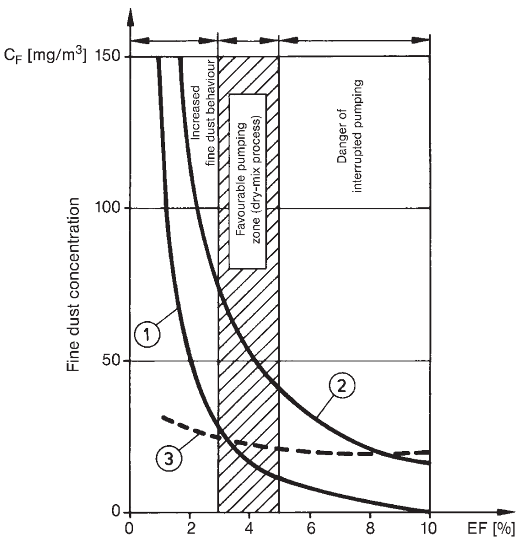


Figure 2-79 Fine dust concentrations depending on the moisture content in the ready mix (dry-mix process) from [120] in the
1. feed area.
2. at the machine.
3. at the nozzle.

Cement. The effect of the cement content of a ready mix on the fine dust concentration was investigated by Handke [120] for the example of the dry-mix process (Fig. 2-80).

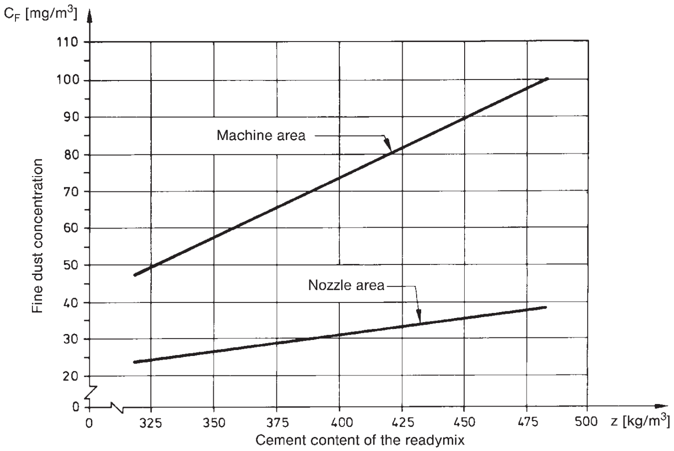


Figure 2-80 Fine dust concentration at the machine and nozzle depending on the cement content (dry-mix process) from [120]

Admixtures. Accelerator. Investigations show that the use of accelerator not only has an effect on the strength development but the fine dust development to be expected can also be considerably reduced. However, according to the results of [322], the effect differs greatly according to the type of accelerator used (Fig. 2-81).

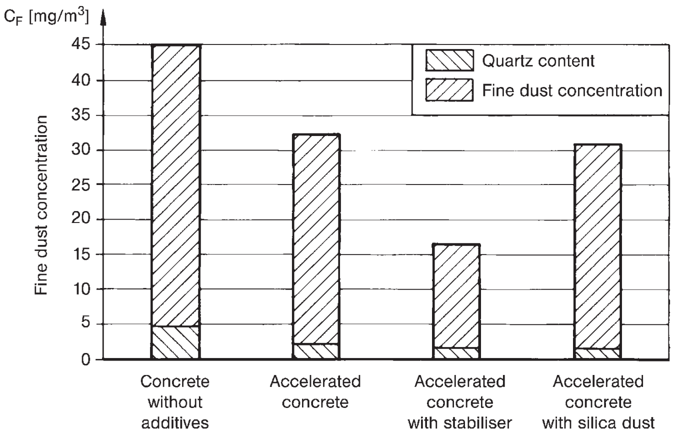


Figure 2-81 Fine dust and quartz contents of accelerated and not accelerated shotcrete with aggregate containing quartz (dry-mix process)

But not only the increase of quartz content plays a role in fine dust development. Occupational health investigations show that the toxicity of accelerated shotcrete dusts is considerably increased [298]. This can surely be explained by the relatively high pH value of over 12 [158].

3. The influence of technological process factors related to the machninery concept

The technological process factors affecting shotcrete production can essentially be categorised into four areas:

- Machine (type).
- Compressed air (energy supply).
- Conveyance pipe (length).
- Type and location of water feed.

Details of these influences on the shotcrete quality, rebound behaviour and dust development are included in the shotcrete handbook [214]. It is assumed that these effects will be optimised before starting construction work through preliminary tests.

4. The influence of technological process factors related to the spraying equipment

The first systematic analysis of shotcrete nozzle guidance was carried out by Guthoff [114]. Information can be found about nozzle guidance techniques in the literature, but no quantitative information is given about shotcrete quality, rebound and dust development. All the publications agree about the optimal angle of the nozzle; they all stipulate the alignment of the nozzle at right angles to the substrate, although the proposed distances from the substrate differ widely.

In order to be able to make meaningful statements about the influence of nozzle guidance, it is necessary to split the nozzle guidance into separate independent parameters in order to be able to investigate the influence of each parameter on the evaluation criteria.

The analysis of nozzle guidance carried out by Guthoff [114] is based on interviews with nozzlemen and the evaluation of videos recorded on sites as well as numerous tests performed on a test rig at the Ruhr-University Bochum. The following parameters were investigated for the dry spraying process:

- *Movement of the nozzle*
  Pattern of movement.
  Movement speed.
  Nozzle distance from substrate.
  Nozzle angle to substrate.
- *Pointing of the nozzle* (Fig. 2-82)
  Shape of pointing movement.
  Size (radius / amplitude) of the pointing movement.
  Speed (frequency) of the pointing movement.

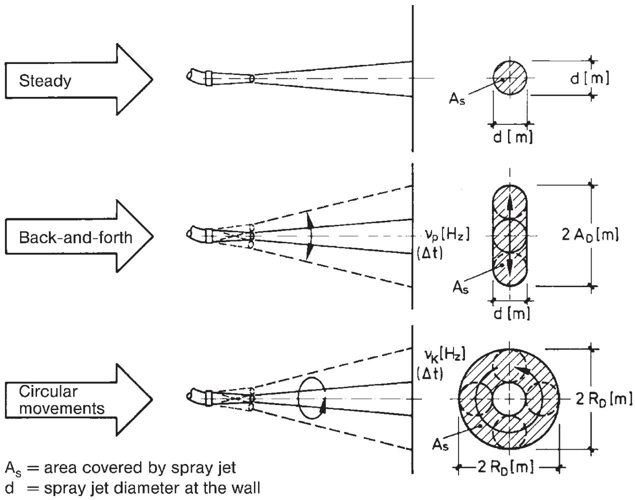


Figure 2-82 Various pointing movements of the nozzle [114]

The results are also applicable to the dry spraying process, as the spraying technique is independent of the spraying process used. Only the absolute values of the evaluation criteria used could differ from the values given here.

In order to obtain a comparison of the nozzle guidance, fixed pointing of the nozzle was selected with automatic guidance of nozzle movement (Fig. 2-83). The evaluation of video recordings and observations on site showed that nozzlemen normally only make slight changes to the pointing of the nozzle over a longer period of time, which means that almost fixed pointing of the nozzle should also be assumed for manual spraying.

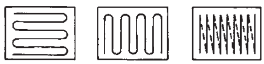
PARAMETER		PARAMETER VARIATION
NOZZLE GUIDANCE MOTION		
Shape	[–]	
Movement speed v_{Dy}	[cm/s]	10 20 25
Layer thickness d_{SB}	[cm]	10 5 4
Nozzle distance a_D	[m]	0.5 1.0 1.5 2.0 2.5
Nozzle angel φ_{Dy}	[°]	60 75 90
NOZZLE DIRECTION MOTION		
Shape	[–]	fixed back-and-forth circular
Frequency v_K	[1/s]	1 3 5
Amplitude/radius R_D	[cm]	5 10 20

Figure 2-83 Test parameter for the influence of nozzle guidance in shotcrete spraying

a) Influences on the shotcrete quality

The quality of the shotcrete is determined from the hardened concrete properties according to DIN 1048, with the cylinder compression strength being the decisive parameter. The spread of compression strengths is also regarded as a measure of the quality of shotcrete [117]. The uniformity of the grain skeleton, particularly the distribution of the coarse grains in the sprayed shotcrete, is an important evaluation parameter for the quality.

Nozzle distance. As is shown in Fig. 2-84, the optimum is a distance of 1.5 m, with smaller distances leading to much greater loss of strength than larger distances. The same behaviour applies for wet spraying whether using the thin or dense flow process.

Nozzle angle. Just small angular deviations lead to a not inconsiderable loss of strength, amounting to almost 15 % for a deviation of 30° (Fig. 2-85). The same behaviour applies for wet spraying whether using the thin or dense flow process.

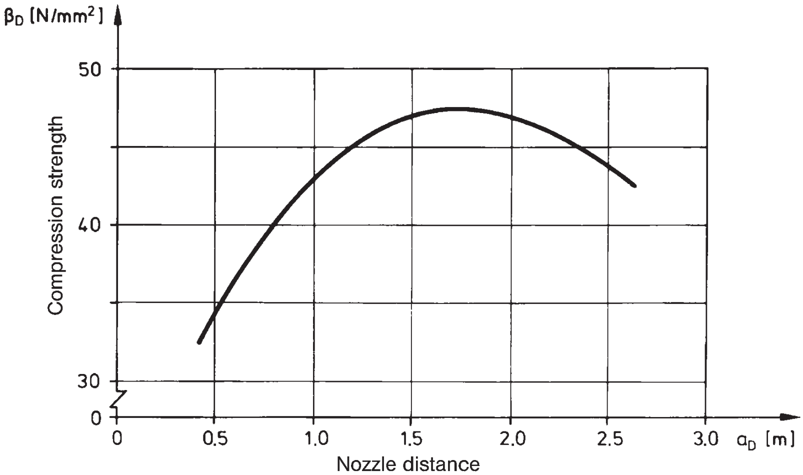


Figure 2-84 Compression strength depending on the nozzle distance (dry-mix process) from [114]

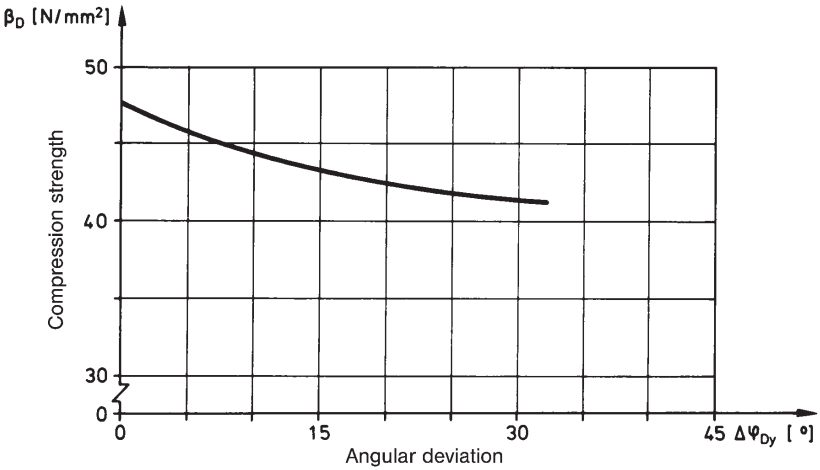


Figure 2-85 Compression strength depending on the nozzle angle deviation (dry-mix process) from [114]

Nozzle pointing pattern. Changing the angle of the nozzle covers a larger area in the same time. This leads to better mixing of the coarse and fine grains, which concentrate in the sprayed jet. Fig. 2-86 shows a clear improvement of the uniformity of the grain skeleton when the nozzle is slewed in comparison with an immobile nozzle. This tendency for better homogeneity is increased when the area covered is increased, as is the case when the jet is continuously rotated. The same behaviour applies for wet-mix spraying whether using the thin or dense flow process.

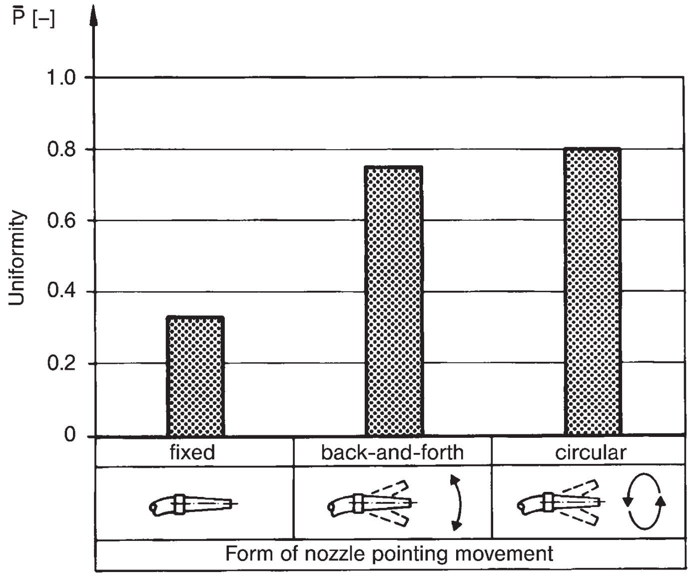


Figure 2-86 Uniformity depending on the pattern of nozzle slewing movement (dry-mix process) from [114]

Similar dependencies apply for the size of the slewing movement and the speed of the slewing movement.

b) Influences on the rebound behaviour

The rebound behaviour is essentially influenced by what happens when the particles impact on the substrate. The impact phase can be idealised as occurring in three stages (Fig. 2-87).

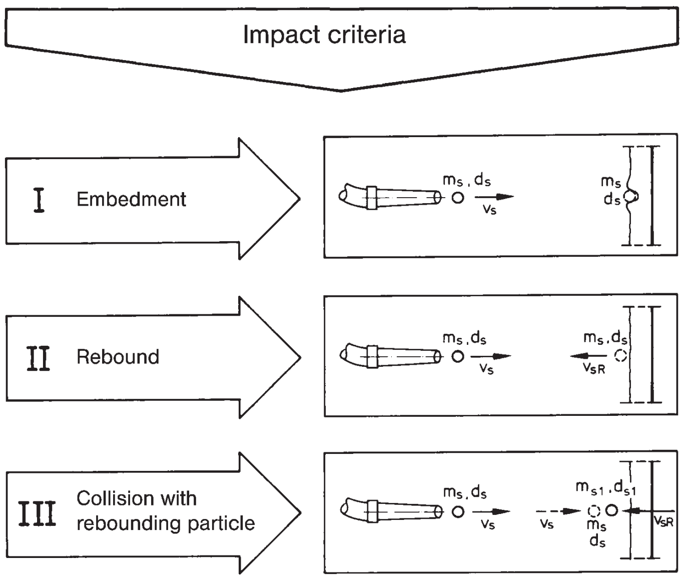


Figure 2-87 Impact stages for a shotcrete particle

Nozzle distance. When the thin flow process is used, whether dry-mix or wet-mix, the optimal distance of the nozzle from the substrate is 1.5 m [63], [114] (Fig. 2-88 and Fig. 2-89).

When the distance is more than 1.5 m, sufficient penetration of all particles into the fresh shotcrete can no longer be assumed, while too low distances destroys the rock mass.

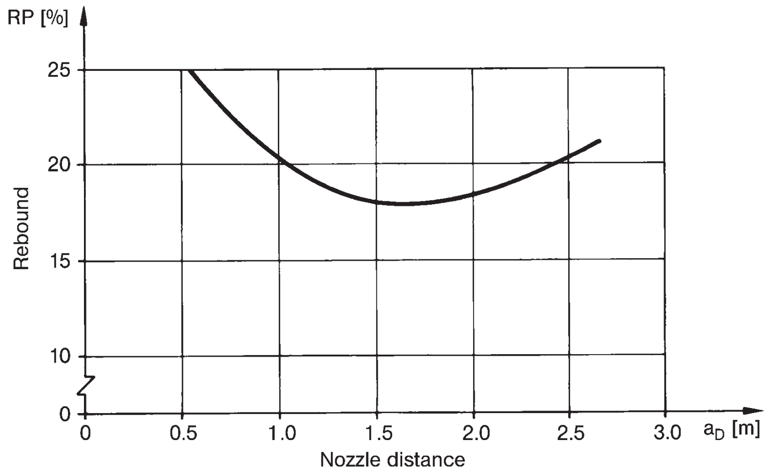


Figure 2-88 Rebound depending on the nozzle distance (dry-mix process) from [114]

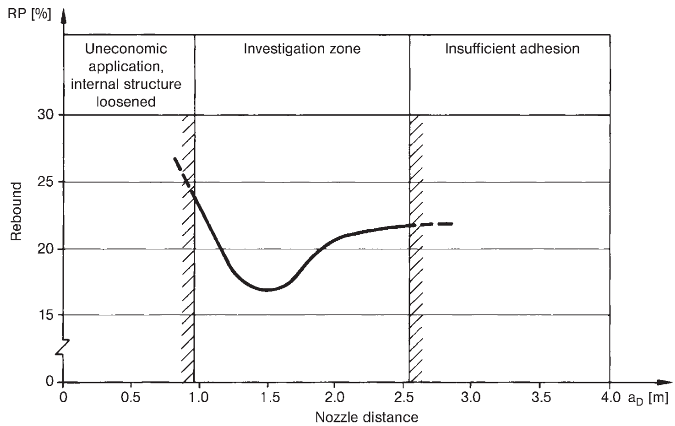


Figure 2-89 Rebound depending on the nozzle distance (wet-mix process with thin flow) from [63]

When the dense flow process is used, the necessary penetration depths of the arriving particles into the fresh shotcrete can no longer be achieved with the same air quantity due to the low material speeds, and this leads to a considerable increase of rebound (Fig. 2-90). A nozzle distance of 0.5 m represents the lowest limit for the process, because the structure of the freshly sprayed concrete is destroyed by the compaction energy at closer distances. The dependencies affecting the strength should be taken into account.

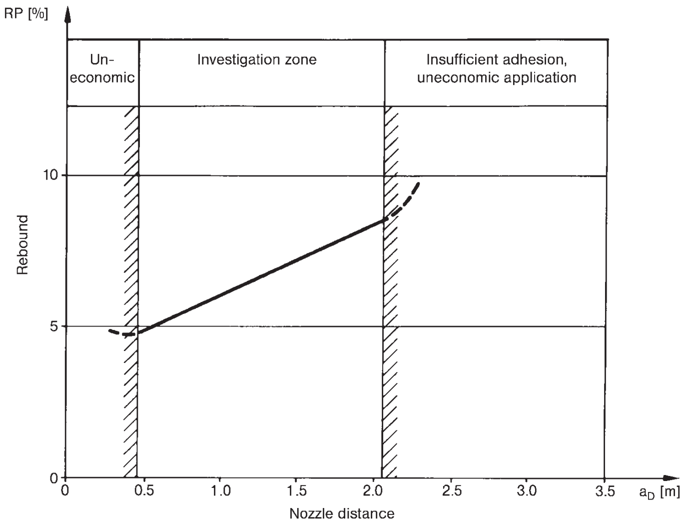


Figure 2-90 Rebound depending on the nozzle distance (wet-mix process with dense flow) from [63]

Nozzle angle. The angle of the nozzle has an extremely strong influence on the rebound behaviour. As can be seen in Fig. 2-91, just slight deviations cause a noticeable increase of up to 30 % at greater deviations. The same behaviour applies for the wet-mix process with thin or dense flow.

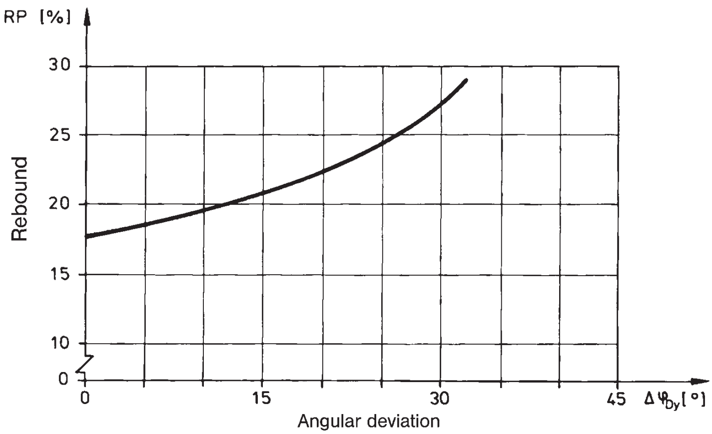


Figure 2-91 Rebound behaviour depending on the nozzle angle deviation (dry-mix process) from [114]

Form of nozzle pointing movement. Different forms of changing the pointing of the nozzle from fixed through to back-and-forth through to circular movement effect a reduction of rebound of 20 % (Fig. 2-92). The much larger area covered by a circular movement reduces the probability of the third stage "collision with rebounding material" (stage III, Fig. 2-87), as the time difference Δt becomes shorter. The same behaviour applies for the wet-mix process with thin or dense flow.

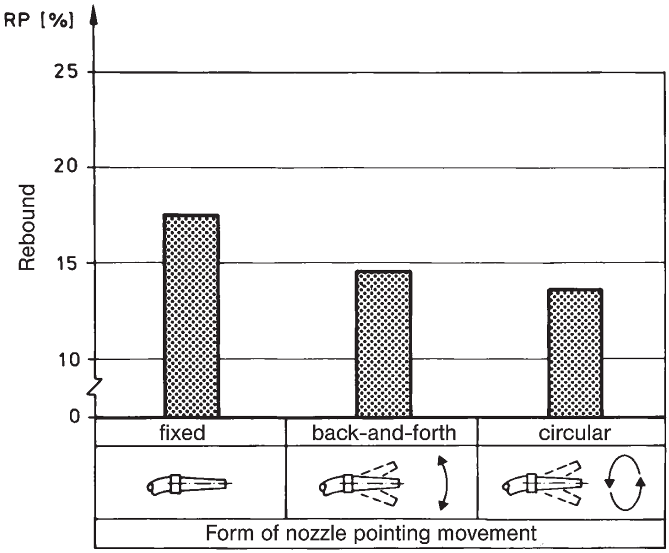


Figure 2-92 Rebound behaviour depending on the form of nozzle pointing movement (dry-mix process) from [114]

c) Influences on dust development

Dust development during shotcrete spraying is essentially determined by the processes that occur between leaving the nozzle and impact on the substrate. These processes have been described by Handke [120] by means of the unloaded and the loaded free jets. The jet emerging from the nozzle creates an induced air flow towards the jet. This flow created a turbulent mixing zone, in which the fine dust particles collide with water droplets and are precipitated. (Fig. 2-93). The size of the turbulent mixing zone depending on the intensity of the induced flow is thus the decisive factor influencing dust development.

The following section deals with the most important influence factors.

Nozzle distance. The sprayed jet emerging from the nozzle enlarges constantly as it travels and forms a cone. Larger nozzle distances thus have the effect of enlarging the surrounding area, which represents the boundary between the sprayed jet and the surrounding air. An enlargement of this boundary surface enlarges the area of the turbulent mixing zone. Increasing nozzle distances thus have the effect of lowering the fine dust concentration through an additional precipitation of fine dust particles. As shown in Fig. 2-94, remarkable reductions of up to 60 % can be achieved, but the loss of kinetic energy resulting from greater nozzle distance has to be taken into account and considered with regard to quality requirements and rebound. But the distance of 1.5 m, which is recommended for both dry-mix spraying and wet-mix with thin flow, also seems an acceptable figure for fine dust concentration. For wet-mix spraying with the dense flow process, the optimal distance is 0.5 m. The same behaviour applies for wet-mix spraying with the thin and dense flow processes.

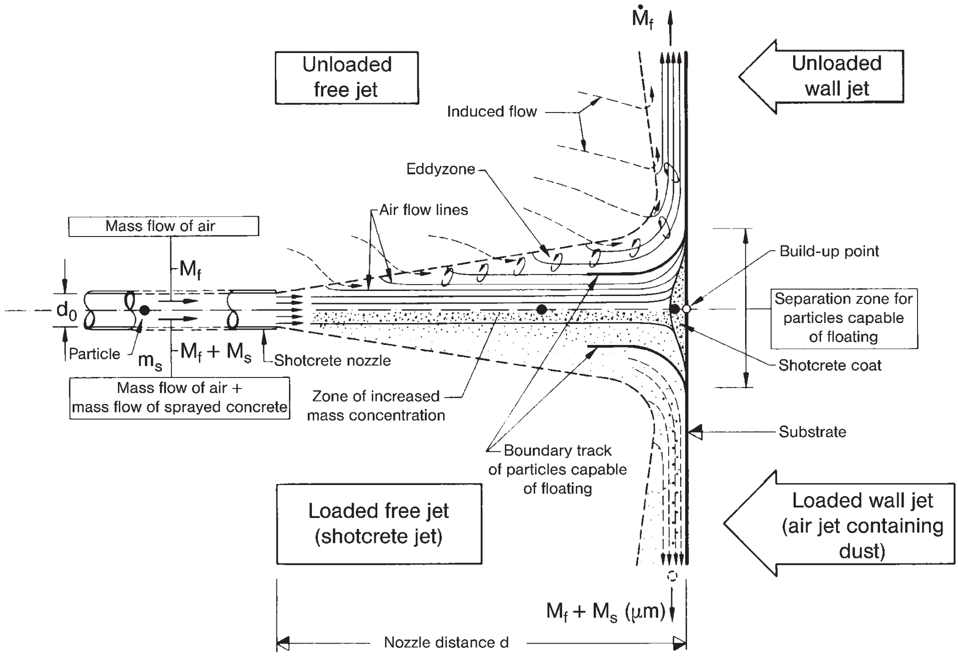


Figure 2-93 Flow pattern of the impact of an unloaded and a loaded free jet on the vertical wall [120]

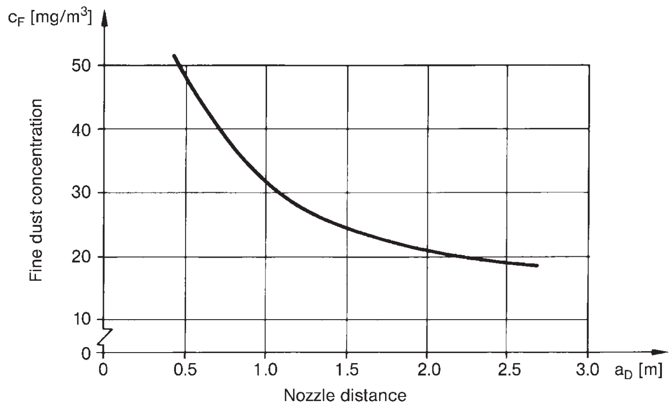


Figure 2-94 Fine dust concentration depending on the nozzle distance (dry-mix process) from [114]

d) Comparison of manual and automatic nozzle guidance

The relationships derived in the last section can be used to make a comparison between conventional manual spraying and automatically controlled nozzle guidance. Manual nozzle guidance requires good conditions. The nozzle should be guided by a qualified and experienced nozzleman and aimed evenly and consistently. The working conditions and the motivation of the workers should be good to very good; the input parameters for Table 2-15 assume these preconditions.

Table 2-15 Comparison of automatic and manual nozzle guidance

		Automatic	Manual
Movement speed	[cm/s]	25	25
Nozzle distance	[m]	1.5	1.3–1.7
Nozzle angle	[°]	90	80
Shape of nozzle motion	[-]	circular	back-and-forth
Amplitude/radius	[cm]	5	7
Frequency	[1/s]	5	1.5
Nozzle guidance coefficient K_D	[m/cm]	29.4	11.6
Radial acceleration a_r	[m/s^2]	11.1	0.6
Compression strength B_D	[N/mm^2]	47.1	42.1
Spread v	[%]	3.3	4.0
Uniformity P	[-]	0.79	0.59
Rebound RP	[%]	13.9	18.0
Fine dust concentration c_F	[mg/m^3]	28.17	32.0

The figures for the compression strength to be expected, their spread and uniformity are given in Table 2-15 as a measure of shotcrete quality. Figures are also given for rebound and fine dust concentration. It can be seen that automation of the nozzle guidance can lead to a considerable improvement of all evaluation criteria for shotcrete. Even a well-trained nozzleman working to the best of his abilities will not be able to achieve comparable values to automatic guidance under the same starting conditions. Fig. 2-95 shows the percentage deviations for manual compared to automatic guidance. The values for automatic guidance are given as 100 % as a reference. The worsening of spread and uniformity are prominent at slightly more than 20 % on one side and rebound at nearly 30 % on the other. The conclusion is that automation can produce homogeneous shotcrete, which also offers economic advantages.

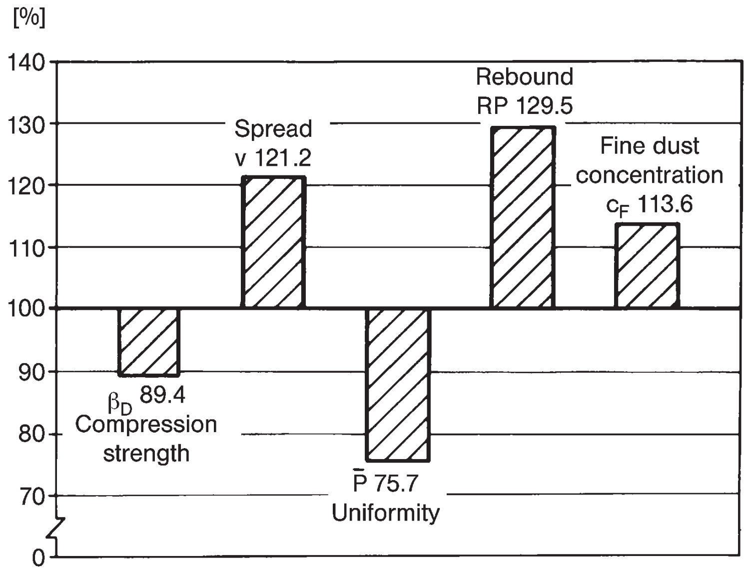


Figure 2-95 Percentage deviations for manual compared to automatic nozzle guidance

2.8.3.5 Quality criteria, material behaviour and calculation methods, quality control

Since shotcrete is applied in very different qualities, it is subject to different requirements, which have to be guaranteed according to the design assumptions.

1. Quality criteria

The many influential factors related to materials and process technology (see section 2.8.3.4) naturally lead to wide spreads. The have an effect on the comparison of shotcrete with normal concrete, which was summarised in an evaluation by [117] (Fig. 2-96).

If shotcrete is to be used as construction concrete according to DIN, the high variation coefficients have to be lowered to the level of normal concrete in order to maintain the material characteristics for the calculation methods. This can only be achieved with the appropriate quality control measures and planning of the entire technology at the design stage with quality tests and quality supervision in the construction stage.

2. Material behaviour of the young shotcrete and calculation methods

While the applicable regulations apply to the final strength of the shotcrete layer, the young shotcrete is subjected to special conditions. This concerns the development of ground pressure with time at the same time as the development of strength and stiffness by the young shotcrete.

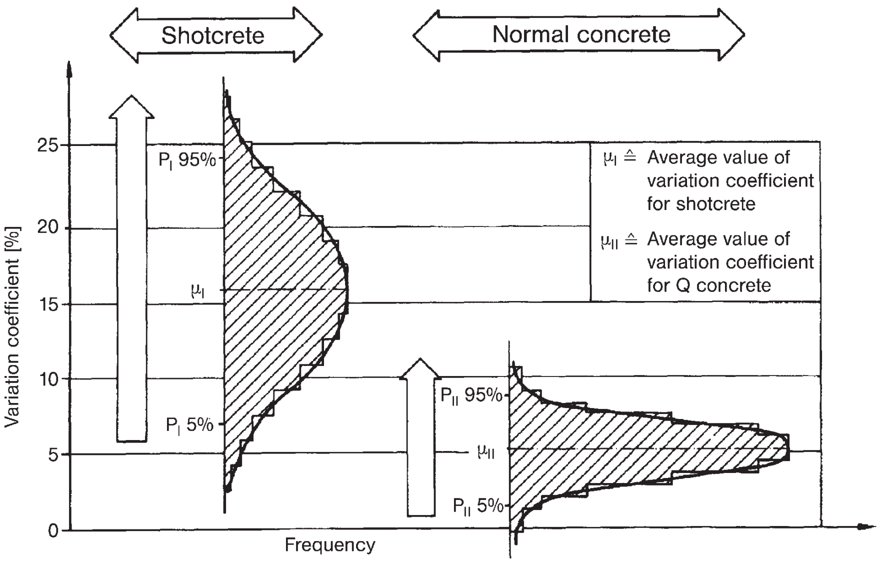


Figure 2-96 Improvement of the spread behaviour for the production of shotcrete under defined conditions [117]

According to M. John [155], the behaviour of the young shotcrete is determined by the following constraints:

- After the shotcrete has been sprayed, it hydrates over time and gains strength and stiffness.
- During this time, the shotcrete is increasingly loaded by the deformation of the cavity.
- The shotcrete has time-dependent behaviour shown as creep and relaxation, which depend on the stress level and age of the shotcrete.
- The stress-strain behaviour of the shotcrete is non-linear; in addition to the results of crack formation, the shotcrete also shows non-linear behaviour under compression.

The practical effects can be qualitatively summarised as follows:

- The shotcrete sheds loading through relaxation. Stress is redistributed into the rock mass.
- The variable stress distribution in the cross-section is reduced by greater relaxation on the more strongly loaded side. The acting moment is thus reduced.
- The behaviour of the shotcrete depends not only on the magnitude of loading at the relevant point in time.
- When the advance is interrupted, the hardening of the shotcrete increases, while the increase of loading is interrupted. The increase of loading after the resumption of the advance acts on a hardened shotcrete layer.
- Under continued loading, there is no relaxation, and no redistribution of stress into the rock mass.

As with all numerical investigations in tunnelling, simplifications are required for the modelling of the young shotcrete. This is permissible as long as the consequences are known and structural stability is not endangered.

There is little reliable knowledge about the development of the Elastic Modulus of young shotcrete. As a summary, a table from M. John is reproduced here [155] giving recommendations for the choice of an Elastic Modulus for young shotcrete (Table 2-16).

Table 2-16 Recommendations for the choice of reduced E moduli (Mpa) for young shotcrete

Shotcrete mix and reinforment	**Tunnelling method and rock mass behavior**			
	Top heading advance with slow redistribution	**Top heading advance with rapid redistribution; side headings**	**Top heading with bench advance**	**Increasing loading**
1-day strength < 10 Mpa sparse reinforcement	1,000–3,000	1,500–3,000	2,500–5,000	9,000–12,000
1-day strength > 10 Mpa moderate reinforcement	3,000–5,000	3,500–6,000	5,000–8,000	10,000–15,000
1-day strength > 10 Mpa heavy reinforcement, steel fibre reinforced	4,000–6,000	4,500–7,000	6,000–9,000	12,000–16,000

The problem here is that lower stiffnesses lead to more economic construction for the calculated verifications of the design states and the lowest possible E moduli are therefore assumed. This can, however, not normally be verified in qualification tests. It is therefore necessary to find the correct figure from experience in order to avoid the risk of manipulated verifications.

3. Quality control

The requirements for the verification of the strength behaviour are laid down in the relevant specification or in regulations like Ril 853 [296] and ZTV-ING [384].

For the early strength, the type of accelerator admixture is decisively important, dependant on its dosage and the addition of further concrete additives. Fig. 2-97 shows these relationships.

For the *verification of the quality of shotcrete,* the standard DIN 18 551 requires a *qualification test* and a *quality test* [68]:

The *qualification test on the ready mix* is intended to determine reference values, if necessary with the addition of a certain water quantity, of consistency, wet concrete bulk density, and 28-day cube compression strength, and additionally for moisture content of a damp ready mix. The individual ingredients of the ready mix are to be monitored and tested according to the relevant standards. For concrete aggregate, admixtures, additives and binder, DIN 1045 Part 2 (2001-07), section 5 [65] applies.

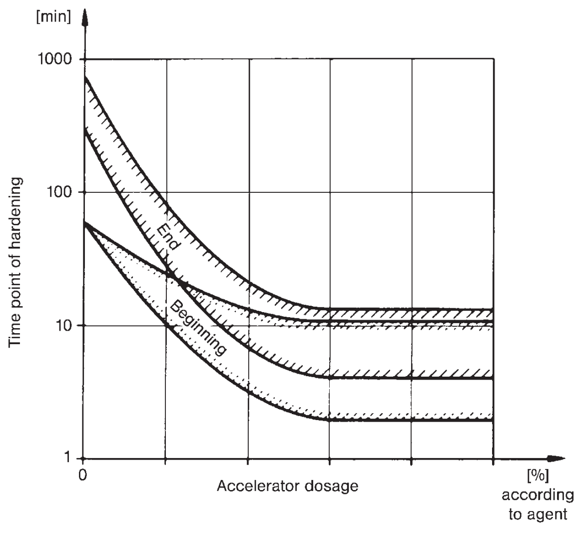


Figure 2-97 Hardening time depending on the dosage of accelerator

For the *qualification testing of shotcrete,* DIN 18551 requires the production of at least 2 slabs 50 cm x 50 cm in the predominant spraying direction for the application with a minimum thickness of 12 cm. These should be produced according to the process used in the relevant application on site. One of the slabs is used for testing the properties of the wet shotcrete. Immediately after spraying, the wet concrete bulk density is determined according to DIN 1048, Part 1 (06.91), section 3.3 [66], the water content m_W, according to DIN 1048 Part 1 (06.91), section 3.42 [66] and the grain fraction < 0.25 mm according to DIN 1048 Part 1 (06.91), section 3.4.1

[66]. When accelerator is added, it may not be possible to determine the water content and the grain fraction < 0.25 mm. The other slab is first to be cured and stored according to DIN 1048 Part 5 (06.91), sections 6.1 and 6.2 [66]. Three testing samples are to be taken from it, preferably 100 mm diameter cores according to DIN 1048 Part 2 [66]. The test samples are to be shortened to a ratio of $h/d = 1$ and stored according to DIN 1048 Part 5 (06.91), section 6 [66] and tested for compression strength after 28 days according to DIN 1048 Part 5 [66].

In addition to the qualification tests required in DIN 18 551 and Ril 853 [296], when accelerator is used, qualification tests are to be performed for the intended concrete without accelerator (zero concrete) and with the intended dose of accelerator. When various accelerators are used, this test is to be undertaken for each accelerator. For more details about the making of the test samples, preparing the samples and the precise procedure for qualification testing, see Ril 853 [296].

The ZTV-ING, Part 5 also requires that the compression strength of shotcrete is to be verified after 6, 12 and 24 hours as well as after 3, 7 and 28 days [384].

In the *quality test of the hardened shotcrete,* the strength is normally to be verified using 100 mm diameter cores 100 mm high. After the samples have been taken according to DIN 1048 Part 5 [66], they are stored and tested after 28 days. The test results are to be evaluated and assessed according to DIN 1048 Part 2 [66]. For the verification of water permeability, cores with 150 mm diameter and approx. 120 mm height are normally taken from the separate slab. Sample taking, storage, test sample preparation and testing are performed according to DIN 1048 Part 2 and Part 5 [66].

According to Ril 853 [296], verification of the development of strength is required in addition to the requirements for the compression strength after 28 days. The testing times for the determination of the development of strength with time are as follows: 3 h, 6 h, 12 h, 24 h, 2 d, 7 d, 28 d. The results of these tests are to be displayed as a graph. The strength cures derived from these graphs serve as a reference for later internal quality control tests.

For *monitoring* (internal and external), DIN EN 206 [80] and the DafStb Guideline for the production and use of dry concrete and dry mortar [55] apply, unless otherwise agreed.

The internal quality control tests required by DIN 18 551 are also to be undertaken after 1 and 7 days according to Ril 853 [296]. The test results gained from cores are to be compared against the target curve produced in the repeated qualification tests. If the results comply up to 24 hours, the early strength development may be taken as in compliance. In case of deviations, additional pullout tests must be performed.

The ZTV-ING, Part 5, requires that the determination of the early strength, in addition to the requirements of DIN 18551, must be carried out at least on one series per 100 m^3 of installed shotcrete, and the strength is to be tested at least every 5 concreting days [384].

Non-destructive testing of concrete in the structure is permissible, although comparable density tests are to be carried out for calibration.

The early strength and the development of the Elastic Modulus are required for the verification of construction states.

The extent and frequency of internal testing are to be taken from DIN 18551.

2.8.3.6 Mechanisation of shotcrete technology

Shotcrete technology has become thoroughly mechanised and automated in recent years. The machinery available has developed during this time from "compressed air sprayers" to highly technical spraying machines. New research results are still needed concerning control and regulation for the maintenance of feed quantities and compressed air components, as research has shown that the quality depends mostly on process-related parameters [214]. At the same time, handling on site has been considerably simplified with the use of manipulators. Even the use of robots would often be practical today, but there are still enough competent nozzlemen in the "traditionally minded" tunnelling industry who can undertake this difficult, dirty and dangerous work, although there is actually scarcely an area of the construction industry where the application of robots would be more suitable. The following section describes the state of development of manipulators and robots.

a) Manipulators

Equipment for the mechanisation of nozzle guidance with remote-controlled manipulators can already demonstrate improved working safety, occupational hygiene conditions and the cost-effectiveness of the construction process, as well as better material quality and stability of the cavity (Table 2-17).

The decisive factor for the constructional arrangement of both the boom equipment and the spraying nozzle is applied kinematics. Due to the particular conditions on a tunnel site (for example collision danger, dusty environment), a simply structured kinematic layout with large reach and small space requirement is ideal.

It is also pertinent that a small number of joints simplifies the control of the device, which helps reduce the time required to learn to operate it. In order to move in three dimensions, at least three degrees of freedom are necessary (which cannot all lie on the same plane). Some kinematic layouts of boom devices with three degrees of freedom are shown in Fig. 2-98.

Table 2-17 Advantages of the use of spraying manipulators

Field	Advantage
– Health and safety	– removal of the nozzleman out of the immediate danger area – no heavy physical work for the nozzleman – increased feeling of safety – omission of scaffolding
– Cost-effectiveness	– reduced rebound – increased output (> 20 m^3/h) – increased utilisation of shotcrete machine – improved advance rate
– Material quality	– optimal nozzle guidance technology
– Stability of the cavity	– early sealing and edge reinforcement – prevention of rock mass looening

While a turning or rotation pair only enables one relative movement between the members through rotation about the pair axis, a sliding or prismatic pair achieves this through translation of the elements to each other.

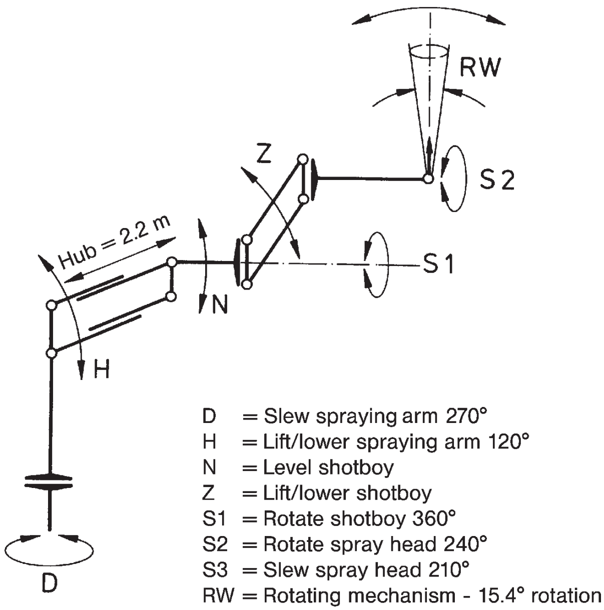


Figure 2-98 Kinematics of a remotely controllable mechanical spraying arm with an independently circling spraying nozzle; Schwing GmbH

Most of the manipulators available on the national and international market today can be assigned with regard to their kinematic systems to the systems listed in Table 2-18. Two examples are shown in Fig. 2-99 and Fig. 2-100.

Table 2-18 Spraying manipulator systems

Base machine	Drive	Boom type	Nozzle
– Tracked vehicle – Wheeled vehicle – Rail vehicle – Hanging monorail	– pneumatic – hydraulic	– slewing an lifting telescopic boom – slewing and lifting boom in combination with a telescopic carriage – main and two auxiliary arms – double-articulated arm construction – sliding column with slewing telescopic boom – hanging scaffold construction	– slewing and tilting – rotary pendulum – rotating

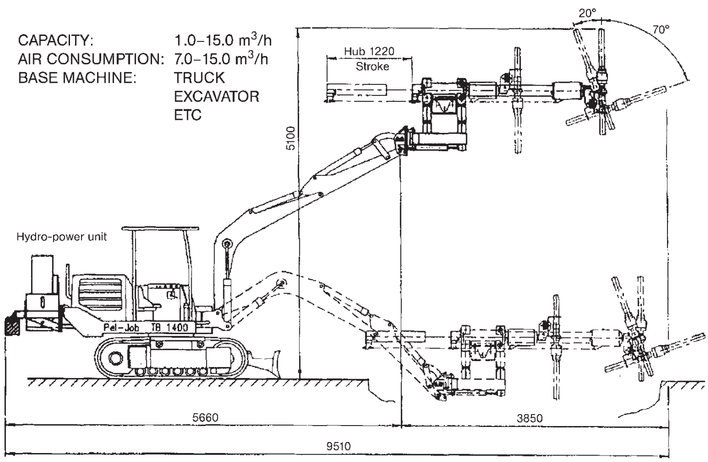


Figure 2-99 Meyco Robojet Modula spraying manipulator arm; Meyco MBT AG

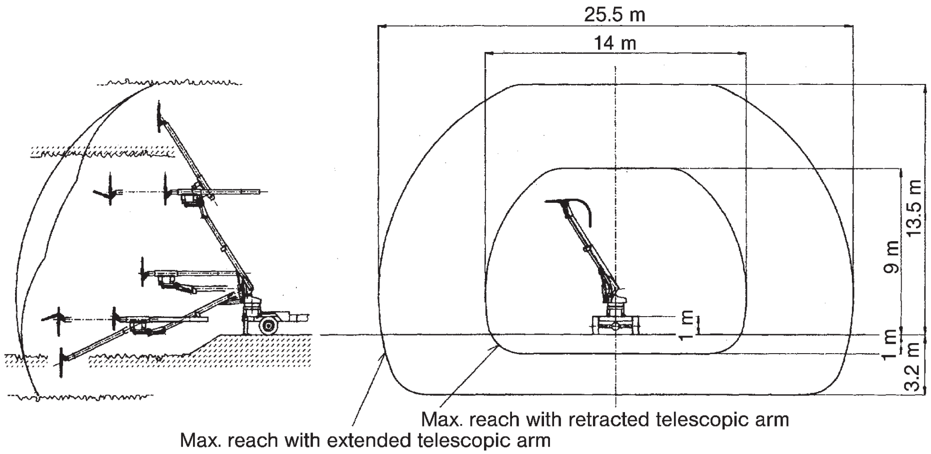


Figure 2-100 Working range of the Meyco Robojet spraying robot arm; Meyco MBT AG

b) Robots

The unattractiveness of the shotcrete workplace on a tunnelling site, which is the result of the extreme exposure of the nozzleman, can only partially be reduced by the use of manipulators, as it is still necessary for the operator to be within visual range during spraying.

Fundamental investigations into shotcrete technology also demonstrate that particularly the nozzle guidance technology has a decisive influence on the material quality including the required uniformity, rebound and fine dust development, and only a shotcrete robot can optimally guide the nozzle [214].

A development in Japan starting in 1988 enabled automatic spraying nozzle guidance according to the previously configured cross-sectional data of the tunnel, with electronic positional fixing of the manipulator with two lasers.

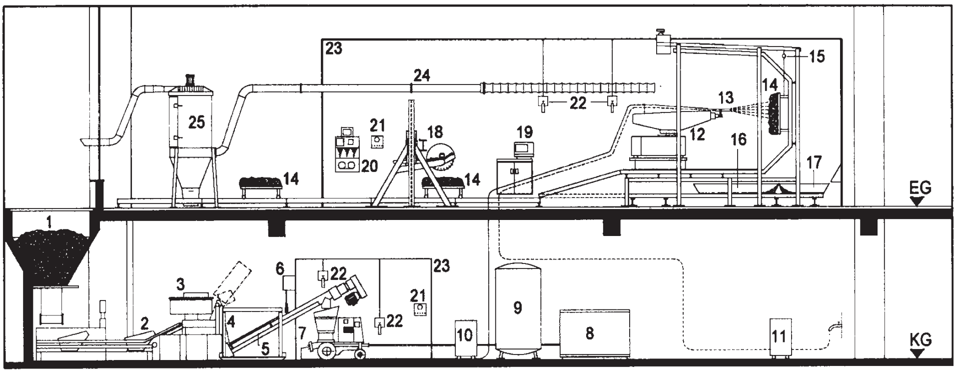
It can, however, be read in reports about site trials of this first robot-type device that considerable problems were experienced with the dimensions, controllability and above all flexibility, which resulted in considerable disruption of the construction schedule. All in all, the inadequate reciprocal adaptability of the three factors machinery, construction process and personnel led to the result that the devices, as with many prototype construction robots in the whole industry, could not gain acceptance for practical use on the construction site and development work was stopped after the prototype trials.

The only spraying robot developed outside Japan, the “SAM” (Shotcrete Automatic Manufacturing), had already been in operation since 1983 at the Ruhr-University Bochum, Chair of Tunnelling and Construction Management as a nozzle guidance system for the investigation of concrete technological and process technical influences on shotcrete production [214]. This device is a standard robot originally intended for stationary industrial production and has been specially converted for shotcrete work at the Ruhr-University Bochum (Fig. 2-101). The device can be used in combination with extra modules for the mechanisation or automation of components like nozzle guidance, nozzle pointing movements, water addition in the dry-mix process, and the dosage of accelerator and mass stream.

The extremely positive effect of the performance of the robot on the material and process technology requirements is demonstrated in Table 2-19, which was produced from the extensive scientific investigations at Bochum.

More detailed information about the possibilities of automation discussed here can be found in [214].

A practical demonstration project has showed that the spraying robot “SAM” in a developed form can be transferred to the construction process (Fig. 2-102). The entire shotcrete equipment including the spraying robot was installed in January 1991 in a tunnel of the Tremonia trial mine in Dortmund [230]. This was the first successful concrete spraying robot application under realistic construction site conditions.



1. Store, aggregate, cement
2. Weighing of aggregate, cement
3. Mixer
4. Stock silo/scales
5. Dosing worm
6. Dosing of additives
7. Shotcrete machine
8. Compressor
9. Compressed air vessel
10. Measurement and regulation (air)
11. Measurement and regulation (water)
12. Spraying robot
13. Spraying nozzle
14. Travelling spraying palette
15. Palette scales
16. Rebound scales
17. Trough for difference quantity
18. Drilling and cutting workplace
19. Spraying robot control unit
20. Data logging
21. Intercom
22. Dust measurement instrument
23. Dustproof housing
24. Ventilation and extraction
25. Extraction and dedusting plant

Figure 2-101 Sprayed concrete test rig with integrated robot SAM [214]

Table 2-19 Overview of solutions in wet and dry processes

		Material technology requirements					Process technology requirements					
		Strength	Early strength	Uniformity	Environmental resistance	Waterproofing	Rebound	Dust development	Flexibility	Output	Time independence	Automation
Dry-mix process	Manipulator	+	o	+	+	+	+	+	+/–	+	o	+
	Robot	++	o	++	++	++	++	++	+/–	++	o	++
Wet-mix process	Manipulator	+	o	+	+	+	+	+	o	+	o	+
	Robot	++	o	++	++	++	++	++	+/–	++	o	++

++ extremely positive effect, + positive effect, +/- not scientifically investigated, 0 no effect, – negative effect

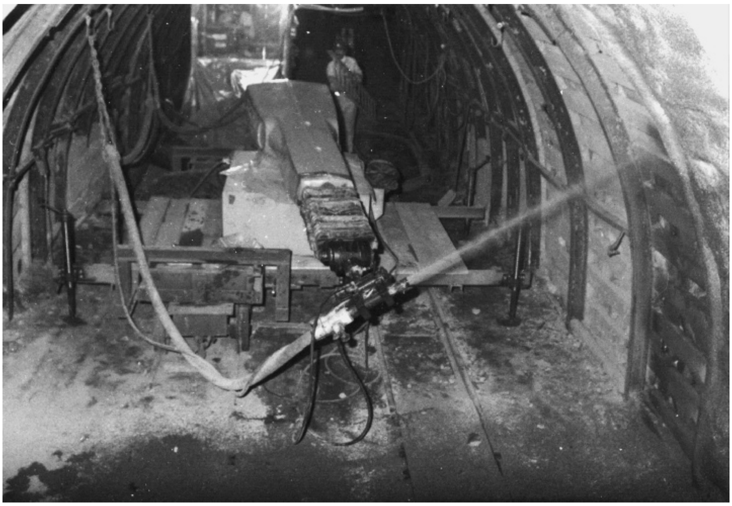


Figure 2-102 Concrete spraying robot in the Tremonia trial mine [230]

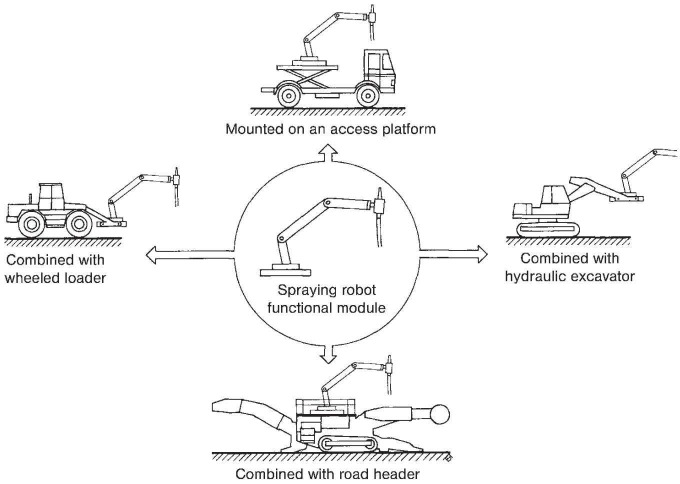


Figure 2-103 Concrete spraying robot functional module as an element in a building block system

The device in its current version (SCOTT, see 2.8.3.6 e)) can be used for the spraying of defined inner linings, for example single-layer shotcrete construction, particularly with the use of steel fibres.

c) Concrete spraying robot functional module

Fig. 2-103 illustrates the "concrete spraying robot functional module" solution, which was designed considering the technological, ergonomic, economic, technical and constructive factors of the construction process [228]. A high degree of adaptability to an automated construction schedule was taken into account.

The system consists of a basic mobile unit, which is manually controlled and positions the concrete spraying robot functional module in the cross-section. Mounted on this is the functional module equipped with automatic control, which carries out the actual nozzle guidance movement from the starting position. Two essential advantages of this design are the simple adaptation of the device to various cross-sections (by exchanging the base unit) and a simplification of the kinematics of the concrete spraying robot functional module and thus of the automation task to be solved.

d) Optimised construction process

Fig. 2-104 shows an example of an optimised construction process with excavation by blasting. The use of a shotcrete robot and the application of the material steel fibre shotcrete and single-layer construction lead to a considerable simplification of the construction schedule. In the excavation phase, each of the machines only has to be positioned once per cycle. The use of steel fibre shotcrete can to some extent make the installation of arches and steel mesh superfluous. For the construction of the permanent lining, which can be installed continuously in an operation independent from the advancing of the tunnel, the same machinery can be used as for the temporary support. Head protection for the miners is also less of a problem since only the shotcrete robot has to work in the unsecured area.

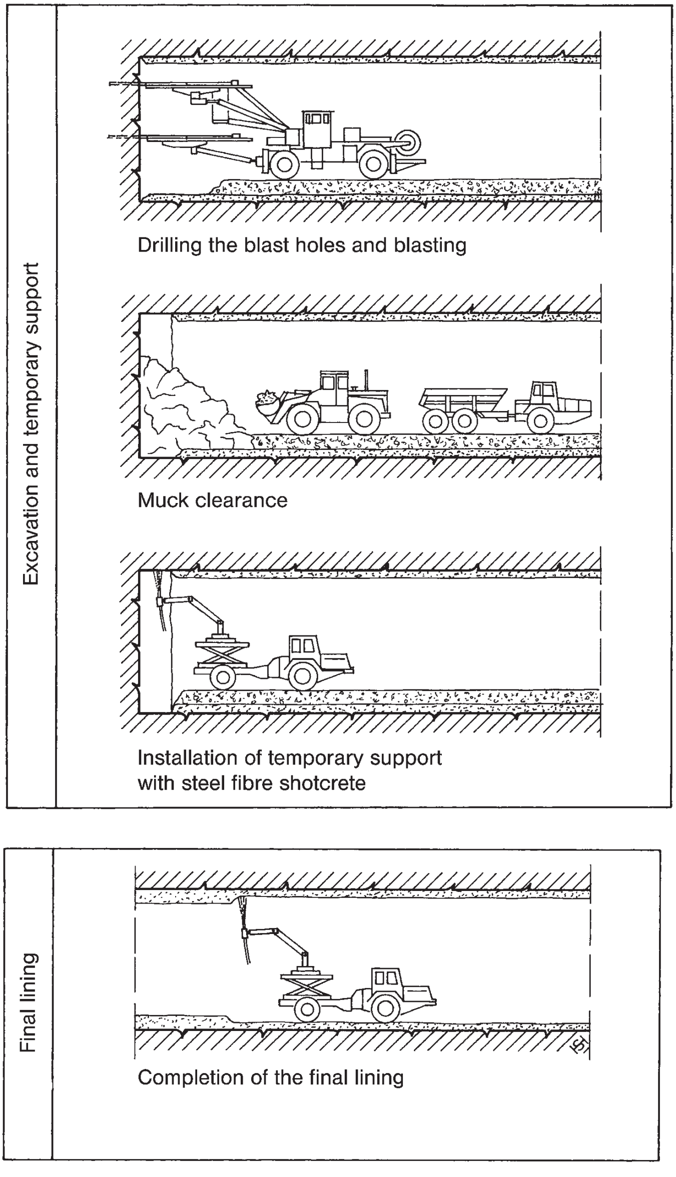


Figure 2-104 Sequence of operations for an optimised construction process with excavation by blasting [228]

In all, this scheme enables continuous operation [215], for which Girmscheid has produced a diagram of the scheduling advantages (Fig. 2-105).

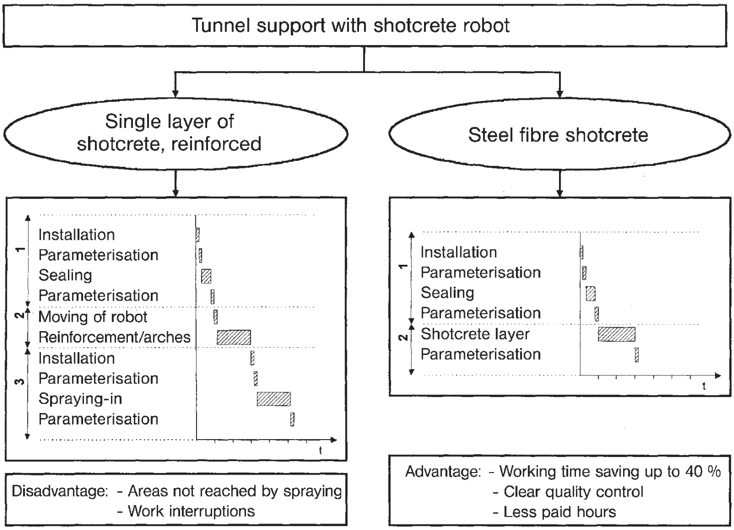


Figure 2-105 Scheduling advantages of tunnel support installed by a shotcrete robot [104]

e) SCOTT sprayed concrete test rig

This sprayed concrete test rig, which has been inexistence for more than 20 years at the Ruhr-University Bochum, enables the production and spraying of shotcrete under defined and constant starting conditions. This is based on instrumentation and process technical conditions, which enable scientific reproducibility of the spraying tests, including parameters of concrete technology and process technology.

The test rig was thoroughly overhauled and upgraded to the latest state of technology in 2010. The heart of the renewed “SCOTT” (**S**prayed **CO**ncrete **T**esting unit for **T**unnelling) test rig is on one hand industrial robots from the company KUKA (Fig. 2-106), which are already familiar from car building, and on the other hand a new concrete pump developed specially for concrete spraying by the company Schwing.

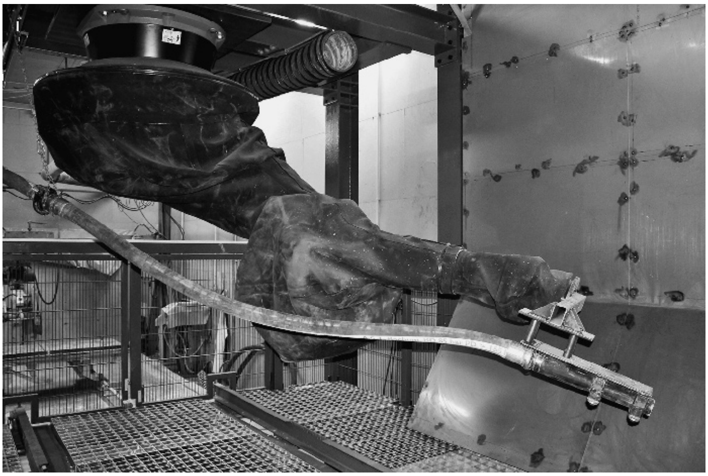
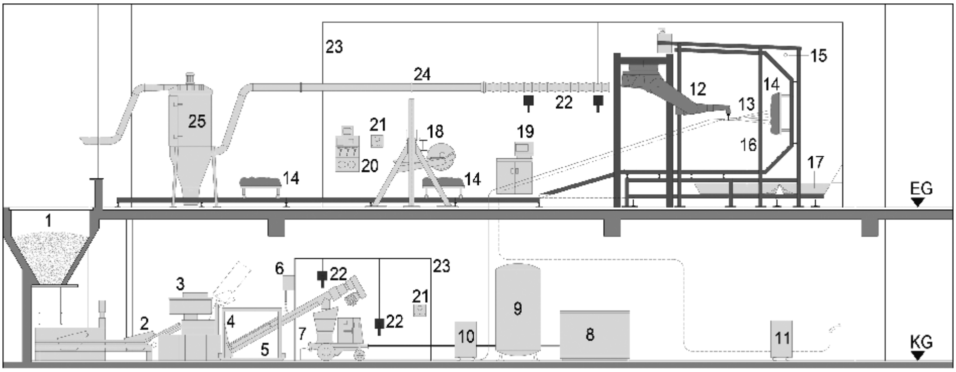


Figure 2-106 Sprayed concrete robot in the starting position

Fig. 2-107 shows a diagram of the overhauled “SCOTT” test rig. The KUKA industrial robot (12) forms the central unit, and can be used to undertake controlled movements of the spraying nozzle (13). The test rig is supplied with concrete mixed exactly to the test specification, which is supplied by the mixing unit (1-5). If required, ready-mixed concrete can be used instead. The mix is than delivered by a Schwing concrete pump (7). The new pump can deliver both a continuous flow of shotcrete and also of accelerator additive, from which a homogeneous test sample can be sprayed onto the intended spraying palette (14). Temporary overdosing of the accelerator is prevented by synchronisation of the quantity of shotcrete pumped and the dose of accelerator.



1. Store, aggregate, cement
2. Dosing of aggregate, cement
3. Mixer
4. Stock silo/scale
5. Dosing worm
6. Dosing of additives
7. Shotcrete machine
8. Compressor
9. Compressor air vessel
10. Measurement and regulation (air)
11. Measurement and regulation (water)
12. Spraying robot
13. Spraying nozzle
14. Travelling spraying palette
15. Palette scale
16. Rebound scale
17. Trough for difference quantity
18. Drilling and cutting workplace
19. Spraying robot control unit
20. Data logging
21. Intercom
22. Dust measurement instrument
23. Dustproof housing
24. Ventilation and extraction
25. Extraction and dedusting plant

Figure 2-107 Schematic diagram of the SCOTT shotcrete test rig

The test rig can also be used to carry out all the usual concrete and process technological investigations into the theme of shotcrete. In addition, further methods of investigation can be simply implemented, for example it is possible to vary parameters like air feed quantity, moisture content of the dry mix, accelerator admixture content, etc. for the production of the concrete mix in order to optimise the shotcrete needed for a particular application. The test rig also offers, in combination with the equipment in the adjacent testing hall, all the conditions required for the normally required quality tests on wet and hardened shotcrete. The programmable control system installed as part of the integration of the new elements and the technical upgrade also enables all relevant data from shotcrete tests to be recorded and evaluated. This includes not only the air quantity but also parameters like rebound quantities, the movements of the robot and flow quantities through the pump.

2.8.3.7 Steel fibre concrete

1. General

Steel fibre concrete has been known since the 1930s. Systematic investigations have been undertaken in the USA, Great Britain and in other western and European countries. The following improvements are offered compared to unreinforced concrete:

- Improved early strength.
- Increased and guaranteed tension strength.
- Increased work capacity.
- Better crack distribution.
- Improved wear properties.
- Improved impact toughness.
- Improvement of the dynamic behaviour.

The production and processing of steel fibre concrete is subject to the following standards and guidelines:

- Bulletin “Steel fibre concrete” of the ‘Deutscher Beton- und Bautechnikverein’ (DBV), issue 10/2001 [58].
- Bulletin “Design basis for steel fibre concrete in tunnelling” of DBV, issue 09/92, revised 1996 [57] (retracted in 2004).

The book “Steel fibre concrete” by the author Maidl [218] can also be mentioned for more detailed technical information.

2. Construction material properties

In addition to the starting mix and the type and quantity of steel fibres, it should always be noted that the properties of steel fibre concrete depend on the process, which is particularly relevant for shotcrete technology for fibre concrete. Testing technology is also important to maintain the characteristic values for the verification of structural stability. The numerous influences do not therefore permit the use of diagrams laid down in advance. The diagrams shown here in Figs. 2-108 to 2-110 serve to display the dependencies. The characteristic values required for structural verification can only be demonstrated in qualification tests on the intended mix. This is certainly a reason why there are particular problems with the applicable standards in Germany. The influence of the testing procedure is not dealt with.

The early strength is dependent on the fibre structure in the concrete. A particular improvement is noticeable in the first few hours after the spraying of steel fibre shotcrete, which is of great significance in tunnelling (Fig. 2-108).

The bending strength of a steel fibre reinforced concrete can be higher then unreinforced concrete by up to 70 %, and the deformation energy required for fracture can be increased by up to 1000 % (Fig. 2-109). Strength and deformation behaviour are, however, essentially dependent on the concrete recipe, fibre content, fibre geometry, shape and dimensions of the test sample and the testing and storage conditions.

The work capacity is one of the most important characteristics for the comparison of steel fibre concrete with unreinforced concrete. Defined as the area below the stress-strain curve, the work capacity gives information about the fracture behaviour and the behaviour after fracture (Fig. 2-110). The comparison shows that steel fibre concrete has more favourable properties than unreinforced concrete.

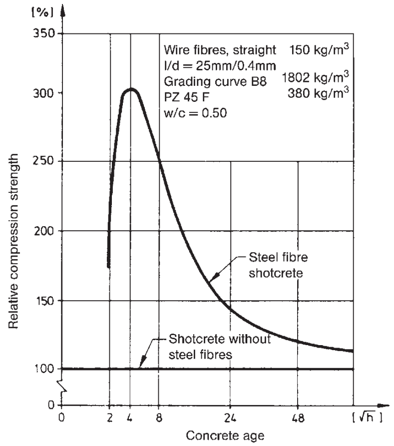


Figure 2-108 Early strength of steel fibre shotcrete relative to shotcrete without fibres, from [317]

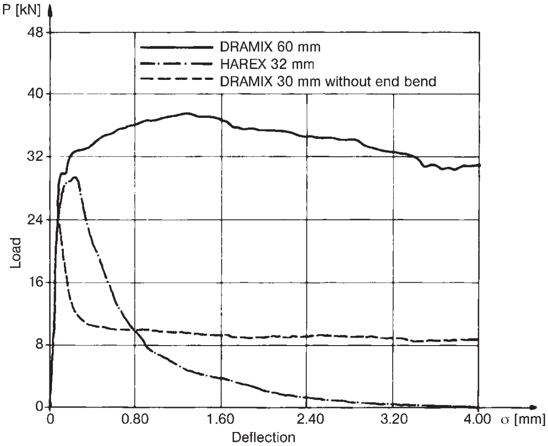


Figure 2-109 Load-deformation behaviour curves for various fibres with the same dosage

Further favourable properties like compression strength, Brazilian tensile strength, Elastic Modulus, impact toughness, wear strength, fire resistance and dynamic loading are not dealt with here; these properties can be of interest for very varied applications [41], [206], [293].

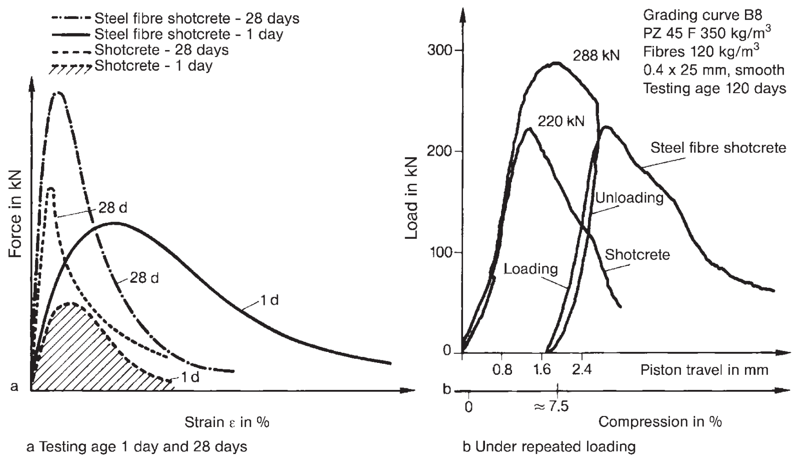


Figure 2-110 Work capacity of steel fibre shotcrete compared to unreinforced shotcrete, tested in uniaxial compression [219]

3. Testing procedure

The tunnel lining is normally loaded by a typical combination of axial compression force and a comparably low bending moment. This loading combination can be resisted by the technically usable bending tension strength of steel fibre concrete.

Table 2-20 shows an overview of the effect of steel fibres on the concrete properties with the associated test methods.

Steel fibre concrete is not a standardised construction material at the moment, although one-off approvals are routine. The bulletin "Design basis for steel fibre concrete in tunnelling" of the Deutscher Beton- und Bautechnik-Verein e. V. (DBV) [58] presents a design procedure based on the bending test with no normal force. Scientific investigations using the Moment-Normal force test rig (M/N test rig, Fig. 2-111) developed at the Chair of Tunnelling and Construction Management have shown that the actual load-bearing behaviour of steel fibre concrete under the loading conditions of a tunnel lining is many times higher (Fig. 2-112). Extensive results are available, describing particularly the influence of fibre type and shape.

Table 2-20 Properties and testing methods for formed steel fibre concrete [226]

Material property	Test method	Effect of steel fibres
Early strength	uniaxial compression test	very great increase
Compression strength	according to DIN 1048	slight increase up to max. 30 %
Tension strength	axial tension test	guaranteed tension strength;
Bending tension test	Brazilian test according to DIN 1048	fibres span the crack -> behaviour after fracture;
Bending tension behaviour	bending tension test according to bulletins of DBV [25], [27], M/n test of the Ruhr University, Bochum	ductile fracture behaviour instead of brittle fracture
Work capacity	bending tension test according bulletins of DBV [17, 18], (compression test according to DIN 1048)	very great increase of up to 1,500 %
Cracking behaviour	bending tension test, M/N test of the Ruhr University, Bochum	more favourable cracking pattern, since more smaller cracks instead of single crack
Behaviour under impact/dynamic loading	no standard test	increase of the energy absorption capacity
Behaviour at high temperatures (e. g. in case of fire)	no standard test	great reduction, or prevention of spalling
Shrinkage:		
free shrinkage	no standard test	little effect
restrained shrinkage	restrained shrinkage test	more favourable cracking pattern
Corrosion behaviour of the steel fibres in:		better than reinforced concrete, since no spalling occurs and the corrosion thus remains limited to the surface
uncracked samples	no standard test	
cracked samples	no standard test	

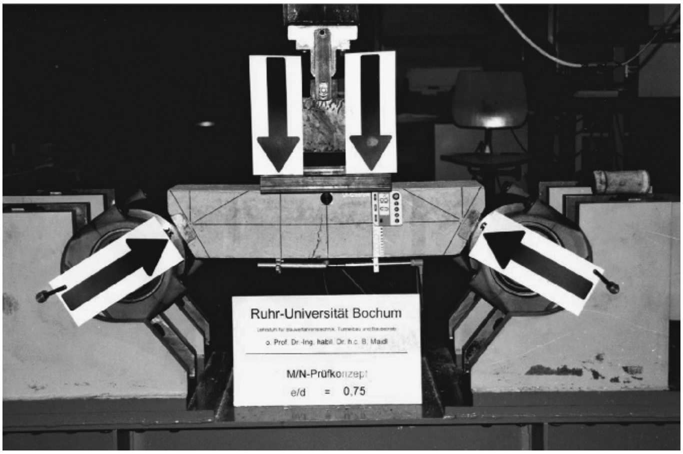


Figure 2-111 Test rig for the investigation of steel fibre concrete under combined loading [218]

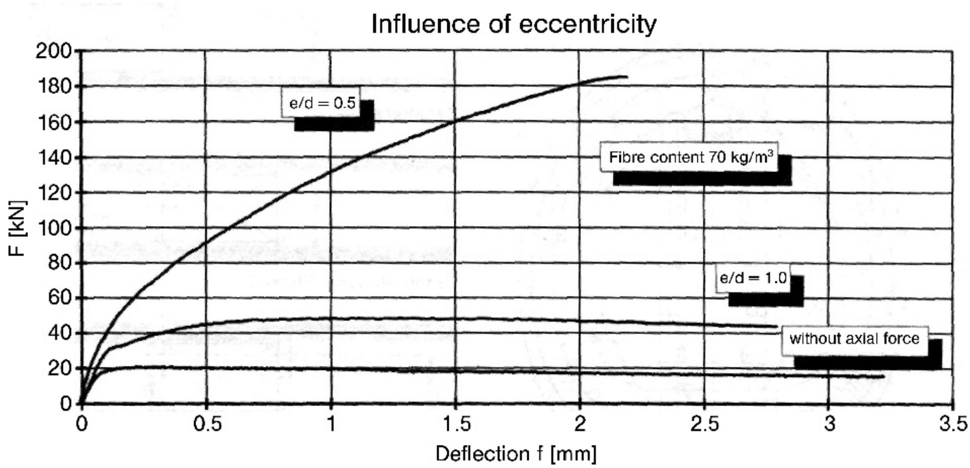


Figure 2-112 Comparison of the load-deformation curves for a bending test with no normal force and a M/N test (*e* = eccentricity of the normal force; *d* = cross-sectional thickness) [97]

Based on these results and the bulletin mentioned above, Feyerabend [97], Dietrich [64] and Ortu [270] developed a design approach, which considers realistic steel fibre concrete properties under the loading combinations of moment and normal force that are predominant in tunnelling and makes it technically usable. See also the structural verifications by Ortu [270].

4. Particular features of the processing of steel fibre shotcrete

In principle, a stage of development has been reached in the process technology for steel fibre shotcrete that enables the processing of the material with conventional machinery with no problems, although increased wear to the spraying machine has to be expected due to the greater abrasiveness of the steel. For the dry-mix process, special metering devices for the steel fibres are no longer needed as long as the fibres used do not tend to form steel fibre lumps and the dry mix is delivered to the site as a ready mix including steel fibres. Such a procedure is to be recommended due to the more uniform quality of the shotcrete to be expected. The production of the ready mix is however also possible on site, given the appropriate planning and supervision.

The diameter of the pumping hose and the fibre length should be appropriate for each other, with the fibre length not exceeding 2/3 of the length of the clear opening width of the hose. This should avoid stoppages. In the planning of the application of steel fibre shotcrete, it should also be considered that steel fibres tend to lead to higher rebound quantities. Qualification tests should therefore be carried out to determine what steel fibre content in the ready mix leads to the required steel fibre content in samples taken from the wall. The loss of steel fibres between the ready mix and a wall sample depends strongly on the type of fibre and can be between 10 and 30%. With appropriate application, it has been possible to demonstrate improved quality and cost-effectiveness for steel fibre concrete in completed structures [218].

5. Cost-effectiveness

Steel fibre shotcrete can alter or improve classic and new tunnelling processes [40], [64], [205].

When used with shotcrete, work activities can be saved (Fig. 2-113). Without it, four work activities were required, but the application of steel fibre shotcrete requires only one. It may also be possible to install less rock bolts due to the support resistance being reached earlier, although it should be noted that steel arches can also be significant for the sense of security of the miners and as an aid to surveying.

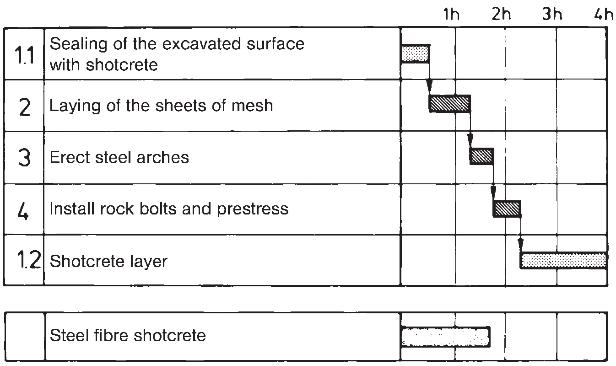


Figure 2-113 Comparison of the number of work activities with the usual shotcrete process (top) and with the use of steel fibre shotcrete

One example of the use of steel fibre shotcrete is the Crapteig Tunnel in Switzerland. During the construction phase, a change was made from a support thickness of 13 cm with mesh to 10 cm of steel fibre shotcrete.

The results were summarised by Bodamer [37] as follows:

- The material costs of the steel fibre shotcrete are only 75 % of the original costs due to the omission of mesh reinforcement and the reduction of the shotcrete layer from 13 to 10 cm (Fig. 2-114 top).
- The susceptible Bündner schist with its safety and deformation problems originally required 4.5 h for each 3 m round for two layers of shotcrete including the laborious

mesh installation. After the changeover to steel fibre shotcrete, the time required for shotcrete application was reduced to 1.5 h, and the time for one round to 9 h. After the changeover, the tunnel advanced with a more favourable rhythm of rounds and shifts (Fig. 2-114 bottom).

– Due to the quicker excavation support, the daily advance rate increased from 4.5 to 6 m, or more than 30 % [37].

This result shows that even if change is slow, the breakthrough of steel fibre shotcrete for temporary support is on its way.

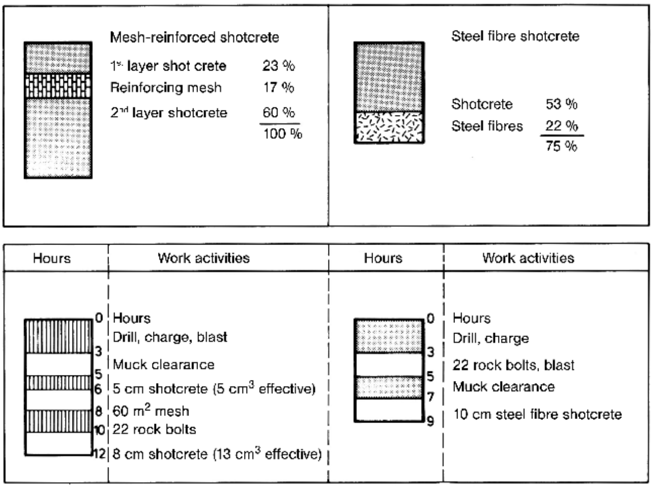


Figure 2-114 Comparison of the material costs (top) and the time required for the excavation support in the Crapteig Tunnel [37]

With regard to properties like the improved early strength, the greater work capacity and the more favourable behaviour after an initial fracture, rock mass behaviour around a tunnel according to Fig. 2-115 should be assumed. Modern tunnelling explains the load-bearing action as a composite of material, i. e. shotcrete, and rock mass, with the largest share being undertaken by the rock mass. Starting with the primary stress state, a secondary stress state will settle as an equilibrium after the excavation of the cavity. Due to the strength being exceeded and the softening of the perimeter of the cavity, the loading is redistributed more and more into the ground mass with its better load-bearing capacity. Rock mass softening of this type is not favourable because it worsens the suitability of the zone to hold rock bolts, along with other problems. If however the perimeter reinforcement is improved by the provision of earlier and thus also higher support resistance, the zone of softening is reduced. The rock mass can be exploited better, even in the near vicinity of the cavity, as a composite with the support.

Increased early strength also has a favourable effect on settlement behaviour. It is known that settlement already occurs before the face is passed through. Thereafter, there a steep increase of settlement is normally observed until the temporary support has been installed, and further movement of the rock mass is known to occur until an equilibrium is established. Most of the movement occurs from the time of opening the face until the support becomes active. If this time period is now shortened through the saving of work activities or by exploiting the early strength behaviour and work capacity of steel fibre shotcrete, the amount of movement can be reduced.

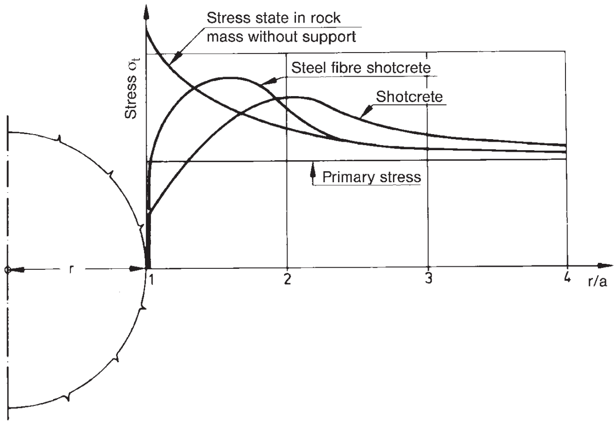


Figure 2-115 Stress states in the rock mass without support, with shotcrete support and with early support with steel fibre shotcrete

A new construction process could then follow the sequence shown in Fig. 2-116: a perimeter reinforcement of steel fibre shotcrete – installed as early as possible – leads to increased stand-up time. Rock bolts can or must be installed according to the rock mass properties. After a minimum time that is operationally favourable and naturally also depends on the rock mass behaviour, the actual structurally load-bearing inner lining is installed. This process could, depending on the waterproofing requirements, lead to a single-pass tunnelling process. The separation into temporary support, which normally has to support the entire loading, and final lining, which is also designed to resist the entire loading, could be dispensed with. This is a demand that has long sought fulfilment.

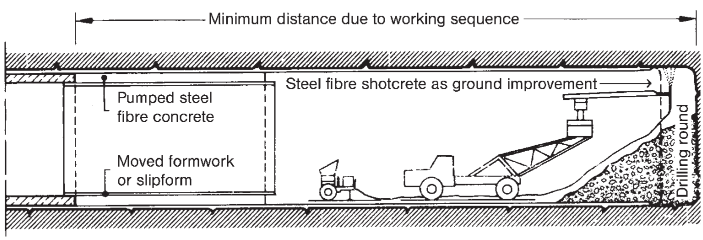


Figure 2-116 Tunnelling procedure with steel fibre shotcrete

2.8.3.8 Working safety

The exposure of the workers performing shotcrete works have led to basic developments, which have not yet been completed today. The sprayed material around the machine and at the nozzle can be damaging to health due to the considerable dust development. The guidance of the nozzle is very heavy work. It has been and still is urgently necessary to constantly improve the spraying process, materials and machines to meet current requirements. Development was driven forward under the research project into workplace improvement in tunnelling through the mechanisation of the dry-mix spraying process, which was supported from 1980 to 1990 by the then Federal Ministry of Research and Technology (BMFT) in the action programme "Humanisation of working life", but since then there has been little activity in publicly sponsored projects.

The German body responsible for accident insurance in civil engineering, BG Bau, have now produced accident prevention regulations for the setting up and operation of shotcrete equipment, which should be complied with [304], [367], [381]. The most important regulations concerning the use of shotcrete in tunnelling are summarised in the following document:

Accident prevention regulations "Construction works" (BGV C 22), version September 2000 [363]:

– Paragraph II. General provisions.
– Paragraph VII. Additional provisions for construction works below ground.

In addition to comprehensive instruction of the personnel about the possible risks in the performance of shotcrete work, suitable protective clothing and face protection should be available. An additional handicap connected with spraying work, particularly using the dry-mix process, is water mist and dust in the breathing air. The safety of concrete admixtures demands particular attention.

1. Dust reduction and dust protection

In mining, the conveyance conditions for the dry mix at the machine are much more difficult than in tunnelling. Oven-dried material or ready mix with a very low moisture content are therefore often used. This leads to very severe dust nuisance at the machine. Apart from the possibility of completely encapsulating the machine, extraction systems could also be used. When the wet-mix process is used, such problems occur less often and are normally confined to the machine.

Around the nozzle, there are several methods of protection. The wearing of dust masks is required. Dust protection helmets with a fresh air supply system have remained in the development phase. One problem that is particularly difficult is dirt on the viewing surface to protect the eyes; solutions with windscreen wipers have not proved successful. At the moment, tear-off foils are normally used, which can be removed when dirty. Experience with such helmets shows that they are worn permanently by some nozzlemen, but they are not sufficiently robust for construction site conditions and with a price of 500 EUR each are too expensive.

The accident prevention regulations already require a limitation of dust concentration of 6 to 8 mg/m³ according to country. Particularly the dust particles, which can enter the lungs, should be kept this low. The chemical industry is constantly working on dust binders.

The dangers of shotcrete work have been summarised by Egger [86] as shown in Table 2-21.

Table 2-21 The dangers of shotcrete work [86]

Area	Danger or exposure from	Cause Origin	Degree of danger Duration of danger Affected area
Machine	Mineral and cement dust	Filling of the feed hopper Outlet of the rotor chamber Cleaning the machine with compressed air	slight danger, continuous in a radius of 10 m
	Dust from corrosive chemicals	Handling of the accelerator	slight danger, continuous in a radius of 2 m
	Noise	Compressed air blowing out of the rotor chambers at the outlet	in closed spaces, medium danger in aradius of 10 m
Hose	Loose and uncontrolled hose ends and connections	Blowing out the pipes without securing them	medium to severe danger, seldom
	Stoppages blowing out	Blowing out stoppages	medium to severe danger, seldom
	Bursting of hose or hose connections	Opening the connection under pressure	medium danger, seldom
Spray nozzle	Mineral and cement dust	Mineral and cement dust finely distributed in the emerging transport air	medium danger, continuous in a radius of 10 m
	Dust and corrosive chemicals	Dust from powder-from accelerator finely distributed in the emerging transport air	slight to medium danger, continuous in a radius of 10 m
	Noise	Air expanding at the nozzle	medium danger, continuous in a radius of 10 m
	Rebound	Rebounding material	slight danger, continuous in a radius of 10 m
	Falling stones	Various	serious danger
	Falling from scaffold or from the bucket	Unsafe location	medium to severe danger

Regarding the dust problem, it has been discovered that the largest part of shotcrete dust has a quartz content of more than 1 %, and thus has to be categorised as silicogenous dust [108]. Whether regular exposure below the permissible value for shotcrete work in the open air on construction sites [108] leads to impairment of lung function cannot be clearly ruled out at the moment. The accelerator dust, which is produced at the feed hopper and at the nozzle in the dry-mix process, but only at the nozzle when liquid accelerator is used, is particularly problematic.

Regarding the noise problem in shotcrete working, noise measurements at the test rig of the Ruhr-University Bochum have determined that the current threshold of the accident prevention regulations (UVV) Noise (BGV B 3, Issue July 1999) [364] of 90 dB(A) is exceeded at the machine and at the nozzle. In the tests, a noise level of about 92 dB(A) was measured at the nozzle and about 110 dB(A) at the spraying machine, almost independent of the type of machine and nozzle (Fig. 2-117).

The measured noise levels mostly result from the expanding air at the machine and at the nozzle as well as general mechanical noise and the impact of the material on the substrate.

In particular, the high noise level at the spraying machine can lead to hearing damage, although shotcrete spraying machines are normally only operated periodically (Fig. 2-118).

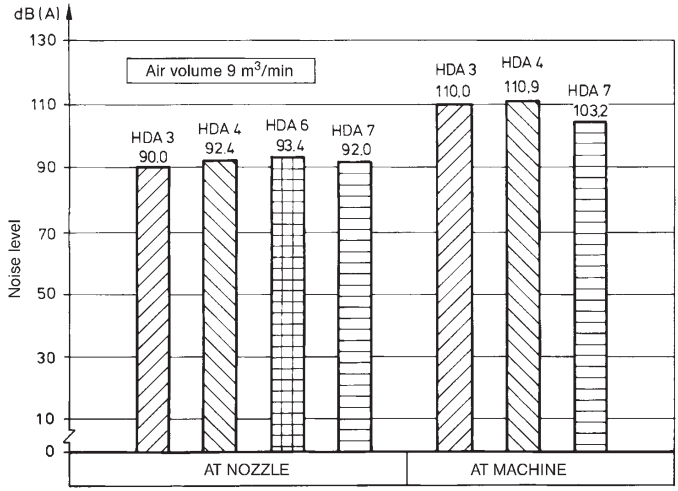


Figure 2-117 Noise level in dB(A) at the nozzle and at the machine (dry-mix process) [232]

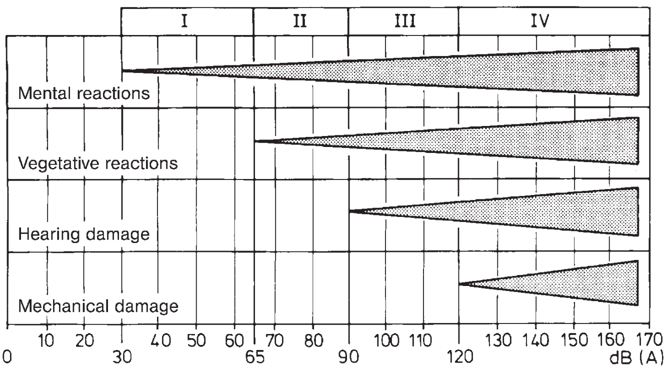


Figure 2-118 Noise levels [181]

In summary, it can be demonstrated that the danger to health through dust, noise and vibration in underground construction is particularly severe. Due to the additional emissions resulting from the excavation of the rock mass and the conveyance of the spoil in the tunnel, and the unfavourable spatial and climatic conditions, working conditions are in need of improvement in all areas of tunnelling, which means drilling, blasting, support work (shotcrete) and mucking. In addition, the particular sensitivity of hearing while under the effect of carbon monoxide should not be neglected [382].

2. Additional conditions for shotcrete work under compressed air

In addition to the positive effects for the environment, any work under compressed air means a health risk for those working under compressed air. This risk can be minimised by taking precautions and systematic checks. The danger potential includes the following risks:

– Dust exposure.
– Decompression sickness affecting workers.
– Increased fire risk.
– Danger of blowouts.

When working under compressed air in tunnelling, the "Regulations for working under compressed air" (DruckluftV) must be complied with [359]. These regulations give detailed information about locking times and medical care. One essential requirement in the regulations is the provision of reserve compressors to maintain the necessary overpressure in the working space when the primary compressor fails.

The reserve compressor must be operated with a different type of energy from the primary compressor and start automatically when the primary compressor fails. When more than two compressors are operating, their capacity is to be selected so that any 2/3 of the compressors can provide the required air quantity. It is worth mentioning with regard to shotcrete work under compressed air that the reduced difference between the pumping pressure of the spraying machine and the surrounding air in the working area reduces the air quantity supplied and thus the dust production.

In order to prevent stoppages, the process technology demands that the air quantity should be set high enough to propel the material. If therefore the same air quantity is assumed under normal atmospheric conditions and under compressed air, then the same fine dust concentration has to be assumed in both cases. The restricted lung function and increased oxygen requirement of workers working under compressed air should, however, be considered in this case, so the health risk increases at the same fine dust concentration. Fig. 2-119 shows the fine dust concentration for dry-mix spraying under compressed air related to the ambient air pressure.

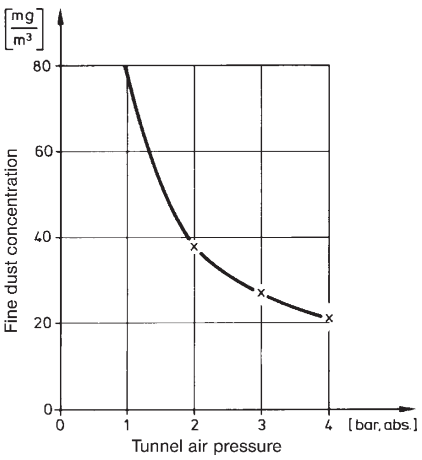


Figure 2-119 Fine dust concentration under compressed air related to the ambient air pressure in the tunnel [322]

Combustible materials ignite more quickly under compressed air due to the higher oxygen concentration, so an increased fire risk is to be expected under the positive air pressure used in tunnelling compared to atmospheric pressure. Fire protection measures therefore demand particular attention on every site where compressed air is used.

2.8.4 Cast concrete

2.8.4.1 Formwork

Cast concrete, also referred to as in-situ concrete, is normally placed into formwork and used as a final lining. Practical considerations limit the minimum thickness to 20 to 30 cm. Installation immediately after the tunnel advance without temporary support has also been satisfactorily solved as a successful process with steel fibre concrete. The use of full-surface slipforming is also under development and will likely increase in the future.

The formwork must be designed for the restricted space and allow construction site traffic to travel through. It must resist the pressure of the wet concrete even if it is not placed symmetrically and without deflecting. The stopend should be designed as easily adaptable and economical. Freestanding constructions are the most suitable. The use of timber formwork with timber support work, which cannot be moved in one piece, is now restricted to short and complicated sections, with reusable formwork units now always being used for longer sections. These normally consist of a wheeled or otherwise movable scaffold with a metal formwork surface. There are even systems suitable for constantly altering cross-sections. Formwork units can be folded together or retracted in various ways so that they can be moved without difficulty through completed sections and assembled formwork units (Fig. 2-120).

Figure 2-120 Lining formwork unit in use for the construction of the new DB line Hannover - Würzburg (Krieberg Tunnel)

The concrete, whether reinforced with bars or steel fibres, is poured in blocks (with block lengths of 8, 10, to maximum 12.5 m) using mobile formwork units. For smaller cross-sections, a full-round formwork unit is used, which means that the invert and the vault can be placed without a working joint. For larger cross-sections, the floating of a full-round formwork unit would be too great, so the invert is concreted in advance and the vault is placed subsequently using a vault formwork unit (Fig. 2-121 and Fig. 2-122).

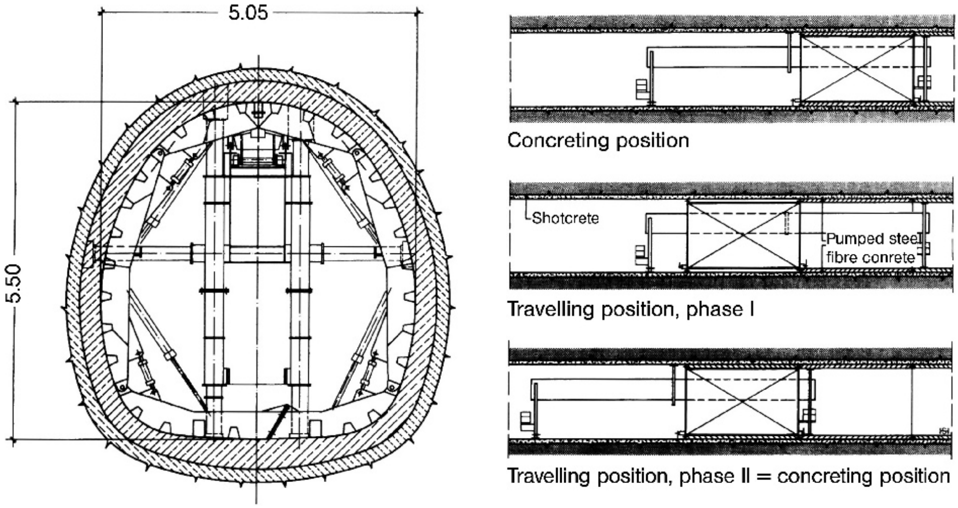


Figure 2-121 Full-round mobile formwork unit for the steel fibre concrete inner lining of the Stadtbahn Dortmund, contract K6a [218]

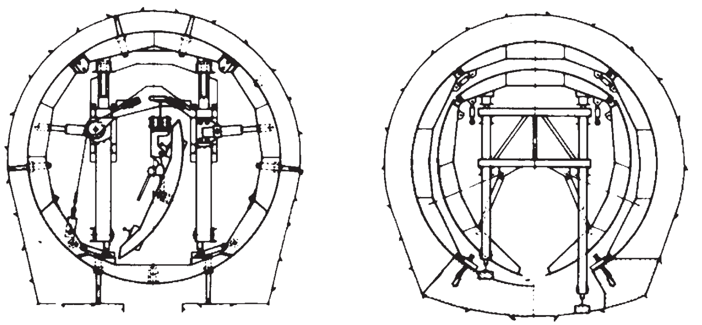


Figure 2-122 Folding formwork unit for the concreting of the full cross-section in one pour (left) and with the invert partially or completely concreted in advance (right)

For large cross-sections, Fig. 2-123 shows an application example.

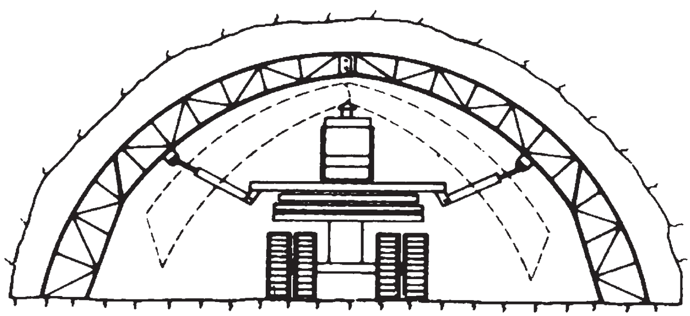


Figure 2-123 Formwork for the top heading of a cavern with fully adjustable support construction, built on a truck, from Icoma

The construction of the formwork carriage should be able to resist loading from concreting and vibration without impermissible deformation of the profile.

One detail, which is being developed constantly, is the stopend. The differences between the theoretical design profile and the actual profile including overbreak in drilling and blasting lead to deviations, which have to be compensated by the stopend. Fig. 2-124 shows an example from the company Bernold, which has folding elements designed for minimum formwork thickness. The extra thickness is formed with boards inserted into a folding frame, which are fixed with timber wedges. When a waterstop is required in the middle, the folding elements can be split. Fig. 2-125 (live picture from Irlahüll Tunnel) shows a conventional construction with wedged formwork panels.

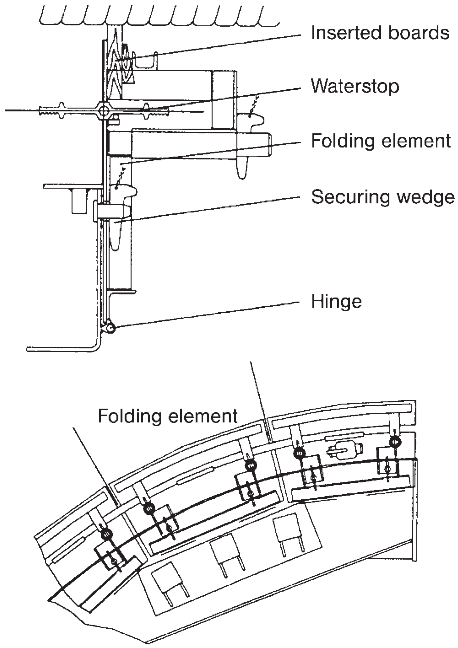


Figure 2-124 Stopend [28]

Figure 2-125 Conventional solution for a stopend with wedged formwork panels, Irlahüll Tunnel, new line Nuremberg – Ingolstadt

The stopend is simpler to carry out with segment temporary support, a form of construction common in Switzerland. In this case there are not such great deviations through overbreak, and the small resulting gap can be closed by inflating robust dilation hoses.

A clean connection to the previously concreted block can be produced with a connection plate (Fig. 2-126), which is also supported by timber wedges.

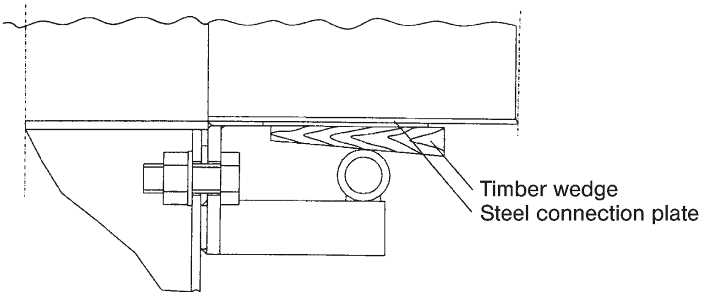


Figure 2-126
Tail plate [28]

2.8.4.2 Concreting

The wet concrete is delivered to the construction site, fed into the hopper of a concrete pump and pumped into the formwork unit through a concrete distributor pipe. The concrete distributor pipe enables uniform distribution of the wet concrete in the formwork unit.

For larger cross-sections, the concrete is often pumped behind the formwork through windows and compacted with internal vibrators if the wall thickness is sufficient (Fig. 2-127). In the crown, or when the wall is thin, the concrete is placed through connections in the formwork surface or through the stopend. A sufficient number of openings should be provided to be able to control concreting.

Figure 2-127 Formwork window

Here are a few important requirements from the Ril 853 [296] and the ZTV-Ing [384]:

– The side of the excavation supported with shotcrete must be checked for dimensional tolerance and evenness and prepared. Pressure-tight tunnels have particular requirements for shotcrete quality with their cast-in items like steel arches, lattice elements, rock bolts etc., but also tunnels with umbrella waterproofing.
– Making good measures to repair early shrinkage cracking is necessary, and the duration is given as three days.

- Subsequent grouting of the unavoidable crown gap is essential, with a grouting pressure of 2 bar [296] but not exceeding 3 bar [384].
- The general quality requirements include not only the standards but also minimum requirements to be maintained. These include, for example, minimum layer thickness, minimum cement content, minimum reinforcement and concrete cover.
- Additional quality requirements apply to construction with waterproof concrete.

2.8.4.3 Reinforced or unreinforced concrete lining

Reinforcement is generally specified in inner-city transport tunnels. It is designed on the basis of structural verifications, as nominal reinforcement, or to comply with the requirements for waterproof concrete. The reinforcement normally consists of mesh.

For underground railway tunnels, waterproof concrete is increasingly being used, and a maximum block length of 8 m is observed, although the current trend is to change to 10 m or 12 m blocks for economic reasons. In order to avoid or keep as small as possible any constraint stresses to the rock mass or to a temporary shotcrete support, the following measures are listed here according to their tasks:

1. Levelling mortar with foil or fleece.
2. Foil, fleece, felt.
3. Limewash or painting of other materials.

As the inclusion of the reinforcement in the temporary support in design calculations is not sufficiently clarified due to questions about corrosion, locational stability and adhesion with the concrete, clients and their consultant engineers normally require two layers of support, with only the inner lining being assumed to act permanently. Economic considerations demand the development of either the inclusion of the contribution of the outer layer in calculations or a change to single-pass lining.

For rail tunnels, the latest version of the Ril 853 makes two layers of support compulsory. Single-pass construction can only be checked and approved as a special case by the client for, for example, escape tunnels.

There follows a discussion of the question of reinforcement of transport tunnels using the example of the tunnels on the new DB AG line Hannover – Würzburg [192], [193], [194].

1. Inner lining with minimum reinforcement

The provision of nominal reinforcement in the inner lining is based on the following considerations: for the constructive resistance to loading from loading cases like temperature, shrinkage, creep, point loads, undefined loads from the rock mass, which are not considered in structural calculations, a constructive nominal reinforcement is often required in Germany. The requirement is groundless when these loading cases are verified and concrete technological measures are undertaken against shrinkage.

For the tunnels on the new high-speed line of DB AG, however, there is a compulsory requirement for minimum reinforcement resulting from the specific loading cases of suction, pressure and self-weight, which apply to the high-speed sections ($v \geq 250$ km/h). In order to ensure the serviceability of the tunnel, a minimum reinforcement must be installed, which is described in more detail in the Ril 853, in order to prevent spalled concrete blocks from falling out.

In Austria and in other countries, tunnels are still often built without reinforcement of the inner lining, unless required to resist loading. In verified loading cases, a reinforcement can be required when the inner lining is subjected to heavy bending, which could lead to bending cracks. For this case, no fracture verification is possible without the inclusion of reinforcement.

It is generally agreed that a small area of reinforcement does not increase the failure safety although the reinforcement does increase the failure deformation of the structure in bending failure. It should be considered whether this improves the safety of the tunnel in case of a catastrophe. The decisive design cases are, for example, collapses of the rock mass, which subject the inner lining to bending. It is doubtful, but cannot be fully ruled out in special cases, whether the rock mass can become stable again after a large deformation of the inner lining. In these special cases, which presume very unfavourable geological conditions, a calculated reinforcement can make a contribution and should be designed and installed according to the loadings to be expected.

It should also be considered whether the bending strength of the concrete section can be significantly decreased or increased as a result of shrinkage or temperature changes and reinforcement is necessary for this reason. Such self-stresses can be kept within limits by suitable concrete technological measures and good curing of the concrete. Should nonetheless cracks develop parallel to the tunnel axis, they can if required be grouted with epoxy resin to bond the edges of the cracks. Such repair work can be omitted if waterproofing has been installed.

Finally, it should be considered that even cracked concrete maintains a certain load-bearing capacity resulting from toothing of the cracks in combination with arch action. It is known from roadbuilding that considerable shear forces can be transferred through the cracked surfaces of dummy joints. A small area of reinforcement, as is today still sometimes installed in tunnel linings, does not noticeably increase the safety of the structure and is thus unnecessary. As the deformations of the rock mass mostly occur gradually and particularly endangered sections have already been given special advance treatment, periodic inspection of the inner lining is preferable to steel reinforcement, which only has a partial effect in case of a catastrophe.

Stresses in the concrete resulting from exposure to fire can under certain circumstances be neglected for the structural calculation of reinforcement in rail tunnels. As long as the construction of the inner lining complies with the guideline drawings and the requirements of the Ril 853, fire protection measures in excess of the requirements of DIN 1045 are not normally necessary.

2. Inner lining of waterproof concrete

According to Ril 853: in-situ waterproof concrete should be produced as concrete of strength grade B25, but as concrete B II. The minimum thicknesses should be:

– For single-layer linings d = 0.40 m.
– For the inner layer of two-layer linings d = 0.30 m.

3. Unreinforced inner lining

The reinforcement can be omitted when

- waterproof concrete is not specified.
- the cross-section according to the waiver to clause 70 is calculated to be fully under compression.
- the lining is outside the frost-thaw cycle zone.
- the pressure from the rock mass is absent or slight, but uniform.
- the shrinkage stresses in the ring direction are exceeded by compression.
- there are no leaching zones in the rock mass.
- no point loads will have to be introduced into the lining or are to be expected.
- no asymmetric long-term behaviour of the rock mass around the cavity is to be expected.
- special measures are specified by the electro-technical specialists for equipotential bonding and the return of traction current in the tunnel.

At high speeds of over 250 kmh, nominal reinforcement is to be provided in every case to ensure concrete cannot fall out. In some tunnels on the new line on the route Würzburg – Hannover, induced loads, point loads, asymmetrical loading and temperature, shrinkage and creep were taken into account in the structural calculations. If the unreinforced lining can be verified for these additional loads, then there is no requirement for reinforcement. If this cannot be verified or the rock mass conditions do not fulfil the requirements, then the section is reinforced. Sections at the entrance to tunnels are generally reinforced.

Apart from the economic advantages, there are also constructive grounds for an unreinforced inner lining, considering the existing construction and the geotechnical conditions of the new high-speed line. According to the Ril 853, concrete according to DIN 1045 can be used in tunnel construction with or without reinforcement. In the example of Sinnberg Tunnel (Fig. 2-128), the reinforcement of the inner lining could be omitted in compliance with the Ril 853 because the required conditions for an unreinforced inner lining could be verified with calculations.

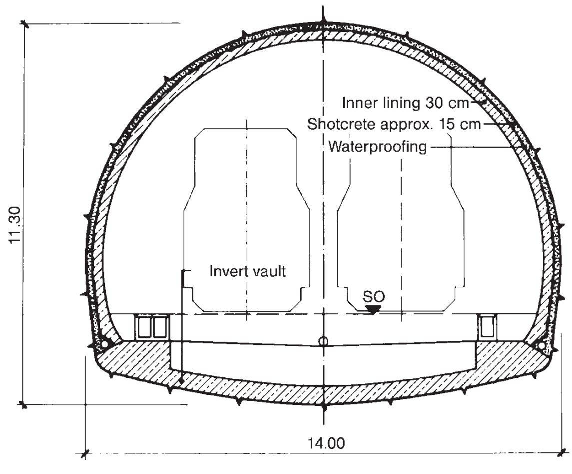


Figure 2-128 Support with unreinforced inner lining. Sinnberg Tunnel on the new line Hannover – Würzburg

2.8.4.4 Factors affecting crack formation

Differences resulting from shrinkage or temperature stresses in the concrete section or in adjacent elements with a solid connection can result in tension stresses in the concrete, which can lead to cracking if they exceed the tension strength of the concrete. The tendency of the concrete to crack must be reduced as far as possible through good mix design and curing of the concrete as well as other measures; on the other hand, the presence of waterproofing foil with fleece – both materials are applied to a locally made good but not geometrically precise outer layer – offer considerably better possibilities for adaptation than direct concreting against the outer layer. If no waterproofing is required, constructional levelling layers are also beneficial in any case for reinforced linings, particularly for waterproof concrete.

Cracking of the concrete (whether resulting from loading stresses or temperature shrinkage) cannot be prevented by reinforcement. It only has an influence on the spacing and thus the width of the cracks that form. In thin unreinforced construction elements, cracks form at a wider spacing with greater width. A dense mesh of reinforcement reduces the crack spacing and thus width.

2.8.4.5 Disadvantages of nominal reinforcement

A critical evaluation of the use of steel reinforcement should also consider the disadvantages:

1. When the construction shown in Fig. 2-128 with waterproofing of the tunnel is used, there is a danger of the reinforcement damaging the plastic waterproofing membrane. In order to prevent this, the waterproofing should be additionally protected. As, however, the accessibility and checking of the waterproofing layer is difficult after the formwork unit has been erected, there remains a danger of damage and thus leaks in the waterproofing layer.
2. In tunnels with full-surface waterproofing between shotcrete temporary support and the inner lining, particular measures have to be taken to secure the position of the reinforcement. The fixing of any reinforcement in the inner lining is very difficult, as in contrast to other construction works, the presence of the formwork around the entire tunnel perimeter means that the reinforcement can only be fixed correctly if either the steel fixers crawl through openings into the space between formwork and outer lining, which is normally only 30 cm, or the reinforcement is suspended and spacers are used to hold the reinforcement in the correct position. According to experience, these unfavourable working conditions often lead to the actual intention of preventing cracking not being achieved.
   The reinforcement is nowadays normally constructed as self-supporting. In order to achieve this, larger tunnel profiles need a relatively high area of reinforcement.
3. Finally, it should be pointed out that the placing and compaction of the concrete is considerably hindered by the steel mesh. There is no doubt that it is technically possible to pump wet concrete between the tunnel formwork and shotcrete waterproofing layer and then adequately compact it. Special measures are, however, necessary to prevent the concrete separating into coarse aggregate and mortar after leaving the pipe and while falling between the reinforcement. Even if this is carried out correctly and expertly, it has to be assumed that the problems of honeycombing (separated zones) and inadequate compaction are considerably worsened by the installation of reinforcement.

4. It should also be pointed out that corrosion protection of the reinforcement demands a dense concrete structure, the maintenance of minimum concrete cover and the pointing of any wide cracks. These measures are also normal practice today, but require additional attention during construction and the repair of faults.
5. Also in case of fire, an unreinforced lining will behave better than a reinforced lining due to the different behaviours of steel and concrete at high temperatures.
6. When tunnelling in rock, considerable overbreak always has to be expected. Unless this is compensated geometrically with expensive measures, the outer reinforcement will not be in accordance with design assumptions.

2.8.4.6 Stripping times

Stripping times are largely dependent on cement type, concrete mix, wet concrete temperature, the external hardening conditions and the construction.

DIN 1045 can serve as a rough guide for the estimation of stripping times in building and civil engineering. In tunnelling, relatively short stripping times often have to be achieved as a result of the chosen process of excavation and concreting. Table 2-22 gives a guide to the influence of cement type, *W/C* ratio and wet concrete temperature at a specified strength of about 5 MN/m^2, which can serve as the lower limit of strength for stripping.

Table 2-22 Stripping times

Cement	*W/C* ratio	Required hardening time in days at a concrete temperature of		
		5°C	12°C	20°C
Z 25	0.4 0.6 0.8	4 9 15	2 ½ 5 9	1 ½ 3 5
Z 35 L	0.4 0.6 0.8	2 5 7	1 ½ 3 ½ 5	1 2 3
Z 35 F Z 45 L	0.4 0.6 0.8	1 2 4	¾ 1 ½ 3	½ 1 2
Z 45 F Z 55	0.4 0.6 0.8	½ ¾ 1	¼ ½ ¾	¼ ½ ¾

2.8.4.7 Filling of the crown gap

The gap above the crown should generally be grouted after concreting due to the settling of the concrete. A cement mortar with flowing consistency is filled through injection connections built into the crown (for smaller diameters 2 to 3 m, for larger 3 to 5 m spacing) until it emerges from the next hole forwards. This hole later has to be drilled out again for further filling if the mortar has already hardened (Fig. 2-129).

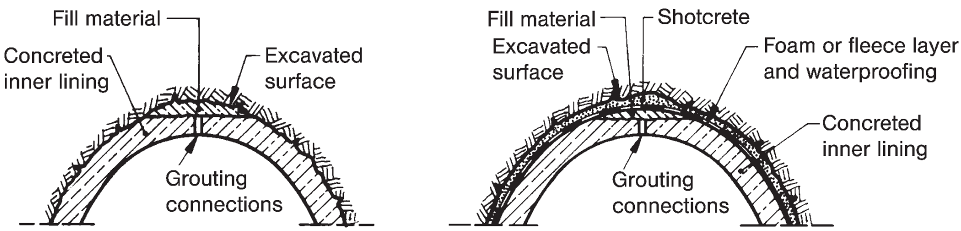


Figure 2-129 Filling of the crown gap for lining without waterproofing (left) and with waterproofing

2.8.4.8 Joint details

There are various constructional details of joints according to the purpose of the joint (Table 2-23). Working joints are determined by the needs of construction and normally have to transfer all forces and moments at the section. They should be located at positions of zero moment if possible. Dummy joints are induced cracks. Movement joints serve to compensate various types of movement.

Table 2-23 Possible movement of various joint types

nearly zero <-			**Movement -> slight <-**				**-> large**
"rigid"			Joint				"weak"
Working and concreting joint	Grouted joint	Dummy joint	Hard (brittle) joint	Movement joints			
				1^{st} type expansion joint	2^{nd} type expansion joint	3^{rd} type movement joint	4^{th} type settlement joint, separation joint
Results from interrupting of concreting. Joint spacings should be laid down before concreting. Ensure bonding to new concrete	As before, but without bonding of the new concrete. this is ensured by painting with bitumen, tar or synthetic coating	Intentional weakening of cross-section (notch) to control crack formation (intentional crack formation)	Spatial joints with hard filling (cement or synthetic mortar), mainly used with small-format construction elements	One-off length alteration due to shrinkage, creep	Repeated, alternating length alteration due to temperature	Additional provision of usually slight reinforcement in all directions (oscillation, rotation etc)	Provision of vertical reinforcement. frequently heavily reinforced, then separation joint

2.8.4.9 Single-pass process, extruded concrete

The idea of simultaneously and continuously concreting the lining behind a tunnel boring machine or a shield is quite old, and has been given new impetus by the use of hydraulics in the machine and steel fibre concrete as the installed material. While processes depending on shotcrete are greatly influenced by the capability of the miners and the nozzleman, shield drives also permit further mechanisation of the lining installation. This section describes developments ranging from reusable formwork to slipforming. Applications so far have already achieved considerably higher advance rates in soft ground with groundwater than the use of shotcrete.

Reusable formwork for unreinforced concrete and steel fibre concrete. Processes were developed many years ago to enable the continuous production of the final tunnel lining in combination with shield excavation.

Folding reusable formwork with lattice arches and reinforcement. The fixing of the reinforcement requires access for manual work at the tunnel walls. If the rock mass will not stand up, the fixing of the reinforcement, erection of formwork and a certain hydration time must take place in the protection of the shield tail. The hydration time can be altered by the used of admixtures.

Reusable formwork with stopend as grouting ring (Fig. 2-130). Y. Matsushita and co-authors [253] reported a method where a reinforced single-layer lining is constructed with a grouting ring as stopend. Comparison with conventional shield methods makes clear the saving of working steps.

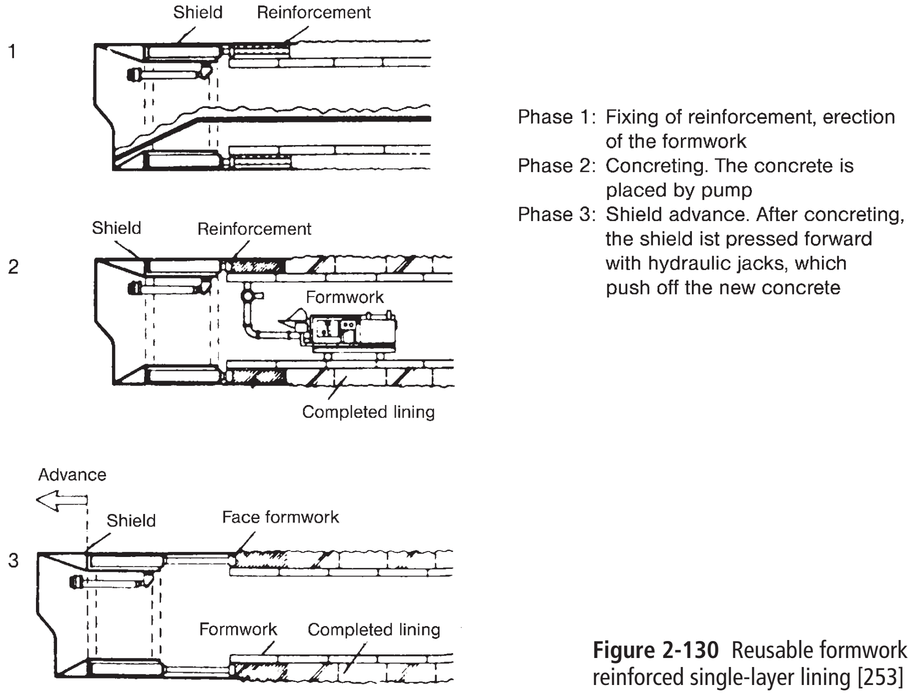


Figure 2-130 Reusable formwork for reinforced single-layer lining [253]

J. Gran [190] also reported a process with a grouting ring. The concreting system was installed in an excavation shield with disc cutter cutting wheel system TSCB and designed for an underground railway tunnel in Prague. The figures given for the time to erect a ring are interesting.

Reusable formwork with moving front formwork and steel fibre concrete. When steel fibre concrete is pumped into the formwork, the process of fixing steel can be omitted. This is what makes it possible to cast an extruded lining with a moving front formwork [229].

Slipforming. Tunnellers have long yearned for a tunnelling machine with attached slipform.

Extruded concrete. In the extrusion process, flowing concrete is pumped through a number of pipes through the stopend shutter moving with the shield tail to continuously fill the annular space immediately behind the shield tail of the tunnel boring machine. The concreted space is bounded to the outside by the surrounding rock mass and to the inside by a steel formwork unit that is moved forwards continually and to the front by the elastically supported stopend shutter (Fig. 2-131).

The formwork unit typically consists of 1.20 m long rings that can be broken down into individual segments (segment formwork). This is provided with a special mechanism to enable it to be rapidly struck, moved and erected again in the protection of the shield tail (Fig. 2-132).

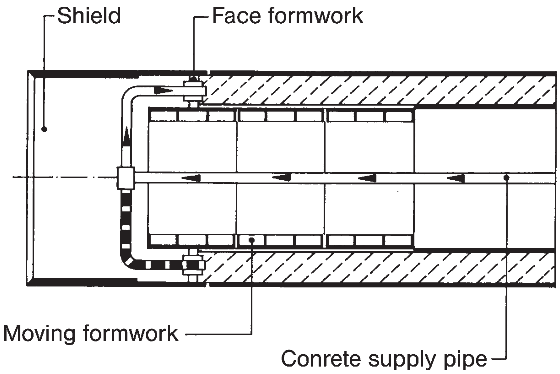


Figure 2-131 Schematic diagram of the extrusion process [41a]

The total length of the reusable formwork is dependent on the strength development of the concrete and the planned maximum advance rate; it is about 15 m. The process was first used for the construction of a main sewer in Hamburg in 1978 [198].

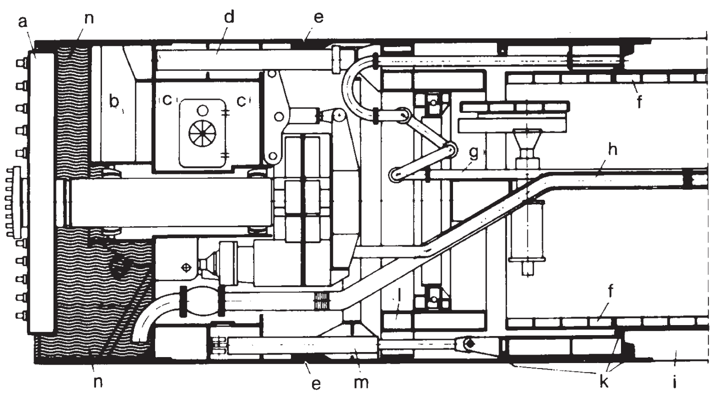


Figure 2-132 Reusable formwork for a single-layer steel fibre concrete. Sewer Harburg-Nord, contract 1 [196]

2.8.4.10 After-treatment

In order to ensure the quality of concrete in the inner linings of tunnels, curing of the formed concrete is compulsory according to the rules of DIN 1045. This curing must, however, be planned according to the special conditions of tunnelling and the specific project. In order to carry this out, mobile curing units are used, which follow directly behind the formwork (Fig. 2-133).

The functional principle of the mobile curing unit is that the unit itself provides an airtight enclosure of the fresh concrete, with seals at the sides of the unit. This creates a space between tunnel wall and the mobile unit called a climate chamber. The gap between the curing unit and the tunnel wall should be kept as constant as possible, and also the temperature and humidity. The latter two parameters are nowadays controlled and checked automatically.

Figure 2-133 Sliding mobile curing unit in the Hellenberg Tunnel on the new line Cologne – Frankfurt [346]

The ZTV-ING Tunnelling and the Ril 853 include important requirements for the treatment of formed inner lining concrete after casting. Some of these are listed below:

- According to ZTV-ING, the mobile curing unit should consist of three independent chambers, each of which should correspond to a block length.
- Mobile curing units should be constructed as self-supporting steel constructions with geometry to the correct profile of the tunnel cross-section.
- The gap between the concrete side wall and the seal should average 10 mm, but not more than 15 cm.
- Curing should not last less than three days. [296]
- Temperature and humidity measurement points are to be provided at three locations in each chamber, and the measurements should be documented continuously.

Modern mobile curing units are equipped with mobile control units, which simplify the documentation of temperature and humidity mentioned in the last point and enable the analysis of the recorded date on-site.

2.8.5 Precast elements, cast segments

Prefabricated elements of steel, cast iron, or reinforced concrete are particularly suitable for the lining of tunnels excavated by shield machines or by jacking entire pipes. Prefabricated lining construction can also be used as the final lining apart from the processes already mentioned in tunnels with shotcrete support.

It is always important that the gap between the rock mass and the prefabricated element is completely filled, either immediately after installation or later, to guarantee all-round support of the entire perimeter. Some types of element only gain their load-bearing capacity through this bedding on all sides. The usual fill materials are pastes, cement grout or in special cases also suspensions and foams. The binders are normally based on cement but can also be bentonite, synthetic resin or other synthetic materials.

Linings of prefabricated elements are normally constructed as a single layer, which means that attention has to be paid to sealing. All joints are critical locations but particularly the corners.

Larger cross-sections, particularly in shallow tunnels, require heavier lining elements, which are prefabricated as segments and assembled on site. Under particularly favourable geological conditions, an elastically bedded tube and lining without stiffness in bending are sufficient; light lining profiles can fulfil the stability requirements.

Since 1970, there has been a continuous development towards the use of reinforced concrete segments. This has led to the situation that steel, cast steel and cast iron are only used for special situations.

2.8.5.1 Steel segments

Steel segments are welded from steel plate and assembled on site. Fig. 2-134 shows an example from the subway in San Francisco.

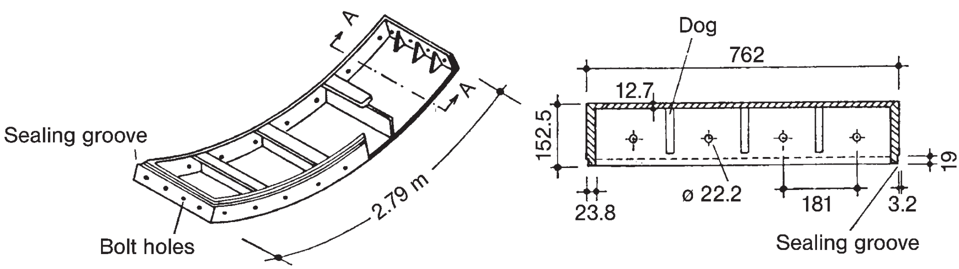


Figure 2-134 Steel segment. Subway tunnel in San Francisco, Bart Line [83]

Steel as a material has the following advantages and disadvantages for the requirements of tunnelling compared to other materials:

Advantages:

- Simple and precise assembly is possible, time saving during tunnelling.
- Standardisation is easily possible.
- Good load-bearing, elastic, plastically deformable within limits, thus safety reserves.
- Watertight joints can be achieved by welding.
- Minimum excavation due to the thin profiles.
- Low weight of the construction elements.
- The material thickness and grade can be altered to suit structural requirements.
- Dynamic loadings can be resisted with special constructions.

Disadvantages:

- Resisting of thrust press forces from shield machines is only possible with additional measures.
- Welding of the elements is laborious and requires skilled workers.
- High precision demands extra expense.
- Problems of corrosion and fire behaviour have to be dealt with.
- Noise protection problems.
- When installed below the groundwater, floating should be considered and verified (possibly leading to additional measures).
- Many clients have reservations against steel (often unfounded).

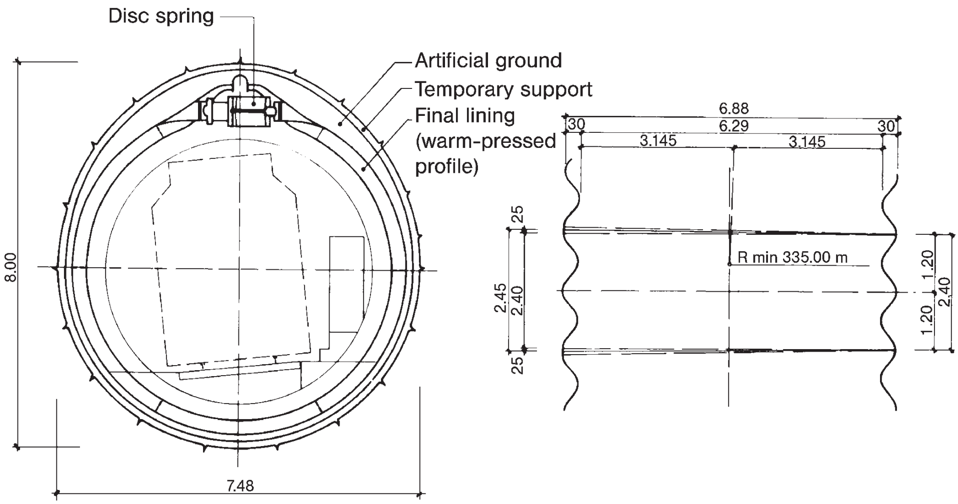


Figure 2-135 Lining of cast iron segments for an urban rail tunnel in a mining settlement area. Gelsenkirchen, 1980 [121]

2.8.5.2 Cast steel segments

Cast steel segments are made in larger dimensions than steel segments. The rings are assembled and welded into rings of 3 to 5 m diameter in an assembly pit. At the installation site, the rings are aligned and welded to each other; this requires the tunnel cross-section to be temporarily supported.

Material quality: cast steel GS17Mn5. Cast steel has all the advantage stated for S355, but also has favourable corrosion properties and in contrast to cast iron segments can be easily welded. Cast steel can thus be used for many special applications and is especially suitable for situations with mining settlement or dynamic ground pressure (Fig. 2-135).

Basic solutions for tunnels in mining settlement areas have been developed by B. Maidl, J. Wolter and K.-G. Krämer [235]. The ring is completely separated by a joint in the crown. A partial pressure and compensation element (Patra element) can overcome mining pressures with crumpling tubes, and mining strains with spring elements.

2.8.5.3 Cast iron segments

Cast iron has a long tradition for tunnel linings. Cast iron segments or tubbings have been used in Great Britain since 1860. Today, only spheroidal graphite (ductile) cast iron or GJS are used (Table 2-24).

Table 2-24 Permissible stresses and safety factors for cast iron segments

Cast iron type	Permissible stresses in N/mm^2 in		Safety factors		Compression strength bending compression	Bending strength bending tension
	bending tension	bending compression	tensile strength in bending tension	yield limit bending tension		
EN-GJL-250	80	160	3.1	–	4.5	5
EN-GJS-500	200	200	2.5	1.75	4	5
EN-GJS-600	240	240	2.5	1.75	3.5	4.5

The joints were sealed until only a few years ago by caulking with lead. Today there are a range of prefabricated joint constructions.

The material cast iron has the following advantages and disadvantages:

Advantages:

- Spheroidal graphite cast iron has similar load-bearing capacity to steel, but not its elasticity and plasticity.
- Corrosion behaviour.
- Processing by casting is simpler than welding. Dimensional tolerances are easier to maintain.

Disadvantages:

- Higher weight.
- More difficult to weld if repair is needed.

The corrugated profile for the most efficient exploitation of the material is shown in Fig. 2-136a. The box profile, for example for tunnels to carry water, is shown in Fig. 2-136b. The comb profile was the forerunner of the corrugated profile and is today only used for special purposes (Fig. 2-136c). Special profiles with high deformation capacity are used in cases of mining subsidence.

a Corrugated profile
b Box profile
c Comb profile

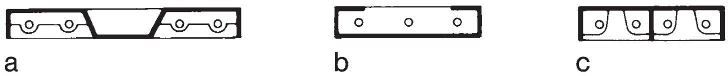
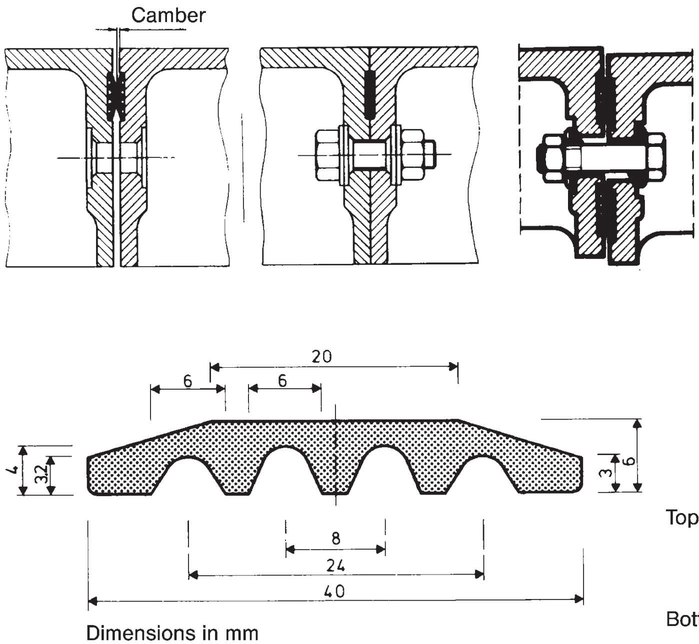


Figure 2-136 Profile shapes for cast iron segments [189]

Ring and butt flange connections are bolted. In order to seal the joints, an endless neoprene strip laid in a groove has proved to be the technically and economically most favourable solution (Fig. 2-137). In addition, hydrophilic materials are used, which swell on contact with water.



Top: Installation of the elastomer gasket from Phoenix AG for cast-in segments [106]
Bottom: Neoprene gasket for steel segments. Elbe Tunnel, Hamburg [91]

Figure 2-137 Flange sealing for cast iron and cast steel segments

2.8.5.4 Reinforced concrete segments

Many years of experience with precast concrete production has also made it possible to produce segments of precast reinforced concrete. Until the 1970s, two layers of lining were usual; the segments used were predominantly block segments. Since then it became standard to realise the required dimensional precision with reinforced concrete segments.

The construction principle of lining with watertight and drained segments is shown in Fig. 2-138.

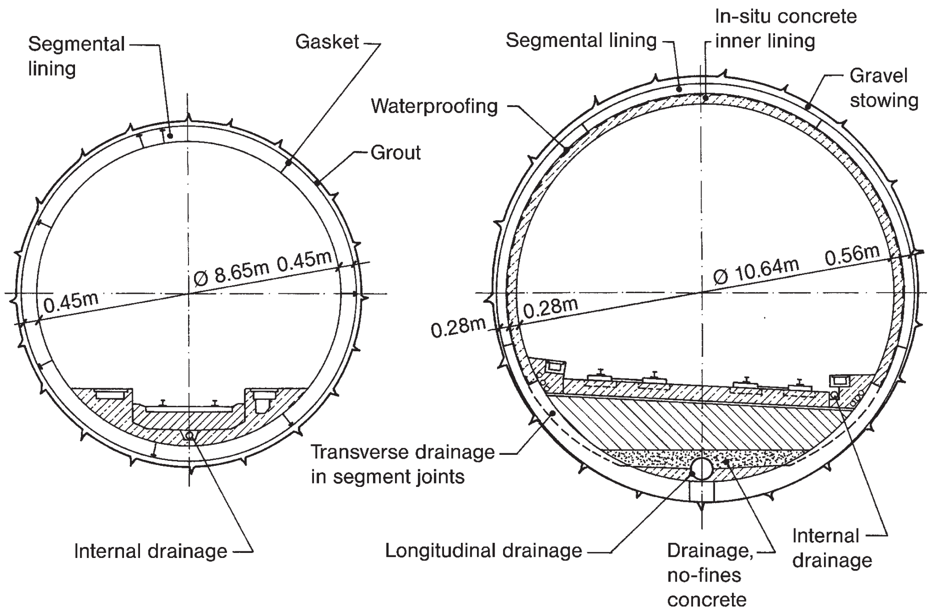


Figure 2-138 The construction principle watertight and drained tunnels with segment lining [242]

Fig. 2-139 shows the spectrum of construction variants of segments for single- and two-layer construction types in tunnelling.

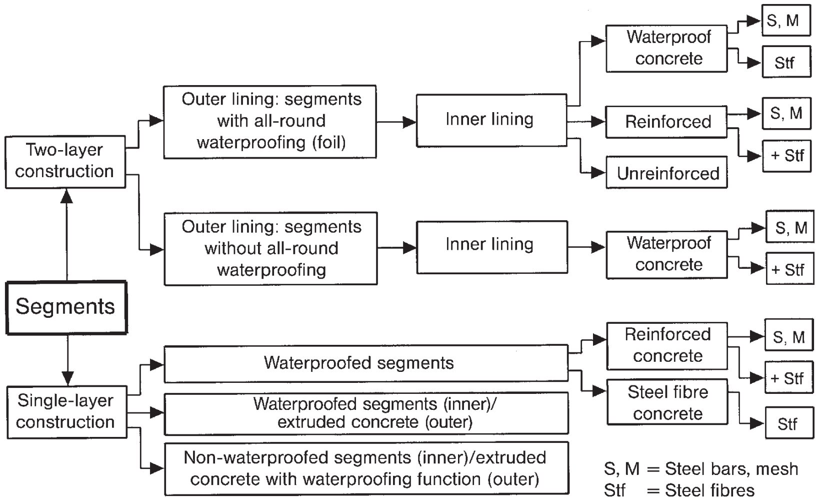


Figure 2-139 Segments for single- and two-layer construction variants [242]

Profile shapes are illustrated in Fig. 2-140.

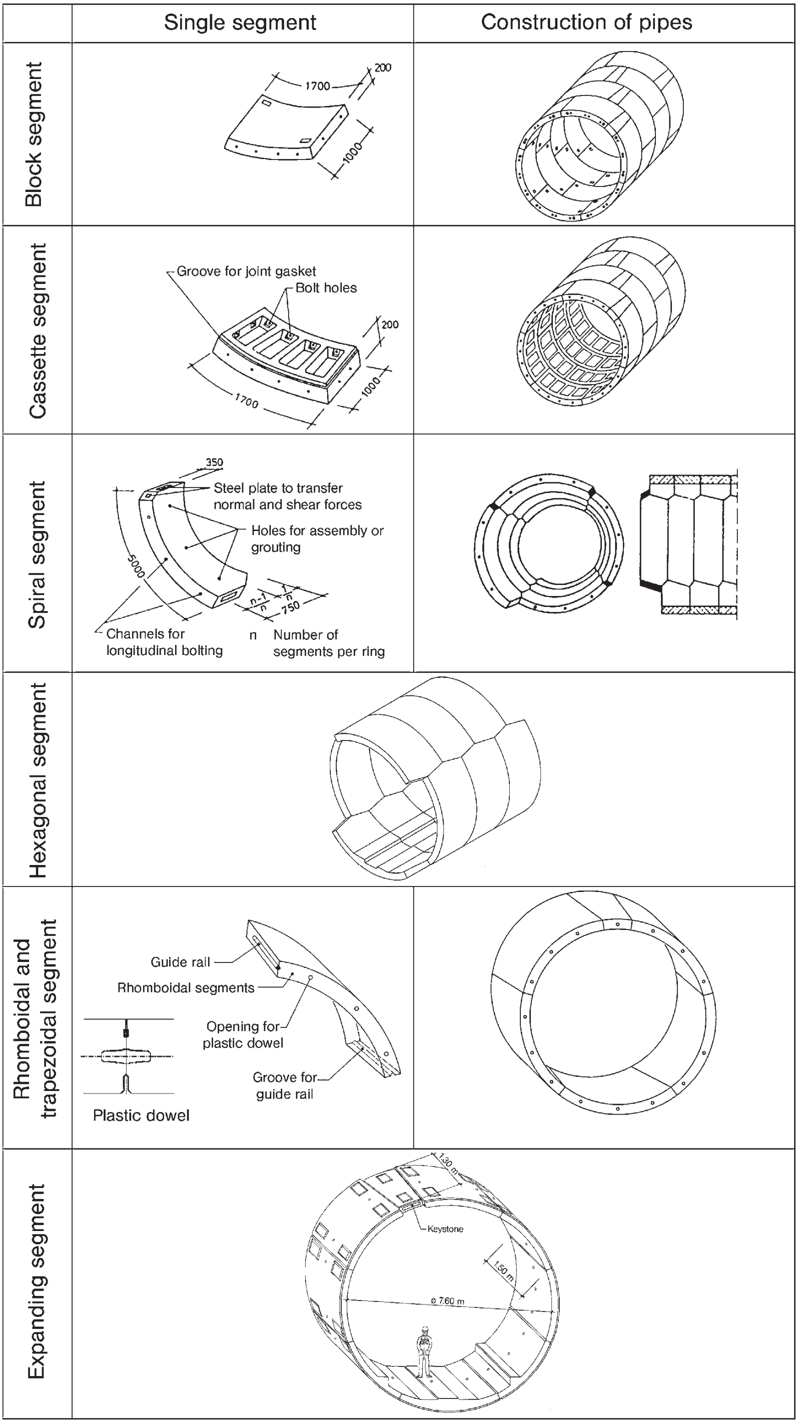


Figure 2-140 Profile types for reinforced concrete segments

2.8.5.5 Geometrical shapes and arrangement

A segment ring normally consists of five to twelve individual segments. Layouts that do not form a closed ring, like diamond or spiral segments, have not proved successful in practice and are now seldom used. In order to avoid gaps when lining curves, tapered rings have to be produced, with mating surfaces that are not parallel. Rings with a taper on one side, as shown in Fig. 2-141, have proved particularly successful. These have a minimum curve radius imposed by their geometry but can also be twisted round to form any spatial curve [276]. Technical innovations in segment production and in erector and jack systems also make possible key segments located below the tunnel sides down to the invert, which means that left- and right-handed rings no longer have to be differentiated. For structural reasons, it is advantageous to make all segments as equal in size as possible, which means that the key segment has about the same opening angle as the other segments, but for ring assembly, a smaller and lighter keystone is easier to handle (Fig. 2-141). It is necessary to consider which system is more suitable for each project. The large key segment is becoming more and more accepted.

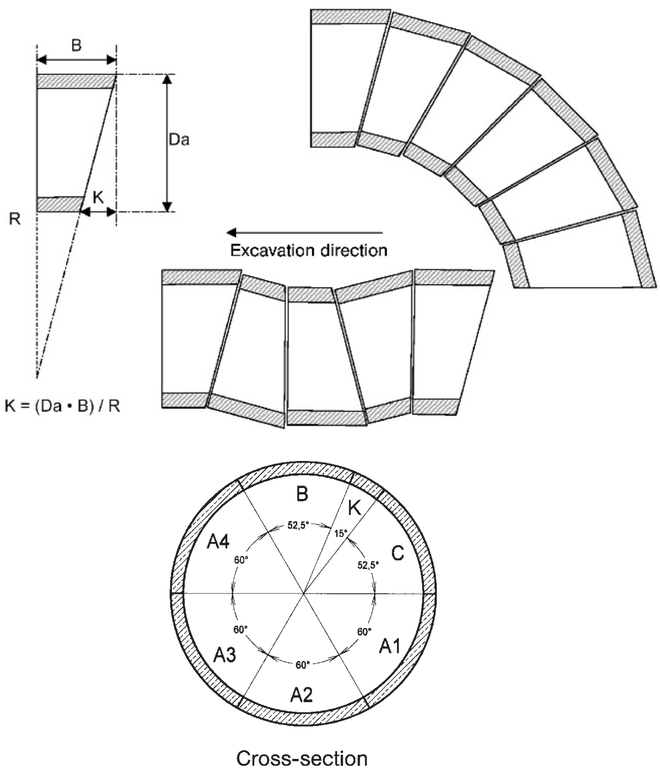


Figure 2-141 Curve ring of segments with tapered circumferential joints [383]

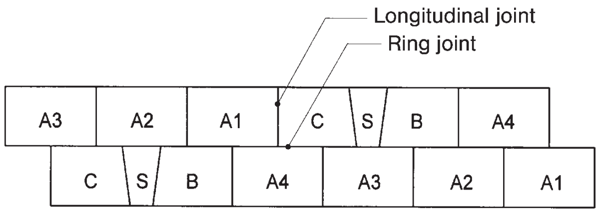


Figure 2-142 Layout of a segment ring [383]

As illustrated in Fig. 2-142, two adjacent rings should not have their joints lined up. This avoids cross-joints, which are the most common location for leaks. It also prevents the circumferential joint offsets adding up over a number of rings and thus increasing the risk of segment damage and leaks.

2.8.5.6 Details of radial joints

In the radial joints, normal forces and bending moments are transferred through eccentric axial forces and shear forces. In order to be able to resist the relevant decisive loading forces, various types of joint have been developed (Fig. 2-143):

- Flat joints transfer normal force over the whole area and can (under compression over the entire area) also transfer some bending moment due to the wide contact surface. Shear force can only be transferred through friction. They have to be aligned during assembly but can permit an offset to compensate inaccuracies without causing concrete spalling.
- Convex-convex ("roller bearing") joints can transfer high normal force even with high joint rotation (high compression strength of the concrete due to Brunel knife-edge bearing), but no moment and practically no shear force. The assembly of this type of segment is extremely difficult, particularly when compression gaskets are used.
- Concave-convex joints ("roller joints") transfer high normal force; only very slight moment and high shear force. In addition, they offer a certain location of the segment being installed during ring assembly.
- Tongue and groove joints ("drawer joints") offer good location during assembly and transfer normal force, moment and shear force. As the flanks of the groove cannot be effectively reinforced, concrete spalling can occur when the play in the joint is only slightly exceeded.
- A special form of tongue and groove is the one-sided tongue and groove detail, for example the internal spur. This is normally only used for small key segments in order to prevent it slipping out.

2.8.5.7 Circumferential joint details

The circumferential joints have to transfer the normal forces from the thrust press reaction and shear forces resulting from lack of fit in the shield tail, from external forces resulting from shield tail grouting and radially asymmetric floating and ground pressure after leaving the shield tail. If the deformed shapes of two adjacent rings are different, the resistance to deformation results in coupling forces (shear forces) in the circumferential joints.

Formerly Kaubit strips were used, now often fibreboard plates are used for defined force transfer in circumferential joints. Plastic inserts compensate production inaccuracies of the concrete surfaces, which could lead to impermissibly high contact stresses.

Flat joints can only resist shear force through friction. There is only sufficient friction when normal force is applied by the thrust presses. If the thrust force is small, the segments can become offset. Elastic fibreboard inserts increase the friction and improve the transfer of shear force.

A cam and socket system in the circumferential joints mainly serves as an aid to centring and to stop the segments falling out during ring assembly. If the permissible shear force is exceeded, the cam should shear off in order to prevent large-scale spalling. Joints with a tongue and groove detail are normally equipped with special “coupling places” intended to resist forces from reciprocal deformation. A new development are joints with loose tongue and grooves. These systems combine the advantages of “tongue and groove” and “cam and socket” systems.

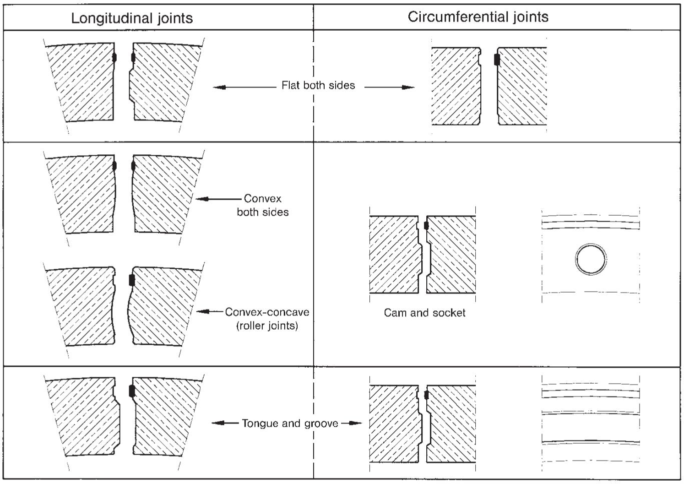


Figure 2-143 Segment joint details [383]

2.8.5.8 Fixing and sealing systems

It is not necessary to bolt or dowel the segments to each other for structural reasons: the ring is closed together (through the radial joints) by ground and water pressure, and the thrust presses apply a prestress along the direction of the tunnel. Fixing of the segments is, however, required temporarily during construction for the following reasons:

- To prestress the segment gaskets in the radial and circumferential joints to ensure waterproofing (avoid the gaskets “breathing out”).
- To connect the rings in the annular gap until the fill hardens.
- To avoid deformation of the segment ring until the annular gap fill has hardened (caused for example by constraints in the shield tail or the thrust pressure in the circumferential joints).

The bolting has to hold the individual segments together against the resistance of the gasket to squeezing; it also makes sure that shear force can be transferred by friction in the joints. The bolting cannot prevent deformation as the forces are too high and the bolt cross-section is usually too small. When hydrophilic rubber gaskets are used or on tunnel drives below the groundwater table, it is often possible to omit the bolting. This applies particularly to two-pass lining systems. The various bolting systems are shown in Fig. 2-144.

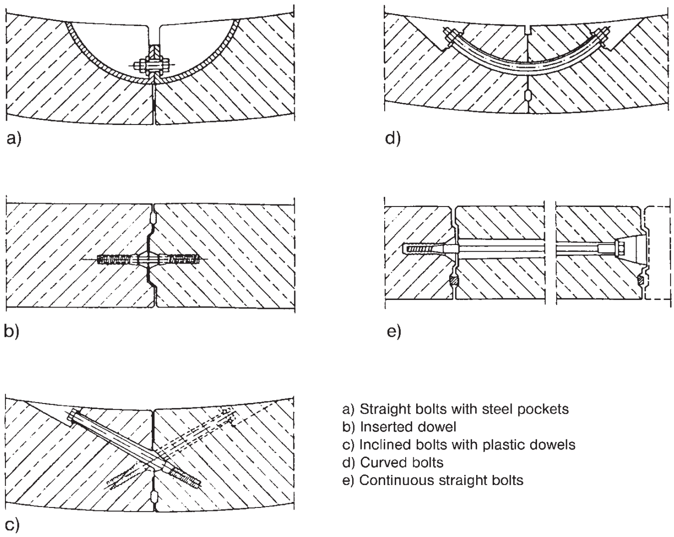


Figure 2-144 Block segment connections [383]

2.8.5.9 Segment gaskets

The most important method of ensuring a watertight tube is the gaskets in the segment joints, and assembly without offsets or spalling and the concrete quality of the segments are also important. Sealing gaskets in the segment joints are made of synthetics, elastomers, neoprene, silicones and hydrophilic rubber (Hydrotite or similar).

The sealing gasket is laid in a continuous groove in the segment joint, which is under normal loading (Fig. 2-145). The compression behaviour of the profile and the dimensions of the groove have to be matched to each other so that concrete does not spall behind the groove due to splitting. The relationships between joint opening and compression force and between testing pressure and joint opening are shown in Fig. 2-145, in this example for a profile from the company Dätwyler.

It can be seen that the compression force in the gasket profile falls with increasing joint opening A (see Fig. 2-145 b). This reduces the achievable test pressure. The reduction of the achievable test pressure is also worsened with increasing joint offset (Fig. 2-145 c).

Hydrophilic rubber makes, which swell in contact with water, have been used relatively often particularly in Asian countries. In Germany, there have only been a few applications (for example the deep inverted siphon at Dradenau), as there have been reservations so far about the long-term behaviour (ageing, brittleness) of the swelling material.

The material is installed as with the already described elastomer gaskets.

The swelling pressure of hydrophilic gaskets does admittedly produce high pressures on the sides of the segment on contact with water. This means that the sealing effect of the materials is independent of any additionally applied compression force and can still be guaranteed even at small joint openings.

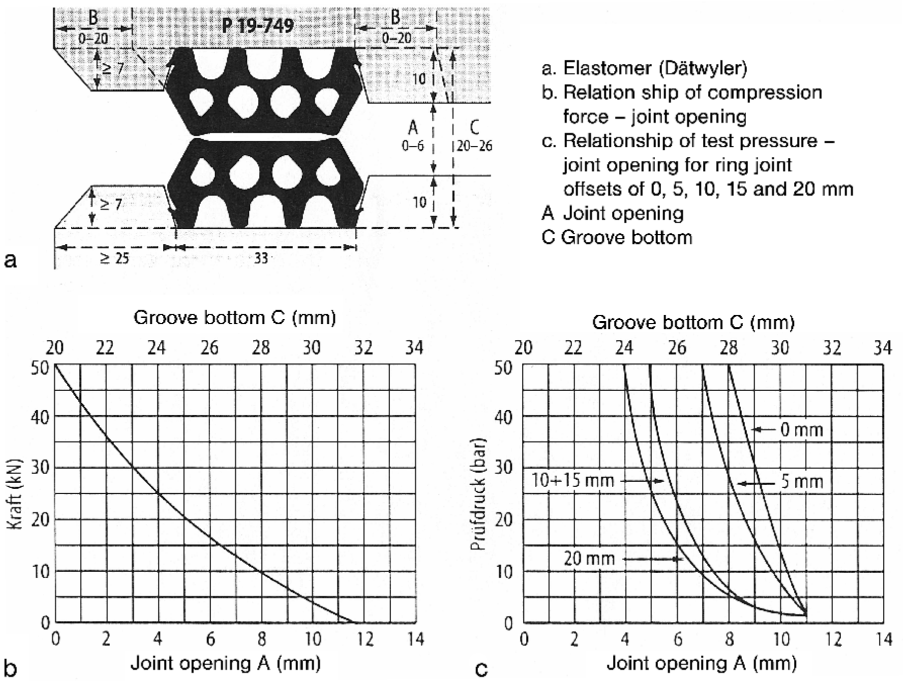


Figure 2-145 Elastomer gasket [383]

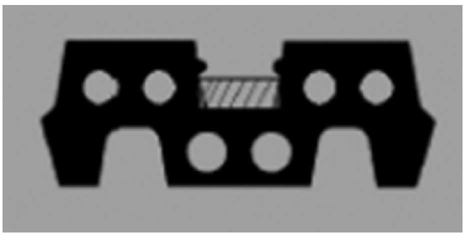


Figure 2-146 Evolutiv seal, elastomer gasket with hydrophilic inserts in the middle; Phoenix AG

Combinations of elastomer waterproofing gaskets with hydrophilic rubber inserts are also now available (Fig. 2-146). The hydrophilic insert is intended to provide additional sealing security, achieving a sealing effect even when the compression of the elastomer is insufficient.

2.8.5.10 Production of reinforced concrete segments

Segments are normally made in special precast concrete works, but is can be more economical on larger tunnel projects to set up a dedicated site production works, as at the Channel Tunnel or the Belt Tunnel, in which case the necessary space should be included in the planning of site facilities. Particularly large projects require detailed logistic planning and implementation for a trouble-free supply of segments.

In order to maintain the stringent precision requirements for segments, only steel formwork is suitable. The tolerance requirements for the production of segments are determined by the effectiveness of waterproofing and the minimisation of constraint loading. Technological progress in formwork technology today has made tolerances similar to mechanical engineering a practical possibility. For rail tunnels in Germany, the stringent dimensional tolerances of Ril 853 apply, of which an excerpt is shown in Table 2-25, and these have already often been included in the specifications of international projects.

Table 2-25 Dimensional tolerances for segments [296]

Segment width	± 0.5 mm
Segment thickness	± 2.0 mm
Segment arc length	± 0.6 mm
Flatness of radial joint	± 0.3 mm
Flatness of circumferential joint	± 0.3 mm
Angle of twist in radial joints	± 0.04°
Taper of the radial joints	± 0.01°

The maintenance of the tolerances is to be checked regularly at the shortest possible intervals in order to be able to detect any warping of the formwork at an early stage. Deviations from tolerances can lead to unplanned constraint loading of the segments during assembly or in service. These can reach a magnitude, which cannot be resisted by the concrete or the reinforcement. If the tolerances cannot be maintained in practice, then the effect on the segments should be investigated.

The taper of the radial joints can be taken as an example. With an increasing angle of taper in these joints, the assumptions made in the structural calculations about the uniform distribution of loading over the width of the segment will no longer be ensured. This results in an eccentric loading in the joint, which would have to be compensated with extra reinforcement.

2.8.5.11 Installation of segment lining

The segments are installed in the shield using an erector. The erector holds the segments either mechanically with a claw construction or pneumatically with a vacuum cushion. The precision attained in ring assembly depends greatly on the exact controllability of the erector.

In order to ensure the most exact possible positioning of the segment, modern erector heads can move in all six degrees of freedom (movement in the x, y and z axes and three rotations). The installation process is speeded up if the movements are mostly uncoupled. The functional principle of the ring erector is shown in Fig. 2-147. The setting head is mounted on a bridge between two telescopic elements mounted on a slewing ring. The angle of rotation is limited to ±200° from centre (pickup position).

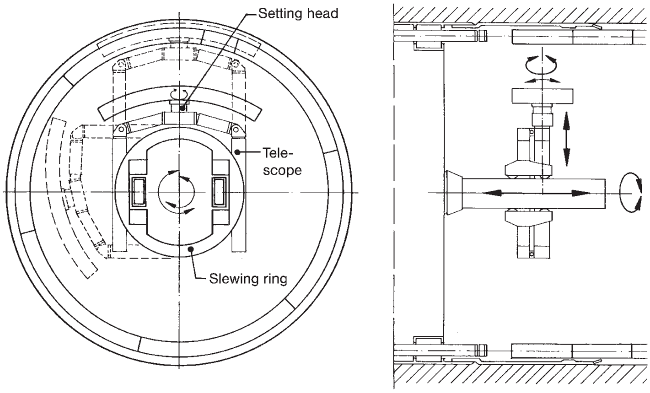


Figure 2-147
Ring erector [244]

2.8.6 Linings for sewer tunnels

General. Cement-bound materials below water are normally adequate for the requirements of urban sewers, but the situation is different when they are acting as the lining of a space filled with sewer vapours above the water level. Not only in tropical countries but also recently increasingly in Europe, cases of microbiological-chemical attack on concrete sewers have reached an alarming extent. This is termed called biogenic sulphuric acid corrosion, and the primary cause is that large sewer pipes often have to be part-full in operation and that foul sewage has a high sulphur content. In contrast to oxygen, volatile sulphur compounds above the water level can be oxidised to sulphuric acid by thiobacillus bacteria, which live in the condensate on the walls of the sewer. The chemistry of sulphur attack of concrete is indeed known, but the causes of increased sulphuric acid production are unknown.

This problem was already latent with brick construction, which was used formerly but seldom practiced today on account of the wage costs. Bricks are generally resistant but the mortar pointing has to be renewed after a certain time.

According to DIN 4030, water, soil and gases are categorised into three degrees of chemical attack; at pH values of less than 4.5, the attack is described as “very strong”. In sewers, pH values down to 1 can be encountered on the walls above the water line. According to DIN 1045, section 6.5.7.4, concrete exposed to “very strong” chemical attack for a longer time have to be protected against immediate contact with the attack substance. At the moment, the degree of attack to be expected on the concrete walls of sewer tunnels is not precisely quantifiable. Until it is, passive corrosion measures, in the form of surface treatment of the concrete, will continue to be applied in cases of “very strong” attack, instead of active corrosion protection measures, which have not yet been proved.

Active corrosion protection

Constructive measures. The hydraulic design of the sewer should ensure uniform flow without sedimentation and with a good fall; good hydraulic shape; and the full exploitation of all measures of concrete technology such as particularly dense surfaces, W/C ratios below 0.45, high concrete strengths (at least B 35) with low cement content, the use of sulphate-resistant or low-alkali cements and aggregates without alkaline silica, poisoning of the concrete or its surface, the use of “sacrificial concrete” with aggregates containing lime.

Operational measures. Regular cleaning of the walls and invert using newly introduced "intensive cleaning processes", ventilation of the air space above the water level in order to considerably lower the humidity, replacement of the oxygen atmosphere by a nitrogen atmosphere, operation with the sewer full.

Alteration of the foul water quality. Technical measures like oxygenation or adding lime to the foul water; statutory measures to influence discharge conditions, particularly to reduce the sulphur freight from synthetic detergents.

Passive corrosion protection

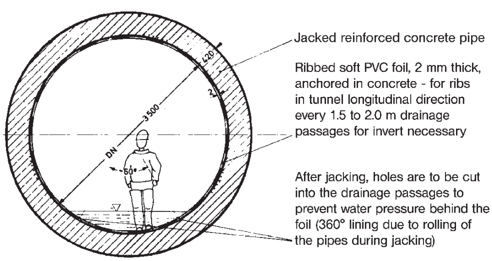
This is to be understood as consistent separation of the corrosive medium from the cement-bound materials. The intention is to find construction materials or combinations of construction materials that can resist the given corrosive and hydrodynamic attack in the long term. Construction methods and processes should be sought, which permit the longest possible period of use with regard to the costs.

Surface protection for sewers. There cannot be any uniform patent solution for surface protection, as each protection measure has to be adapted to suit the construction process. Protection can be installed with the concrete or subsequently. The former has the advantage of being cheaper but with the disadvantages that it can be damaged by later construction work or is not always immediately usable. External sealing of the structure is normally not possible or problematic. For all corrosion protection measures, the external pressure of the groundwater should be taken into account due to the capillaries, even if the tunnel is of waterproof concrete.

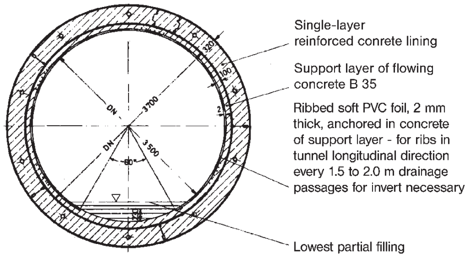
Internal linings can be of 2 to 3.5 mm thick soft PVC foil (Fig. 2-148 a,b) or hard PVC profiles (Fig. 2-148 2 c), of PE panels 2 mm thick for subsequent installation, of hard PVC panels or polypropylene panels 6 to 8 mm thick (Fig. 2-148 d), of V4A corrugated sheet metal 1 mm thick, composite constructions of cast-in stoneware or plastic pipes (Fig. 2-148 e) or also coatings of solvent-free two-part synthetic resins with average thickness of 3000 µm for subsequent application in reinforced concrete shafts. All steel components (like pipes, sluices, valves, support construction) must have surface protection. Stainless steel should be specified for steel accessory elements like ladders, guide rails, fixing profiles, balustrades, handrails and sealing bars. The surfaces of ductile cast iron pressure pipes should also have surface protection, for example by coating, in sections of pipe runs, which can be empty during pumping pauses. In a pressurised sewer pipe in Hamburg, the internal surfaces of the pipes and fittings made of ductile cast iron were coated with epoxy resin with a layer thickness of 600 µm, the first such application in Germany.

As the corrosion protection should be continuous, the detailing of the joints should be paid particular attention. The joints of pipes for pipe jacking have an external secondary seal for the construction state and an inner primary seal for operation.

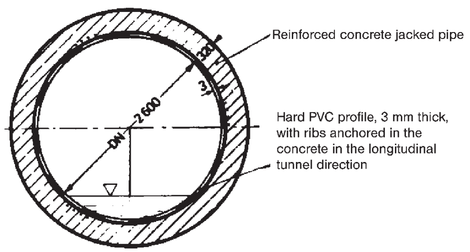
Pipe connections. The pipe connection is an essential construction detail of pipe jacking pipes. The pipe connection must function in the completed state, i. e. in operation, and also during the jacking process with the angles of deviation, which may occur, above all when jacking below the groundwater table. At the same time, the connection also has to transfer the jacking forces during pipe jacking and also resist the angles of deviation and shear forces resulting from steering.



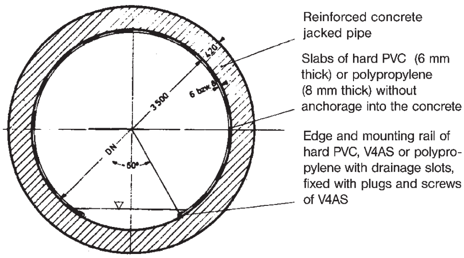
a) Lining of ribbed soft PVC foil installed during the production of reinforced conrete pipes for jacking



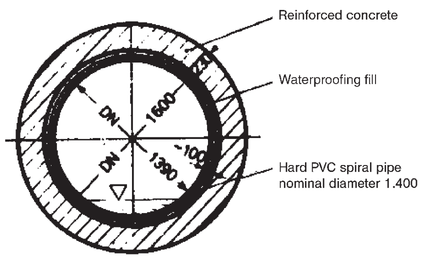
b) Subsequent lining of ribbed soft PVC foil on a support layer of concrete



c) Lining of ribbed PVC profiles in the production of reinforced concrete pipes for jacking



d) Subsequent lining of reinforced concrete pipes for jacking with panels of hard PVC or polypropylene



e) Subsequent lining with inliners of hard PVC spirals, stoneware or polyester resin conrete pipes

Figure 2-148 Possible methods of lining the inside of sewer tunnels

The jacking forces are transferred from pipe to pipe through inserts, which have to both compensate for any unevenness and also resist high compression stress without large transverse strain. While rubber and plastic inserts lead to spalling of the edges of the face surfaces, timber inserts have long proved successful (without knots). The pipes are guided by external sleeves of material capable of resisting tension, which can either be loose or permanently installed, typically steel guide rings of normal steel S235 JR. For the smallest cross-sections, stainless steel or glass fibre-reinforced plastic are also being tried. The seals are normally the usual sliding rings or circular section rings of synthetic elastomers.

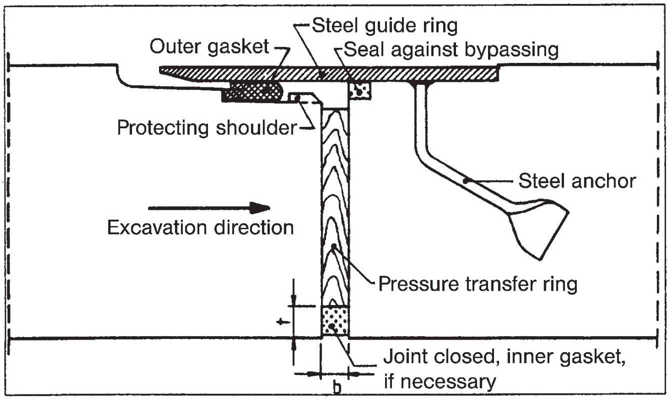


Figure 2-149 Principle of a pipe connection with fixed guide ring for concrete, steel fibre concrete and reinforced concrete pipes [18]

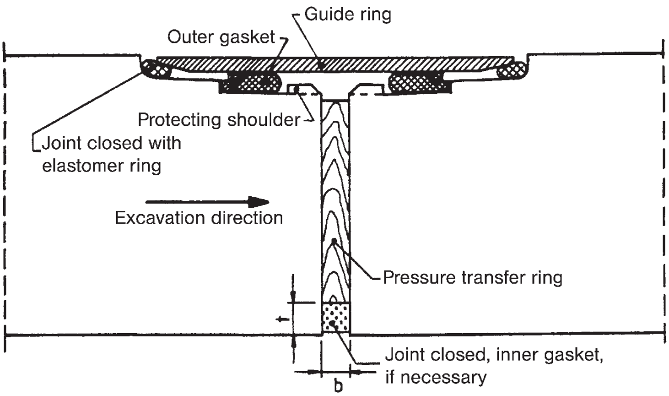


Figure 2-150 Principle of a pipe connection with loose guide ring for concrete, steel fibre concrete and reinforced concrete pipes [18]

For larger accessible pipe jacks, particularly of larger section sewer pipes, a primary seal is installed on the inside and a secondary seal on the outside of the pipe wall (Fig. 2-149 and Fig. 2-150). The secondary seal in this case has the main task of preventing groundwater and liquefied soil from penetrating into the joints or into the sewer. The primary seal, which is installed with great care after the pipe has been jacked, serves to prevent the foul water penetrating into the joint or into the surrounding ground. For smaller inaccessible sections, it is not possible to install an internal seal after jacking and this has to be taken into account in the detailing. In tunnels for the transport of liquid or gas, the seal then has to undertake both tasks, preventing water from the outside entering the tunnel and preventing the foul water escaping.

Mastic material. The joint is, according to DIN 52460 02.00 “Sealing and glazing – Terms”, a space between construction elements that is intended or unavoidable due to tolerances, and has to be detailed and sealed in accordance with the constructive requirements. The sealing of joints, which has to function under all possible loadings and exposure, is normally provided by sealing profiles or mastics.

Sealing profiles are prefabricated, elastically deformable bands or strips (mostly of synthetic rubber), which are laid in the joint before erection or pressed into the joints of the completely erected structure. The sealing effect is based on the pressing forces of the adjacent elements in both cases.

Mastics are sealing materials, which are inserted into the joint space in a plastic condition and harden or set in the structure after insertion. In the joint, they set and seal the joint through adhesion to the sides of the joints. Hardening at the back of the joint should be prevented, as otherwise even the slightest joint movement could lead to tearing of the mastic. The requirements to be complied with by mastics used in sewer and tunnel construction are not the subject of a specific standard, but in some cases exceed the requirements laid down in DIN 18 540 T 1 and T 210.73 for mastics used in general building. Particularly for the joints of sewer mains and pipes, not only absolute waterproofing should be specified to prevent contamination of groundwater but the penetration of water or liquefied soil from outside must also be prevented.

The following actions have to be considered for the sealing of joints in sewer mains and tunnels:

Chemical and biological attack can arise from the effects of climate and soil, as air, gas, change of liquid and temperature, groundwater, aggressive materials, roots, micro-organisms.

Mechanical action resulting from soil mechanical phenomena, the installation process and the structure itself and any dynamic loading from traffic. The mechanical actions resulting from movement of the construction elements and water pressure mostly affect mastic materials as shear and tension at the adhesion surface and in the mastic itself (Fig. 2-151).

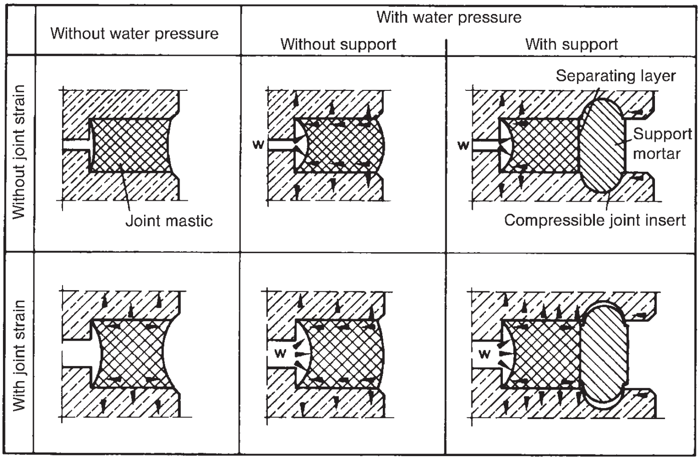


Figure 2-151 Mechanical actions on mastics due to joint movement and water pressure [105]

A mastic can be described as plastic, plasto-elastic, elastoplastic or elastic according to its proportion of elastic and plastic behaviour (Fig. 2-152). This relationship is derived from the resilience (the value in percent, to which a non-ideal elastic sealing compound returns after being strained), which is determined according to DIN 52458 9.75 for a specified magnitude and duration of loading:

- A resilience of less than 10 % is plastic. Plastic joint mastic compounds are not suitable for the sealing of permanently loaded movement joints. They are endangered by movement, which can lead to tearing (so-called chewing gum effect).
- A resilience of 10 to 50 % is plasto-elastic.
- A resilience of 50 to 90 % is elasto-plastic.
- A resilience of more than 90 % is elastic.

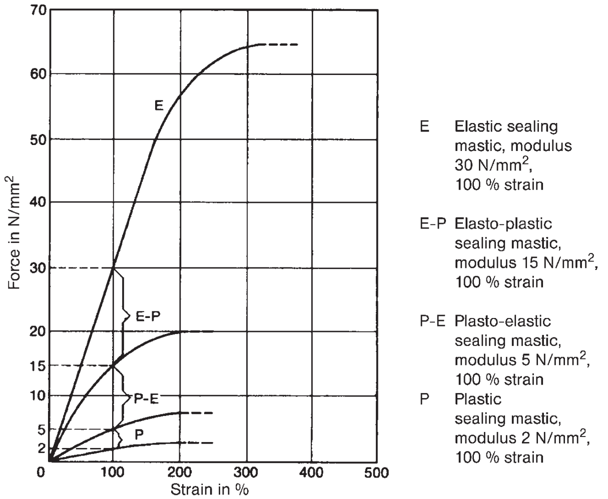


Figure 2-152 Categorisation of mastic compounds according to their stress-strain behaviour [139]

Joint mastics are normally named and differentiated according to the chemical composition of their raw materials, from which significant statements can be made about their properties and features. However, it should be noted that many of the available compounds contain inactive fillers, which could also influence the properties. According to the chemical basis of the raw materials, the types are polysulphides (also known as thiokols or thioplastic, SR), polyurethane (PUR), elastified epoxy resins (EP), silicon rubber (Si), butyl rubber (IIR), polyisobutylene (PIB) and polyacrylate. Only polysulphide and polyurethane mastics are considered here because the other types cannot be used for sewer mains and tunnels due to the following reasons:

- Plastic or plasto-elastic mastics are pressed out of the joint by water under pressure. These compounds also have a small plastic strain capacity and are also much inferior to other elastic mastics in their other properties.
- One-part jointing mastics with elastic behaviour are unsuitable due to their long hardening time. In a joint 24 to 32 mm deep, they need many weeks or months in a sewer or tunnel until they reach their final elastic state.
- Elastified epoxy resins are still being developed as a flexible sealing compound. The elastification of epoxy resins, which are otherwise rigid after hardening, through the addition of flexibilisers (rubber flour, asbestos fibres, quartz flour) is not yet possible without disadvantages for the other properties.

Polysulphides. Elastic sealing materials with plastic components are mostly used. The setting process is catalytic hardening, and lasts two to seven days depending on temperature. Preparation of the jointing surfaces with an appropriate primer is almost always specified. The practical movement capacity is normally over 10 % of the joint width. The lifetime is given as 20 to 25 years, derived from experience in building. In tunnelling, the lifetime can be expected to be longer, as there are no climatic influences like rapid changing of temperatures, humidity and UV radiation. The chemical resistance is very good. Polysulphide compounds do not, however, have the required stability against water pressure, and have to be additionally supported. They are also unsuitable for sewer construction because they have no resistance to the microbiological processes that occur in normal domestic foul water. Polysulphide compounds are apparently degraded by anaerobic microorganisms, which in the course of time can lead to the total loss of the originally applied mass.

Polyurethanes. The mechanical behaviour is predominantly elastic. The setting process is through the formation of chemical cross-links, and lasts one to seven days depending on temperature. During this time, two-part polyurethane compounds are very sensitive to moisture. Polyurethanes also almost always require the application of a primer. The values given by the manufacturers for practical movement capacity are 10 to 25 %. In this range, joint movement and water pressure can be resisted, although proper adhesion to the sides of the joint and even and correct application have to be ensured. Polyurethane-based joint sealing compounds are characterised by very good wear resistance, resistance to chemical attack and flexibility, even at low temperatures (constraint behaviour at changing temperatures), and also possess good resistance to microbiological attack.

2.8.7 Yielding elements

When convergences are large, solid shotcrete support layers can suffer enormous deformation leading to spalling or destruction of the support. The resistance of a solid support layer creates compression stresses, which are described as ground pressure. The maximum compression strain that shotcrete can accept is about 1 %, so the relatively stiff support can only permit a slight radial displacement. The ground pressure from the deformation of squeezing ground, however, only declines slowly.

A tunnel support layer with yielding elements, on the other hand, enables controlled yielding of the layer. Key developments for the application of yielding elements were made in Austria [323a], [259a], [291a]. The ground pressure is redistributed into more distant zones of the rock mass. The initial strength of the shotcrete is therefore exploited without overloading the young shotcrete. With increasing deformation and concrete age, the resistance of the yielding elements should rise. This regulates the final deformation of the support layer and the necessary support resistance can be provided by the shotcrete layer.

The yielding elements are installed in longitudinal slots within the shotcrete lining. These are distributed around the tunnel perimeter, as is shown in Fig. 2-153 for the top heading of the second drive of the Tauern Tunnel.

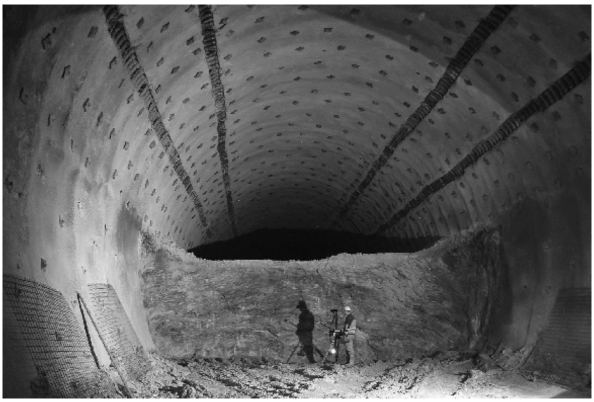


Figure 2-153 Arrangement of yielding elements in the top heading (Tauern Tunnel).

At the time of publication of this edition, three types of yielding elements are available on the market. The yielding system LSC (Lining Stress Controller) from Dywidag Systems International (Fig. 2-154 a), which consists of a group of steel tubes connected by head-and baseplates. When the elements are compressed, a rotationally symmetrical cylinder buckling effect is exploited. Guide tubes arranged concentrically with the yielding tubes serve to limit the buckling developing outwards and inwards and thus optimise the working curve. The beam-shaped yielding system hiDCon (High Deformable Concrete) from the company Solexperts (Fig. 2-154 b) consists of a high-strength concrete matrix with porous aggregates like for example gypsum or glass foam granulate. The necessary coherence of the matrix is provided by special reinforcement layers in the form of plates, reinforcing rings and the addition of steel fibres. The Wabe yielding element from Eisenhütte Heintzmann, Bochum (Fig. 2-154 c) consists of circular hollow steel profiles, which are connected in layers by intermediate plates. The cavities in the structure between the connecting plates create space for deformation, so that the yielding of the yielding element approximates to the sum of the internal diameters of the round steel profiles. If required, extra steel profiles can be inserted subsequently into the cavities of the yielding element.

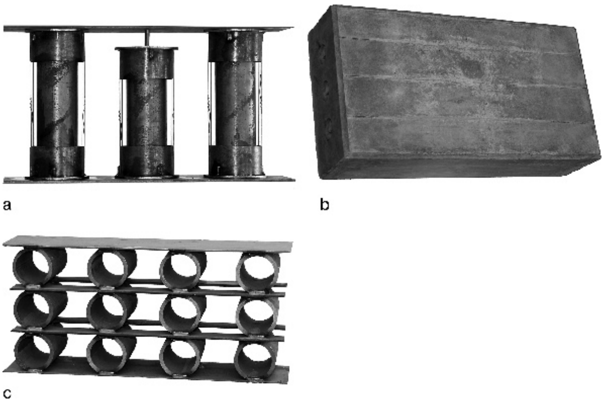


Figure 2-154 Available yielding elements
a) LSC yielding element
b) hiDCon yielding element
c) Wabe yielding element

In tests at the Chair of Tunnelling and Construction Management at Ruhr-University Bochum, the various yielding elements were tested under the same conditions in various installation situations [269a]. As an example, Fig. 2-155 shows load-deformation curves for the three elements. The curves can be adapted to suit the requirements of a particular project through variation of the geometry and construction of the relevant element, which applies for all types of element.

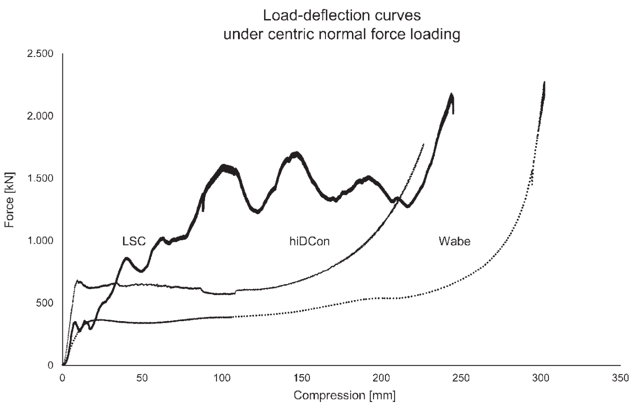


Figure 2-155 Examples of working curves for yielding elements [375a]

3 The classic methods and their further developments

3.1 General

The classic methods of tunnelling were no less audacious than modern methods and the experience of the engineers of the time was at least as good. Timber and stonework were the support materials available at the time [162]; steel and concrete only arrived later. A study of the specialist literature from the turn of the 19th to the 20th century shows quite clearly that the fundamentals of modern tunnelling were already well understood. Anyone who has concerned themselves with the career of Ržiha would have to agree that the most important of the 21 features of the NATM given by L. Müller had already been included in the art of tunnelling by Ržiha and later by Heim and Andreae [122], [302], [303], [320], [386]. A study of the literature also shows that the basic processes were already almost completely understood in the 20th century. The only actual innovations have been the use of new materials like shotcrete, steel fibre shotcrete and the use of high-strength steels for anchoring. The continued development of drilling, blasting, mucking and surveying have also made it possible to minimise deformation of the rock mass, which has led to lighter and cheaper support.

The classic tunnelling methods with masonry lining had a severe disadvantage regarding contact with the rock mass. Either during or after the building of the masonry, the gap between the masonry and the rock mass was filled more or less "completely" (Fig. 3-1). This work had to be carried out under very poor accessibility conditions, and it was often not even possible to remove the timber installed as temporary support. It was not possible to prevent such cavities collecting water like a drain, transporting fines out of the joints and leading to rock falls. The result was the development of additional long term loosening with the associated high ground pressures, which had to be considered in the design calculation assumptions. At the end of the 19th century, concrete became established as a support material, and careful placing with subsequent filling of the cavity above the crown largely solved the problem.

In mining, backfilling behind the support is still practised. The support normally consists of steel arches with lagging of timber, concrete planks or mesh. The gap to the rock mass is then back-filled with excavation spoil with more or less effective contact to the rock mass.

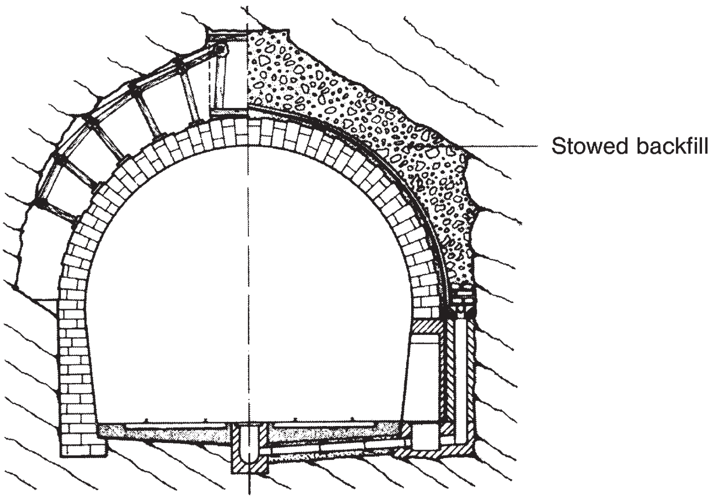


Figure 3-1 Backfilling behind the masonry lining of an old tunnel [320]

The application of shotcrete as a support material introduced a process, which could provide temporary support in direct contact with the excavated rock. The factor of time, which has an important influence on rock deformation and the development of pressure, could be greatly reduced. This enabled excavation is difficult geological conditions, even of the full face. With the first application in Frankfurt clay, this method of support later even became a normal process in soft ground [172]. Compared to the conventional processes used previously, this was an enormous step forward. Nevertheless, when the several stages of installing shotcrete support are considered (preliminary sealing, installation of the first layer of mesh, installation of steel arches, second layer of mesh, spraying), it can be seen that the process still has potential for faster working. The relatively long hardening time of the concrete should also be considered. Even with accelerator admixtures, early strengths of about 5 N/mm^2 can only be guaranteed after 6 to 8 h. The time factor can be favourably influenced with the use of steel fibre shotcrete, which also provides high early support resistance. Steel fibre shotcrete has useful properties for the further development of tunnelling processes.

The following section describes the most significant basic principles of classic tunnelling methods through the example of former tunnels. The technological history is also continued until today. It is clear that the classic methods still have their areas of application. The differentiation of the various terms is sometimes difficult. The usual names according to countries are used here combined with the most important process principle in each case, like German or core method, Belgian or underpinning method.

The listing begins with full-face excavation. The excavation of the entire section is certainly the oldest process. Ground behaviour and excavation method were and still are closely related. If the excavation method is suitable for the ground conditions, then it only remains to decide the most economical method of operation using modern machinery. The section “Full-face excavation” does not cover shield tunnelling or mechanised full-face excavation, which are dealt with separately.

3.2 Full-face excavation

The decision whether to excavate the full face (Fig. 3-2) is based on three questions:

1. The stand-up time of the rock, depending also on the size and shape of the cross-section.
2. The time required to install temporary support depending on the stand-up time of the rock. If the rock will stand up for a longer time, then the support can even be omitted, although head protection for the miners and machinery should always be considered.
3. The equipment used, like drilling jumbo or platforms, roadheaders or excavators, which have to reach the entire face, but also the machinery used for muck clearance in order to load and transport away the muck. The minimum profile for economical driving is 5 m^2. Smaller sections hinder the work of miners and machinery so that the cost rises despite the smaller excavation quantity and requirement of support materials.

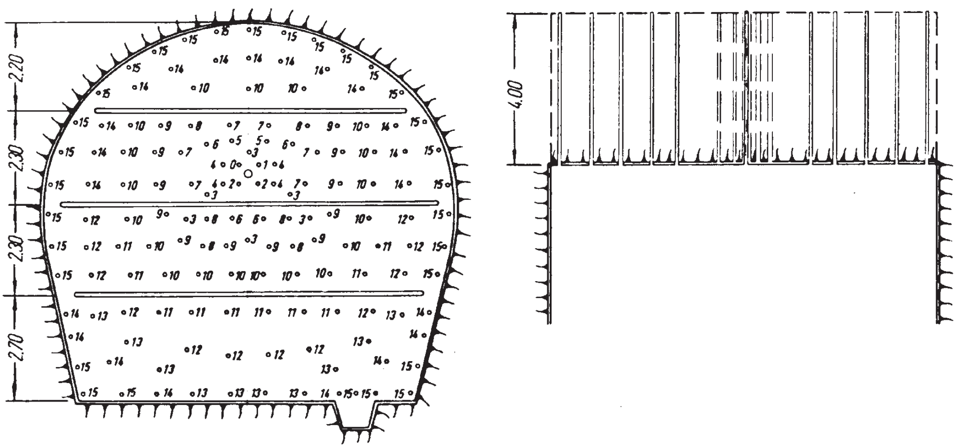


Figure 3-2 Full-face excavation. Blasting pattern for the Mont-Blanc Tunnel (French side), 1964 [51]

Advantages:

– No multiple redistribution of stress and thus more gentle to the rock mass.
– Free working space, mechanical excavation can be used.
– More efficient working is possible, leading to shorter construction times, and also gentle on the rock mass due to the quicker installation of support.
– Clear advance making good organisation and economy possible.

Disadvantages:

– Can be dangerous in rapidly changing or worsening rock conditions and in case of water ingress.
– Poor adaptability if mechanised.

3.3 Partial-face excavation

The division of larger sections was formerly often necessary to overcome the geological conditions. The support materials available at that time, mostly consisting of timber frame sets with lagging, only permitted small partial excavation sections. In contrast, sections are now divided when it permits more economical use of machinery. The excavation of a partial section is always followed by the installation of temporary support. This leads to extra cost, an effect on the rock mass, constant rebracing and typically also to loosening and increased ground pressures. Three reasons can lead to a decision to excavate a section in parts:

1. The stand-up time of the rock mass is not sufficient for the excavation of the entire section.
2. The time required for the installation of temporary support is not sufficient to support the entire excavated section.
3. Machinery like drilling jumbo or platforms, roadheaders or excavators cannot reach the entire face, or the machinery used to clear the spoil cannot reach the entire pile of excavated material and thus requires partial sections.

3.3.1 Bench excavation

Partial-face excavation for the third reason was formerly practiced both from the crown down and from the invert upwards. Excavation methods starting from the crown form the basis of bench excavation, which changed continuously with the development of support measures and machinery. The advantages and disadvantages of bench excavation (Fig. 3.3) are almost identical to those listed for full-face excavation, except that an appropriate selection of machinery can improve adaptability (Fig. 3-4).

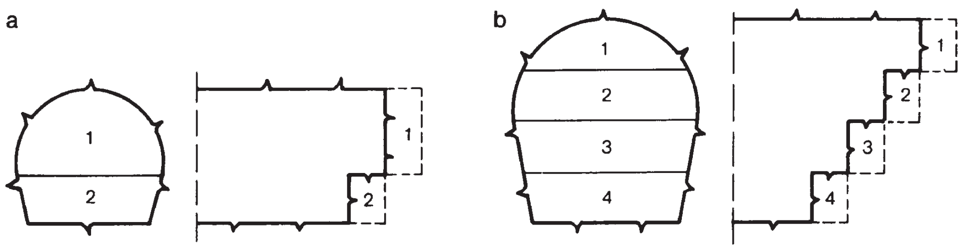


Figure 3-3 Bench excavation sequences for the excavation of small sections (a) and larger sections (b)

3.3.2 The Belgian or underpinning tunnelling method

The Belgian method was first used in 1828 for the canal tunnel at Charleroi in Belgium. Later, this method was used successfully to drive a large number of important Alpine tunnels like Mont Cenis (12.5 km) and the old St. Gotthard (15 km).

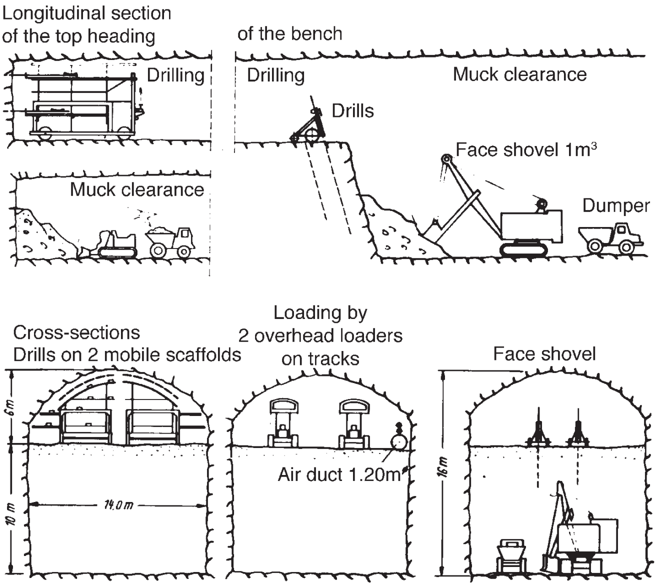
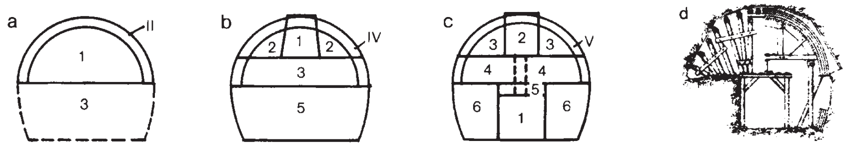


Figure 3-4 Bench excavation of larger sections. Possible machinery applications [251]



a For short and small tunnels with medium cross-sections: Full face excavation of the top heading

b For larger cross-sections and short distances

c For large cross-sections and long distances

d Classic application at the Hönebacher Tunnel, from F. Řziha [302]

Figure 3-5 The Belgian method [133]. Arabic numerals give the excavation sequence and Roman numerals the support sequence

The excavation starts with a top heading (Fig. 3-5). Then the support is installed (formerly a vault as the final lining) starting from the spring lines. Once the support has been installed, there are various methods of excavating down to the invert. Normally, a central trench is cut and the material below the vault is removed in sections. The excavation below the heading vault is carried out from a continuous trench or from cross-cuts, by sinking individual shafts or subsequent digging of half or all the bench, and abutments are built to underpin the vault as shown in detail in Fig. 3-6. There are many known variants of the top heading or pilot tunnel.

Advantages:

– The early support of the crown reduces loosening.
– Adaptation is possible to changing geological conditions.
– Continuous operation is possible, thus flow production and cycle planning.

Disadvantages:

– High abutment pressures in the spring lines leading to a danger of settlement before underpinning is complete, particularly in loose or plastic ground.

– Very susceptible to side pressure.
– The invert is closed very late.
– Risk of damage to the heading when blasting the benches.

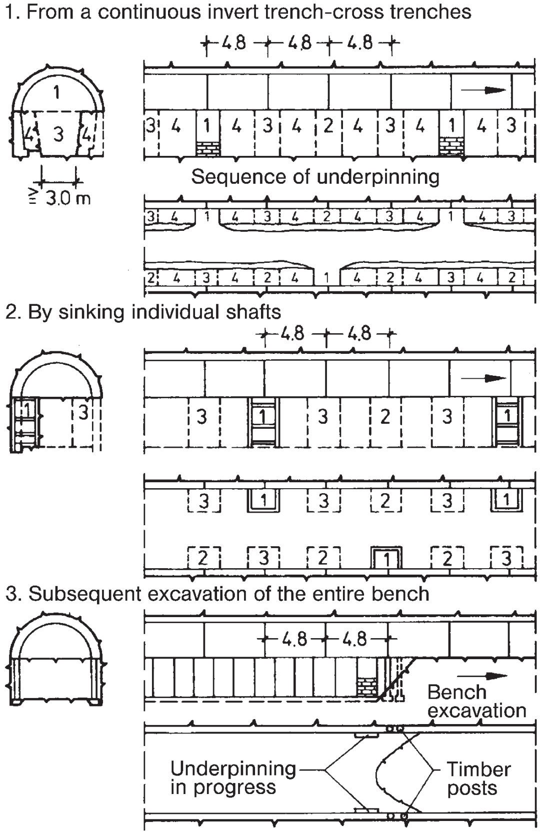


Figure 3-6 The Belgian method. Working sequences to underpin the crown vault [133]

Modern applications of the Belgian method

Since its development, the Belgian method has often been used in variable to slightly squeezing rock [208]. One further development is the Kunz support system, which consists of excavation arches sitting on the side walls, over which lagging plates are forepoled. The arches are supported with spreader beams on the heavier inner arches, which are supported from the ring sill with vertical round timbers. The crown vault is then constructed to be as stiff in bending as possible with continuous reinforcement cages at the spring lines (Fig. 3-7).

Recent developments in shotcrete and anchor support can remedy some former disadvantages. Any advance excavation of a top heading followed by the installation of support can in principle be assigned to the Belgian method of tunnelling. In order to resist the reaction forces at the feet of the top heading for a longer time, the vault is often supported on thickenings called elephant's feet. Under difficult conditions, like passing directly below buildings, the sides can be stabilised by grouting or ground freezing. Examples of such further developments are described under shotcrete construction in Chapter 4.

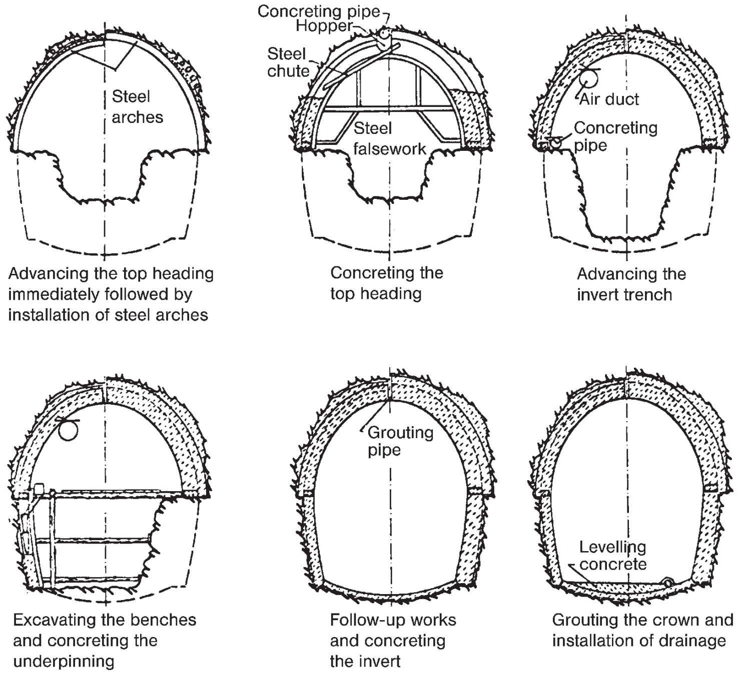


Figure 3-7 Belgian method. Rail tunnel in Thailand, 1966 [208]

3.3.3 The German or remaining core tunnelling method

The German method was originally developed in France in the Tronquoy Tunnel in 1803 and the Pouilly Tunnel in 1824 with the essential feature being a central core left standing, and is the oldest method in the literature. With the construction of the Königsdorfer Tunnel in 1837 and the Triebitzer Tunnel in 1842, the method was perfected under extremely difficult ground conditions as an alternative to the English method of the time. Several further famous tunnels were constructed by the German remaining core method, like the Semmering railway between 1848 and 1851 among many others.

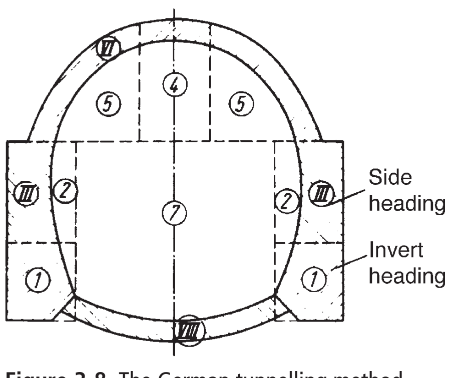


Figure 3-8 The German tunnelling method. Sequence of excavation and support [203]. Arabic numerals denote the excavation sequence; Roman numerals denote the support sequence

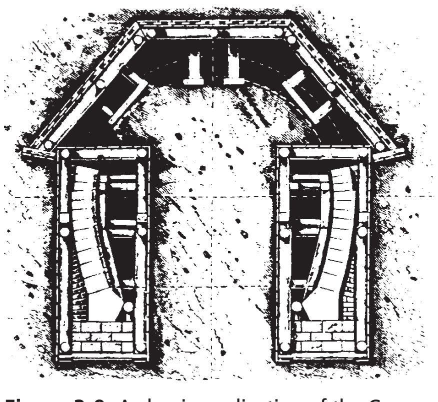


Figure 3-9 A classic application of the German method in the Triebitzer Tunnel 1842/44, from F. Ržiha [302]

Small and manageable sections are driven around the perimeter of the tunnel section, normally starting at the bottom and continuing upwards. The final lining is installed in these sections. This creates a vault in these sections, which is only connected later (Fig. 3-8). The core (No. 7 in Fig. 3-8) remains standing until the vault (VI) can bear loading and serves to support the support work and also to support the face, which makes the area of the face considerably smaller and thus supports it better (Fig. 3-9). There are many known variants with advance pilot tunnel in the invert or crown.

Advantages:

– The invert heading can be used as a pilot tunnel for the investigation of the ground conditions and also to provide relief drainage for the further excavation.
– Low settlement due to the support provided by the core.
– Less susceptible to side pressure due to the support provided by the core.
– Core supports the face; face support can be omitted.
– The vault over the crown is built on solid abutments, which reduces settlement.
– Economic core excavation.

Disadvantages:

– Stresses in the rock mass are not contained and the rock mass is loosened due to any protective shells that could form in the rock mass being cut through.
– Frequent underpinning and thus replacement of bearings lead to increased risk and settlement.
– The invert is closed later, unless the invert is excavated in advance.
– Expensive excavation of small sections, and installation of support.
– Slow advance rate.

Newer applications of the German method. Newer applications between 1960 and 1980 have also been successful but were all under difficult ground conditions (Fig. 3-10 and Fig. 3-11)

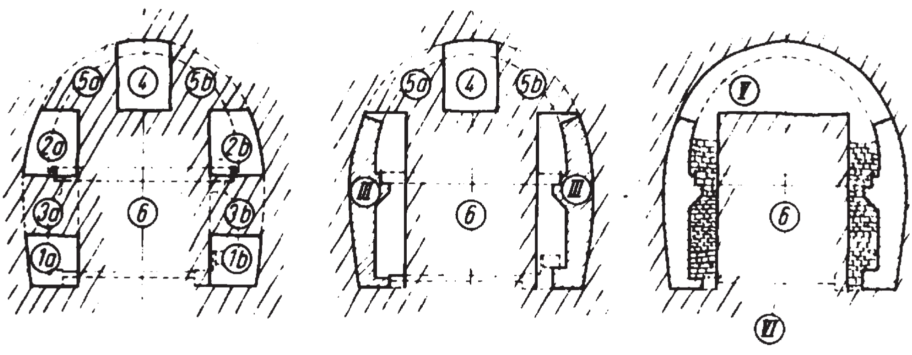


Figure 3-10 German tunnelling method: Kiesberg Tunnel in Wuppertal 1967. Arabic numerals denote the excavation sequence; Roman numerals denote the concreting sequence [26]

L. Müller [260] had a poor opinion of the basic principle of the German tunnelling method, which is understandable regarding earlier applications, but to some extent also regarding more recent applications. More recent developments have only lessened the stated disadvantages. Chapter 4 “Shotcrete tunnelling” describes recent developments with examples.

The range of possible applications of the German tunnelling method is shown in Fig. 3-12.

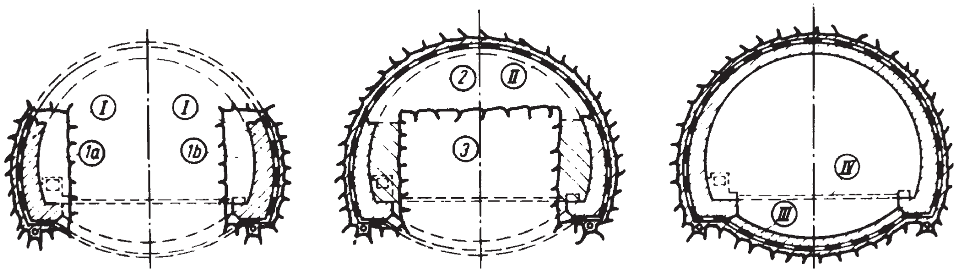


Figure 3-11 German tunnelling method: Hölzern Tunnel near Weinsberg, 1970. Arabic numerals denote the excavation sequence; Roman numerals denote the support sequence [27]

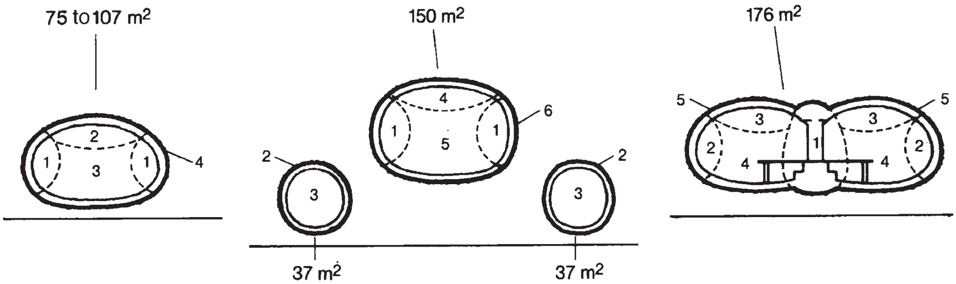


Figure 3-12 Core tunnelling method. Examples of multi-track running and station tunnels [368]. The final lining is normally installed later after the completion of temporary support

Pipe screens can also be considered as a core tunnelling method. These consist of jacked pipes arranged so close to each other that they form a canopy (Fig. 3-13). The core is then excavated under the protection of this pipe screen. Pipe screens can be installed longwise or crosswise. The diameter of the pipes formerly had to be large enough to permit a man to work inside, but the development of mechanised driving of small tunnel sections means that smaller sections can also now be driven economically.

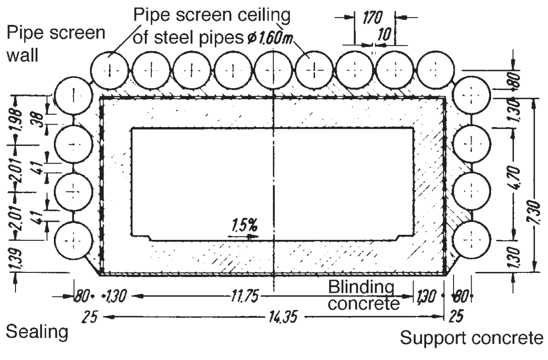


Figure 3-13 Core method, longitudinal pipe screen. Second Gildehof Tunnel in Essen, 1969 [203]

In addition to the principle of core tunnelling method, the construction of a longitudinal pipe screen is also a feature of the longitudinal beam or English method of tunnelling. The pipe screen often has the purpose of temporary support for the excavation of the core. Under the protection of the pipe screen, the final lining can then be constructed in sections.

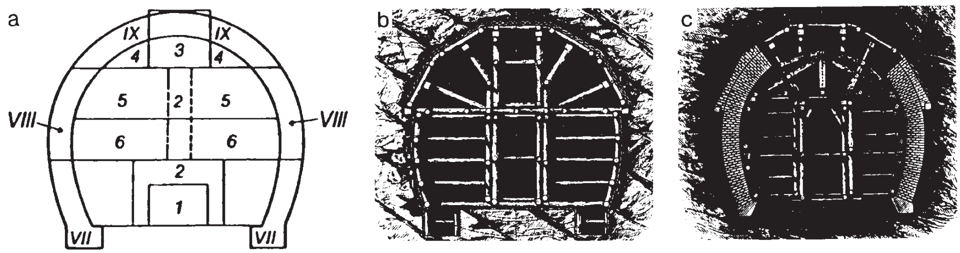
3.3.4 The Austrian or upraise tunnelling method

The (old) Austrian tunnelling or upraise method was also described as rafter or crossbeam method due to the type of timbering, in contrast to the longitudinal beam (English method, section 3.3.6). This process was first used for the construction of the Oberau Tunnel in the railway line from Leipzig to Dresden in 1837. In 1839, the method was used in Austria for the building of the Gumpoldskirchner Tunnel, and after 1848 above all for Alpine tunnels, where it was derived from the basic principles of the previously developed English method. There are numerous variants of excavation sequence, and further development has continued until today. The following terms can be differentiated:

1. Old Austrian tunnelling method.
2. Newer Austrian tunnelling method.
3. New Austrian tunnelling method (NÖT, NATM).
4. Newest Austrian tunnelling method (with deformation slots).

The last two methods belong to the modern processes, which are handled in detail in sections 3.3.5 and 4.2.1.

The main characteristic is that the entire section is excavated in slices from the crown to the invert – in few or many partial sections – and temporarily supported in sections until the entire section has been excavated and supported (Fig. 3-14). Then the permanent lining (formerly masonry) is installed from bottom, i. e. from the invert abutments, upwards to the crown. The invert vault is mostly not provided or inserted later. The many operational variants are interesting (Fig. 3-15). The reason for this is the desire to develop as many starting points for excavation as possible in order to reduce the construction time by enabling as many men to work as possible. In this way, remarkably short construction times were achieved compared to modern tunnelling.



a Sequence of excavation and support
b, c Classic application at the Main Semmering Tunnel fom F. Ržiha [302]

Figure 3-14 The old Austrian tunnelling method. Arabic numerals denote the excavation sequence, Roman numerals denote the support sequence [302]

Advantages:

- Pilot tunnel is possible for early investigation of the ground and water drainage.
- Good possibilities to increase the number of excavation locations and to detach sequential working steps.
- The excavation in slices makes complete ring building possible, even with invert stiffening, which was however seldom practised.

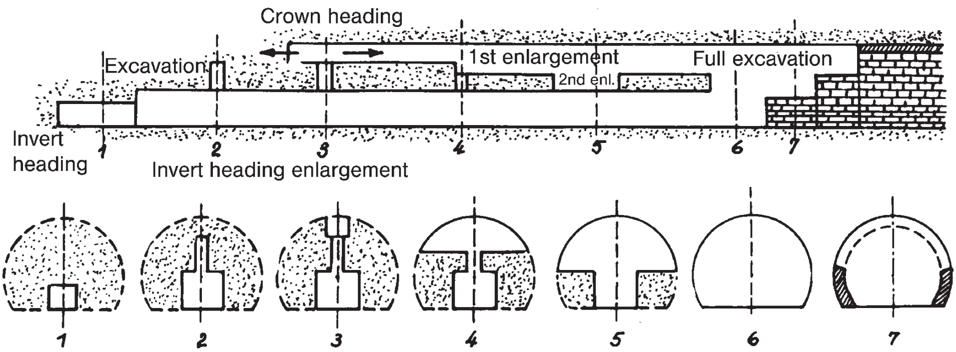


Figure 3-15 The Austrian tunnelling method. Operational method with top heading [133]

Disadvantages:

– Restricted working conditions due to the heavy use of timbering and falsework.
– Susceptible to settlement during rebracing.
– The rock mass is supported on the timbering for a long time, which can lead to settlement.
– The constant alternation between excavation and support work leads to slow advance rates at one excavation location, no flow production is possible.

The further development to the "newer Austrian tunnelling method" consisted of using iron for the essential support elements. Above all F. Ržiha [303] had a large part in this development with the "new method" of ring construction. Later, the ring construction idea was further developed to the Kunz support system. The Kunz support system was often only used in the crown, in which case the system has to be assigned to the underpinning methods. No more recent applications of these old and newer Austrian tunnelling methods as further developed processes that are still practical today are known to the authors or have been superseded by shotcrete methods. All construction methods using shotcrete will be dealt with in Chapter 4 "Shotcrete tunnelling".

3.3.5 The New Austrian Tunnelling Method

The New Austrian Tunnelling Method is abbreviated NATM (German: NÖT). The construction and operational method is illustrated in Fig. 3-16.

In Germany, this method is assigned to the shotcrete construction methods, but in Austria, the method is also often described as cyclical tunnelling or process [268] in contrast to continuous tunnelling, which denotes mechanised tunnel drives.

The NATM in its original sense, that is according to the patent of L. v. Rabcewicz [291], still included the basic principles of the "old and newer" Austrian tunnelling methods: the entire cross-section is excavated and temporarily supported, and the inner lining is only installed after a relatively long time. The support formerly consisted of timber framing; today shotcrete, rock bolts and steel beams are used. The NATM as understood today depends on three essential construction elements. Shotcrete and rock bolt technology had already been developed much earlier.

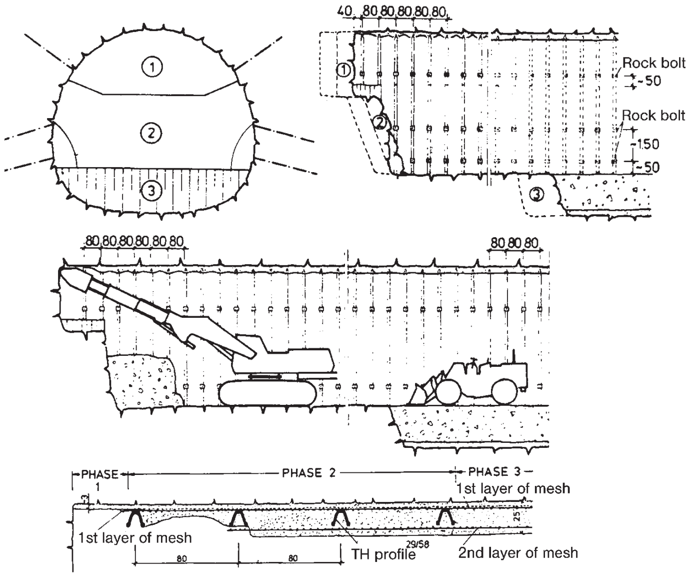


Figure 3-16 NATM. Construction and operational method for tunnelling below buildings in Bochum, 1971. Bottom – sequence of shotcrete works [350]

Shotcrete

– Ensures full contact of the support to the rock mass without cavities.
– The support not only has the function of an independently load-bearing construction, but also improves the excavated surface of the rock mass.
– A composite construction is created "rock mass – concrete" with most of its structural action in the rock mass.

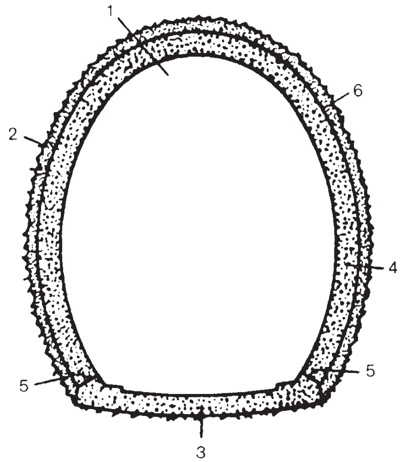
Rock bolts

– The composite action of the rock mass and the concrete is improved by the installation of rock bolts (dowelling and pulling together).
– The influence of inhomogeneities like bedding and jointing can be reduced.
– The structural ring of the composite construction can be extended further into the rock mass.
– Loosening due to the excavation process (blasting) can be somewhat reduced.

Steel or lattice beams

– Head protection after installation.
– Immediately load-bearing after spraying.
– Ring reinforcement of the shotcrete layer.
– Useful for geometrical and surveying requirements.

The philosophy of the NATM is based on a patent of L. v. Rabcewicz [291] from 1948. A temporary thin-walled support should deform to relieve the ground pressure and move it deeper into the surrounding rock. The final lining is thus subjected to less loading and can be installed later as a much thinner construction (Fig. 3-17). The deformations are observed by measurement and the results can be included in calculations and structural verifications [288]. The text of the patent states a few centimetres.



1 Excavation
2 Relatively weak support vault
3 Invert
4 Load-bearing vault
5 Abutments in the invert to support the load-bearing vault
6 Waterproofing

Figure 3-17 The new Austrian tunnelling method according to the patent of L. v. Rabcewicz [291]

The arguments for the NATM have remained of a largely philosophical nature and cannot be considered as a single unit, as they would sometimes be contradictory. The 21 basic principles as stated by L. Müller [260] are:

1. The essential load-bearing element of a tunnel construction is the rock.
2. In order that the rock can bear the redistribution of mass (and stress) caused by the opening of the cavity, it must be left in a condition, in which it does not lose its original strength (or only within unavoidable limits).
3. As unloading deformations are much worse for the rock to bear than additional loading, uniaxial and biaxial stress states must be avoided, even temporarily.
4. As a consequence of these requirements, rock deformation must on the one hand be permitted to such an extent that resistance is provoked by shape alteration around the artificial cavity and a load-bearing ring is created in the rock, which surrounds the cavity as a protective zone and prevents the rock moving further into the cavity; on the other hand, the displacement of the rock must be limited so that its deformation does not reach an extent or quality, which could lead to loosening due to overloading and thus to softening and loss of load-bearing capacity.
5. The purpose of the excavation support has this purpose and no other, to support the excavated surfaces. Its function is to control deformation of the rock through the provision of a reactive support resistance or by developing an active support pressure as described above. It does not essentially have to carry rock that has lost its own load-bearing capacity (which was formerly required), but it has the main purpose of preserving the rock in a load-bearing condition and protecting it from avoidable loosening and softening.
6. In order to optimally fulfil this aim, the support must be installed at the right time – which means, not as early as possible, but not too early and not too late – and become effective (an effect, which is mostly only gained after further deformation of the innermost shell of rock has taken place). The time that the rock needs for its deformation must be exploited and selected so that its support resistance prevents softening deformation of the rock but the formation of a protective zone is aided.
7. To this end, the specific time factor for the rock must be taken into account and the rock must be correctly assessed in this regard.

8. This assessment can be carried out through preliminary tests in the rock encountered or by measurement of displacement and deformation in the tunnel during construction.
9. In order to support the rock, shotcrete is generally used due to its planar action that guarantees the necessary mechanical bond and can be adjusted for speed, mostly in combination with rock bolts and reinforcement mesh, often also with tunnel arches; it does not act as an arch bearing the rock, but essentially to seal the rock as part of a composite construction consisting of concrete, steel and rock. In many types of rock, it is sufficient to use just one type of support like shotcrete with arches but without rock bolts, with rock bolts but without arches or also shotcrete or rock bolts on their own.
10. The shotcrete support, which has a particular structural function in the overall construction due to its strength and form altering properties quite apart from its bond with the rock, is generally applied as a thin layer and remains weak in bending, as only secondary bending stresses should occur in the layer; the relief of bending stresses as the initial deformation occurs and provokes the support resistance is ensured by the behaviour of the layer being as plastic as possible and capable of creep, and suitable composition of the shotcrete.
11. According to the basic concept that an underground cavity can be structurally considered as a thick-walled pipe, the closure of the ring at the correct point in time (invert closure) will ensure that the pipe is actually complete at the time when its structural function is required.
12. The outer layer (in some cases also the anchoring fulfilling this purpose) can be considered as a part of the permanent construction as long as it is not susceptible to being destroyed by corrosion or is protected against corrosion.
13. The time, within which this has to occur, the ring closure time ("invert closure time"), is an essential construction factor and is estimated through tests before starting work in geological situations that are geotechnically difficult to interpret, and controlled and modified through measurements during construction.
14. The design of the tunnel must take into account that we consider it structurally as a pipe, therefore shapes as round as possible – circular or oval – are preferred to avoid stress concentrations, which could occur at corners and notches. For this reason, angled designs of the outer lining at the spring point of the vault and wide abutment foundations should be avoided if possible.
15. Particular attention is also paid to intermediate construction states. In order to limit stress redistribution as much as possible and avoid intersections of protecting layers, excavation should be carried out with as few intermediate stages as possible and full-face excavation should be clearly preferred, or at least full-face excavation with advanced top heading.
16. In order to improve safety or to include an isolating layer, a second inner layer can be provided. This should also generally be kept thin so that it remains as free of bending stress as possible; radial force should be transmitted between the two layers but no friction or shear force.
17. Strengthening of both the inner and the outer layers should generally be provided not by thickening but by reinforcement of the layer; steel arches laid in the shotcrete as reinforcement are particularly suitable for structural strengthening. Strengthening of the entire construction can also be achieved by increasing the number of rock bolts or their length, which reinforces the structural ring in the rock.

18. The stabilisation of the entire system and its safety, the necessity of strengthening or the permissibility of reductions of lining thickness are evaluated according to measurements of convergence of displacement around the tunnel.
19. In order to design the outer layer, measurements are made of concrete stress and the contact stresses between support layer and rock, which serve both to check the structural safety and to provide a preliminary estimate for the design of the following section of the tunnel. In practice, this is dealt with by providing a sufficient number of profile types, which can be used according to the results of the deformation measurements.
20. The design of the inner lining can be undertaken from two points of view: if the outer layer is designed to be so strong that it provides most of the stability of the overall system, it is designed to provide an additional safety reserve. If the outer layer on the other hand was designed so weak that it is only capable of slowing rock deformation until the installation of the inner lining, then the inner lining has to be designed to undertake the remainder of the stabilisation function in addition to the required safety surplus. In case the outer layer has to be expected to rot in the course of time, it should be made as weak as possible and the inner lining should be designed to provide the necessary structural function on its own.
21. The outer support layer – if appropriate also the inner lining – is also protected against external water pressure and flow pressure in the surrounding rock by providing sufficiently watertight drainage.

The basic principles were essentially already understood in earlier tunnelling. The most important of them, including the necessity of closing the invert early, had already been dealt with by F. Ržiha and others before the start of the 20th century [122], [303], [302]. The significance of a deformable thin support layer had been demonstrated by W. Schmidt in 1926 [316], see also J. Spang [337]. The works of L. v. Rabcewicz, L. Müller, F. Pacher and others involved with the new Austrian tunnelling method should however not be underestimated. They introduced an important step in the development of modern tunnelling. Together with new understanding from the young field of rock mechanics in combination with the latest developments in support technology, a movement was created, which has enormously enriched tunnelling.

Today there are numerous methods of tunnelling using shotcrete with rock bolts, which do not belong to the NATM. These are mostly further developments of classic methods, but due to the use of rock bolts and shotcrete they are falsely assigned to the NATM with arguments that are not scientifically founded and comprehensible [166].

Since the awarding of the patent, there has been plenty of material for discussion about the name, the actual innovation and the scientific foundations for it. As the entire theme is very complex and sometimes subjected to national interests, objective discussion is scarcely possible.

The opinion of the author is that the NATM as a process only has an independent existence when the historical development is taken into account. The works of Kovári [166], [167] are interesting on this subject. In contrast to the interpretation of the author stated here that the NATM can be defined as a construction method due to its historical development, Kovári considers the basic principles of structural action and comes to the conclusion that the “thought structure” of the NATM that has been preserved over the years cannot stand up to critical scientific examination.

Numerous works have been produced as part of the search for verifications of the NATM. As examples of these, the works of L. v. Rabcewicz [290], [286], H. Kastner [161], L. Müller [260], F. Pacher [271] and J. Golser [107] should be mentioned. The shear failure hypothesis of K. Sattler (Fig. 3-18) and the failure mechanisms according to G. Feder (Fig. 3.19) are also briefly described [95a], [287], [305].

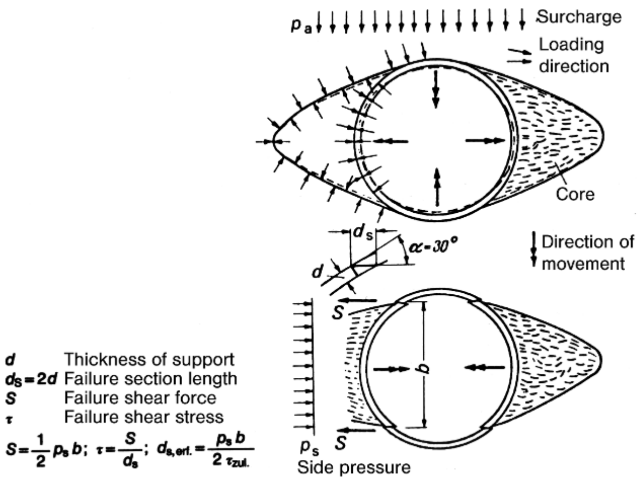


Figure 3-18 Shear failure hypothesis of K. Sattler [305]

K. Sattler demands:

– Movements in the rock mass are to be avoided as far as possible.
– The temporary support should be completed as quickly as possible (ring closure).
– Shotcrete with rock bolts results in distributed loading; point loads are to be avoided.
– Bending deformation and bending tension cracks do not occur, therefore no bending stresses can be present.
– From practical observations, the shear force at failure is considered decisive for design purposes.

G. Feder investigates failure mechanisms depending on the structure of joint bodies and the various depths. The purpose of his work is not only to describe the behaviour to the rock mass but to formulate it in relation to the support pressure or the support stiffness (formerly described as the lining resistance). There follows a description of the failure states:

State 1: There may be a few tension cracks but the sides are still compact.

State 2: A closely limited zone in the sides is squeezed. The areas above and below the squeezed zone remain compact and form "jaws", which "bite" into the squeezed zone as the crown settles and the invert heaves.

State 3: Failure of the side "jaws" and thus final collapse.

States 1 and 2 can be stabilised with suitable support materials, state 3 cannot.

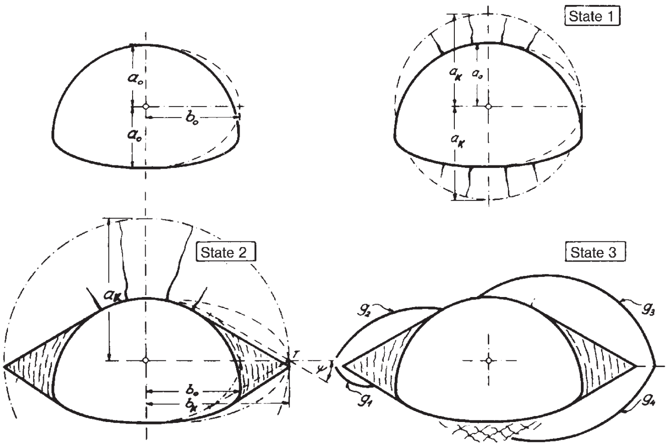


Figure 3-19 Failure states under anisotropic conditions, from G. Feder [95a]

3.3.6 The English tunnelling method

The English tunnelling method, also called longitudinal beam or trestle method, was developed in 1830 for the construction of the first English railway line. Similarly to the Austrian method, the entire cross-section is excavated and temporarily supported, then the masonry is built from bottom to top, although the process only allows the building of 3 to 6 m wide rings (Fig. 3-20). The contrast to the old Austrian method is that the timbering does not consist of rafters or crossbeams but long or trestle timbering.

The advantages and disadvantages correspond to those of the Austrian method but with a serious additional disadvantage, that the length of the crown beam cannot be too short for reasons of stability. This was disadvantageous compared to the Austrian method with regard to the variation of round lengths and thus adaptability to changing geological conditions.

Further developments of the English method were concerned with new materials like steel and concrete. A sort of longitudinal crown beam is still used today, particularly for tunnels near to the surface and buildings on the surface that are sensitive to settlement. All types of longitudinal pipe screens refer to the principles of the English tunnelling method.

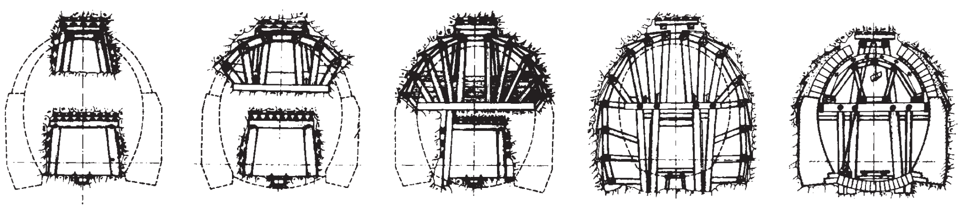
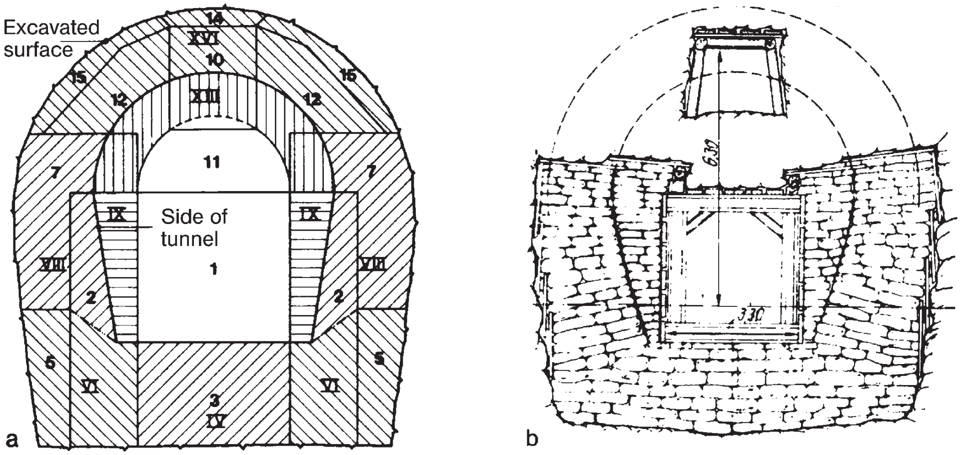


Figure 3-20 The English tunnelling method [320]

3.3.7 The Italian or packing tunnelling method

This method of tunnelling was developed in 1867 for the construction of the railway line Foggia – Naples and improved in 1893 for Alpine tunnels in order to overcome very difficult geological conditions (strongly squeezing rock with water ingress). Excavation took place in sections with immediate installation of the final lining, even inside later cavities (Fig. 3-21). The support material was initially timber, and later steel and materials for the final lining (masonry) were used, which were broken out later after the closure of the ring.



a Sequence of excavation and support
b Classic application at the south side of the Simplon Tunnel 1898 – 1906 [351]

Figure 3-21 The Italian tunnelling method. Arabic numerals denote the excavation sequence, Roman numerals denote the support sequence

Advantages:

– Support can be installed after short lengths when the stand-up times are short.
– Small rock movements can be achieved.
– Lower consumption of timbering materials, but more masonry.
– Adaptation to changing geological conditions is possible.

Disadvantages:

– Very slow advance rates.
– Very high costs.
– No mechanisation is possible.
– Difficult removal of the support materials in the cavity to be used later.

No more recent applications of further developments of the classic process are known. But some individual elements like the temporary packing of the already excavated cavity as support have been repeated in some conventional processes.

The tunnelling method developed by Lunardi [191] in 1980 is primarily used in Italy under the name ADECO-RS (Analysis of COntrolled DEformation in Rocks and Soils). This process is dealt with in section 4.7. The core consolidated in advance certainly shows parallels to the older packing method.

3.4 Classic shield drives

According to its traditional development, shield tunnel driving also belongs to the classic tunnelling methods. Detailed shield processes were designed and described long before the engineering development of the classic tunnelling methods.

The development of shield tunnelling is immediately associated with the name of Marc Brunel. The inducement was the commission to construct an all-year traffic connection across the River Neva in St. Petersburg. The bridge piers suffered heavy damage each year from ice floes from the Ladoga Lake. In his project preparation, M. Brunel developed a tunnel solution, although he had formerly proposed a suspension bridge. He secured his ideas in 1818 with a shield patent. All shield designs from M. Brunel are characterised by division into cells. A worker can work independently in each of these cells with complete protection (Fig. 3-22). While in one method, the cells were permanently fixed in the shield skin and the entire shield skin was pushed forwards with (already hydraulic) presses after the completion of excavation, the cells in the other method were independently movable. All modern closed full shields are based on the first of these methods; the second was never developed to be ready for use, unless the blade shield can be regarded as a further development of the idea.

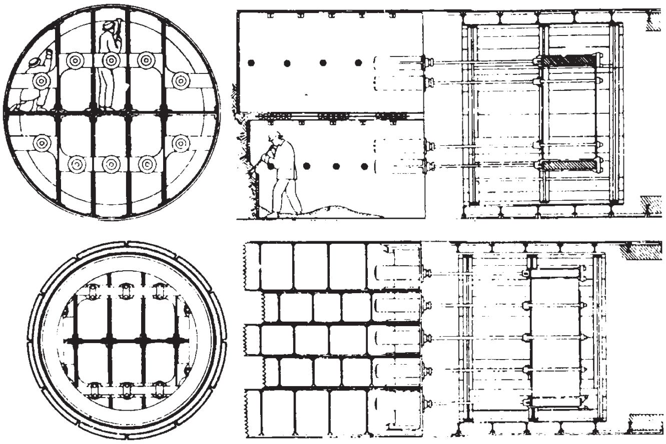


Figure 3-22 Shield from M. Brunel, patented in 1818. The cells are installed fixed in the shield skin (top) or individually movable [339]

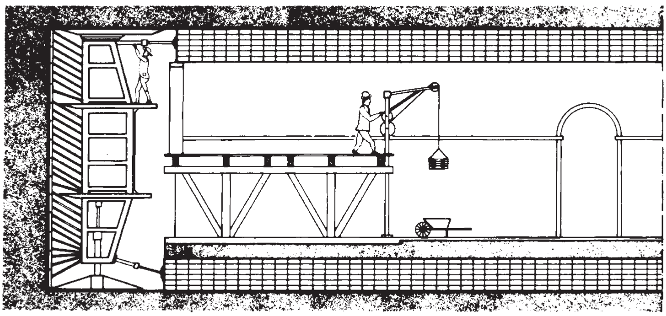


Figure 3-23 The shield from M. Brunel in the Thames Tunnel 1825 [302]

The Thames Tunnel project in London finally offered M. Brunel an opportunity to implement his ideas (Fig. 3-23). The shield was rectangular and consisted of twelve adjacent frames, each of which was divided into three chambers, each providing space for one miner to work, making altogether 36 miners. The system worked like this: Firstly, the crown piles were driven into the ground with screw jacks. The breasting plates at the face were removed starting from the top and the soil was dug out 6 inches deep, the breasting was installed again and supported with screw jacks. The entire frame was pushed from the masonry, which had been built behind. The works in the Thames Tunnel started in 1825 with great difficulties. The tunnel was only completed in 1843 after 10 severe water inrushes.

Two years after the first serious accident in the Thames Tunnel, Colladon suggested the use of compressed air in a letter to Brunel in order to avoid such incidents. Brunel however had other worries with the restarting of works under the Thames and rejected the idea. Compressed air working was thus developed further for caisson foundations before it was used for tunnel driving.

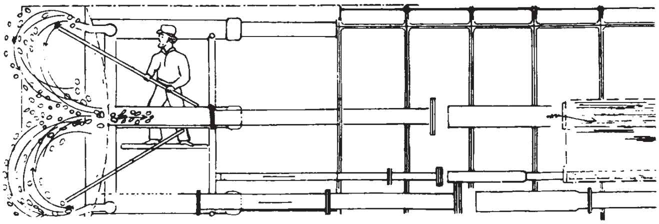


Figure 3-24 Slurry shield from Greathead, patented in 1874 [339]

H. Lorenz secured a patent for the stabilising effect of bentonite under pressure for face support, but the idea was not successfully tested until 1950, by E.C. Gardner for a water tunnel with a diameter of 3.35 m. Greathead had already designed a slurry shield in 1874 (Fig. 3-24). A two-layer steel skin with a shield tail construction ensures the sealing requirements for water and compressed air. The bentonite is pressed into the shield space through two pipes and pumped out through the central pipe. The soil is excavated by the squirting action of the bentonite and with manual assistance. This could be considered as the forerunner of all shields with the face supported with slurry, particularly the Hydroshield and Thixshield with suction head.

3.5 The classic tunnelling machines

The development of mechanical tunnelling machines was already being worked on with great energy before the invention of dynamite. The Mont Cenis Tunnel project was of high political, economic and military interest for Napoleon, but only in 1845 were the works on the rail tunnel from Turin to Genoa under Mont Cenis actually started. Henri-Joseph Maus, a Belgian engineer with a good reputation from the construction of railways in Belgium, was entrusted with the works at the age of only 37.

H.-J. Maus had been given a commission to build a tunnel 12,290 m long using gunpowder, hand drills and scarcely usable ventilation equipment. He started immediately in 1846 with the design and construction of a mechanised tunnelling machine, the first ever built. On a metal framework, H.-J. Maus installed 116 hammer drills. The basic idea was to split the rock into blocks by wedging between parallel drilled holes and remove them in a certain system (Fig. 3-25). The machine was used successfully for the construction of the Mont Cenis Tunnel. Even if the development of modern cutting machinery is not recognisable in this machine, the drills and the equipment for drilling blasting holes at least were developed further and initial experience of mechanised full-face excavation was gained.

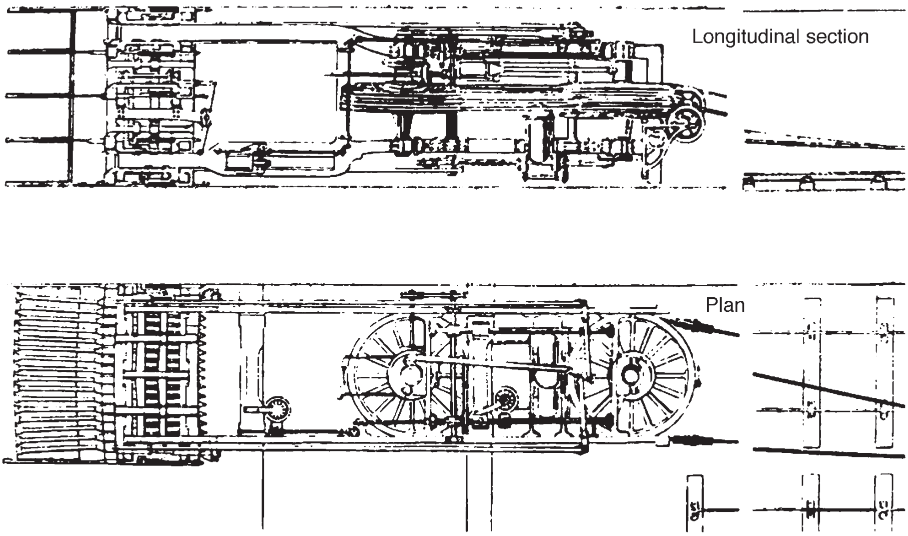


Figure 3-25 The tunnel driving machine of H. J. Maus for the Mont Cenis Tunnel, 1846 [339]

In 1856, C. Wilson patented a tunnel boring machine, which was equipped with disc-type cutters (Fig. 3-26). The cutters were fixed on arms and intended to break out the face according to a certain arrangement of tracks. The spoil was to be moved backwards by the screw effect of the head. C. Wilson's machine was built but did not prove satisfactory.

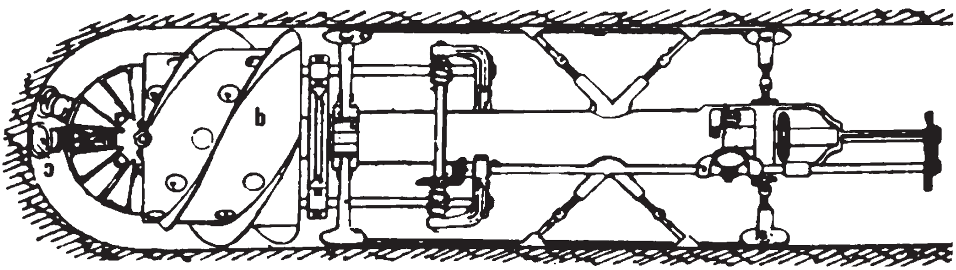


Figure 3-26 Tunnel boring machine from C. Wilson, tried out in the Hoosac Tunnel, patented in 1856 [339]

With a new machine patented in 1857, Wilson changed his concept with a machine, in which many features of modern machines are already recognisable. A stable cutting head, already with rolling groups of discs, which are assisted in the cutting pattern by the creation of free surfaces, with others being arranged in the cutting direction at right angles. Two or three additional roller discs are mounted centrally in order to achieve a preliminary cut. The excavated spoil is transported by slopes and scrapers into a collecting container and then down a slide to a wagon. C. Wilson's overall concept was to cut out an external slot and then break out the centre. The remainder was to be broken with gunpowder after the machine had been withdrawn. This concept is interesting and is missed today for rock types, such as occurred in the tunnels on the new railway line from Hannover-Würzburg of DB AG. However this second machine concept did not produce satisfactory results either when tried out in the Hoosac-Tunnel in 1862.

The plans for a Channel Tunnel between England and France resulted in an immediate impetus for the development of tunnelling machinery. These are associated with the developments of Beaumont, an English lieutenant. In 1864 he developed the first tunnelling machine with a combination of hammer drills and discs. He also initially intended to produce a perimeter slot and blast the core later. This machine was tested successfully, but Beaumont's developments continued further. Finally he succeeded in designing, building and testing a tunnelling machine, which was used both on the French side and on the English side of the Channel Tunnel (Fig. 3-27). In the years 1882 and 1883, the machine bored 1,840 m of tunnel on the French side and 1,850 m on the English side, but the project was stopped in 1883 for political reasons.

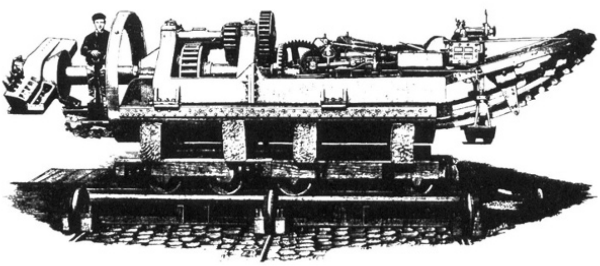


Figure 3-27 Tunnelling machine from Beaumont, patented 1880 It was used on both side of the Channel Tunnel [339]

The development of roadheaders came much later. Fig. 3-28 shows a U. S. patent from C. T. Drake from 1903. Above all the mucking is interesting. The cut spoil is transported inside the cutting head with a screw conveyor to a handover cylinder and there falls into a transport skip.

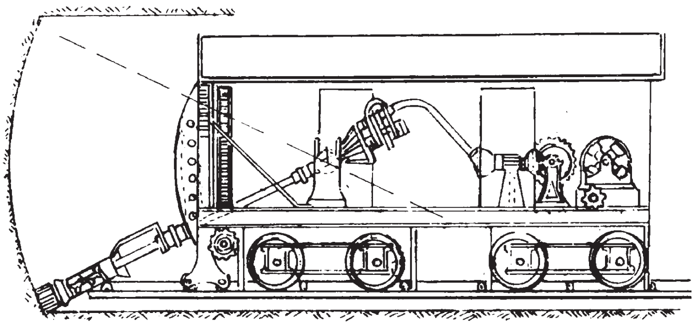


Figure 3-28 Roadheader tunnelling machine from C. T. Drake, patented in 1903 [339]

4 Shotcrete tunnelling

4.1 General

Tunnelling is characterised by dramatic leaps of development, but each age makes use of the experience of previous ages. We can still recognise the principles of classic tunnelling methods today in modern methods, which use the same excavation sequence. Only the support materials used, shotcrete, rock bolts, steel support elements, steel liner plates and prefabricated elements have superseded the former timber and masonry. There are numerous tunnelling methods today that are still based on the principles of classic methods.

Definitions, classifications etc. therefore have to consider historical developments (Fig. 4-1).

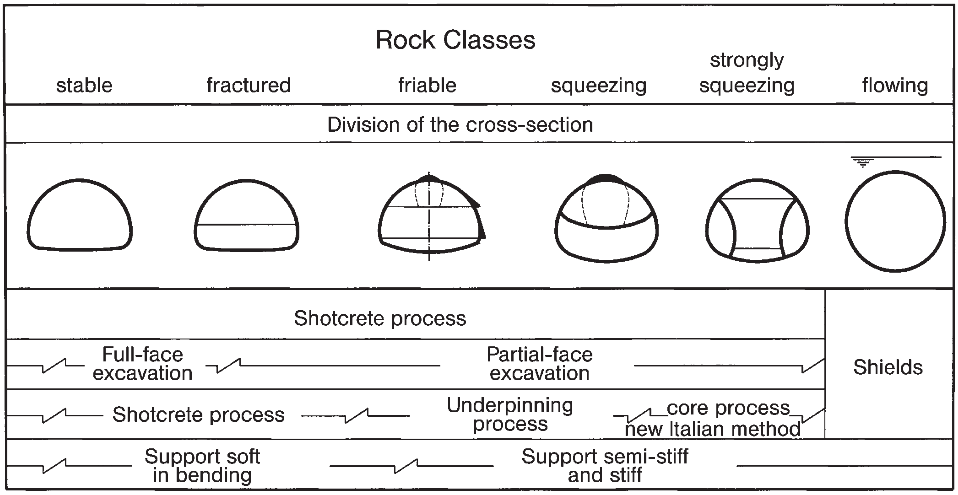


Figure 4-1 Correlation of shotcrete tunnelling and other methods into rock classes

For tunnels driven using conventional shotcrete methods, the construction process technology and the selection of suitable support measures are of great importance. The technical and economic success of a tunnel drive is influenced not only by the geological and hydrological conditions, but also the following factors:

– A suitable construction method, i. e. division of the cross-section.
– A suitable operational method, i. e. construction planning along the tunnel including all the logistics.

Numerous construction methods have been developed and these are illustrated in Fig. 4-1 according to the division of the entire cross-section for partial excavation. The basics of full-face excavation are dealt with in section 3.2.

The decision whether to divide the cross-section is essentially determined by the geological conditions, although larger sections may still be divided even in rock that will stand up for reasons of process technical or to suit the machinery. Another factor is that changing the process during the drive can be expensive and time-wasting, particularly a change from top heading to side headings or vice versa when the change has to take place within a section of a drive.

4.2 Top heading process

The top heading is excavated first and the bench and invert are brought up behind as required. The various methods are shotcrete tunnelling, underpinning method, crown pilot heading method and shotcrete tunnelling with longitudinal slots.

4.2.1 Shotcrete tunnelling method

For a divided cross-section in stable rock with properties ranging from fractured to friable, the shotcrete method is used. After excavation by blasting or mechanical excavation, support is installed that is weak in bending, consisting of shotcrete and steel arches or lattice elements in combination with rock bolts. For the already mentioned reasons, the cross-section is typically divided into top heading, bench and invert, with the advance of the top heading and bench being decided dependent on the geological conditions encountered, the external requirements and the deformation measured during the drive. The top heading and bench can be carried out with or without invert vault according to the conditions of the rock mass. Fig. 4-2 shows the situation in the Himmelberg Tunnel in friable rock, where a support core and an invert vault were necessary.

4.2.2 Underpinning method

In very friable to squeezing conditions, it can be advantageous to use the underpinning method. Excavation in this case also starts with the top heading, although the shotcrete layer is applied thicker to limit deformation and is thus designed to be stiffer (Fig. 4-3).

There are a range of variants for the underpinning method, which are based on the geological conditions and the operational requirements (see also Section 3.3.2). In principle, any tunnel drive with the top heading excavated in advance and support that is no longer weak in bending can be categorised as an underpinning method. The basic difference between the two processes is summarised in the following section:

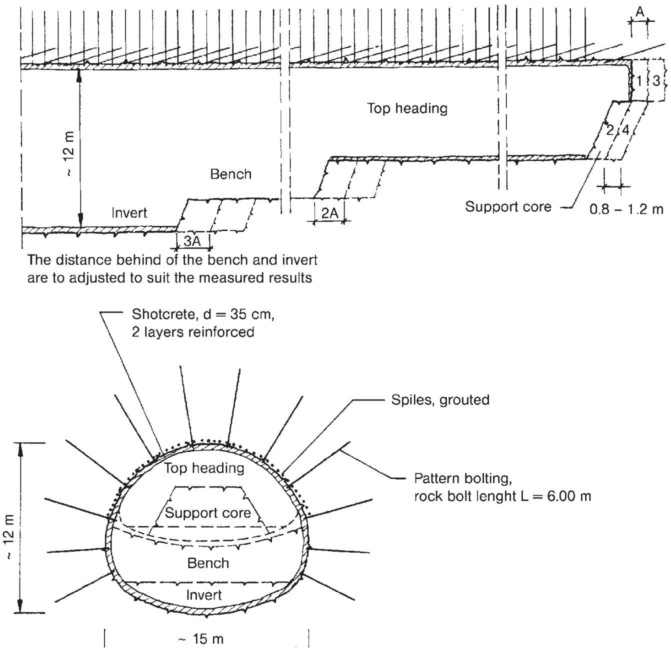


Figure 4-2 Application of a top heading in the Himmelberg Tunnel in friable rock.

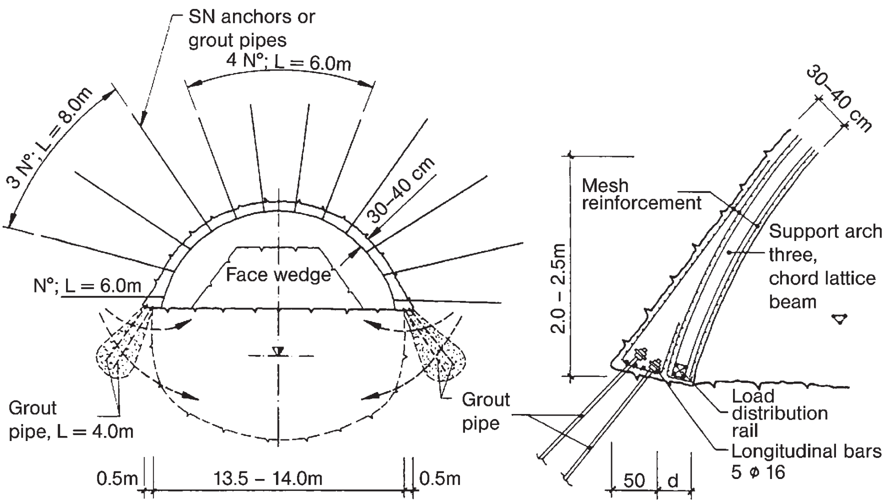


Figure 4-3 Support measures – top heading and the thickening at the foot of the top heading for fault zones encountered during the construction of the Landrücken Tunnels, central contract

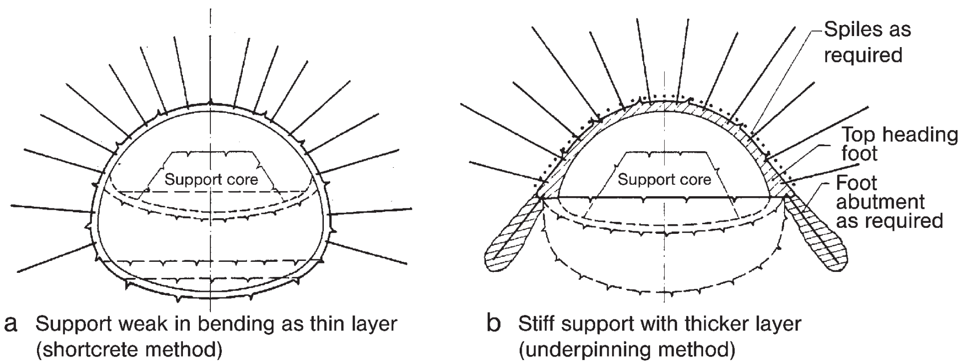


Figure 4-4 Differentiation of shotcrete and underpinning methods

– In shotcrete tunnelling (Fig. 4-4 a), the tunnel drive can be planned with the thinnest possible shotcrete layer with little bending resistance, with more deformation being permitted in order to form a structurally active ring in the surrounding rock mass. The dimensions of the top heading and bench, which are excavated in advance (excavation area, round depth, top heading invert vault, invert vault), are then closely related to the dimensioning of the temporary support and the advance rate, which are dependent on the stand-up time of the rock mass.
– The underpinning method (Fig. 4-4 b) is used in friable to squeezing conditions, particularly with shallow cover, in other words conditions under which the formation of a structurally active ring in the rock mass cannot be guaranteed. But specification requirements for low settlement also demand thicker shotcrete layers, foot thickening, foot piles and perhaps grouting. Under the stated conditions, a shotcrete layer that is weak in bending is no longer achievable and thus not responsible. The use of the underpinning method can comply with the desire for greater safety. In the course of excavation, the shotcrete support layer in the top heading is underpinned with the tunnel structure being verified with calculations, so this case can no longer be considered a support layer that is weak in bending with closed invert, but an underpinning. If there is a danger of the abutments of the top heading support failing, they should be strengthened with grouting or piles.

If the conditions become worse still, particularly combined with the support of a large face, then the cross-section will have to be divided further according to Fig. 4-5. The division can either be by excavating a half side or by advancing a heading in the crown, in which case the crown beam process represents a special development.

Alternatively, the crown beam process permits the retention of an advanced top heading with the use of various types of screen or canopy for support as described in more detail in Chapter 2.

4.2.3 Crown pilot heading with crown beam

This process was developed by B. Maidl in 1982 for the Westtangente Tunnel in Bochum [217] and was also used very successfully in the Schulwald Tunnel in very friable to squeezing ground types (Fig. 4-6). In this tunnel, it was not practical to drive the entire top heading as the stability of the face could no longer be ensured [25].

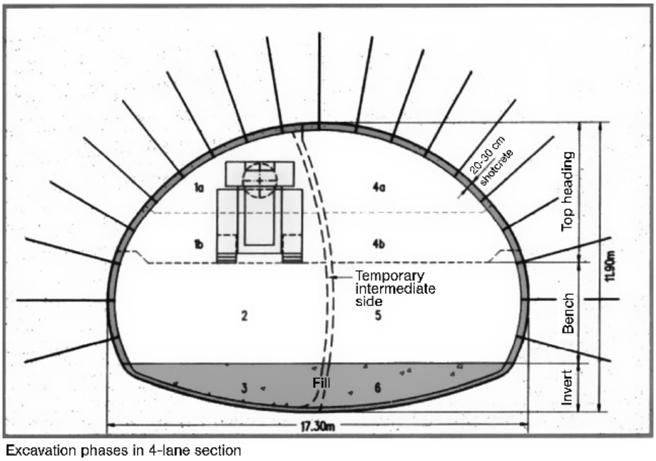


Figure 4-5 Division of the cross-section for the Altstadt Tunnel, Arnsberg.

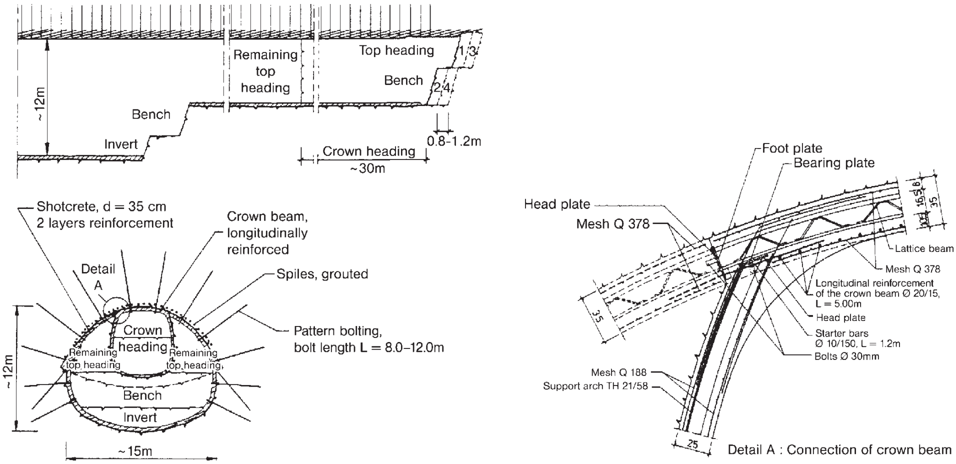


Figure 4-6 Application of the crown pilot heading method with crown beam at the Schulwald Tunnel

The crown pilot heading has about half the cross-sectional area of the top heading and is advanced about 30 to 40 m in front of the widening. If required, the crown pilot heading can also be divided into top heading and bench/invert with inclined face, and the anchoring of the face of the crown pilot heading and an invert vault could also be necessary depending on the geological conditions encountered. The crown pilot heading also has three layers of longitudinal reinforcement with 32 20/15 diameter bars running along the tunnel in 35 cm thick concrete, which acts as a longitudinally load-bearing crown beam.

The construction sequence for excavation and support in the Schulwald Tunnel was as follows:

I. *Crown pilot heading:*

– Top heading: Excavation and sealing shotcrete $d = 7$ cm as face support,
Outer layer of reinforcement, erection and spraying of the steel arches.

Installation of spiles if required, $l = 3.0$ m.
Shotcrete in crown d = 25 cm, sides $d = 20$ cm.
Installation of rock bolts, $l = 8.0 – 12.0$ m.

– Bench: Same steps as top heading but no spiles.
Next top heading round, next bench round.
Ring closure.
Second reinforcement layer in the sides and shotcrete $d = 25$ cm.
Construction of crown beam: inner reinforcement (steel mesh and 32 bars dia. 20/15, $l = 5$ m), shotcrete crown $d = 35$ cm.

II. Remainder of top heading:

Excavation left and sealing shotcrete 7 cm as face support.
Outer layer of reinforcement, connection and spraying of the support arches.
Reinforcement and shotcrete, top heading feet.
Installation of spiles if required.
Shotcrete $d = 25$ cm and setting of the anchor plates.
Round right, next round left, next round right.
Installation of the foot piles, inner reinforcement layer and shotcrete to $d = 35$ cm, alternating sides.
Removal of the support at the sides of the crown heading.

III. Bench:

Excavation and sealing as required.
Outer reinforcement layer, connection and spraying in of the support arches.
Installation of rock bolts, $l = 6$ m.
Inner reinforcement layer and shotcrete to $d = 35$ cm.

IV. Invert:

Support and sealing as required.
Installation of the temporary drainage.
Outer reinforcement layer, shotcrete.
Inner reinforcement layer, shotcrete $d = 35$ cm.

The advanced crown pilot heading reduces the size of the face and thus reduces the danger of face collapses. (Fig. 4-7). It is also useful for advance probing and water drainage. The crown beam provides additional support while the rest of the top heading is driven, with its load-bearing action along the tunnel having the effect of limiting settlement in the unsupported area (which can be shown to make up the largest part of overall settlement) when the remainder of the top heading is driven. The bearings for the crown beam are provided by the pipe-like support to the crown heading at the front and by the already load-bearing support at the back. The excavation of the remainder of the top heading can be performed synchronously or alternately, with the top heading support bearing being provided either by thickened footing (with or without foot pile) or as an invert vault. The excavation of the bench required special measures at the Schulwald Tunnel because of the squeezing rock mass. The footings of the top heading were already highly loaded and not capable alone of supporting additional loading from the underpinning during the bench advance. This demanded particular skill of the miners and special attention to the underpinning of the top heading feet in order to ensure the necessary safety for the excavation of the bench and invert. One important measure at this stage of construction is the fastest possible closure of the invert for the entire cross-section, meaning that the bench and invert have to be advanced immediately after each other. In this case, it had to be borne in mind that the cross-sectional area of more than 100 m² would scarcely allow for emergency support with timber if a collapse had occurred.

Figure 4-7 Crown pilot heading method in the Schulwald Tunnel.

In this method, the advance can be changed over from top heading to crown heading immediately, and a change from crown heading to top heading advance only requires repetition of the enlargement to the face of the crown heading. This makes the crown heading method an especially adaptable construction process, which can thus also be used when unfavourable geological conditions, which make the excavation of the entire top heading impractical, are encountered unexpectedly without having been prepared. Without having to change to the core method with side headings, this method of tunnelling is to be recommended under very changeable geological conditions and particularly when tunnelling below buildings that are sensitive to deformation [185].

It should be mentioned that with hindsight, the use of a shield machine would have been an alternative worth considering for the Schulwald Tunnel with 4.5 km to be driven, particularly because a division into two bores and thus a shield drive without more risk would have been practical and interesting in terms of time and cost savings compared to the solution described above.

4.2.4 Shotcrete tunnelling with longitudinal slots

When deformations are very large, into the range of metres, the shotcrete layer with the integrated construction elements like steel or lattice arches and rock bolting can be preserved from destruction by providing systematic deformation slots (Fig. 4-8 and Fig. 4-9). Calculation methods can be found in section 3.7.

The width of the slots can be up to 40 cm. It can be assumed that the convergences that occur will deform the steel arches, which are fitted with yielding lap joints and thus represent a type of predetermined failure point.

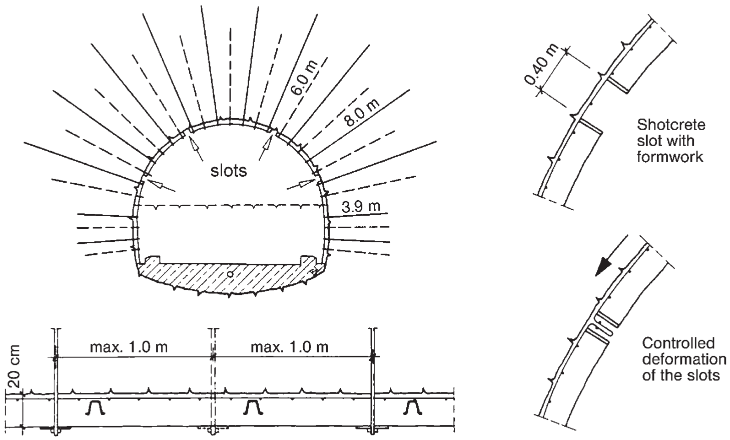


Figure 4-8 Construction of a temporary support of shotcrete with longitudinal slots [154].

Figure 4-9 Outer support layer with deformation slots.

Most of the lattice arches used can deform freely in the slots, which have not yet been sprayed. The shotcrete layer is thus saved from destruction and can form a vault in the surrounding rock mass to activate the best possible load-bearing action. The slots can also, as described by Schubert [324] from the Galgenberg Tunnel, be provided with timber or dilatation tube elements (Fig. 4-10) in order to transfer a controlled normal force. See also Section 2.8.7 Yielding elements.

Figure 4-10 Dilatation tube element in the Galgenberg Tunnel [324].

4.3 Core tunnelling method with side headings

In zones with considerably lower rock strengths due to deep-acting weathering or in loose ground, in other words ground that exerts pressure, particularly when the horizontal forces are high, the core tunnelling method with side headings (or side wall drifts) is advantageous. This process was used, for example, for some sections of the Himmelberg Tunnel (Fig. 4-11).

In this method, similarly to classic tunnelling (see section 3.3.3), two or more headings with a controllable cross-section are first driven around the section perimeter at the sides. Then the top heading is excavated and supported, with the side headings serving as bearings for the top heading vault. Then the support arches of the top heading and the external side arches are connected.

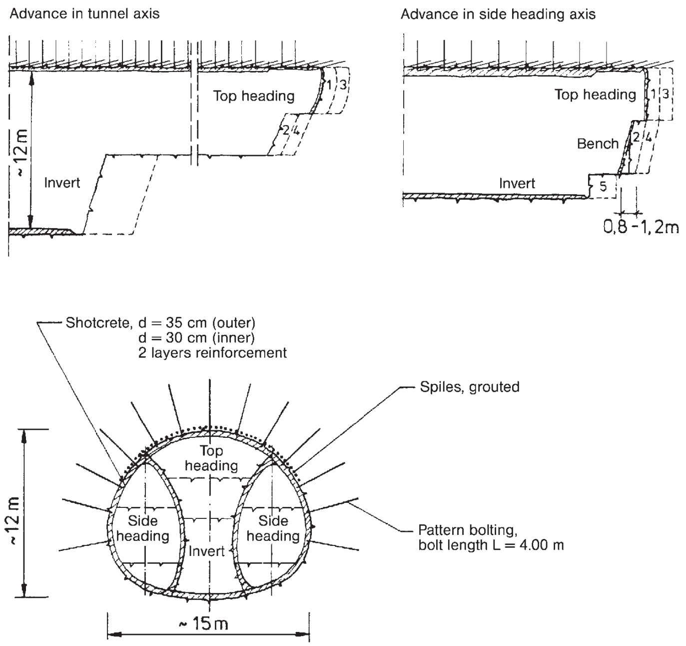


Figure 4-11 Application of the core tunnelling method with side headings at the Himmelberg Tunnel.

In these sections, the final support can be brought up afterwards when the settlement specification is particularly stringent. This produces a vault in these sections, which is connected up later.

The core remains standing and serves to support the face since its area is much smaller and thus better supported (Fig. 4-12), so the core tunnelling method with side headings can be considered as leading to low settlement and less susceptible to side pressure, with the advanced side headings serving both for investigation and drainage. If the geological conditions improve, changing from this method to a top heading advance is laborious. This is due to the fact that the change does not take place just inside the top heading section but affects the entire excavation and support concept and thus the entire logistics. A tender should therefore only specify a classification with side heading advance when a change to top heading advance during the drive can be largely ruled out. The trend today is to extend the economic range of applications of the top heading method through the use of pipe screens as advance support or the crown beam method. Side heading drives should only be used in cases with really difficult conditions or high side pressures and stringent settlement requirements, and when no change of construction process during the drive is expected.

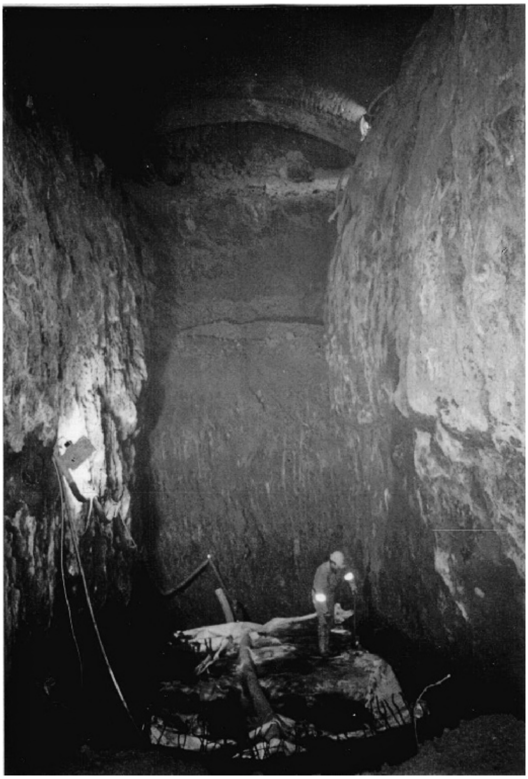


Figure 4-12 Core excavation in the Himmelberg Tunnel.

4.4 Special processes using shotcrete

There are numerous processes using shotcrete.

4.4.1 Compressed air

In recent years, shotcrete has been combined with compressed air in order to avoid open water drainage. This process has been used many times in Munich. The combination is based on the research work of J. Schreyer [322].

4.4.2 Ground freezing, grouting

In order to tunnel through heavily water-bearing fault zones, ground freezing (in addition to grouting) often offers a practical solution [272]. Ground freezing is used to produce a vault or a canopy before starting excavation, under the protection of which a temporary support of shotcrete is installed in the course of excavation. The frozen ground can function to improve load-bearing, waterproofing or both simultaneously. As the process is expensive, it is only worth considering for really difficult conditions [231], [201]. Ground freezing has hydrological advantages over grouting, since it is only a temporary measure during the construction phase. The effect of the frozen ground on the shotcrete has been tested many times in practice. Ground freezing through drilling in the axial direction (in long tunnels, drilled in sections or in short tunnels, drilled through), depends greatly on the achievable length of hole and the driling tolerances (Fig. 4-13). Ground freezing through transverse drilling can be carried out as vault freezing (Fig. 4-14) or as sealing wall freezing (Fig. 4-15) (drilled from pilot headings or from the surface). If ground freezing is considered as load-bearing, then it is in principle a type of canopy process.

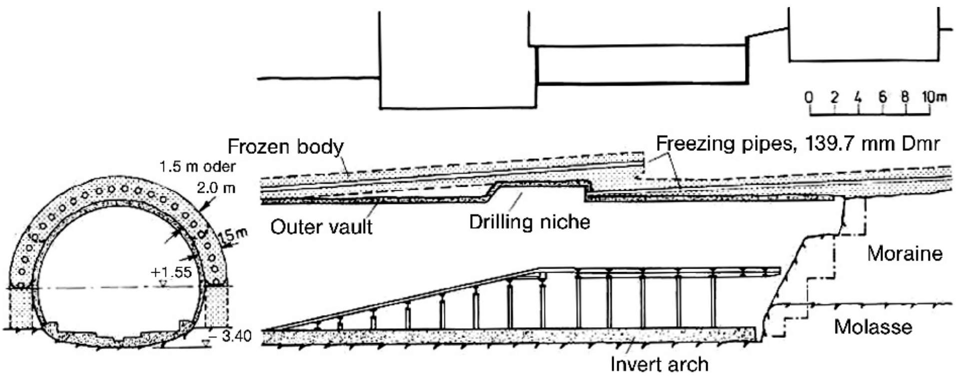


Figure 4-13 Shotcrete with ground freezing vault (longitudinal drilling in the top heading) Milchbuck Tunnel, Zürich, 1980 [350]

Grouting is, in contrast to freezing, a permanent measure and can have a great effect on hydrological conditions. In principle, this also forms a longitudinal or transverse canopy (Fig. 4-16). The process and the drilling precision requirements are similar to ground freezing. When the ground is suitable and can be uniformly grouted, grouting can be cheaper, but is still limited to special difficult cases as with ground freezing [223], [224], [225], [259]. There are many ways of providing advance support in connection with shotcrete such as pipe screens, grouted and jet grouted screens, which are described in Section 2.5.4.

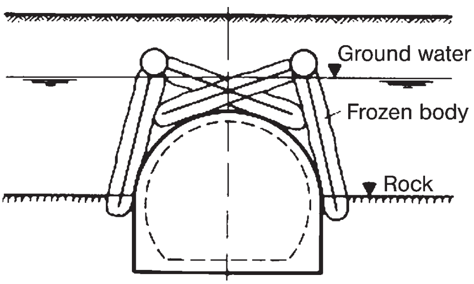


Figure 4-14 Vault freezing. Advance heading with transverse freezing. Stadtbahn Essen, contract 17, 1980 [299]

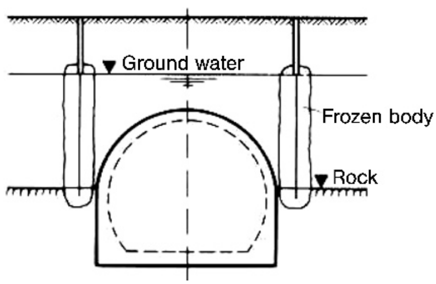


Figure 4-15 Sealing wall freezing. Frozen walls as a groundwater barrier, created from the surface [299]

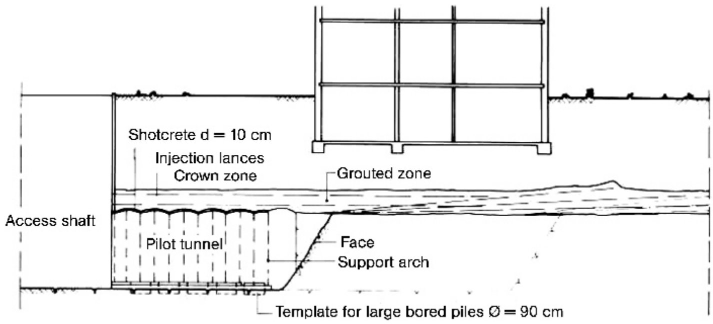


Figure 4-16 Grouting. Driving of the crown heading (pilot heading). Stadtbahn Duisburg, contract 6/7, 1981 [371].

4.5 Shotcrete in mining

4.5.1 Tunnel support

The common method of support in coal mining consists of support sets (for example steel arch support with lagging mats and back-filling with cuttings), which do not provide sufficient contact with the rock mass to transfer force, which means the support is only loaded after the rock mass has subsided. As the rock mass is loosened and softened by this subsidence, the support set has to be designed for a large part of the ground pressure.

With the increasing trend of mines to become deeper, the amount of steel used per metre of gallery has increased constantly. Instead of back-filling with cuttings, the gap to the rock mass is often filled with pumpable materials. Additional measures are often carried out relatively late, after the softening and loosening of the rock mass has already occurred. If however the loosening of the rock mass and the associated deformation of the cavity have already set in, then they can only be slowed down by laborious measures like deep-reaching grouting of the rock mass. Galleries very often have to be re-supported after a certain time for safety reasons and to preserve their operational function, an expensive process.

In a large-scale trial at the Nordstern mine, shotcrete tunnelling with all its basic ideas and principles was applied for the first time in mining [3]. The basic idea was to largely exploit the load-bearing capacity of the rock mass with the installation immediately after excavation of a support capable of transferring force to prevent the loosening of the rock mass and thus preserve its load-bearing capacity.

The tunnel lies at a depth of about 1,100 m in the horizon of the Zollverein coal seam. The rock structure consists of an intercalated sequence of shales, sand shales, sandstones and coal seams with flat bedding. In cross-section, there are mostly shales of low strength with thin intercalated coal bands. Immediately above the crown at a distance of 0.5 to 3 m is a seam 1.30 m thick along a length of many 100s of metres. The tunnel also passes through two overthrust zones with a throw of about 35 m. The constructed cross-section is a compromise solution, having to balance rock mechanical and tunnel driving requirements (Fig. 4-17).

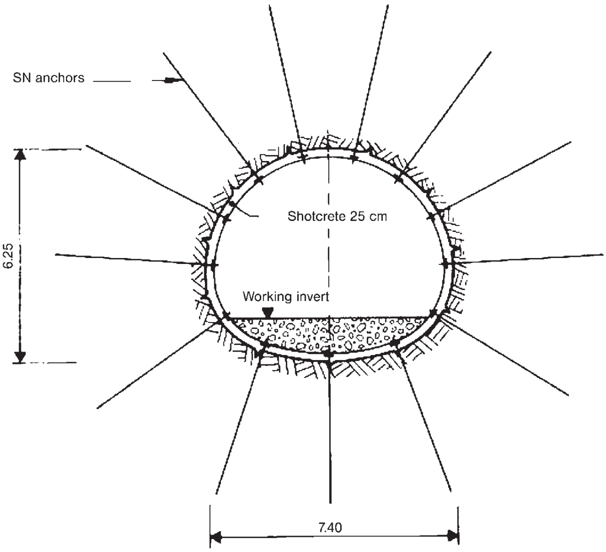


Figure 4-17 Gallery at the 12th level of the Nordstern mine, application of shotcrete tunnelling 1981 [3].

Operational experience at this mine has shown that support consisting of a shotcrete layer in combination with pattern bolting seems more suitable than shotcrete in combination with steel arches. The disadvantage of using arches is they buckle under heavy loading due to the poor bonding to the shotcrete layer and lose their load-bearing capacity. Rock bolting on the other hand is more elastic and can move with a certain amount of rock mass deformation leading to a prestress, which increases the load-bearing capacity of the rock mass.

It can be assumed that the rock mass will be better able to resist the dynamic loading resulting from later extraction than typical mining support. Damage cannot be ruled out, but the measurement programme should enable early recognition and targeted introduction of support strengthening [216].

For future applications, a combination with shotcrete with longitudinal slots as described in Section 4.2.4 should be planned.

4.5.2 Shaft insets

In order to support shaft insets, brickwork or constructions of in-situ concrete or precast elements have been used until now. All these support measures could only be installed after the creation of the entire cavity. In contrast, to support a cavity with systematic bolting and shotcrete is more flexible and can be adapted during the advance to the requirements of structural investigations [159], [207].

Single-layer construction. A single layer of shotcrete is not normally capable of resisting the large deformations to be expected without damage. With deformation slots, deformations can be resisted without damage to the shotcrete layer (see 4.2.4). These deformations have to subside before the installation of the final lining. In mine shafts, however, the maximum deformation normally occurs after the construction has been put into operation.

For the shaft inset in the General Blumenthal mine, the requirement that the deformations should be resisted without damage was omitted as a design criterion for the two-layer support, as it was safe to assume no later effects of extraction on the shaft inset. This made it possible to construct a simpler, single-layer support construction (Fig. 4-18). The single-layer support consists of a 20 cm thick reinforced layer of shotcrete. The reinforcement was installed as bars and bent on site. Along the intersection lines between shaft and the entrance to the collecting station, a reinforced 40 cm thick shotcrete thickening was provided. The density of the rock bolts was specified at one bolt per square metre for 3.45 m long bolts.

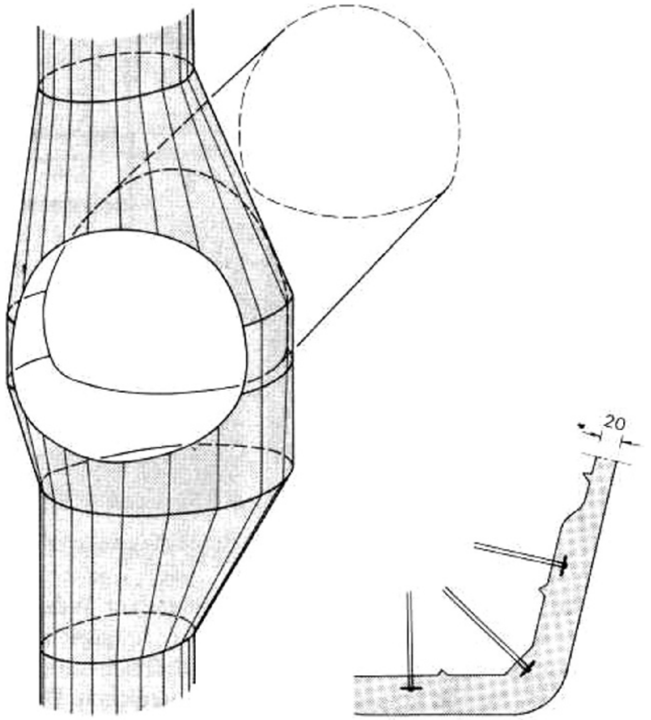


Figure 4-18 Idealised geometry of the shaft inset at the General Blumenthal and section through the single-layer support, 1978 [207].

Two-layer construction. The Prosper 10 shaft inset was supported in two layers (Fig. 4-19). Rock bolts and shotcrete layer undertake the temporary support of the cavity and form a load-bearing part of the outer lining.

In order to stabilise the rock surface, 5 cm of shotcrete was applied immediately after the completion of excavation. Then fully bonded 24 mm rock nails 2 m long were installed on a 1 m grid. As a third support element to anchor the rock mass, end-anchored rock bolts were installed in sections on a 2 m grid. As these had to reach into the calculated unloosened zone of rock, they had various lengths and inclinations. Dywidag system single-rod anchors with 32 mm diameter and 650 kN test load were installed in 115 mm diameter holes according to DIN 4125 “Ground and rock anchors” as permanent anchors. The adhesion length in the unloosened rock was specified at 2.5 m after preliminary tests, which took the unfavourable conditions into account. In the free-play section, the anchor is protected against corrosion in a pre-filled sleeve and the space around it is filled with a bentonite-cement mix of low strength. When the anchor is installed centrally, this makes cleavage displacements possible without loading the anchors in shear, which ensures that the mine support load-bearing function of the anchor is largely preserved. The anchors are prestressed to 80 % of the working load, because only slight elastic deformation has to be expected in the prevailing sandstone and shale strata.

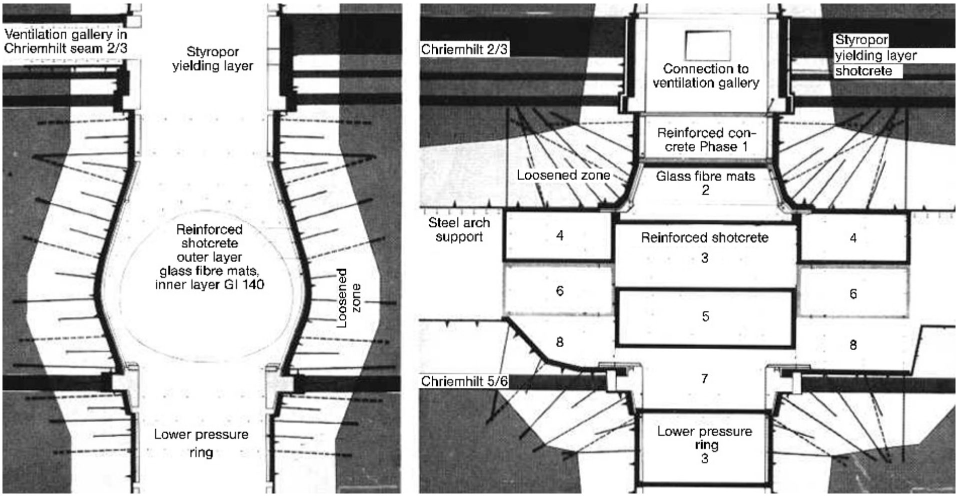


Figure 4-19 Cross-section and longitudinal section through the two-layer shaft inset at Prosper 10, 1980 [207]

The fourth support element, the shotcrete layer, was also applied in sections according to construction progress. It is 10 cm thick and reinforced with Q 188 mesh. Due to the ingress of highly aggressive water, highly sulphate-resistant cement 45 F was used. The shotcrete layer is largely waterproof. In order to avoid water pressure building up behind the shotcrete layer, 16 pressure relief pipes were installed in the upper tapered transition zone.

The subsequent 80 mm thick intermediate layer consists of glass fibre mats according to DIN 4102 “Fire behaviour of building materials and building components”, installed in two layers with lapped joints. In order to provide protection against the penetration of cement slurry, it was covered with bituminous sheets. The selected make is siliconised and thus highly water-repellent. In order to drain any water running down from the upper part of the shaft, openings are provided below the load-bearing ring.

The inner lining is not structurally necessary in the construction phase. It is therefore installed in a second phase after the creation of the entire cavity, starting from the lower foundation. Due to the flexible glass fibre intermediate layer, shotcrete was not used for the inner layer. In-situ concrete would also have been very laborious due to the barrel-shaped geometry. It was therefore decided to install a prefabricated load-bearing steel scaffolding made of GI-140 profiles with an erection spacing of 50 to 80 cm and fill in-between with concrete B 35 with a thickness of 28 cm using PZ 45-HS. An intermediate layer of mesh and expanded metal served as sacrificial formwork, and on the inner side, SZ elements. Both sides of the formwork are held together in tension. The SZ elements have a corrosion protection of 2 cm shotcrete.

4.6 Outlook for further development

Shotcrete as a support material has made construction processes possible, which can keep the temporary support in contact with the rock mass. The effect of time as a factor, which is decisive for displacement of the rock mass and development of pressure, could considerably be reduced. This would make excavation even of the full face possible in difficult geological conditions.

The technical advantages and the good adaptability of this process can cope with almost all rock classes except for running ground, but some points should be noted for further development:

1. The advance rates in highly squeezing ground, particularly in loose ground below the groundwater, fall to less than 2 m/d, which is no longer economically viable.
2. The performance of the work requires qualified people from design and supervision to the miners and nozzleman in the tunnel. Few suitably qualified people are available.
3. The miners are exposed to danger while installing the temporary support until the shotcrete has attained a certain strength.
4. The current legal regulations concerning working time complicate the continuous progress without interruption of the advance that is necessary for this method of construction.
5. The verifications of structural stability in construction states are often difficult to perform [202].

Precast support elements produced by extrusion, also in combination with shields and tunnelling machines, will show the direction for further development. Concerning materials, steel fibre shotcrete, steel fibre concrete, fibre concrete, plastic concrete but also plastic will be included in the development. The application of new and improved drills, excavation machinery, clearance machinery, anchoring equipment, shotcrete machines, robots and concreting devices should improve the method and help to improve working conditions.

If the use of the shotcrete method succeeds in untangling the working steps, which have until now had to be performed consecutively, and permit parallel working, this process will be able to compete with mechanised tunnelling. Starting in the Vereiner Tunnel and continued in the Lötschberg and Gotthard Tunnels, this development is already under way. See also section, 6.3.2

4.7 The new Italian tunnelling method (ADECCO-RS)

Since the end of the 1970s, great efforts have been made in the development of this construction process, particularly by Lunardi and Pelizza [273]. The intention is to develop a process, which can counter the disadvantages of the NATM (shotcrete tunnelling) in regard to safety and cost-effectiveness when working in very friable and particularly squeezing ground (Fig. 4-20).

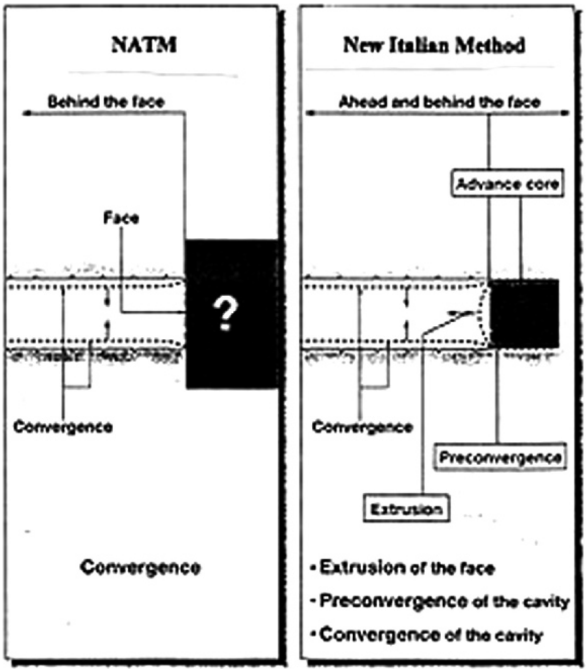


Figure 4-20 Comparison of the NATM with the new Italian tunnelling method [84].

Even with moderate overburden and particularly in inner cities with great sensitivity to settlement, the NATM with its soft shotcrete support layer cannot fulfil the safety requirements, which as already described in Section 4.4 can only be fulfilled with a stiff support layer. In addition, the increasing support measures and the time taken to install them mean that the advance rate slows considerably. At advance rates of less than 2 m/d, it is no longer possible to demonstrate cost-effectiveness, because the individual measures always have to proceed consecutively. An alternative is offered by full-face excavation with advance support measures as described in Section 2.5.4. or the Italian method described here also offers advantages. When the full face is excavated in combination with advance support of the face, the ring can be closed earlier than with the excavation of the entire cross-section in parts. This keeps the deformation of the rock mass low. The full cross-section that is already available at the face permits the use of large and powerful machines, in contrast to top heading or side heading advances. As only one working step is necessary for the entire round, there is no more interaction between the individual operation points. This makes good logistics possible, a contribution to cost-effective tunnelling. The ground improvement can, for example, be in the form of glass fibre anchors in the face and/or jet grouted screens.

4.7.1 Theoretical model

This model derives from Lombardi [187], later Lunardi [191] in the mid 1970s, when it was demonstrated that a previously consolidated core in front of the face and an inner lining concreted immediately behind it have a favourable effect on the deformation curve and thus also settlement at the surface (Fig. 4-21).

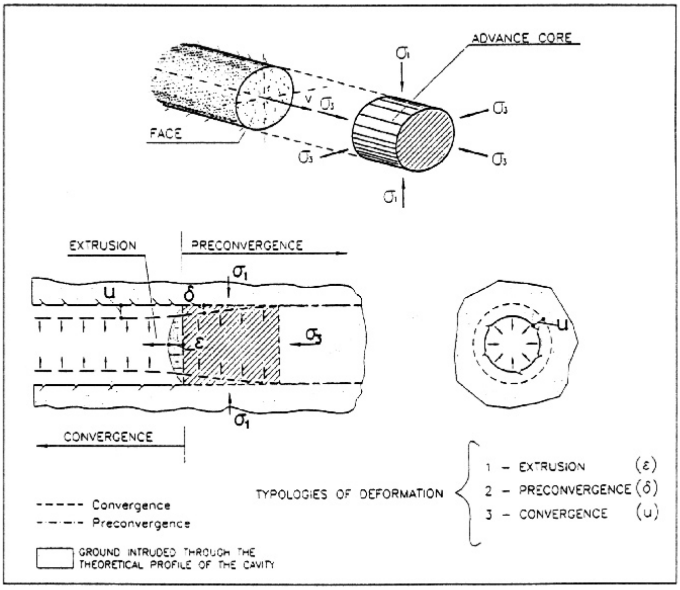


Figure 4-21 The advance core as a way of interpreting the deformation [191].

This model idea was initially investigated through in-situ measurements in numerous tunnels, and simultaneously the possible technical processes for its implementation in practice were produced and tried out. Today this process has been developed ready for practice and offers in the opinion of the author the most economical way of making use of the advantages of full-face excavation in squeezing rock, when all advance support measures have been exploited with the processes described in Section 4.2 and the shotcrete tunnelling process as described in Section 4.4 would have had to be changed over to side heading advance with core method.

Unfortunately, there are not yet any application examples in Germany or Austria, so in following the procedure is described using the example of the new line from Bologna – Florence.

4.7.2 Procedure through the example of the new line from Bologna – Florence

For the tunnels on the new line from Bologna – Florence, the selection of necessary ground improvement measures was derived from a description of the stand-up time of the face based on the preliminary geological, geotechnical and hydrogeological investigations. Firstly, the tunnels were divided into three essential zones according to their different face behaviour (Fig. 4-22) [191]:

– Behaviour type A (stable face).
– Behaviour type B (face with short stand-up time).
– Behaviour type C (unstable face).

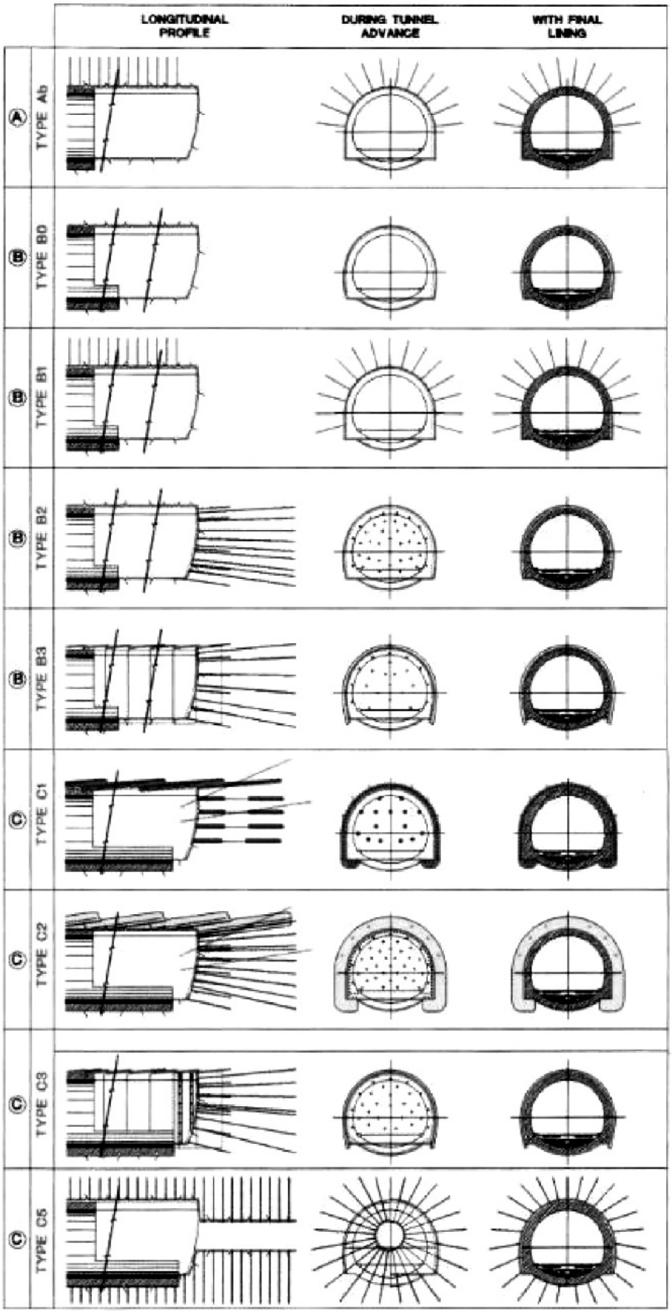


Figure 4-22 Construction process used depending on the behaviour of the ground at the face of the tunnels on the new line Bologna – Florence [191].

Behaviour type A. The face was stable, making no ground improvement measures necessary. Reinforced shotcrete with a thickness of 25 cm was used here as temporary support. The final lining was of unreinforced in-situ concrete, which was 60 cm thick both for the inner lining and the invert arch.

Behaviour type B. A face with a short stand-up time was forecast for 60 % of the tunnel length. These were further sub-divided into main cross-section types B1, B2 and B3. For type B1, no advance support measures were intended but the temporary support of shotcrete was thickened to 30 cm and doubled steel arches were provided. The final support consisted of an inner lining 80 cm thick and a 90 cm thick invert vault, installed at a maximum distance of three tunnel diameters from the face.

For type B2, the inner lining thickness increased to 90 cm and the invert vault thickness to 100 cm, being installed at a spacing of maximum 1.5 tunnel diameters behind the face (Fig. 4-23).

In addition, the face of tunnels in this cross-section category was supported in advance with the installation of 15 m long glass fibre anchors with an overlap of at least 5 m. Glass fibre anchors were used to consolidate the core of the advance and thus support the face in cases of larger cross-section, shallow overburden below buildings and friable ground (Fig. 4-24).

Grouting at up to 100 bar could be carried out for further ground improvement through the tubes fixed to the anchors. The properties of the glass fibre enable the excavation of the tunnel with normal tunnelling machinery. This makes it possible to plan in advance and economically tunnel through various ground types.

For cross-section type B3, the poor stand-up time of the face made an intensive arrangement of glass fibre anchors in the face necessary. The inner lining thickness remains 90 cm and the invert vault thickness 100 cm, but the invert vault in installed only 4 to 6 m behind the face.

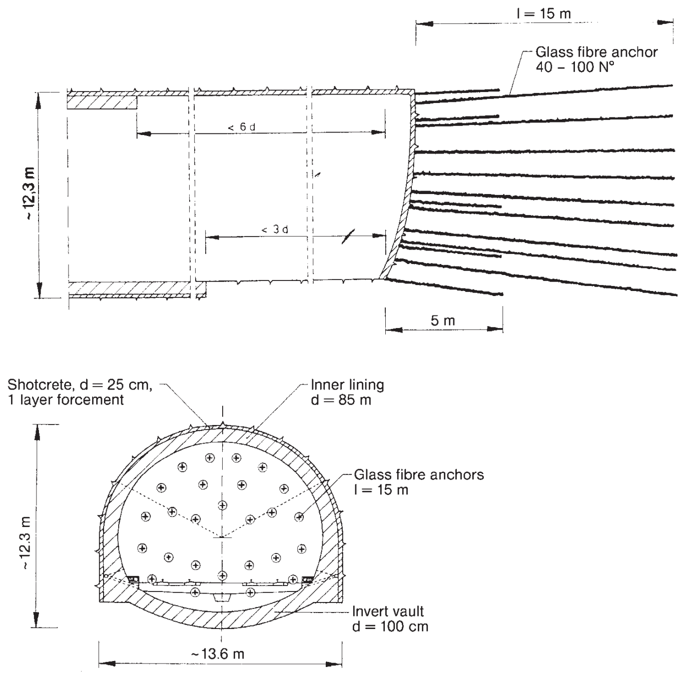


Figure 4-23 Application of the construction process for cross-section type B2 in the tunnels on the new line Bologna – Florence.

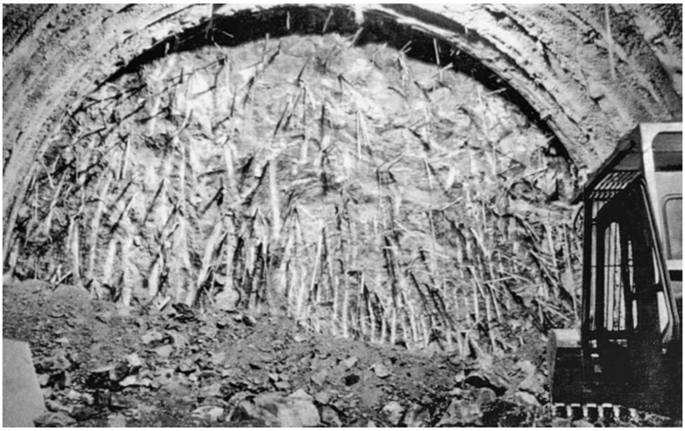


Figure 4-24 Use of glass fibre anchors to support the face in the tunnels on the new line Bologna – Florence.

Behaviour type C. Due to the unstable face, this type of cross-section demands further ground improvement measures, with two sub-divisions being differentiated. Type C1 has a 13 m long jet grout screen to improve the ground (Fig. 4-25). The tunnel can be advanced under the protection of this canopy around the tunnel.

In addition, jet grout columns reinforced with glass fibre anchors were installed in the face to provide extra stability. The length of the glass fibre anchors here was 15 m with a minimum overlap of 5 m. The thickness of the inner lining was adapted to suit the local conditions and varied between 40 and 140 cm, with the thickness of the invert vault being constant at 100 cm. The invert vault was installed at a maximum of 1.5 tunnel diameters behind the face.

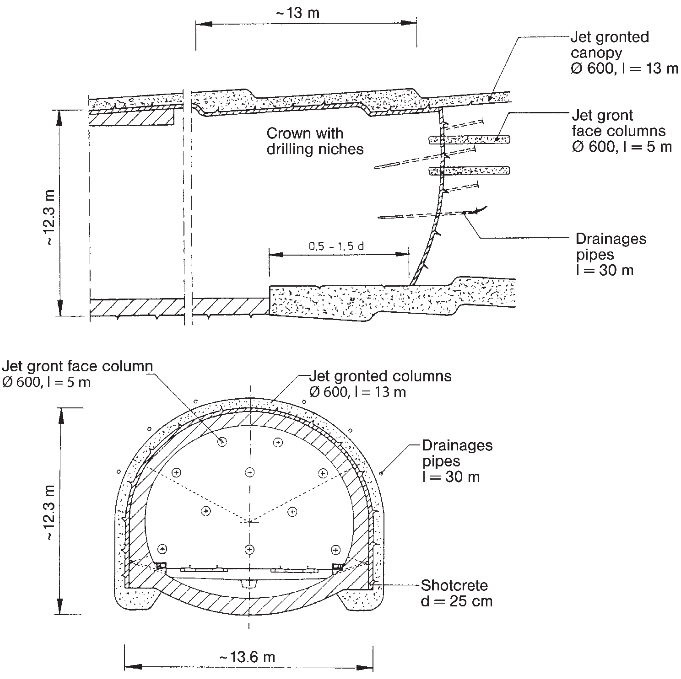


Figure 4-25 Application of the construction process for cross-section type C1 in the tunnels on the new line Bologna – Florence [43].

For cross-section type C2, jet grout columns reinforced with glass fibre tubes were used, and the face was additionally improved with groutable glass fibre anchors in addition to the jet grout columns in the face (Fig. 4-26). The temporary support consisted of reinforced shotcrete with double steel arches and the final support was provided by the inner lining 90 cm thick and an invert vault 100 cm thick, installed at a spacing of 1.5 tunnel diameters behind the face.

Figure 4-26 Jet grout screen and jet grout columns in the face as ground improvement measures in a tunnel on the new line Bologna – Florence.

5 Drill and blast tunnelling

5.1 Historical development

The development of tunnelling by drilling and blasting was mainly determined by the development history of the two essential components. Both the continuous development of equipment, particularly for drilling, and the development and improvement of explosives helped drill and blast to achieve a considerable significance, which continues today.

The term “blasting” as generally used means the violent loosening or destruction of a natural or artificial coherence. The property of “bursting” was already known to the ancient Egyptians in 2100 BC (Table 5-1), who made use of the swelling and expanding properties of dry wood or leguminous fruits [360]. These ancient bursting methods can however not really be regarded as the predecessor of the explosives used for some centuries in tunnelling, as these have an explosive and mostly also detonating effect.

The first explosive powder mixes based on saltpetre were probably already known in the culture of ancient China. The invention of dynamite by the Swede Alfred Nobel in 1867 provided another significant impetus to blasting in tunnelling. This explosive had a five times greater blasting effect than the gunpowder that had been almost exclusively used before, was relatively safe to transport and in contrast to gunpowder, could even be used in wet conditions. The construction time of the St. Gotthard rail tunnel was reduced thanks to the use of dynamite to 10 years (1872 to 1882), remarkably short even by today‘s standards.

At the same time, another major leap in development also benefited compressed air drilling tools. In the St. Gotthard Tunnel, drills from Sommeiller, Ferroux, Turrettini and Colladon, McKean as well as Dubois and François were used [95]. At the start of tunnelling, advance rates of only 0.7 to 0.8 m/d were achieved despite the use of Sommeiller drills (Fig. 5-1) and the use of dynamite, but this was later increased to an average 4.3 m/d.

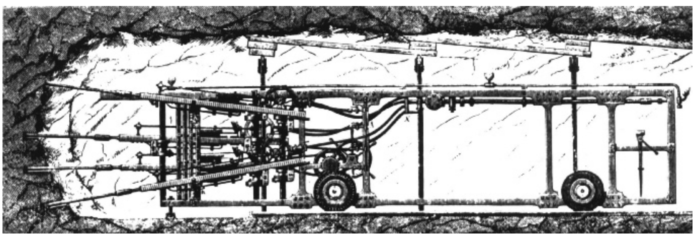


Figure 5-1 Arrangement of the Sommeiller drill for the Mont Cenis Tunnel [302]

Table 5-1 Timeline of the history of explosives

Date	Event
2100 BC	Fire setting in mining in Spain; bursting agents in Egypt
1800 BC	Fire setting in mining in Austria
552 AD	"Black powder" was used for fire setting in Italy: finely ground charcoal with explosive action
1232	"Fire powder" containing saltpetre, the first explosives in hand grenades in China
about 1250	Black powder recipes (fire compounds containing saltpetre) in Europe
1330	Application of gun powder for firearms by Berthold Schwarz
1360	Gunpowder accident; explosion in the Lübeck city hall
from 1430	Use of gunpowder in mines and underwater blasting by the military
1533	State accident prevention regulations for gunpowder manufacture
1573	Blasting in mining with gunpowder by the Italian miner G.-B. Martinengo near Vicenza in Friaul (Upper Italy)
1627	Documented underground blasting in mining with gunpowder by Kaspar Weindl in today's Slovak Ore Mountains
1790	Introduction of cavity blasting
1844	Electrical detonation in the form of battery detonation
1845	C. F. Schönbein invents guncotton (nitrocellulose)
1846	A. Sobrero invents nitroglycerine
1863	A. Nobel invents his "patent detonator" based on gunpowder
1867	A. Nobel invents dynamite, consisting of nitroglycerine and kieselgur
1893	Gelatin safety blasting explosive class I
1938	Sheathed safety blasting explosive class II
about 1950	Start of development of safety blasting explosive class III
1954	Decisive tests with ammonium nitrate/fuel oil (ANC explosives) in the USA
1969	Start of the development of emulsion explosives in the USA
1986	Electronic detonation

The development of the first hydraulic drill, which Brandt designed for the machinery manufacturer Sulzer Brothers and patented on 5 February 1878 in the USA [339], a maximum advance rate of 6.2 m/d was achieved in the Simplon Tunnel (Fig. 5-2).

The opening of the second Simplon Tunnel marked the last major tunnel for a while, as the main rail routes now had the shortest routes and the countries north and south of the Alps were connected. The rapid development of the car led to further tunnels under the Alps, which were all excavated by drill and blast, like the Arlberg road tunnel (constructed 1974 to 1978) or the St. Gotthard road tunnel (constructed 1969 to 1980). Modern progress in mechanisation and the power of the machinery available has led to the perfection of drill and blast tunnelling.

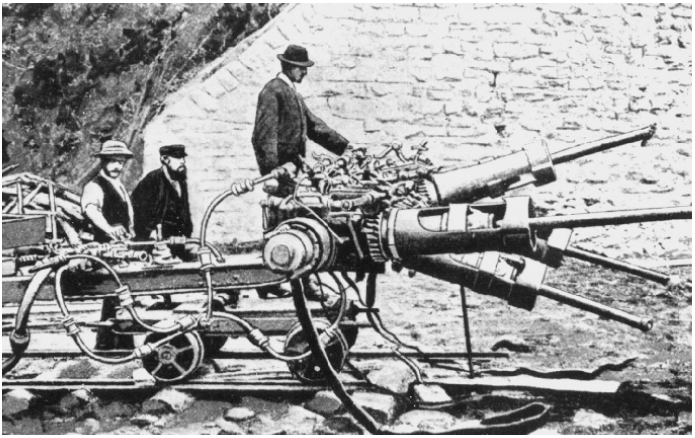


Figure 5-2 Hydraulic rotary drill from A. Brandt [263]

This development of drill and blast has made it the normal technology for major transport tunnels. Even if mechanised tunnelling continues to gain acceptance, drill and blast will maintain its advantages of flexibility and cost-effectiveness.

5.2 Drilling

5.2.1 General

The term drilling generally denotes the use of drills to make holes. The hardness, toughness and strength of the rock to be drilled determine the requirements for the drilling tools. Modern drilling equipment is needed in all branches of construction. It is however necessary to define the term drilling more closely.

This book only describes drilling to create holes for the purpose of tunnelling. Drilling and blasting are therefore always considered together. Any optimisation of the drill and blast excavation process should take both technologies into account.

Drilling technology is used to drill holes for various purposes in tunnelling (Table 5-2).

Table 5-2 Purpose of holes drilled in tunnelling

Shot hole drilling	Holes for explosive charges, empty holes to limit the extent of the effect of blasting
Anchor drilling	Holes for rock bolts for ground improvement and support, fixing bolts for construction operations
Relief drilling	Holes to relieve groundwater or groundwater pressure
Probe drilling	Holes in front of the tunnel into the rock mass around the tunnel to discover voids (probing) and aquiferous strata
Geotechnical drilling	Holes for the installation of geotechnical instruments

5.2.2 Drills

Drilling technology. In tunnelling, drilling equipment is a means to the end of blasting, which is intended to loosen the rock to the correct profile and without excessive damage to the surrounding rock. This action can normally only be achieved with the appropriate mechanical equipment. Drilling technology and the associated machinery have experienced rapid and exciting development in the last two decades, mostly due to the change from air to hydraulically powered drilling equipment. The most decisive improvement has been the considerable increase of drilling performance (Fig. 5-3).

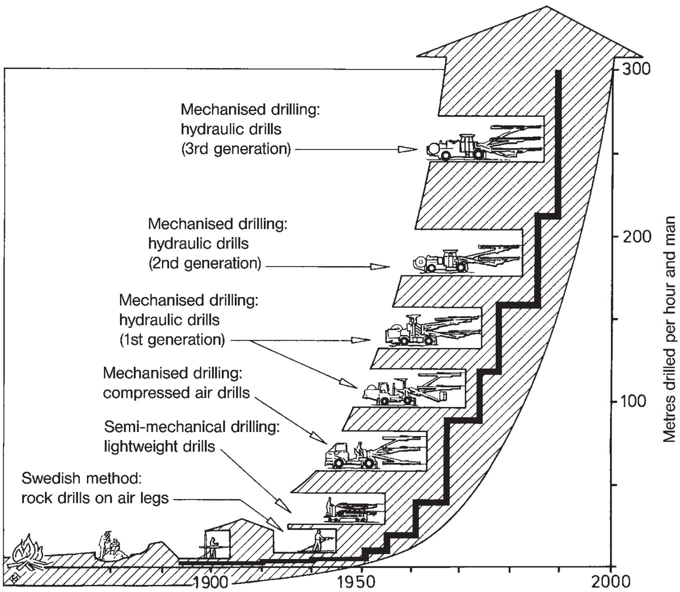


Figure 5-3 Development of mechanical drilling equipment as achievable drilling performance in m/h

In the past, blasting technology had to be planned within the limitations of the available drilling technology. Modern machinery is shortening ever more the time taken for drilling, which lies on the critical path, so that drilling technology today is opening up new methods of blasting, and it is now practical to adapt drilling work to optimise the success of blasting.

The high-performance drilling machinery in modern drill and blast tunnelling can be mounted on wheels, tracks or rails, according to the cross-sectional size and shape. The decision also has to take into account the intended method of muck clearance.

Drilling process. A drill can work by hammer drilling, rotary drilling or rotary hammer drilling according to functional principle. These processes differ mainly in the type of rock destruction and energy transfer. In rotary drilling, the rock at the end of the hole is cut by the rotational movement and the pressure on the tip (Fig. 5-4 a). In contrast, in hammer drilling the rock is destroyed by percussion forming a notch (notch impact) (Fig. 5-4 b).

Hydraulic rotary hammer drills have a percussion mechanism and a rotary drive. The substrate is essentially broken by the percussion action, with thrust force being applied to keep the bit in permanent contact with the end of the hole while drilling and the rotary drive also contributing to breaking as the hammer piston withdraws. The common description rotary hammer drilling is not actually quite appropriate in hard rock as the proportion of notch impact breaking performed by the percussion mechanism is much higher than the cutting action of the rotary drive.

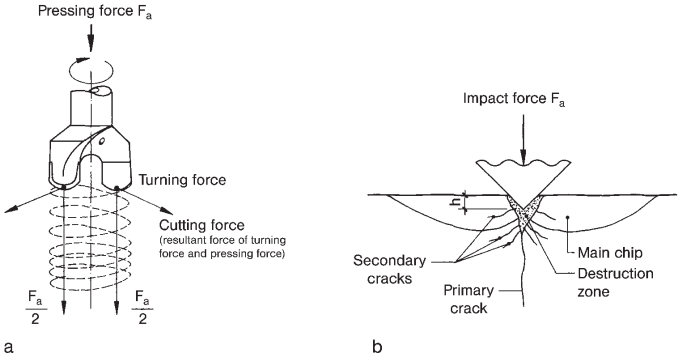
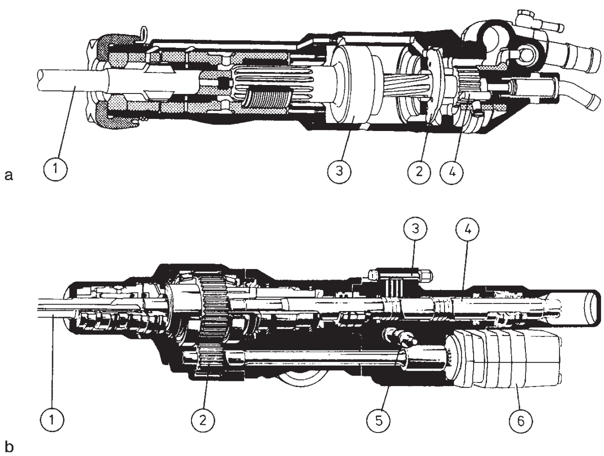


Figure 5-4 Drilling process of cutting in rotary drilling (a) and impaction of a notch in hammer drilling (b), [285]

Hammer drills can be categorised by weight. Light, hand-held hammer drills weighing up to about 35 kg are normally pneumatic, while heavy hammer drills are hydraulic. For subsidiary work and where limited space is available, light pneumatic hammer drills are used. Rotary hammer drills are also sometimes used for probe drilling in loose ground. In modern tunnelling, most drilling is undertaken by heavy pneumatic rotary hammer drills mounted on mobile drilling rigs, also called jumbos or boomers.

Pneumatic hammer drills work on the percussion drilling principle, the rock being broken solely by percussion. The cutting tips of the drill bit break out notches in the end of the hole. Hammer drills work intermittently, with effective breaking only being performed by the hammer impact. As the percussion mechanism is withdrawn, the drill rod is moved before the next impact, i. e. slightly rotated. The rotation angle depends on the stroke of the hammer piston and hammer drills thus do not rotate continuously (Fig. 5-5). Hammer drilling is characterised by loose contact between the drill bit and the end of the hole as the drill rod rotates.

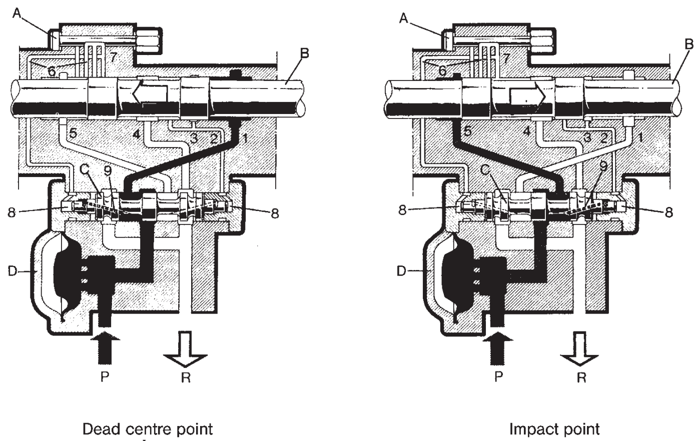


a) Compressed air drill: with each movement forwards, the impact piston 3 hammers to the adapter 1 of the drill rod. After the impact, the compressed air is fed to the back of the piston by the control valve 2 and fed through the mechanism 4 as the pitson returns. The pressure ist 4 to 6 bar.
b) Hydraulic drill: The impact piston 4 is driven by the hydraulics under a pressure of 90 to 250 bar according to type. The torque of the drill rod 1 is provided by the auxiliary rotation motor 6 through the gearing 2. The stroke length is controlled by the regulating stopper 3 and the impact mechanism is controlled by the valve piston 5.

Figure 5-5 Function of a pneumatic hammer drill (a) and a hydraulic hammer drill (b)

All hammer drills achieve their best performance with the optimal thrust force, which depends on the type of rock, the operating air pressure and the skill of the drill operator. When the optimal thrust force is exceeded, the angle rotated with each impact becomes smaller and the piston stroke and the effectiveness of breaking are reduced. Manually guided pneumatic drills on air legs can achieve thrust forces of up to 500 N, although this should not be assumed as continuous.

Hydraulic hammer drills, also called rock drills or drifters, work on the percussion drilling principle. Rotary hammer drills are a combination of rotary drill and hammer drill. They have separate percussion mechanism and rotary drive. Rotary hammer drilling is characterised by continuous contact between the drill bit and the end of the hole (Fig. 5-6). According to the performance class, drifters mounted on feed rails with high thrust forces and high torques of between 120 and 980 Nm can perform uninterrupted breaking even between impacts. The proportions of breaking work performed by the percussion mechanism and the rotary drive depend on the percussion frequency and energy, rotation speed, torque, thrust force, drill bit and the rock type and its properties. Hard rock is broken more by notch impact, soft rock more by cutting.



1 Main channel to rear cylinder space
2 Control channel from rear cylinder space
3 Bleeding channel for valve changeover
4 Return channel
5 Main channel to front cylinder space
6 Control channel from front cylinder space
7 Bleeding channel for valve changeover
8 Reversing surface
9 Holding channel

A Adjustment stopper for stroke length
B Impact piston
C Valve piston
D Accumulator
P Pressure hose
R Return hose

Figure 5-6 Hydraulic hammer drill, impact cycle, valve control and stroke length adjustment [17]

The high weight of hydraulic hammer drills of between 50 and 180 kg and the required thrust force necessitate the mounting of the drill on a feed rail with the drill being advanced by a hydraulically driven chain, cable or telescopic mechanism. Hydraulic drills are relatively long, since the mass required for the percussion energy can only be achieved through the piston length. The length of the drill does however reduce the usable length of the feed.

Hydraulic hammer drills can be adapted to suit variable geological conditions through the hydraulic coupling of percussion mechanism, rotary drive and feed system. When working overhead (rock bolting, installation of instruments), most drills have a pressurised lower part (sealing bush) to prevent the penetration of dirt and water into the drill.

Feed mechanisms. In addition to the drive of the hammer drill, a feed mechanism is required for the appropriate drilling performance. The feed has to provide sufficient thrust force to ensure uniform, permanent thrust of the drill bit onto the rock. Feed mechanisms include air legs, carriages and feed rails. Stopers are special devices for overhead drilling (Fig. 5-9).

Air legs (drill supports) are used for drilling in very restricted space and to support manually guided drills for special purposes. Such light hammer drills supported on telescopic supports are especially useful for emergency bolting after an electric power cut and for smaller-scale drilling work behind the advance. Due to the low weight, an air leg is driven by compressed air and also has a water connection for hole flushing and tool cooling (Fig. 5-7).

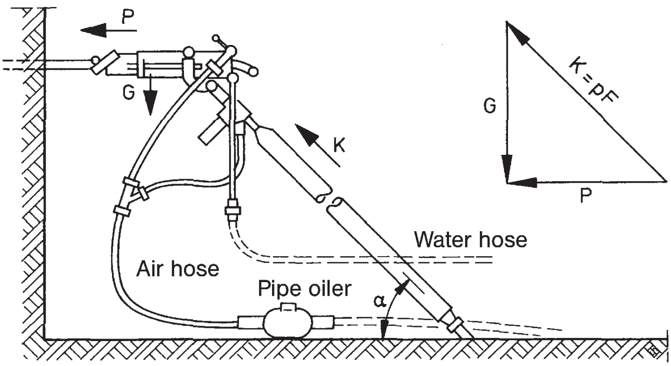


Figure 5-7 The system hammer drill – air leg

The weight of the hammer drill is between 25 and 28 kg, and the telescopic air leg weighs 13 to 21 kg. Air leg hammer drills are very robustly built and are used to drill holes of 27 to 44 mm diameter [17]. The air supply appropriate for the revolution speed is controlled with a lever on the drill and the feed is controlled by a knob on the air leg. Return drill columns enable one-hand operation with central control of drill and support. The operating pressure is 4 to 7 bar, and the air consumption of the drill is 48 to 97 l/s. Depths of up to 6 m can be drilled.

Drill feeds are required for the mechanical support of heavy hydraulic hammer drills, whose drilling performance is determined by the highest possible precisely regulated thrust force. A sliding feed carriage is driven forward while drilling on a feed rail (drill feed) by a spindle or hydraulic cylinder, and a second carriage moves backwards to withdraw the drill. Between the two hydraulically driven carriages runs the hammer drill mounted on a third carriage. The drill carriage is positioned against the two feed carriages by chains or cables running round pulleys (Fig. 5-8).

In this way, the drill carriage can transfer the thrust force to the drill rod with the appropriate elasticity similarly to a block and tackle. The front carriage has a bush to guide the drill rod and the back carriage has rollers for the hydraulic hoses to supply the drill. The thrust force is 10 to 20 kN with hydraulic pressures of 20 to 110 bar. In order to support the drill against the rock surface while drilling, there is a mounting element (steel mandrel, rubber buffer) at the end of the feed. Telescopic feeds have a second feed carriage, on which the drill carriage can be extended forwards by its own hydraulic cylinder independent of the feed length of the drill.

Drill feeds have to be constructed to be robust and stiff against twisting. They are available in fixed lengths or with telescopic extension. Telescopic feeds enable the selection of the drill rod length for top heading, rock bolt and bench drilling. In order to keep manipulation times as short as possible, the drive should be able to withdraw quickly. The feeds used on drilling jumbos in tunnelling can weigh 250 to 650 kg depending on the type of jumbo and drill.

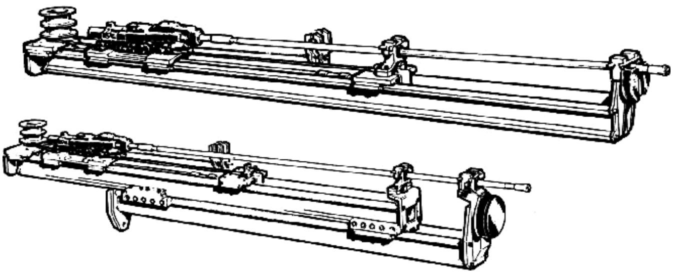


Figure 5-8 Drill feeds, simple and telescopic [17]

Stoper drills are a special version of the column hammer drill for drilling overhead in mining and vertical shafts. A stoper forms a unit with hammer drill and telescopic support (Fig. 5-9) and has a reduced power setting to start the hole. Stopers weigh about 35 to 40 kg and have an air consumption of 4 to 6 m³/min at 6 bar. The telescopic travel is 720 to 920 mm for a single telescope and 1,370 mm for a double telescope.

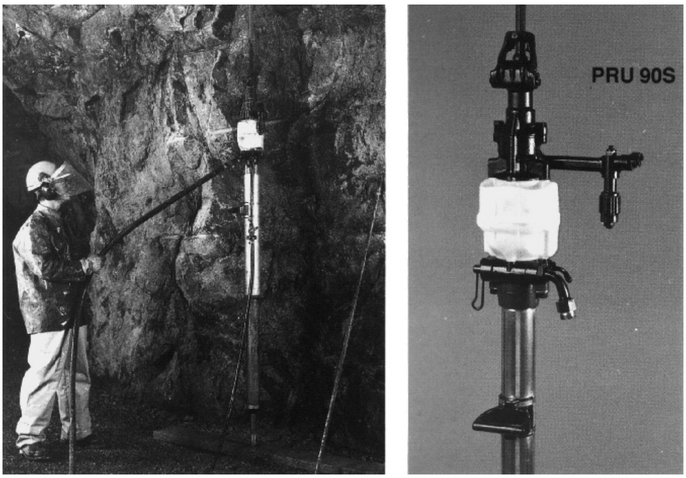


Figure 5-9 Pneumatic overhead hammer drill – stoper [332]

Drilling boom. The boom mounted on the carrier vehicle supports the feed machinery described above together with the drill or another accessory such as for example an access basket. The drilling boom has the task of bringing the drill feed into the correct position and holding it steady. It can be slewed in all directions to place the feed at any position in the cross-section. The number, length and construction of the booms are determined by the size and shape of the excavation cross-section. The thrust and positioning forces of modern booms are 7 to 20 kN.

Cross-sectional areas of 20 to 100 m² can be covered by a hydraulic drilling boom. Drilling rigs generally have two arms, but three are required for larger excavation cross-sections of up to 170 m² (Fig. 5-10).

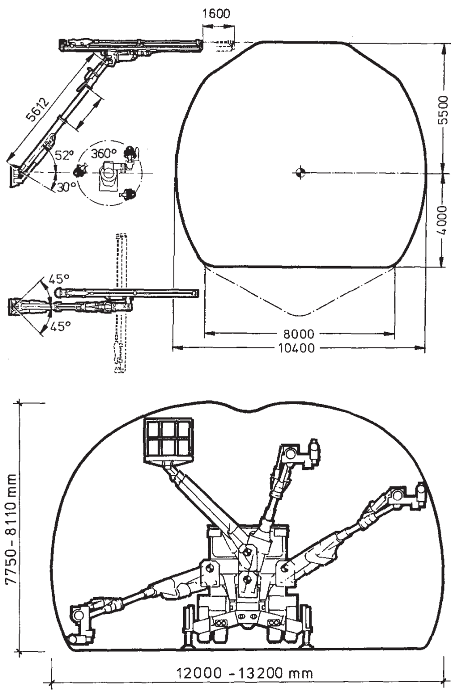


Figure 5-10 Working range of a boom (top) [22] and a drilling jumbo with three arms and a working basket (bottom) [17]

The application of a drilling jumbo determines the selection of the appropriate booms and the degree of freedom of movement. In tunnelling, boom and feed must fulfil the following requirements:

a) Number and direction of holes:

Top heading:	axial and slightly inclined (lookout) blasting and anchor holes, inclined cut holes (wedge, cone), spiles.
Bench:	axial and inclined blasting and anchor holes.
Invert:	vertical blasting and anchor holes.
Tunnel sides:	radial anchor and mounting holes.

b) Constructional requirements (Fig. 5-11)

Lifting and slewing cylinders on the support bracket.
Telescopic travel 800 to 1800 mm (single-stage).
Axial front rotation (rollover) of the telescopic cylinder 360°.
Slewing of the feed carrier ±98° (left – right).
Inclination of the feed carrier +20° (upward) and -100° (downward).
Parallel automatic control during all telescopic, rotating and slewing movements.

If the appropriate booms have been fitted for the cross-section of the tunnel, there should be no “drilling shadows”, or parts of the cross-section that cannot be reached by the booms. Modern booms have direct control, which enables minimal shifting times because any position in the working range can be moved to directly, i.e. diagonally by the shortest route. This is enabled by the provision of a side-by-side layout of two hydraulic cylinders directly behind the rotation mechanism, which can be operated simultaneously and independent of the rotation of the lifting and slewing movements to place the drill feed (Fig. 5-11).

The automatic parallel cut feature enables the automatic maintenance of the chosen feed direction during the shifting process to the next hole position, which normally avoids the need to align the feed again. The exact parallel direction of the drilled holes enables the rock to be broken out parallel, which permits longer round lengths if the cross-section remains constant. The current trend is to greater automation.

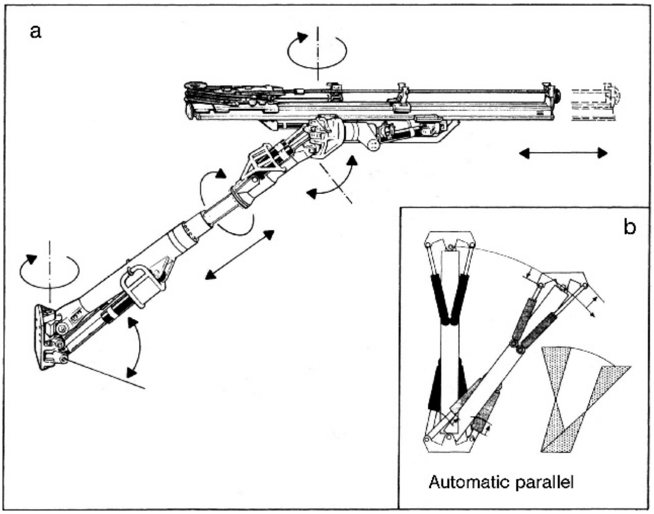


Figure 5-11 Possible movements of a boom with feed (a) and kinematics of the parallel cut feature (b), [17]

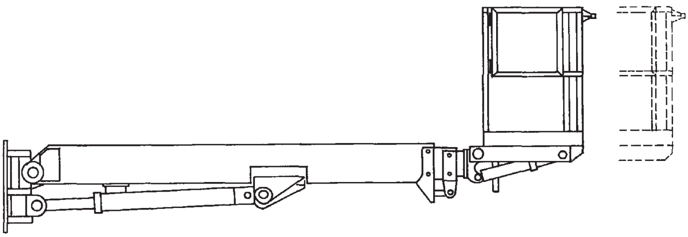


Figure 5-12 Hydraulically telescoping working platform on a drilling jumbo [17]

Working platforms are required for loading charges, support and rock bolting work at height. A working platform is mounted on a multiply telescopic auxiliary boom mounted on the carrier vehicle, and can be moved similarly to a drilling boom (Fig. 5.12). With a vertical movement of 60° upward to 35° downward and a sideways slewing range of ± 45°, the telescopic range can reach up to 11 m according to the machine. With self-weights of 1.5 to 1.9 t, loads of 300 to 400 kg can be transported. The charge loading basket provides space for two miners and has automatic control to keep the working platform horizontal. The platform can be controlled both from the drilling jumbo and from the working platform.

Drilling rigs, also called jumbos or boomers are the mobile carrier unit of a drilling rig. They form a unit with the individual devices drill, feed, boom and working platform and serve as tool carrier. A drilling jumbo has a drive motor, electrical systems, the cable drum for electric supply, auxiliary power units, if required also a high-pressure water pump for Swellex rock bolts, and the control cab. Drilling jumbos are independently mobile and can travel on wheels, tracks or rails. The most common type in tunnelling are wheeled diesel-powered drilling jumbos with electro-hydraulic drilling equipment. Large drilling rigs can have up to three booms (= number of feeds and drills) and two auxiliary booms (working platforms). The individual components such as drills, feeds, booms, working platforms and tool carrier are variable and can be customised to meet the requirements of a particular application (Fig. 5-13).

Drilling work is controlled and monitored by the operators, who are normally the miners, from a central control position. Modern developments [264] provide a choice of semi-automatic (computer-assisted) or fully automatic (computer-controlled) control of the drill, boom and feed. Computer-assisted drilling jumbos [373] show the miner (driller) a pre-programmed drilling pattern (blasting pattern) for the next round on the monitor. The holes are displayed in three dimensions with starting point, angle of inclination and depth (Fig. 5-14).

The actual drilling point and the setting of the feed can be controlled by a miner working under cover. Drilling can proceed fully automatically with self-regulating percussion and feed settings and automatic reaction to the drill jamming. Every process however can be manually controlled if required. All parameters relevant to the drilling are saved automatically and can be retrieved. The geodetic data comparison necessary for the self-control is achieved by positioning the jumbo with a laser beam using a target on the boom.

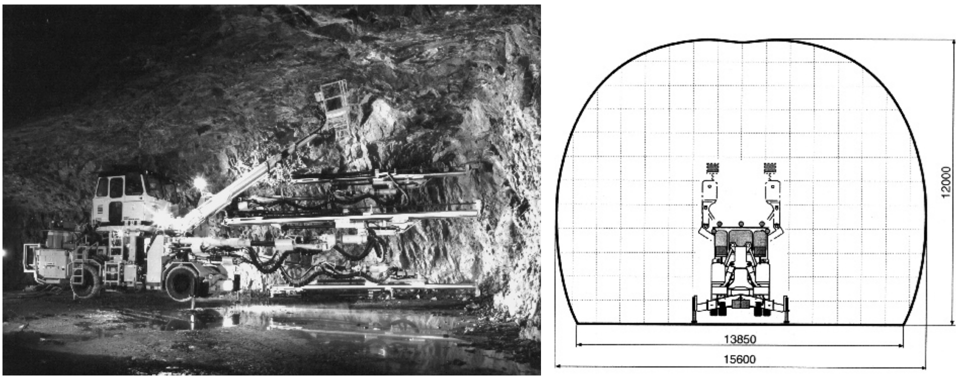


Figure 5-13 Three-armed, wheeled high-performance drilling jumbo with working range [17]

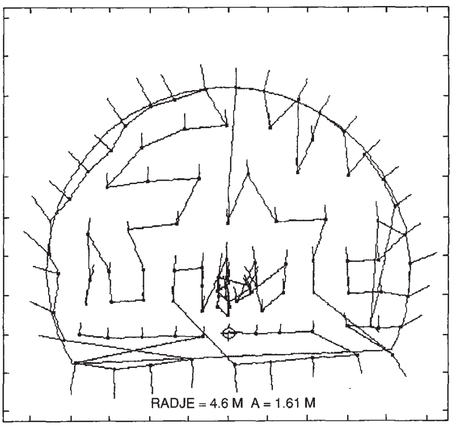


Figure 5-14 Documentation of the recorded drilling sequence [372]

Fully automatic drilling of the complete drilling pattern is possible in principle with the latest machines, although in practice the shifting and drilling processes are watched by experienced miners due to the mostly heterogeneous geological conditions and in order to coordinate several drill booms. The trend of modern developments is to partial and full automation of the individual drilling actions, which are controlled by the microprocessor-controlled jumbo and restrict the activity of the miner to monitoring. A significant step in the direction of full automation can be expected with the installation of a profile measurement system on the drilling rig. This can control optimised drilling with a continuous comparison of tunnel data with the actual drilling performance. Further details of automation are given in section 5.7.

5.2.3 Drill bits

Drilling is a physical process of rock destruction, which can be categorised into four destruction mechanisms (Fig. 5-15). A stress state depending on loading is produced around the contract point of the drilling tool (cutter, button) with the rock [17]:

1. Under the drilling tool (cutter, button), the compression loading creates a zone of fine ground dust.
2. Starting from this ground zone, radial cracks appear in the rock due to the action of splitting tension loading.
3. When the stress in the rock increases and there are sufficient radial cracks present, the shear loading breaks out large chips of rock.
4. The course of movement of the drill bit causes dynamic cyclical loading of the rock.

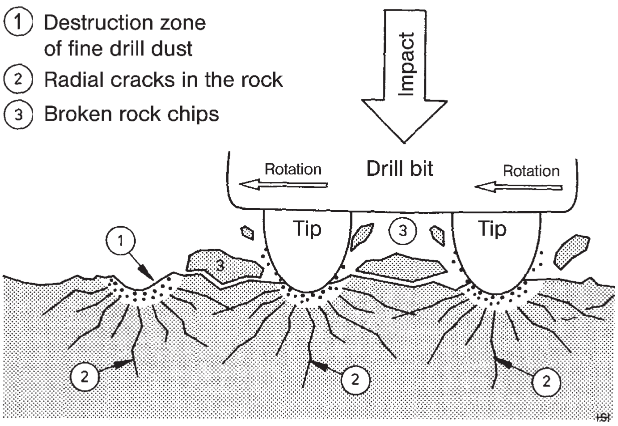


Figure 5-15 Schematic diagram of the destruction mechanism of rock by a button cutter [356]

The drilling tools (drill bit, drill head with hard metal cutters) are the part of the drilling equipment that carry out the actual drilling and grinding of the rock [356]. The term cutter-head can be considered independently of drilling diameter and is used both for small holes (mm) for blasting holes and also for large-scale boring (m) by tunnel boring machines TBM (Table 5-3). The term drill bit is generally used for the drilling of holes for blasting and rock bolts.

Table 5-3 Types of rock drilling and boring tools

Type of drilling	Drilling tool
Hammer drilling	Hammer drill bit single chisel cutter cross chisel cutter, X-cutter etc. button drill bit
Rotational boring	Rotating drill bit disc cutter winged drill bit core cutterhead
Rotational hammer drilling	Rotating hammer drill bits button cutter bit single chisel cutter cross chisel cutter, X-cutter etc.

Drill shaft. The force transfer (impact energy, rotation) and the feed of flushing medium (water, air) between the drill and the drill bit on the rock at the end of the hole is through the drill shaft. The universal term drill shaft can include individual and individually exchangeable shaft elements for various depths of hole and hole diameters, which apart from the monobloc drill can be fitted together with threaded connections to make a multi-part drill string. The individual components are:

– shank adaptors inserted into the drill.
– connecting sleeves.
– drill rods (extension rods).
– drill bits.

Four basic material properties of the drill shaft and drill bit steel are of great importance for the effectiveness and lifetime of drilling equipment (rock drills) [96]:

- Stiffness of the drill rod – minimisation of energy losses.
  – to avoid the hole running off.
- Durability – under compression and tension stresses.
- Toughness – drill shaft – against fracture under high bending loading.
  – hard metal – under compression and tension stresses.
- Wear resistance – drill rod – hardness to ensure long lifetime of the thread
  – hard metal – hardness against extreme impacts, protection when drilling hard rock.

Adapters. The impact energy of the percussion piston in the hammer drill and the torque of rotation are transferred into the drill rod through the shank adapter. If the cross-sectional area of the percussion piston and the adapter are the same, the energy is transferred optimally. The sealing element of the hammer drill and the bond of the drill rod fix the impact surface against the piston. Rotation is transferred through a special toothing behind the collar of the adapter. Shank adapters must be made to high precision due to the high loading of the contact surfaces (impact surface, grooves, threads) and possess very high wear resistance. For this reason, adapters are hardened by case hardening. The high carbon content results in good wear resistance (hardness), durability and stiffness.

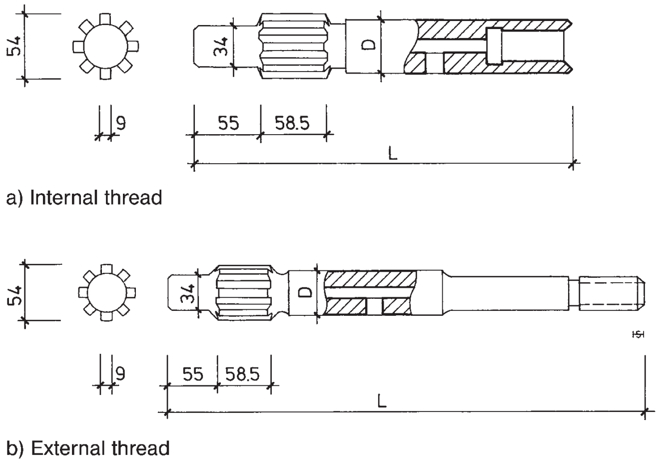


Figure 5-16 Adapters for blasting and rock bolt drilling with a) internal and b) external thread [14]

Adapters with an external thread are connected to the drill rod with sleeves. A thin neck is often provided before the external thread to ensure greater elasticity for the acceptance of high bending stresses under high thrust forces. Adapters with an internal thread do not have a sleeve, the drill rod with external thread is directly inserted. This reduces the projection, which increases the flexibility of the drill in small tunnels (Fig. 5-16).

Connecting sleeves. Adapters with an external thread need a connecting sleeve to make a connection capable of transferring force into the drill rod and ensure optimal transfer of percussion energy. The connecting sleeves have a rib in the middle to stop them moving against the rod.

The selection of a thread depends on the drilling conditions. In tunnelling, R-thread or T-thread can be used. R-threads have a coarse pitch and need less increase of rotation pressure to tighten (force transfer). T-threads are less susceptible to insufficiently tight fitting in unfavourable conditions, as they have more profile and wear volumes with their finer pitch (Fig. 5-17).

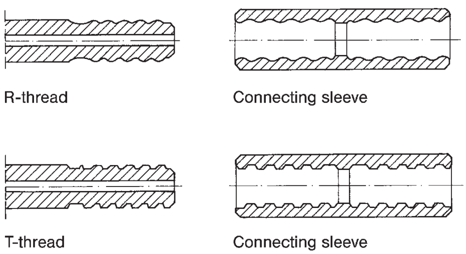


Figure 5-17 Connecting sleeves with R- or T- thread [13]

Drill rods. The connecting elements between the hammer drill (adapter) and the drill bit (cutterhead) are connecting sleeves and drill rods (extension rods). The rods are connected with sleeves to make a continuous drill rod unit of the required length (Fig. 5-17). These transfer the percussion energy, rotation and the flushing medium (air, water) from the drill to the drill bit. In tunnelling, six-sided drill rods are mostly used. The hexagonal section is larger than that of a comparable round rod in order to make the rod harder (Fig. 5-18). The profile with edges also swirls the flushing flow better than round rods and a spanner can be applied to any free length if the rod jams. As with the adapters, most drill rods have a necked transition between the hexagonal shaft and the drill bit thread to better resist bending stresses.

Depending on the rock formation, either drill rods with external thread and casehardened rods (wear resistance for hard rock) or high frequency hardened rods (elastic for jointed rock mass) are available. The largest possible diameter of sleeves and rods ensures the drilling of straight holes. Particularly with small-calibre holes, the drill bits used must however permit free rotation of the drill rod in order to enable an effective flushing flow to remove the debris. In order to dissipate heat produced at the threaded connections and also prevent entry of debris into the threaded connection, drill rods have a flushing groove at the side of the drill bit, through which the flushing medium reaches the thread.

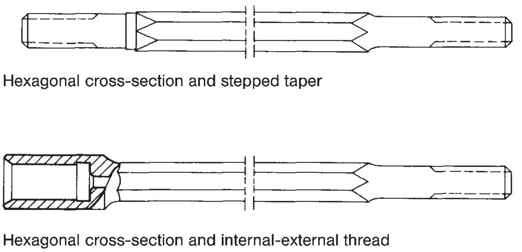


Figure 5-18 Drill rods with external / internal thread

In order to speed up changing of the drill rod and to improve the transfer of percussion energy, special drill rods can also be used with one internal and one external thread for direct connection of the rods without sleeves (Fig. 5-19). This reduces the unavoidable loss of percussion energy at the transfer through the sleeves, as the overall number of connections is less. These drill rods are especially suitable for drilling with automated rod changing for longer holes (probe drilling, bench and extraction drilling).

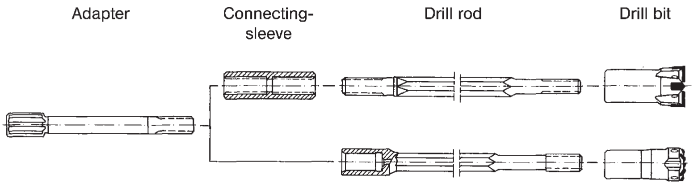


Figure 5-19 Drill rod connections – adapter, sleeve, extension rod, drill bit [15]

Monobloc drills. In addition to drill rods with changeable drill bits, monobloc drills made in one piece with solid forged adapter and drill bit can also be used, depending on the cross-sectional area and drilling equipment. The omission of the detachable drill bit and adapter connections reduces the loss of percussion energy and leads to fewer breakages at these points. This enables a longer length and higher drilling speed to be achieved. On the other hand, when a monobloc drill does break, the keeping of spares is more expensive and regrinding is more laborious.

Drill bit. The drill bit consists of a tool holder made of tool steel, into which the actual hard metal tool inserts that perform the grinding work are set (cold pressing, soldering). For drilling hard rock, quite different shapes of drill bit can be used depending on the application.

The hard metal inserts are mostly buttons or chisels. In modern tunnelling, button drill bits are mostly used. Button drill bits are mostly fitted with six to nine buttons, which can have different shapes. The most common are spherical (round) buttons for fine-grained, abrasive, hard to medium-hard rock. These are wear-resistant and also tough. Ballistic (parabolic, cone-shaped) buttons are suitable for coarse-grained, less abrasive rock. With their bullet-like shape, they penetrate deeper into the bottom of the hole, which results in better drilling performance although this advantage is balanced by increased button wear in harder rock with shorter regrinding intervals. Particularly robust drill bits have larger calibre buttons at the edge of the bit than in the centre (Fig. 5-20).

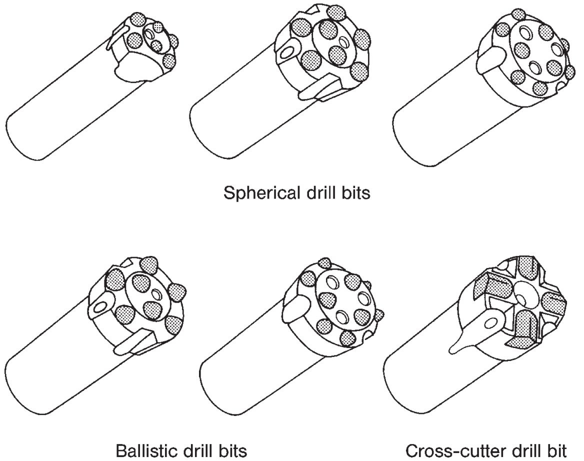


Figure 5-20 Various forms and construction types of drill bit for tunnelling [13]

All buttons can be reground up to ten times using special grinding machines, which can also be operated on site, and this considerably increases the drilling performance particularly in hard and abrasive rock, or at least moderates any loss of performance. The regrinding intervals depend on the type of bit and the type of rock, but the bit should always be reground when the wear surface is a third of the button diameter.

The body of the bit can be flat (hard, compact rock) or have a back cutter. This shape of drill bit enables backwards drilling in case the sides of the hole are unstable in softer, jointed and changeable rock. The better longitudinal guidance of this bit also produces very straight holes, which is particularly interesting for bench drilling.

The front of the drill bit can be flat or with a drop-centre (dished). Flat drill bits have more mass and are thus particularly suitable for very hard rock. Drop-centre bits have better flushing characteristics and produce better drilling performance in straight holes, but are used less in tunnelling with relatively short hole depths, being used mainly in bench drilling (in quarries).

The various types of button drill bits vary not only in construction, shape and number of buttons but also in the number and layout of flushing holes and side flushing channels. The flushing system that is used determines the rapid removal of debris (rock broken up and ground by drilling at the bottom of the hole) and has a significant effect on drilling performance and tool wear. It is important where the flushing flow exits the cutterhead (flushing drilling) and how the debris to be flushed away can get past the cutterhead into the space around the drill rod (Fig. 5-21). The optimal drill bit should therefore be selected for each rock type.

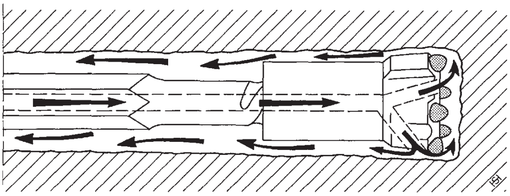


Figure 5-21 Effective debris removal (flushing flow) at the cutterhead of a button drill bit [13]

Cutter drill bits are used less often today, mostly in softer rock. They enable straighter hole guidance than button drill bits, but the drilling progress is slower and the regrinding intervals are shorter. Cutter drill bits can have crossed cutters with diameters between 35 and 64 mm or X-cutters with diameters between 64 and 89 mm.

The body of the drill can be flat (hard, compact rock) or have a back cutter. This bit shape enables backwards drilling in case the sides of the hole are unstable in softer, jointed and changeable rock. The better longitudinal guidance of this bit results in very straight holes. The drill bit diameters for blasting holes lie between 38 and 45 mm, for SN bolts (grouted anchors) 48 mm, for Swellex bolts (anchors expanded with high-pressure water) between 32 and 52 mm.

Enlargement drilling. When the charges are detonated, a free area in the form of a cut is necessary in the centre of the face for the blasted rock to break into. This can be created with conventional drilled holes as a wedge-shaped cut or in modern tunnelling often in the form of a parallel cut consisting of large diameter holes without charges. The large-calibre cutting holes are enlarged from a normal hole in stages. This type of enlargement drilling can be drilled more quickly by a high-performance jumbo than the boring of the entire diameter in one stage.

Before the enlargement, a guide hole is drilled with a normal drill bit (dia. 35 to 45 mm). Then an enlargement or reamer bit (dia. 64 to 127 mm) is used to enlarge the hole to the desired size (Fig. 5-22). To enlarge the hole, the hollow reamer drill bit is inserted into a tapered guide adapter screwed to the drill rod. The seating of the drill bit on the taper ensures the transfer of force from the drill rod. The guide adapter has a chisel-shaped cutter so that is can penetrate into the end of the guide hole and the enlarged hole can be bored to the same depth as the other cutting holes. A special tool is required to loosen the reamer bit from the taper of the guide adapter (knock-of). For special cases, guide adapters with an external thread are used, onto which a normal drill bit can be screwed to lock the reamer bit. The enlargement can also be bored in one or more stages.

Flushing. The drilling speeds possible today in high-performance drilling of 4 m/min and more demand effective flushing to reliably and effectively remove the debris. If the flushing is ineffective, the drilling speed falls.

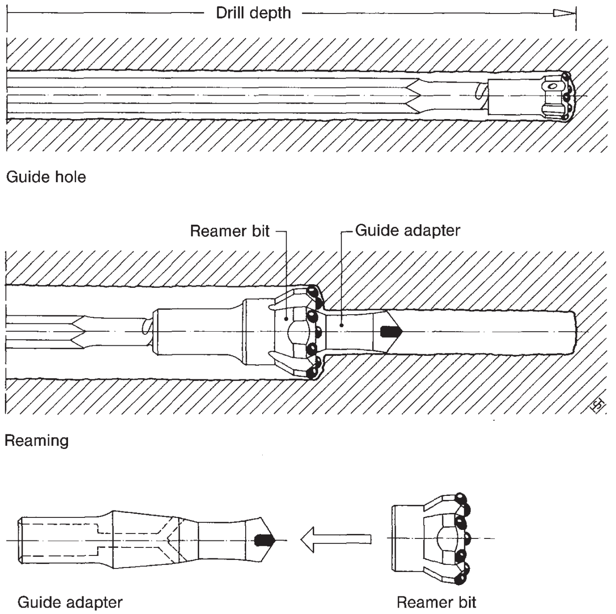


Figure 5-22 Enlargement boring to create a parallel cut for large-diameter holes: 1. Guide hole, 2. Reaming with guide adapter [13]

With central flushing, the flushing medium is fed through a flushing pipe directly into the impact surface of the adapter. With separate flushing, the flushing medium is fed through a sealed flushing head, which is fitted to the adapter with side flushing channels. This enables higher flushing pressure without water penetrating into the hammer drill and harming the lubrication. Modern rock drills have a stainless flushing head integrated in the hammer, which permits flushing pressures of up to 20 bar (Fig. 5-23).

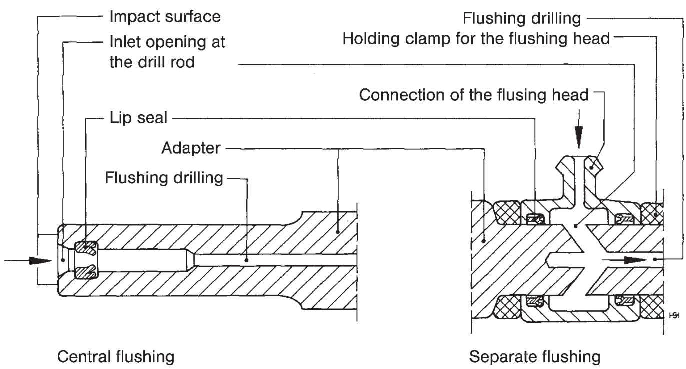


Figure 5-23 Removal of debris and feeding of the flushing medium in the adapter with central and separate flushing

5.2.4 Wear

The drill bit with its fitted tools (buttons, cutters) suffers the worst wear as the leading element of the drilling system. Wear in this case denotes the deterioration of the tool during drilling. Wear is given as the tool life considering the lifetime of a drill bit. The tool life is normally measured in drilled metres per drill bit (m/drill bit). If wear exceeds a certain degree, the drill bit can no longer work effectively and has to be replaced.

Wear to drill bits depends mainly on the content of wear-relevant minerals in the rock [338]. This means minerals with low fissibility from mineral hardness 6 according to MOHS (for example hornblende 6, pyrite 6.5, quartz 7, corundum 9) [281]. In practice, wear and particularly the forecasting of wear are difficult to quantify, as uniform rock formations are seldom encountered. It is normally only possible to determine the average quartz content from laboratory tests for selected homogeneous zones. The wear on drill rods, sleeves and adapters can be considered less significant than drill bit wear, as these drill steel elements are not performing the immediate destruction work and typically last five to ten times as long as button drill bits (Table 5-4).

In addition to the quartz content, various geological factors can also have an influence. For example, investigations from the Inntal Tunnel discovered that the degree of toothing of the microscopic fabric and the porosity of the rock were significant, but these are only recorded locally and can therefore scarcely be predicted reliably for larger zones. Another factor causing wear, although difficult to quantify, can be the effect of high formation water ingress, in which case the fine mineral content washed out of the rock mass can have an effect similar to grinding paste in the drilling debris [127].

Quantitative drill bit wear depending on the abrasiveness of the rock can best be illustrated in the form of a classification of six wear grades with assigned evaluation criteria for the most common wear effects on drill bits.

Table 5-4 Classification of wear types and wear patterns on button drill bits [356]

Wear class	Wear pattern	Typical causes of wear
1	normal wear (blunting) of the hard metal buttons	very abrasive rocks with high compression strength
2	calibre wear (grinding) of the carrier material of the drill bit	abrasive rocks with low compression strength
3	brittle fracture (high shear loading) of buttons, hard metal damage	rocks with locally very high compression strength, jointed rock mass
4	buttons completely broken out, destruction of the hard metal	rocks with very high compression strength, jointed rock mass
5	total wear of the drill bit down to the base of the hard metal buttons	rocks with very high compression strength and/or abrasive rocks
6	button shaft breakage below the button surface	material defect, violent damage, unsuitable button material

5.2.5 Performance

Numerous rock properties determine the success of drilling. In addition to the compression, tension and shear strengths, elasticity properties of the rock also play an important role. The orientation of any jointing structure (anisotropy) has a great influence on the drillability of the rock mass at the bottom of the hole; the highest drilling speeds are achieved when the cleavage is at right angles to the drilling direction (Fig. 5-24 a), as the cleavage offers an ideal surface to break out. If the drilling axis is parallel to the cleavage (Fig. 5-24 b), poor drilling progress is achieved. These effects have to be considered in the rock mechanical investigation of samples from the tunnel route.

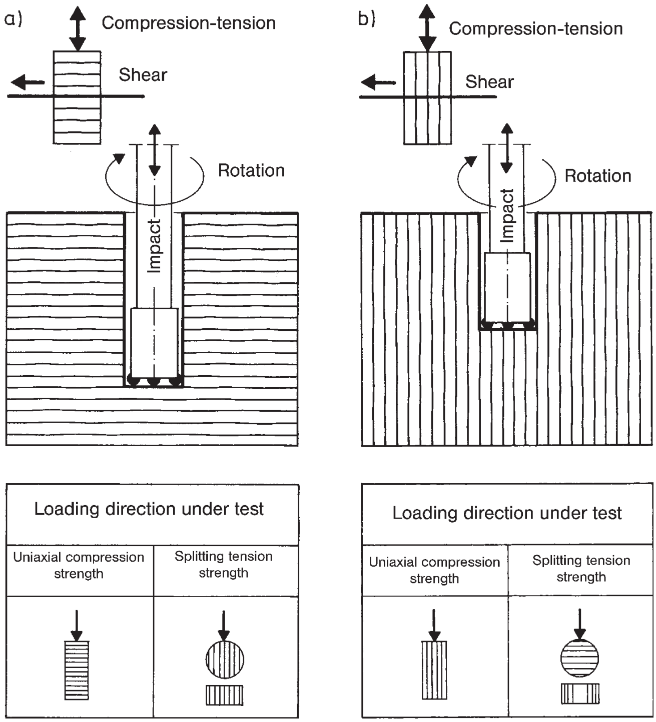


Figure 5-24 Schematic diagram of the drilling process with different orientation of the cleavage structure [356]

In conventional tunnelling in hard rock, the drilling performance of blast hole production is an essential factor affecting the overall advance rate, next to charge loading, spoil clearance and support work. The advance rate is generally measured in metres per unit of time (day, week, month). The drilling performance for each round cannot normally be described in this simple way because only one hole can be drilled by each drill at a time and the overall time taken for each round can only be recognised from the production of all the holes for one round depending on the number of drills working.

Drillability. The overall advance rate is decisively influenced by the drillability of the rock formations encountered [210], [211]. This term is often used in construction but is not strictly defined. Drillability is normally understood as the sum of the interacting effects of drilling progress and drill steel consumption.

Drilling progress is determined mainly by the drilling speed that can be achieved while drilling the blasting holes in the face, and the drill steel consumption is mainly determined by the wear of the drill bits and the other drill steel elements that are less susceptible (adapters, sleeves, extension rods). In addition to these technical factors, the human factors affecting the drill operators should also be considered. The sum of these three factors gives the drillability (Fig. 5-25).

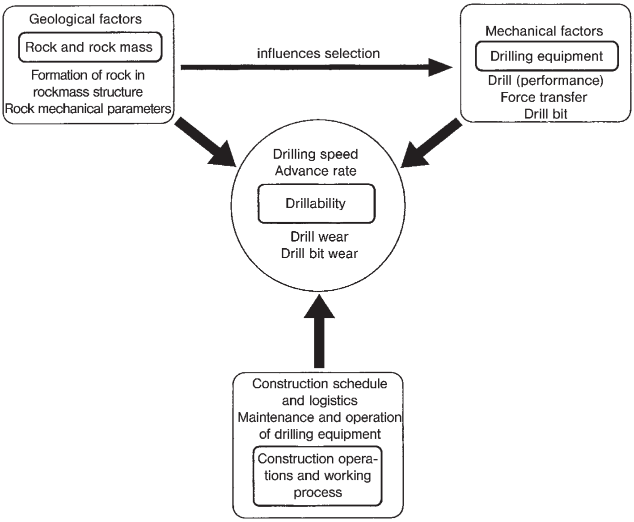


Figure 5-25 Factors influencing the drillability [356]

The factors with an influence on the wear of drill bits have already been described in section 5.2.4.

Factors influencing the drilling speed. The drilling speed is subject to a range of different influences:

– **Machine technology parameters**
  Construction of the drill bit.
  System and performance of the drill.
  Material quality and design of the force transfer.

– **Rock mechanical parameters**
  Destruction work.
  Uniaxial compression strength.
  Splitting strength.
  Modulus of elasticity.
  Dry bulk density.

– **Geological factors**
  Degree of jointing in the rock mass.
  Spatial layout of the cleavage (overall anisotropy).
  Degree of toothing of the microscopic fabric.
  Weathering conditions (or hydrothermal decomposition).
  Material-specific properties.

Increased rock mechanical parameters generally lead to decreased drilling speed. For the evaluation of drilling progress related to a single hole and particularly related to the overall drilling performance of a round, influences of operational procedure can determine the total time of drilling operations.

– **Human factors (experience, skill)**
  Foreman (selection of drill bit, drilling pattern).
  Driller (boom operation, placing drill, drill rod withdrawal).

Assistant driller (delivery of drill steel, rod changing, sleeving).
Blaster and miners (charge loading work, tamping).
Setting of rock bolts.
Equipping and changing times.
Operation errors.

– **Other influences (obstructions)**
Water ingress (flushing, drilling against pressurised water).
Surface structure of the face (drilling point).
Stability of the drilled hole:
Jamming of the drill rod (collapse of the drilled hole).
Collapse of the drilled hole (after withdrawing the drill rod).
Collapse of the drilled hole (during charge loading).
Repeated drilling (new drilling of displaced holes).

As a result of such factors, drilling and still more the following work of loading and tamping charges can be slowed down by considerable difficulties. If the rock quality is very bad or the conditions are very changeable within a round (different rock formations within one round depth), the loading of the charges can be a factor that determines overall progress (redrilling of holes due to the sides falling in, displacement of geological sequence, loading charges under water ingress), particularly when holes have to be redrilled.

The factors influencing drillability can be illustrated in the form of a classification according to the decisive parameters drill bit wear and drilling speed (Table 5-5).

Table 5-5 Classification of drillability according to wear and drilling speed [356]

Drillability	**Wear**	**Drilling speed**	
description	description	description	drilled metres/min 1)
light	very low	very high	4-6
normal	low	high	3-4
difficult	medium	medium	2-3
very difficult	high	low	1-2
extremely difficult	very to extremely high	very to extremely low	< 1

1) Values refer to the drills AC COP 1238 ME and COP 1440

5.2.6 Costs

The costs of a tunnel drive are a combination of numerous cost components, each of which can be split into components of time and performance on the one hand and wage, material and machinery costs on the other. In this section, only the costs for drilling blasting and rock bolt holes are considered. As all costs that arise are specific to one project, there can be no general consideration.

When considering drilling costs, it is noticeable that the stated cost components resulting from conditions affecting drilling, primarily determined by the geology, have an over-proportional significance compared to the usual cost development in civil engineering. The cost of drill steel predominates, the different wear rates affecting drill bits being decisively dependent on geology.

In close relation with the classification of drillability, a table of trends affecting drill bit wear (Table 5-6) can also provide a starting point for coarse cost estimation [356].

Still more seriously, nearly all cost components can be influenced by the time factor (drilling time = part of the construction time). The influence on the performance actually achieved is due above all to obstruction of drilling and charge loading resulting from difficult geological conditions.

Table 5-6 Trends of drillability – drilling speed – drill bit consumption

Drillability	Drilling speed		Wear	Tool life	Drill bit consumption	
description	description	m/min	description	hole metre/bit[1)]	drill bits/ day	drill bits per 1000 m top heading advance
light	very high	> 4	very low	> 2500	< 0.6	< 60
normal	high	3-4	low	1500-2500	0.6-1	60-95
difficult	medium	2-3	medium	1000-1500	1-1.5	95-145
very difficult	low	1-2	high	500-1000	1.5-3	145-290
extremely difficult	very low	< 1	very high	200-500	3-8	290-720
extremely difficult	very low	< 1	extremely high	< 200	> 8	> 720

1) refers to button drill bits with diameters o 43 to 48 mm, mainly diameter 45 mm

5.3 Blasting

5.3.1 General

The term explosion denotes a violent exothermic reaction (oxidation process) of explosive substances detonated by an external energy supply, in which great quantities of gas are suddenly liberated.

Explosive materials include explosives and detonators in addition to pyrotechnic materials and propellants.

Primary explosives are high explosive materials, which detonate from flame. Since they are dangerous to handle, they are not used as explosives but only as a primary charge for detonation.

The general term explosive also includes detonation and firing agents.

Explosives can be divided according to their chemical composition into single compound and mixed explosives, according to their composition and ease of initiation into blasting agents, powder explosives and explosives.

Powder or low explosives can be initiated by just a flame and explode (deflagrate) with a velocity of 400 m/s, high explosives have to be initiated by an explosive detonator and explode at 3,000 to 7,000 m/s.

The development of explosives has had the primary purposes of improving the safety properties and improving the effectiveness and performance of explosives, although of course economy (reduction of cost) has to be considered as well.

The continued progress of development led from gelatin ammonium nitrate explosives to the inexpensive ANFO (ammonium nitrate/fuel oil) blasting agents, which are fully free of blasting oil, mostly resistant to impact and friction and thus extremely safe to use.

Then the so-called slurry or water-gel blasting agents were developed, which mostly contain ammonium nitrate and gel additives in a highly concentrated aqueous solution. In order to make them more sensitive, either explosives like TNT, RDX or other components, mostly aluminium, are added. Due to their water content, slurry blasting agents are largely safe to handle but also difficult to initiate, requiring a booster detonator.

The latest development in the field of aqueous explosives free of explosive material are the emulsion explosives.

Detonators serve to cause an explosion, and can be divided into primary explosive detonators and secondary explosive detonators.

Non-explosive detonators cannot detonate alone and are used to initiate the detonation of a primary explosive detonator. Non-explosive detonators include time fuses, time fuse igniters, simple electrical detonators and shock tubes and initiators (retarders) for non-electric detonation.

Primary explosive detonators are capable of detonation and are used to detonate high explosives. Primary explosive detonators include blasting caps, fuses (quick fuse, detonating fuse), detonation delays, electrical detonators and non-electrical detonators.

The slow fuse (Fig. 5-26) consists of a black powder core with marking threads surrounded by a textile hose (double or triple braiding of jute or cotton) and has an outer casing of plastic or tar for waterproofing.

If the powder core of a slow fuse is ignited, for example by a fuse igniter (simple igniter, match head lighter, pull wire), then it burns with a duration of 120 + 10 s/m. At the end of the slow fuse, a flame emerges, which causes a blasting cap to detonate.

Today the use of slow fuses is restricted to single charges and avalanche blasting; they are insignificant in tunnelling.

The blasting cap developed by Nobel has been in use for many years in basically unaltered form and still forms the basis for today‘s electrical and non-electrical detonators.

The blasting cap (Fig. 5-26) consists of a thin-walled casing of metal (aluminium or copper), the inner housing with holes and the primary charge and the secondary (main) charge).

Only the primary charge contains primary explosive, for which lead azide or lead nitroresorcinate are used, and the secondary charge is typically PETN.

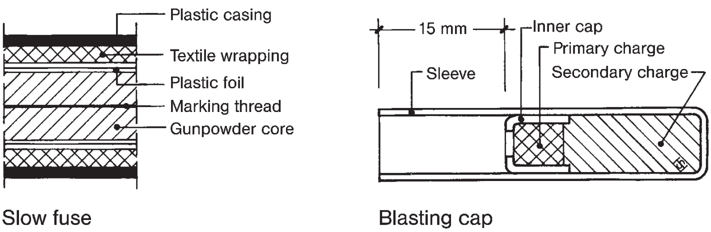


Figure 5-26 Detail of a slow fuse and a blasting cap

The slow fuse is inserted into the end of the blasting cap and crimped with suitable blasting cap pliers. The flame from the slow fuse passes through the holes in the inner housing and detonates the primary explosive in the primary charge. The primary charge than causes the secondary charge to detonate, which is sufficient to detonate the surrounding high explosive.

Development work has led to non-electrical, electrical and electronic detonators.

5.3.2 Explosives in tunnelling

All the explosives used in modern tunnelling are high explosives.

If this sort of explosive is given an initiation impulse, i. e. is caused to detonate by an explosive detonator, then it undergoes an explosive oxidation reaction under the heat development in the explosion gases. This process proceeds with detonation speed through the column of explosive packed in the drilled hole.

This reaction of the explosive in the hole initially acts on the surrounding rock as a percussion, which propagates in waves through the rock and leads to complete crushing of the rock in the immediate vicinity of the detonation and to crack formation further away (percussion phase). Secondary fracture remnants then force the expanding gas from the main reaction into the cracks and widen them (gas phase), which squeezes and throws the rock.

Neighbouring charges detonated at the same time affect and reinforce this destruction of the rock.

Gelatin ammonium saltpetre explosives. All the gelatin ammonium saltpetre explosives used today consist of the main components gelignite and ammonium saltpetre, with further components being aromatic nitrogen compounds, sawdust and colourings.

Presuming good conditions for detonation in the hole, gelatin ammonium saltpetre explosives have a high explosive force, which increases according to their nitroglycerine content (about 15 to 30 % for modern commercial explosives).

Due to both their good water-resistance and handling safety and their high explosive effect, they have largely displaced the dynamite formerly used in tunnelling. They are used above all in hard rock, where their high explosive effect (brisance) is important for the success of blasting.

Powder-form ammonium saltpetre explosives

1. Donarit

Explosives sold under the name Donarit are powder-form ammonium saltpetre explosives with nitroglycerine content of 2 to 8 %. They have less density than the gelatin ammonium saltpetre explosives, a greater displacement effect on the rock and are sensitive to damp due to the hygroscopic properties of the ammonium saltpetre and can only be stored for a certain time.

In tunnelling, Donarit is seldom used due to its disadvantages, mostly depending on the properties of the rock and in weak rock.

2. ANFO (Ammonium nitrate/fuel oil) explosives

ANFO blasting agents have a content of about 5 to 6 % of fluid hydrocarbon, spindle oil or diesel, instead of nitroglycerine.

ANFO explosives have less detonation capability when they get damp or are compacted. They are extremely safe to handle, which means that they cannot be safely detonated by a No. 8 blasting cap. They therefore need a booster charge for example of gelatin explosive, or a more powerful blasting cap.

ANFO explosives are almost always used in loose form. They are the cheapest form of explosive.

In tunnelling, ANFO explosives are seldom used due – compared with gelatin explosives – to their lower detonation velocity, the greater displacement effect, the lack of damp-resistance and the negative oxygen balance when they react.

Emulsion blasting agents. Emulsion blasting agents are an emulsion of water in oil. Ammonium saltpetre dissolved in water is emulsified with mineral oil, which greatly enlarges the contact area between the oxidation agent ammonium saltpetre and the fuel mineral oil. This leads to increased detonation capability compared to the ANFO blasting agents made from the same components.

The sensitivity of emulsion explosive mixtures is improved by the inclusion of air pockets (micro-hollow balls containing gas or air), so that they can be detonated by a blasting cap and thus described as explosives.

The consistency of emulsion blasting agents can be altered by the addition of paraffinated waxes from viscous, pumpable fluids to solids, which are sold in cartridges like gelatin explosives.

Emulsion blasting agents contain no explosive materials and are therefore extremely safe to handle and unintended detonation, for example by drilling into residues, can be virtually ruled out [258].

They can liberate similar amounts of energy to the conventional gelatin explosives but have better detonation behaviour, as the detonation effect and speed do not depend on the enclosure conditions to the same extent as with gelatin explosives.

Emulsion explosives are very suitable for storage and result in low concentration of pollutants in the explosion fumes. Measurements in the laboratory have shown that the reaction of emulsion blasting agents only produces about one tenth to a fifth of the carbon monoxide and a fortieth to twentieth of the nitrous gases compared to those measured for gelatin explosives [275] (gelatin explosives from the former Eastern Bloc countries typically show considerably higher values). Measurements of pollutant concentrations carried out in the Semmering pilot tunnel after blasting with emulsion explosives and with gelatin explosives showed that the use of emulsion blasting agents led to about half the quantity of carbon monoxide and a third of the quantity of nitrous gases compared to the peak values produced by gelatin explosives [274].

All the stated advantages make emulsion explosives very suitable for use in tunnelling, although the geological properties can be relevant, for example for cutting charges in hard rock, where emulsion explosives have about 20 % less density compared to gelatinous explosives.

Safety blasting explosives are gelatin explosives, which do not detonate explosive dust-air mixtures when they are detonated. This is achieved by the addition of extra additives compared to the normal mixture, mostly salt (sodium chloride), which reduce flame formation and explosion temperature.

5.3.3 Detonators and detonation systems in tunnelling

For reasons of clarity, this section deals with detonators and detonation devices and aids together.

Electrical detonation. In an electrical detonator (Fig. 5-27), the detonation wires are connected to two contact plates, which are connected at their ends by a thin wire called the bridge wire. Around the bridge wire is a pyrotechnic priming charge called the blasting cap. When electric current flows through the bridge wire, it glows and ignites the blasting cap, producing a powerful flash.

Similarly to a slow fuse, this sort of simple electrical detonator can be inserted into the explosive cartridge as a primary explosive detonator. The flame passes through the hole of the inner housing and detonates the primary charge, which detonates the secondary charge.

These sorts of primary explosive detonator – the simple electrical detonator and the blasting cap combined in a metal casing – are produced in factories all over the world and available as electrical detonators.

The metal casing normally consists of aluminium. For applications in locations at risk of firedamp, copper casings are used instead due to the high combustion heat of aluminium [307].

Instantaneous detonators and delay detonators differ in their time of action.

Instantaneous detonators are electrical detonators, which detonate at the moment of initiation. Delay detonators only detonate after a predetermined delay after initiation. This works through an intermediate circuit of a pyrotechnic detonation mix between blasting cap and primary charge.

Long period delay detonators and short period delay detonators (millisecond detonators) are in use.

Long period delay detonators are available as half-second detonators, in Germany also as quarter-second detonators, in many graduations with delays differing by 500, 250 or 100 ms.

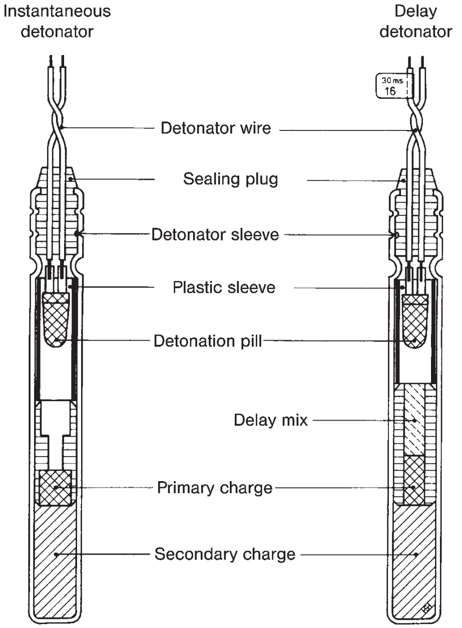


Figure 5-27 Electrical instantaneous and delay detonators [138]

Short period delay detonators are in use as 20, 25, 30, 40 and 80 ms delay detonators and with numerous graduations.

Detonator manufacturers colour code the detonator wire insulation according to the usage in the destination land, so that the type of detonator is recognisable from the ignition wiring. For example, in Austria and Germany, a detonation wire of an instantaneous detonator is white, half-second delay is red, 20 to 40 ms delay from light green to dark green and 80 ms delay detonators have light blue wiring. The delay can also be read from a label fixed on a detonation wire, and the same number is embossed on the bottom of the detonator casing.

One danger affecting electrical detonation is that outside sources of energy (electrostatic charge, extraneous electricity, lightning, electromagnetic waves) could reach a magnitude that would lead to unintended detonation of the electrical detonator.

Unintended detonation of electrical detonators is prevented by the manufacture of electrical blasting detonators to be antistatic (antistatic wire insulation and a combination of protective sleeves and safety spark paths inside the detonator).

Detonators are rated according to their susceptibility to outside sources of electricity as unsusceptible detonators (U detonators, also called Fiduz detonators in Austria) and highly unsusceptible detonators (HU detonators, also called Polex detonators in Austria).

Unsusceptible detonators offer sufficient safety against unintended early detonation in locations with increased risk of static electricity, for example with the use of explosive charge loading devices or at extremely low air humidity. According to the manufacturer, the current that securely leads to no detonation is between 0.45 and 1 A.

Highly unsusceptible detonators also provide protection against unintended early detonation from atmospheric electricity (in high mountains, near transmitters), the effect of electrical rails or high-voltage transmission lines or extraneous electricity from high-voltage equipment. Considering the detonation wires are of copper with its low resistance, they have sufficient safety against shunt connections. The current, which securely leads to no detonation, is between 4 and 5 A according to manufacturer.

In Central Europe, electrical detonation is mostly used in tunnelling, traditionally mostly long period delay detonators, and recently increasingly 80 millisecond detonators (40 millisecond detonators only in exceptional cases).

For safety reasons, only highly unsusceptible detonators are used in tunnelling.

Blasting machines. The detonation current for electrical detonation is generated by a specialised blasting machine (Fig. 5-28). Nearly all blasting machines used now are capacitor blasting machines, in which the detonation current is generated by a generator powered by a hand crank, stored in a capacitor and discharged at the time of detonation. In smaller machines, the discharge is automatic once the correct voltage has been reached. In larger machines, a glow lamp shows when the required voltage has been reached, and initiation is by pressing a button or replugging and then turning the handle.

Blasting machines are delivered in various types with different performance and limit resistance.

Blasting machine testing instruments. Each type of blasting machine has an associated test instrument to check its performance without having to connect the machine to the firing circuit (Fig. 5-29). When the blasting machine is operated, a test lamp lights up on the test instrument to show that the blasting machine is delivering the correct performance.

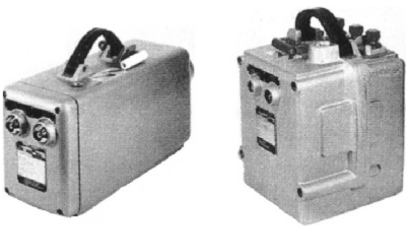


Figure 5-28 Various types of capacitor detonation machines for electric detonation [307]

Figure 5-29 Firing circuit tester (left) and blasting machine test instrument (right) [307]

Firing circuit tester. Another testing instrument in use is the firing circuit tester (Fig. 5-29). These are special ohmmeters with protective resistors, which prevent the battery current entering the firing circuit during the test. Firing circuit testers are used to test continuity to the individual electrical detonators and to test the firing circuit for shunt connections and short circuits and resistance before initiation.

Most of the testers in use today have digital displays apart from older analog instruments.

Firing circuit, connection wires. In order to initiate electrical detonation, blasting lead wires and connection wires still have to be installed. The wires connect the ends of the detonator wires to the blasting lead and if required also to each other. The blasting wires, insulated multicore steel or copper cables, provide the connection between the blasting machine, which is under cover or out of range of the explosion and the detonator wires at the blasting location.

Electronic ignition. A modern development of the electrical detonator is the electronic detonator (Fig. 5-30). Instead of the pyrotechnic ignition mix of the electrical detonator, the delay in an electronic detonator is provided by an integrated circuit before the blasting cap. This enables the ignition time to be configured practically to the microsecond according to the geological conditions and the blasting parameters. Blasting machines have been developed for electronic detonators, which permit free programming of the ignition intervals.

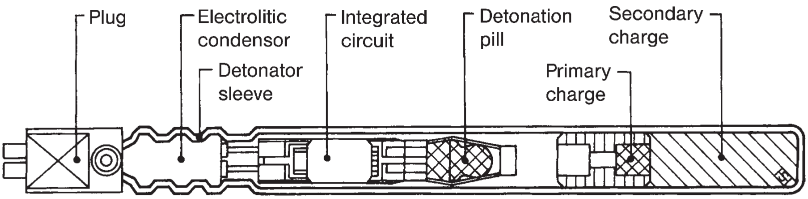


Figure 5-30 Electronic detonator [275]

The sequence of the detonation system is as follows:

1. Unlocking of the detonation circuit.
2. Charging of the capacitors.
3. Programming of the ignition time intervals.
4. Transmission of the detonation signal.
5. Delay period in the detonator.
6. Discharge of the capacitors through the priming cap.
7. Detonation.

The appearance and the layout of the firing circuit are practically identical using electrical or electronic detonators, but there is a considerable price difference with electronic detonators costing many times more than electrical.

The precise detonation within milliseconds of the electronic detonators used for contour shots can achieve an improvement of profile accuracy and reduce trimming costs for over- or underbreak [124]. Positive experience with electronic detonators was gained during the driving of the Mitholz Adit as part of the Lötschberg project in Switzerland between 1995 and 1997 [345].

Blast box. The detonation is controlled by a special blast box (Fig. 5-31), which permits the programming and control of a parallel circuit of up to 200 electronic detonators. The device works with a low operating voltage (approx. 10 Volt), so that the danger of shunt connections in the firing circuit is extremely low.

Figure 5-31 Blast box for electronic detonation [85]

Non-electrical detonation. With non-electrical detonators (Fig. 5-32), detonation is initiated through a shock tube, also called a signal tube, which is a plastic tube with a diameter of about 3 mm coated on the inside with a highly explosive powder, normally HMX (with the addition of aluminium powder) with a quantity of less than 20 mg/m.

When the detonation is initiated, the coating detonates and the front propagates with a speed of about 2,000 m/s inside the shock tube, a detonation process that is not noticeable on the outside of the shock tube.

The flame of the percussive front detonates – analogously to the blasting of an electrical detonator – the delay element and then the primary and secondary charges of the non-electric detonator.

Non-electrical detonators are also available with various delay periods with delays from milliseconds.

When used, particular care should be taken that the shock tube is not mechanically damaged, for example by contact with sharp edges, otherwise the susceptibility of the high explosive coating can lead to failure or misfires.

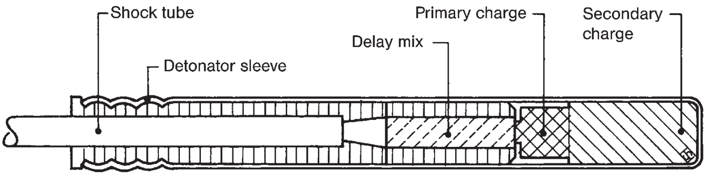


Figure 5-32 Non-electrical detonator [138]

Shock tube starting device. A shock tube starting device (Fig. 5-33) distributes the detonation energy to the individual detonators.

The shock tube starting device or initiator consists of a plastic block and – inserted into it – a certain length of shock tube, which is sealed at the end. In the plastic block is the transfer cap, a blasting cap with a very reduced primary charge (the blasting power is about 1/7 of that of a normal cap). The detonation is initiated through the connected shock tube after a selected delay produced by an intermediate pyrotechnic and detonates the transfer cap. The detonation of the transfer cap is sufficient to detonate all the shock tubes inserted into the starting device and thus detonate through the shock tubes all the connected non-electrical detonators or the next starting device with the selected delay. This permits a building block system to detonate any number of shots with the intended delay from one initiation.

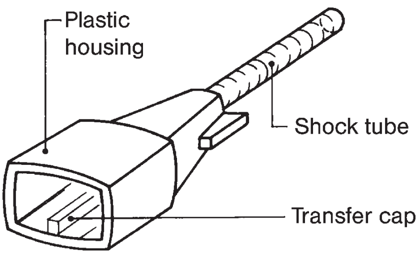


Figure 5-33 Shock tube starting device [138]

Start pistols. Non-electrical systems can also be detonated using the normal explosive detonators like blasting caps, detonators or detonating cord. A start pistol (Fig. 5-34) has an impact pin and blank cartridge, whose energy is sufficient to detonate the shock tube inserted into the start pistol.

Figure 5-34 Start pistol [307]

Detonating cord (quick fuse, detcord). Detonating cords consists of a multiply woven textile cord filled with high explosive, normally PETN, protected against damp by an outer plastic tube. Detonating cords can be detonated by a blasting cap or a detonator, and the detonation front then propagates along the cord at about 7,000 m/s. Detonating cords are used above all to transfer the detonation from the blasting cap or detonator to the explosive, but they can also be used as an explosive.

Detonating cords are used with an explosive mass of 4 to 100 g/m (more in exceptional cases).

In tunnelling, heavy detonating cords are also used very successfully as an explosive for contour shots to cut a profile without too much damage to the surrounding rock, although this is associated with health and safety problems due to the negative oxygen balance when they are used and the high content of toxic gases in the fumes produced.

Delay connectors. In order to achieve the desired delay, delay connectors can be used with detonating cord, which delays the detonation front propagating in the detonating card by 20 or 40 ms.

5.3.4 Transport, storage and handling of explosives

Transport to the site. The transport of explosive materials is a public safety problem and is regulated by various national and international laws and regulations for various forms of transport.

For road transport, the "European Agreement concerning the International Carriage of Dangerous Goods by Road", commonly known as ADR, applies and for rail transport the CIV or "Regulations concerning the international carriage of dangerous goods by rail" or RID applies, and there are also similar regulations covering shipping and air freight.

All these international regulations contain

– Classification of dangerous goods.
– A system for labelling dangerous goods.
– Packaging regulations.
– Transport document regulations.
– Regulation of the means of transport.
– Regulation of the training required for the transport of dangerous goods and
– Regulations concerning combined transport.

Due to the extensive regulation of the transport of explosive materials to the construction site, explosive materials are normally delivered not by the blasting sub-contractor but by the supplier of the explosives. For this reason, no further details are given here about the transport of explosives to the tunnel site.

The contractor responsible for the tunnel does, however, still have responsibility for storage and safe-keeping of explosive materials on the site and their transport from the magazine to the point of use at the tunnel face.

Also of interest for the user are the internationally regulated packaging requirements for explosives, as the labelling on the packaging makes clear the contents and the danger potential.

Packaging regulations. In all international regulations, explosives are categorised as class 1. The following section gives the essential provisions, which are listed in Appendix A of the ADR.

Danger. Explosive materials are categorised into sub-classes according to their danger potential, from sub-class 1.1. (the materials and objects are mass-explosive, which means the entire charge explodes almost instantaneously) to sub-class 1.6. (extremely insensitive explosive materials).

This classification of materials and objects is given in the UN testing handbook based on practical tests.

Compatibility. There is also a classification into compatibility groups. Explosive detonators are normally in compatibility group B, high explosives and detonating cords are in compatibility group D and non-explosive electrical detonators and shock tube starting devices are in compatibility group S.

Materials in compatibility group B must categorically not be stored together with materials in compatibility group D.

Classification. The combination of sub-class and compatibility group is described as the classification code.

Some examples of this classification are:

All high explosives have the classification code 1.1.D, emulsion and ANFO blasting agents are included in 1.5.D.

Detonating cords belong to classification 1.1.D or 1.4.D, blasting caps, detonators and delay connectors 1.1.B or, if not mass-explosive, to 1.4.B.

Labelling. The packaging must show the UN number (from the UN recommendations), the description of the material (emphasised in italics) according to the ADR, for explosives the trade name and the hazard label, which will show the classification code.

The hazard label has an orange diamond shape with black surround, which shows

– for sub-classes 1.1., 1.2., 1.3., a bomb in the upper half and in the lower half the classification code with a small number 1 in the bottom corner and
– for the other sub-classes the sub-class in the upper half and compatibility group in the lower half and also a small number 1 in the bottom corner.

Storage on site. As considerable quantities of explosives are required on a tunnel site, the provision of a dedicated explosive store on the site is unavoidable in most cases.

The magazine should generally be under ground, but can sometimes be above ground.

1. Health dangers

The health dangers from the storage of explosives are essentially limited to gelatin explosives.

In case of fire, it should be borne in mind that explosives burn with the formation of particularly poisonous gases.

2. Accident dangers

Due to the good handling safety of the explosives used in tunnelling, the risk of unintended detonation from external action is indeed improbable, but naturally cannot be ruled out with absolute certainty.

Detonators containing explosives are a much more serious accident risk because they can contain high-explosive primary charges, which can be detonated by flame or impact.

In case of fire, it should be considered that the stored explosives do initially burn but such a fire could result in detonation leading to a catastrophe.

3. Precautionary measures

The appropriate artificial and/or natural ventilation of the magazine can lower the concentration of nitroglycerine vapour sufficiently to exclude any danger to employees.

The primary intention should be to avoid any dangerous mechanical or thermal actions affecting the explosive materials in the site magazine. The secondary intention should be to limit the consequences for people and the effect on the surroundings in the catastrophic case of an unintended detonation.

The first principle of planning a site explosives magazine should therefore be to set it up firstly to be protected against mechanical actions like rock fall, stones thrown by blasting etc., but also secondly that in the catastrophic case of an unintended detonation, the effects on the surroundings from shock wave, vibration and thrown objects but also poisonous gases, should be limited to an absolute minimum.

In every case the explosives magazine should be at an adequate safety distance from transport routes and working and accommodation areas or other buildings. The door of the magazine must be lockable and should face in a direction that would result in the least danger and the least destruction in case of an explosion.

The magazine should also be constructed to offer sufficient resistance against unauthorised entry or mechanical (flying objects, rock fall) and thermal (fire) actions. This can be achieved by placing the magazine in rock or stable ground and by using appropriately robust building materials. Appropriate lightning protection should be provided and the electrical installation should be explosion protected.

The management on site should make sure that no fireplace or oven is situated in the magazine and no materials should be stored apart from explosives. In order to prevent danger from mechanical actions, the handling of explosives (opening the cartons, boxes and packets, issue of the explosives etc.) should take place outside the actual magazine. Any naked flames and smoking should be forbidden in the magazine.

High-explosives and detonating cords are stored separately from explosive detonators with a sufficient distance so that – in case of a catastrophe – any detonation of detonators cannot affect the stored explosives and detonating cords.

Explosives should also not be stored in their entire quantity but in the smallest quantities possible to and separated by such a distance that – in case of a catastrophe – their detonation cannot propagate.

Explosives magazine at the surface (Fig. 5-35). In order to store the large quantities of explosives required, the magazine is set up either in natural rock or stable ground or has an adequately dimensioned cover of fill. An earth bank should be placed in front of the access to the magazine and detonators should be kept in a dedicated magazine with a lock.

Underground explosives magazine. The access to the magazine is provided with many turnings and buffer spaces at the corners in order to reduce the effects of an unintended detonation as far as possible. There must be a sufficient safety distance from the magazine to the tunnel or other protected space (Fig. 5-36).

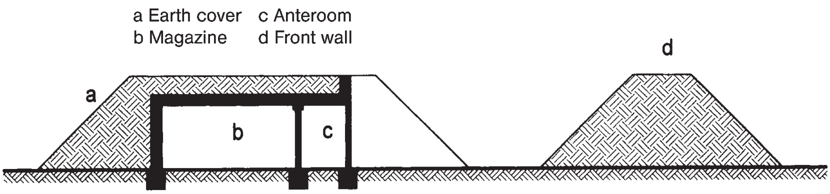


Figure 5-35 Explosives magazine at the surface

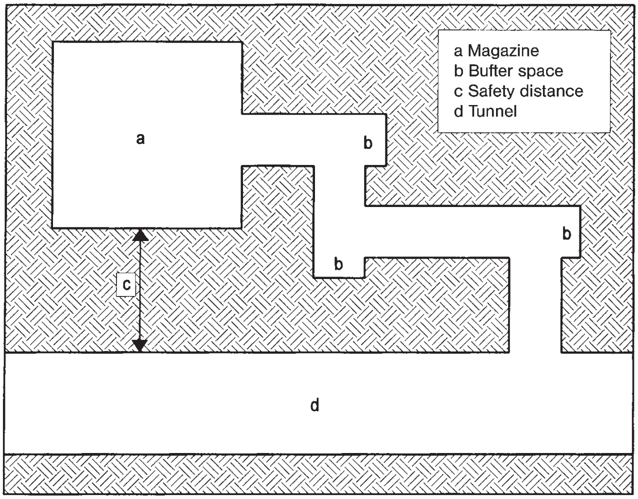


Figure 5-36 Underground explosives magazine

Intermediate magazine. In addition to the actual explosives magazine on the site, it can be practical in a longer tunnel not to collect the explosives required immediately before using them but to transport the explosives required before the start of a shift from the magazine to an intermediate storage point near the tunnel face.

This type of intermediate magazine is laid out for one day‘s requirement and storage can be in a room (container) or smaller quantities in a box (“shot box”).

The same basic principles also applies to such an intermediate magazine, so it should be at a sufficient distance from the tunnel face to be protected against any mechanical or thermal effect, that the magazine should be locked, that explosives and detonators should be stored as far from each other as possible and that explosives should not be stored together with other objects or materials.

Transport on site. Explosives can be transported from the magazine on site to the place they are used at the tunnel face by carrying or on a vehicle.

1. Health risks

The health risks from carrying explosives result from the risk of contact with nitroglycerine through the skin or by inhalation.

2. Accident risks

The potential danger from the transport of explosives results from the unintended detonation of an explosive and above all explosive detonators, which contain primary explosives.

The danger is acute when electrical detonators are transported loose in vehicles without their delivery packaging or other container, in which case contact of the wires with the metal of the vehicle could result in detonation.

3. Precautionary measures

All vehicles used for transporting explosives should be marked as such so that it is clear to all parties that this is an explosive transport.

Explosives containing nitroglycerine should not be touched with the bare hand and protective gloves should be used. The risk of inhaling nitroglycerine vapour can be regarded as slight as long as explosives are transported in their original or other suitable containers.

In order to minimise the risk of accident, the basic principle of separating explosives and detonating cords from detonators applies here as well. Explosives and detonating cords on the one hand and detonators on the other should therefore always be transported in different vehicles or carried by different people. This basic principle should only be contravened for small quantities, which is provided in all national regulations. Small quantities of explosives and detonators can be transported together in a container made of spark-resistant material, but in separate compartments of the container.

All explosives should generally be transported in unopened delivery packaging. If this is not possible, explosives should be transported in suitable containers of spark-resistant material.

When transported in a vehicle, explosives are stowed so that they cannot shift dangerously, secured against sliding, bumps and impacts and against falling out.

Smoking should naturally be forbidden for safety reasons at all times during the transport of explosives.

Handling

1. Health dangers

Nitroglycerine. Gelatin explosives can contain explosive oils like nitroglycerine or a mixture of nitroglycol and nitroglycerine, and there is risk to health involved with hand contact or inhalation of explosive oil vapour.

Nitroglycol and also nitroglycerine can penetrate into the body through the skin or through the windpipe, which is vasodilatory and attacks the coronary blood vessels. The effect of reducing blood pressure can lead to headaches, fainting, feeling sick and loss of consciousness.

Fumes. In the course of working in tunnelling, workers are exposed to numerous potential health risks, including in addition to possible exposure to nitroglycerine the serious problem of inhaling fine dust (primarily resulting from the spraying of shotcrete using the wet and particularly the dry process), diesel motor emissions (nitrous gases and particles), and the toxic gases released by the reaction of explosives (see section 5.6.2).

2. Accident risks

Drilling into explosive residues. The main danger results from the remains of blast holes left over from the previous round. If the driller drills into these and they still contain residues of explosives, the energy from the drill could set off a detonation. Tests have shown that gelatin explosives will probably explode when drilled into, whereas emulsion explosives probably will not explode.

Explosives in muck heaps. Another dangerous possibility is that explosive, which did not explode but was ejected from the hole by blasting, can end up in the muck pile. This danger can be considered slight for cut shots and stoping shots, for which the cartridges are stemmed, even if not all the holes are used. The danger is more real for contour shots, in which the small-calibre cartridges are not stemmed unless the holes are stopped with umbrella plugs.

Particularly dangerous is the extremely rare case that a detonator has not detonated despite being fed the measured current (defective delay element) and the priming cartridge is thrown out into the muck heap. In this case, the mechanical clearing of the muck can result in mechanical damage to the detonator and an explosion.

Loading charges before the completion of drilling. Above all when the tunnel cross-section is very large and the blasters are not at work drilling, work sometimes starts on charging before the drilling has been completed in order to save time. The danger in this case is that an adjacent charged hole is drilled into, causing the charge to detonate.

Firedamp. Another special danger in tunnelling is firedamp, a mixture of methane and air. As methane forms an explosive mixture at a concentration of 5 to 15 % in air, there is a danger even in the absence of blasting work that a spark can cause this mixture to explode, but blasting would surely ignite the explosive mixture with unintended strengthening of the effects of blasting.

Stray currents. Rail transport with electric locomotives can result in a risk of extraneous electricity, which under some circumstances can have enough energy to produce an unintended early detonation of electrical detonators.

Lightning. A lightning strike could also lead to unintended early detonation of electrical detonators if the tunnel has not yet penetrated sufficiently deep into the ground.

Tunnelling from two directions. If two drives are approaching from opposite directions, blasting carried out in one drive can result in danger in the other such as the collapse of blocks of rock etc.

3. Protection measures

In order to protect the miners from health risks resulting from explosive oils, carbon monoxide and NOx gases, the basic principle of the EU directive is that the dangers should be attacked at source – explosives whose use leads to the lowest concentrations of toxic gases in the fumes should be used whenever possible. This undoubtedly speaks for the preferred use of emulsion explosives.

Explosive oil. When emulsion explosives, which contain no explosive oils, are used there can be no ingestion of nitroglycol or nitroglycerine through the skin or the respiratory system. The headaches, which miners frequently complain when charging gelatin explosives, cannot occur when emulsion explosives are used. If gelatin explosives have to be used, any contact with the bare hands should be avoided by using protective gloves.

Fumes. When emulsion explosives are used, their explosion produces a fraction of the dangerous components in the fumes produced by blasting, specifically about half the carbon monoxide and a third of the NOx concentrations compared to gelatin explosives.

In order to further minimise the concentrations of harmful pollutants in the fumes, it should be ensured that the correct detonators are used to initiate explosions (danger of weak initiation). Suitable, fresh and intact explosives with sufficient oxygen balance and the lowest possible production of pollutants should be used. If powder explosives are used, then it is important that they do not become damp and are not tamped.

Further reduction of the concentration of pollutants can be achieved by careful stemming of the shots, especially with water-stemming ampoules, and sufficient spraying of the muck pile where NOx gases could collect.

In any case, adequate ventilation will ensure an appropriate thinning of pollutants.

Drilling into the holes from the previous round. The danger that explosive residues in the remains of the blasting holes from the previous round are drilled into can primarily be reduced by ensuring that the priming cartridges are always at the depth of the holes. This will practically rule out explosive residues remaining after blasting. It is also important to make sure that the remains of holes from the previous round are never drilled again.

Explosives in the muck pile. The use of stemming, particularly in contour shot blasting holes with the provision of umbrella plugs, can practically rule out the ejection of cartridges. The pile should also be closely inspected for unexploded residues before clearing.

Charging before completion of drilling. Unintended drilling into already charged blasting holes can be ruled out by making sure that a safety margin of at least one blasting hole depth is preserved between the drilling point and the nearest charged blasting hole.

Firedamp. Where firedamp is encountered, the unintended detonation of this explosive mixture of methane and air – just a spark can ignite it – has to be avoided. In all tunnels where the geological conditions mean that the occurrence of firedamp cannot be ruled out in advance, instruments should be provided to continuously monitor the concentration of explosive gas. When 50 % of the lower explosion limit for methane is reached (i. e. 2.5 % methane in the air), blasting work should be stopped for safety reasons until the appropriate ventilation has reduced the concentration below this level.

Also wherever firedamp occurrence has to be expected, only safety blasting explosives (and the intended detonators with copper casings) should be used. These do indeed have the disadvantage of less explosive effect but do not detonate firedamp, As an additional measure, the priming cartridge is loaded last, and all blasting holes are carefully filled with stemming.

Stray currents. The danger of early detonation by extraneous electricity is primarily avoided by using highly unsusceptible detonators.

Rails, pipes and cables running parallel are connected electrically at a certain spacing, normally every 50 m, to earth them. The firing circuit and the detonator wires should have an adequate spacing from rails, pipes etc., normally at least 50 cm.

Lightning. When electrical detonators are used near the surface, all blasting work should be stopped when a thunderstorm is advancing and any charges ready for detonation should be removed.

Driving from two directions. When two opposing drives become dangerously near, normally within 25 m, the miners in the other tunnel drive should be alarmed before blasting to enable them to leave their place of work and go to a safe place. When the two drives come within 10 m, work should only be undertaken from one side.

5.3.5 Charge determination

In order to achieve a blast with the desired optimal effect, the correctly designed charge is the most important precondition in addition to the correct arrangement and detonation of the charges.

The determination of charges is part of the preparation of blasting work. It is also of great significance when damage occurs to clarify the situation and determine responsibility.

It is not possible to calculate charges for all blasting and in many cases, also in tunnelling, charge quantities are mostly determined from experience.

Blasting terms

1. Confinement

Confinement is understood as the condition of the rock mass depending on both the primary rock stress and the number and size of free surfaces. In tunnelling, confinement is loosened by blasting a cut to create free surfaces for the subsequent stoping and contour blasts. Further charges are also arranged to create free surfaces for the following charges.

2. Burden

The burden w is the most important factor for the calculation of charges (Fig. 5-37). The burden is defined as the shortest distance from the depth of the blast hole to the nearest free surface.

In the theory of blasting, a concentrated charge at the depth of the blast hole creates under ideal conditions a V cut, in which the radius of the cone corresponds to the burden.

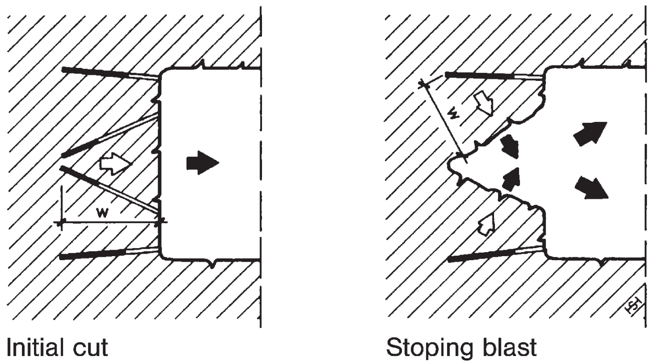


Figure 5-37 Burden for initial cut and stoping blast [123]

3. Round length, round depth

The round length (round depth) depends mainly on the excavation cross-section of the tunnel to be blasted and the rock conditions and thus the support measures required. A rule of thumb for circular cross-sections is that the round length in smaller tunnels is about half of the tunnel diameter, and in larger tunnels about two thirds, or using a parallel cut the round length is about equal to the tunnel diameter (Fig. 5-38). The longest round lengths are about 4 to 4.5 m, and greater lengths mostly fail due to the greater confinement of the rock mass or because drilling is not cost-effective.

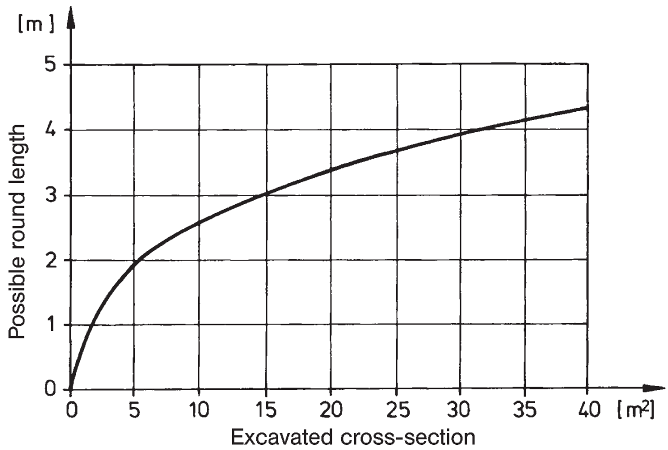


Figure 5-38 Rule of thumb values for possible round lengths depending on the excavated cross-section [376]

In general it can be stated that no generally valid method of calculation of round length is possible due to the numerous factors. The most favourable round length for blasting and the possible round length considering the rock conditions and the resulting requirement of support measures need to be balanced. In general, greater round lengths bring time and cost advantages.

4. Blast hole diameter

The normal hole diameter in tunnelling is 36 to 43 mm, although recently holes have also been drilled with 45 mm diameter or even larger. Large-diameter holes are bored with a diameter of 75 to 200 mm.

5. Cartridge diameter

For cut and stoping blasts, cartridges with a calibre of 30 and 35 mm are used. In order to shorten the charge loading time, cartridges up to 700 mm long and more are available.

Since an air buffer between cartridge and side of the blast hole is necessary to cut the rock cleanly (see section 5.3.6.3), long small-calibre cartridges (more than 1 m long) are used for contour blasts. For gelatin explosives, these cartridges are made with a minimum diameter of 17 mm, for emulsion explosives 25 mm.

6. Hole spacing

The spacing of the blast holes for cuts depends on the type of cut intended.

For stoping blasts, the spacing of the holes is the same as the burden to be thrown. The spacing of blast holes depends on the rock strength and the size of the tunnel section, but particularly on the calibre of the explosive. The normal explosive calibre in tunnelling is capable of throwing about 80 to maximum 110 cm of burden, giving a maximum spacing of stoping blasts of 110 cm.

For contour blasting, the holes are arranged as close to each other as possible around the edge of the profile to achieve the cleanest possible cutting of the rock. The spacing is normally 60 cm for thin cartridges or 40 cm for detonating cord.

Basics of charge calculation. Most charge calculations are based on the charge L being proportional to the volume to be blasted V.

In the simplest case, a charge L throws a cone shape with approximately radius r and height w (Fig. 5-39).

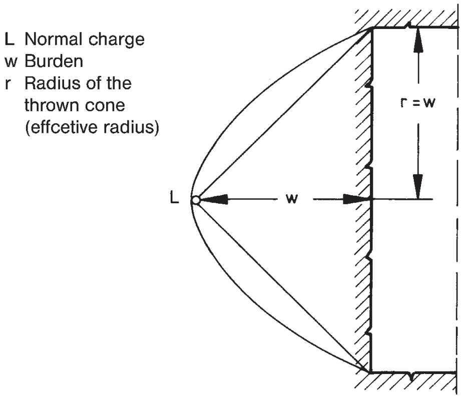


Figure 5-39 Cone thrown out be the explosion of a normal charge

For the case where a cone has an angle of 90° at the peak, this gives $r = w$ and thus a volume proportional to w^3.

With the introduction of a special blasting constant q, this gives an equation for the charge (Chalon formula) [123]:

$$L = q \cdot w^3$$

As the intended blasting result of a right-angles cone is not achieved in all cases, a factor n was introduced, which is a function of the ratio of burden to cone radius.

$$L = q \cdot w^3 \cdot n$$

In this way the charge can be calculated for any shape of cone, under the precondition that the charge is installed at the innermost point of the volume to be blasted.

The formula from Chalon was extended by Lares and Weichelt and the specific blasting constant q was determined as the product of rock, explosive and charge space properties [123], [138].

$$q = f \cdot e \cdot s \cdot v \cdot d$$

with

f = strength factor of the material to be blasted.

e = mechanical value of the explosive (“brisance number”).

s = structural factor for the material to be blasted.

v = confinement of the material to be blasted.

d = tamping of the charge column.

The extended Chalon formula is mostly used for extraction blasting in quarries.

Charge design in tunnelling. The charge in tunnelling could certainly also be calculated using the above formula for cut, stoping and contour holes, working from the appropriate burdens for each hole type.

The use of the Chalon (or other) formula has, however, not become accepted in tunnelling. The rock encountered has too many and too variable properties, which is a general problem for their consideration as factors for purposes of calculation.

In tunnelling, therefore, it is usual to base the blasts on values obtained from experience gained in similar tunnels.

The following basic principles should be considered:

- Cut blasts have to overcome the largest burden, and are therefore charged with the largest charge of the most powerful explosive.
- Cut and stoping holes are normally charged at about two thirds of the hole depth.
- Contour holes are charged with small-diameter cartridges or with detonating cord; for longer round lengths, the provision of normal diameter cartridges at the depth of the hole can be necessary.
- For the determination of the hole spacing, the heaving effect of the explosive should be taken into account; the heaving effect of a 35 mm diameter cartridge is about 80 to 110 cm.
- The consumption of explosive depending on the excavation cross-section and the type of rock can be estimated from the values in Table 5-7.
- The drilling pattern and the charges for subsequent blasting can be corrected according to the results of the first blasts. Changing rock conditions can also be considered in this way.

Table 5-7 Rule of thumb values for explosive consumption [kg/m^3] according to tunnel cross-section and type of rock [85]

Cross-sectional area up to	6 m^2	10 m^2	40 m^2
Soft rock	0.8 to 1.5	0.6 to 1.3	0.3 to 1.0
Marl, loam, clay	Donarit		
Medium-hard rock	2.0 to 2.8	2.0 to 2.5	1.2 to 1.7
Sandstone, limestone, shale	Gelatin explosives, emulsion explosives		
Hard rock	2.5 to 3.5	2.5 to 3.0	1.5 to 2.0
Hard limestone, dolomite, granite	Gelatin explosives, emulsion explosives		
Very hard rock	2.8 to 3.8	3.0 to 3.5	2.0 to 2.5
Hard granite, gneiss, basalt	Gelatin explosives, emulsion explosives		

5.3.6 The drilling and firing pattern

In tunnelling, three types of holes and charges are differentiated:

– The cut to break out the confinement of the face.
– Stoping to enlarge the cavity opened by the cut and attain the intended round length.
– Contour blasting to enlarge the hole cleanly to the intended profile.

Cuts. Two types of cut are used in tunnelling, cuts with angled holes (V cut or fan cut) and with parallel holes (parallel cuts).

1. Angled cuts

The most common types of angled cut in tunnelling are the cone cut, the wedge cut and the fan cut, although many more types are possible by combining these.

When holes are drilled with a drilling jumbo in a smaller tunnel, angled holes cannot be used because the feeds on the drilling jumbo are too long in relation to the available space to be applied at the intended angle.

One advantage of angled cuts is that less drilling accuracy is demanded that with parallel cuts, and slightly inaccurate drilling can still deliver a satisfactory result (round length, muck pile).

It can generally be stated that the trend is to prefer the simplest blasting patterns. Sophisticated drilling patterns are hardly used today with the primary demand being for rapid advance. For this reason and also due to the blasting success, the most common type of angled cut is the wedge cut.

*Conical cut***.** The angled holes are arranged to produce a conical cut (Fig. 5-40). The charges loaded near to each other at the depth of the holes work together with the effect of one concentrated charge. For longer round lengths, one conical cut is normally insufficient and a second staggered cut has to be blasted.

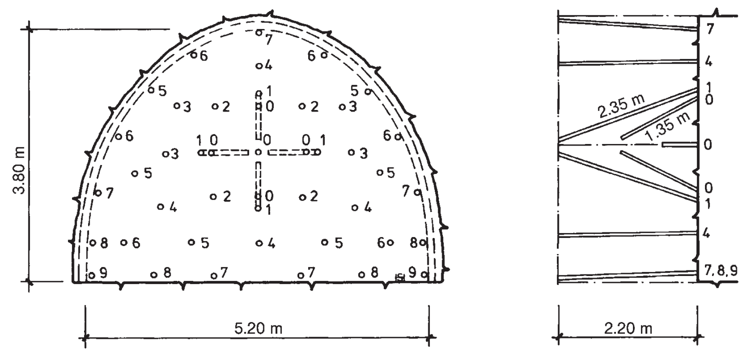


Figure 5-40 Conical cut

*Wedge cut***.** Instead of a cone, a wedge of rock is blasted from a number of vertical or horizontal opposing pairs of holes (Fig. 5-41). For longer round lengths, a second wedge is cut in front.

The wedge cut is popular with blasters on site due to its simple shape, although its relatively strong throwing effect should be considered, especially when the blasting of the wedge is not staged but blasted all together as a whole in one stage.

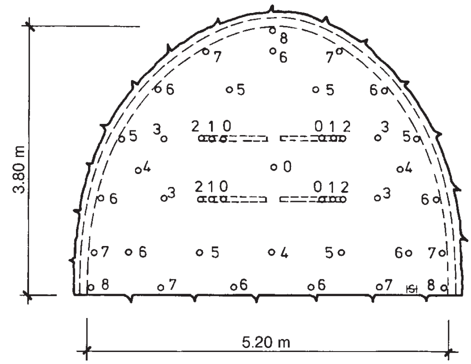


Figure 5-41 Wedge cut

Fan cut. The first holes for this cut are drilled at an acute angle to the face and set either vertically or horizontally with little burden, and the following holes are arranged like a fan until the full round length has been reached (Fig. 5-42).

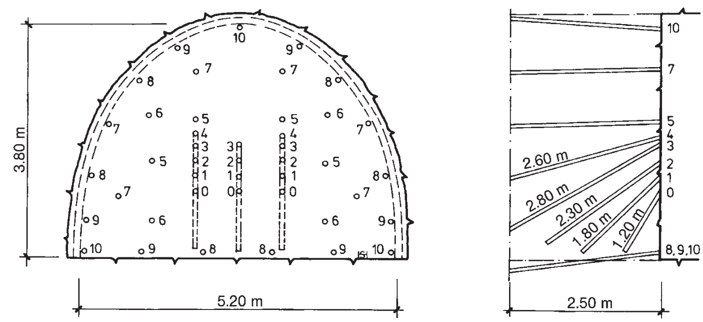


Figure 5-42 Fan cut [376]

2. Parallel cuts

Parallel cuts only use holes parallel to the advance direction and thus at right angles to the face. In order to achieve the intended blasting success, exact drilling to the entire depth is essential, which is only practical using drifters mounted on feeds. Particularly when using drilling jumbos in restricted tunnels, this method enables longer round lengths than angled cuts. Types of parallel cuts include the burn cut and the parallel cut with large diameter uncharged holes, and many further variations are possible.

Burn cut. The burn cut is the oldest form of cut with parallel holes. The success of blasting in this case does not depend on the throwing of the burden as with the angled cuts, but the disintegration effect of the explosive on the rock between the blast holes.

Extra uncharged holes are drilled with a spacing of 8 to 15 cm between the charged holes to enable the ejection of the disintegrated rock between the blast holes. If however the charge is too powerful, then the rock sinters instead of being thrown out. Burn cuts are no longer used today.

Parallel cut with large diameter holes. In this type of cut, the charged blast holes are arranged at burden around the large diameter uncharged hole and thus the actual cut is not blasted but bored. The methods used today include the spiral cut (Fig. 5-43) and the double spiral cut.

In the spiral cut and the double spiral cut, the blast holes are arranged in a spiral around the large diameter uncharged hole. One special type of double spiral cut is the Coromant cut (Fig. 5-44), in which two overlapping holes are bored using a template instead of one large diameter hole. The Coromant cut has proved especially effective for small tunnels with high confinement.

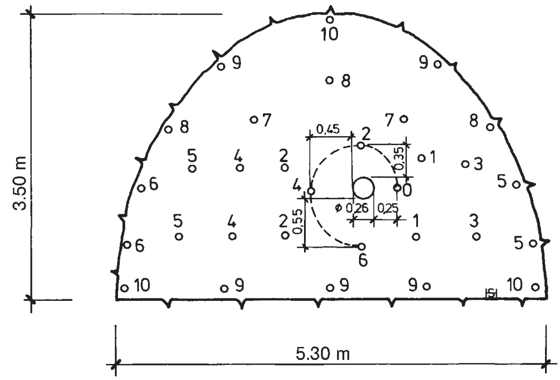


Figure 5-43 Spiral cut [376]

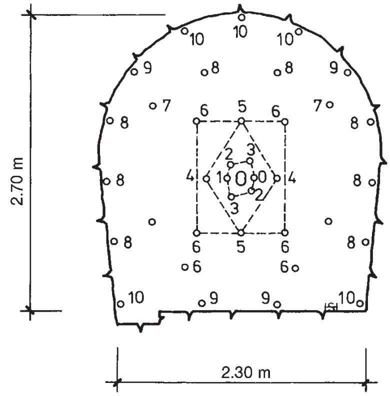


Figure 5-44 Coromant cut [376]

Stope holes. After the initial cut has created a second free surface, the stope holes can be blasted at burden into the cut.

For the most common explosive diameter in tunnelling of 35 mm, it is possible to throw a burden of about 80 to maximum 110 cm in very good rock conditions.

Contour holes. The contour holes have the task of producing the most exact profile possible by smooth blasting. Both overbreak and underbreak lead to increased costs and are thus to be avoided.

1. Smooth blasting

The principle of smooth blasting has the following advantages:

– Precise profiles are created, saving both the increased filling costs resulting from overbreak and the scaling work resulting from underbreak.
– The remaining rock mass is treated gently and cracks and loosening are avoided.
– The vibration of the surroundings is greatly reduced.

These effects are achieved by providing holes at close spacing around the planned contour, or around the profile of a tunnel.

These holes are only lightly charged with thin cartridges or detonating cord so that an air buffer remains between the cartridge and the sides of the holes. This means that the rock is not thrown out or disintegrated, but a fracture gap is created between the holes that ensures the loosening of the rock.

The detonation of the contour charges should be timed as simultaneously as possible so a simultaneous and thus uniform crack can form starting from each hole to the adjacent hole. Simultaneous blasting of the contour charges does, however, have a greater vibration effect.

2. Small diameter cartridges

Specially made small diameter cartridges of gelatin explosives and emulsion explosives with diameters of about 20 to 25 mm smaller than the hole diameter are used for contour blasting. These must however be larger than a critical diameter necessary for secure detonation of the explosive column. This critical diameter is 17 mm for gelatin explosives and 25 mm for emulsion explosives. Contour holes are usually closed with umbrella plugs to prevent the ejection of cartridges.

3. Detonating cord as charge

Heavy detonating cords can be used as explosive for contour blasting and have very good blasting properties for smooth blasting. The high-explosive PETN ensures the appropriate powerful detonation shock wave, which opens a fracture between the contour holes but avoids cracking of the surrounding rock mass and so produces a relatively flat and precise excavation profile [163], [282], [283].

Compared to gelatin explosive, less weight of detonating cords is required for contour blasting – the charge in each blasting hole is only about one third. On the other hand, slightly closer spacing of the holes is normally necessary, so more holes are required. The cutting of the detonating cord to length and the closing of the cut ends to prevent the explosive core running out also require more preparation work.

Then disadvantages of using detonating cord are the higher purchase price, the negative oxygen balance when they explode and the relatively high concentration of carbon monoxide in the fumes (about twenty five times that measured for with emulsion explosives). It should, however, be considered that this is relative to the smaller quantity of explosive in the detonating cord compared to the charges used for cut and stoping blasts.

The firing sequence. Delay detonators are always used. The selection of the delay numbers is based on the principle that the previous charge must have enough time to throw. For this reason, only half-second detonators were used formerly but more recently, 80 millisecond detonators have been used. This normally gives enough time for the throwing of the material, and also favourably influences the individual blasts through overlapping of the ground vibrations and the effect of the gas pressure on the following charges. Only for blasting cuts is it recommended – above all in hard rock – to omit a time step to ensure time to make sure of the throwing of the previous charge.

One fixed rule for the selection of a firing sequence is to detonate the cut charges with the lowest delay numbers. Stoping and contour charges are given higher numbers, which increase with increasing distance from the cut. The contour charges are set off last.

The selection of delay numbers for the individual charges is shown in Fig. 5-40 to Fig. 5-44.

5.3.7 Charge loading

Charge loading equipment and aids

Pneumatic devices. Pressure vessels or ejector units are available for loose blasting agents without cartridges, being used to blow the powder blasting agent (ANFO) into the blast hole.

Pump trucks. Pumpable slurries can be delivered to the site in specialised trucks and mixed from the components on the truck and pumped into the holes.

As however ANFO blasting agents and pumped slurries are seldom used in tunnelling, such trucks are seldom used.

Pneumatic loaders. Pneumatic loaders are available for the charge loading of gelatin explosives or emulsion explosives, in which the explosive cartridge is loaded and shot into the hole with compressed air at up to 5 bar.

For safety reasons, pneumatic loaders are not used when a charge is already in the hole. As however charges in tunnelling are mostly detonated at the depth of the hole, the use of pneumatic loaders remains restricted to special cases.

Loading poles and scrapers. The simple aids used overall to ease charge loading work include loading poles made of plastic or wood, which are used to check the passage of the hole before loading charges and also used to push in and tamp explosive cartridges; and scrapers, which are used to clear material from the hole and also to pull out jammed cartridges, with the exception of priming cartridges.

Primer or detonator cartridges. The cartridges, in which the detonators in tunnelling are initiated, normally electrical detonators or in exceptional cases non-electrical detonators, are called primer cartridges. If various types of explosive are used in one blast hole, then the more powerful explosive should always be chosen for the primer cartridges. The primer cartridge is only prepared immediately before insertion into the hole. A hole is made in the end and the detonator is inserted so that it is fully surrounded by explosive. The base of the primer cartridge always points toward the explosive column.

In tunnelling, only highly unsusceptible detonators are used for electrical detonation. The wires of the electrical detonator are tied in a loop around the primer cartridge and pulled tight. When non-electrical detonators are used, the shock tube is fixed to the primer cartridge with adhesive tape or binding wire. In both cases, the intention it to rule out the detonator pulling out of the primer cartridge.

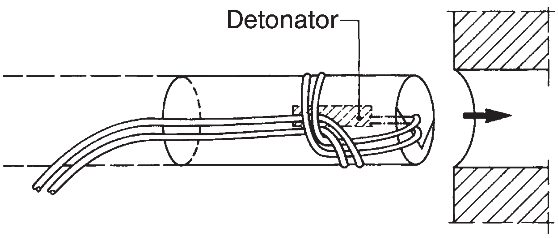


Figure 5-45 Primer cartridge [138]

With regard to the arrangement of the primer cartridge in the explosive column, it can be loaded as the first cartridge at the depth of the hole, as the last cartridge in the mouth of the hole or, in case firedamp is to be expected, as the cartridge next to the first at the depth of the hole.

The technology of blasting suggests that blasting should start from the depth of the hole – detonation starts at the location where the most work is to be done, but there are also safety aspects. When detonation takes place at the mouth of the hole, this can lead to partial misfires, for example that a charge is torn off by the explosion of the adjacent cartridge and thus to the remains of holes containing unexploded explosive residues, which can lead to serious accidents when the old hole is redrilled, although this is forbidden. Detonation from the depth of the hole can indeed also lead to hole remains, but unexploded residues in these holes are extremely unlikely.

One disadvantage of electrical detonation starting from the depth of the hole can be that the detonator wires down into the depth of the hole may have to be inserted past sharp edges. This could damage the insulation of the wire or even worse cut the wire, leading to the charge not receiving an electrical impulse. This type of problem should, however, be noticed when the firing circuit is checked for continuity with a tester and remedied by loading a new primer cartridge.

When non-electrical (shock tube) detonation is used starting from the depth of the hole, the thickness of the tube makes it improbable but not totally impossible that the shock tube is mechanically damaged by sharp rock edges. This could lead to a misfire due to the susceptibility of the explosive in the shock tube to moisture, but cannot be detected by visual inspection.

After consideration of the advantages and disadvantages, detonation from the depth of the hole has become the established practice in tunnelling and is almost always used today.

If detonating cord is used to detonate, then the first cartridge must be firmly bound with the detonating cord (in Austria, the cartridge bound with detonating cord is described as detonating cartridge in order to accentuate the difference from a primer cartridge). This can be done by

– running the detonating cord parallel to the cartridge and binding adhesive tape or wire around them.
– by winding the detonating cord around the cartridge and then knotting the cord.
– or by pulling the detonating cord through a pre-prepared longitudinal hole through the cartridge.

Charge loading. Before the start of charge loading, the loading pole is pushed into the hole to check that it passes, and also to check the depth and direction, and any material in the hole is removed with the scraper or by blowing out the hole with compressed air. Then the primer cartridge is prepared and pushed down to the end of the hole with the loading pole. It is important that the base of the primer cartridge with the detonator is pointing in the direction of the mouth of the hole (Fig. 5-45).

As the primer cartridge is pushed in, the wires are held taut to prevent mechanical damage to the insulation as a result of contact with sharp edges.

In order to charge the hole as densely as possible and thus achieve the best explosive effect, it is advantageous with both gelatin explosives and emulsion explosives to tamp the cartridges by pressing repeatedly with the loading pole.

In order to load the small-diameter cartridges used in contour holes, a slotted plastic pipe can be a useful aid. The thin cartridges are jammed in the pipe to form a rigid rod, which is easy to push into the hole.

Stemming. The loading of stemming (sand, clay, water) at the mouth of the hole is generally practiced, particularly in extraction blasting. Due to the stemming, the energy released by the explosion is used to throw the material, particularly when the explosive column is short.

Another advantage of stemming is that the concentration of harmful components in the fumes produced is lower and that cartridges cannot be torn out of the hole too early. Any loose material can be used for stemming, but stemming cartridges are normally used. Stemming cartridges containing water have an additional safety benefit of reducing the dust produced by blasting and at least partially precipitating NOx gases as they are produced.

Despite these blasting and safety advantages, blasting without stemming is still common practice in tunnelling. Due to the length of the explosive column, the cartridges near the mouth of the hole have a similar effect to stemming, which guarantees correct explosion of the cartridges in the depth of the hole and thus the desired round length. The fact that this leads to large fragments in the muck pile is only of significance in tunnelling when the muck is removed on a conveyor belt. The omission of stemming also naturally leads to time and cost savings.

Contour holes charged with small-diameter cartridges are closed in every case in order to prevent the ejection of the thin cartridges, which are not tamped in the holes, after detonation from the depth of the hole. Due to the smaller charges in the contour holes, stemming of normal thickness could lead to inadequate crack formation between the holes next to the stemming and require more subsequent work. For this reason, contour holes charged with small-diameter cartridges are normally closed with umbrella plugs. When detonating cord is used to charge contour holes, the holes are not normally closed since the ejection of detonating cord from the depth of the hole is unlikely.

Detonation

1. Electrical detonation

The wires projecting from the blast hole are connected to the wires projecting from the adjacent holes to produce and thus connected in series.

This firing circuit is then tested with a firing circuit tester; the resistance is measured and compared to the total resistance calculated from the resistances of the individual detonators and the connection wires. Since highly unsusceptible detonators are always used, there is no safety problem measuring the resistance and continuity before clearing the danger area. If the resistance measured is greater than that calculated, then the wires are mostly poorly connected. If a smaller resistance is measured, then either there is a short circuit or individual detonators have not been connected.

After any defects have been remedied and the measured resistance is the same as the calculated resistance, the firing circuit is connected to the firing line with connection wires. The firing line runs to a safe place where the blasting machine is placed.

Before connecting the firing line to the blasting machine, the resistance of the whole circuit ready for detonation is measured and compared to the calculated total resistance (sum of the resistances of the detonators, connection wires and the firing line). It should also checked that the limit resistance of the blasting machine is not exceeded. After all these checks have been made and the scatter area has been cleared and closed off, the detonation is initiated.

2. Electronic detonation

After the firing circuit has been checked, the electronics of the entire detonation system first have to be released by a coded signal to the programming and control device; then follows the loading of the capacitors of each individual detonator and the programming of the firing delays. The last step is the actual detonation, which can also be initiated by a coded signal to the blasting machine.

3. Non-electrical detonation

When non-electrical detonation is used, the still closed shock tubes projecting from the blast holes are collected into starting device and thus combined in steps into a single shock tube, which leads to the initiator (or other device to initiate detonation) in a safe place.

It is not possible to check the resistance of the competed firing circuit as is done with electrical methods. Checking is thus restricted to visual checking of the tube connections.

5.3.8 Time calculation

In order to estimate the time required for a blast, the individual working steps are investigated [211].

Charge loading. The time required for charge loading can be split into a constant component for each hole t_1 and a component dependent on the length of the hole t_{1L}.

The time required for the following actions is independent of the hole length or nearly so

- for a worker – and with him the required explosives and other aids – to go to the drilled hole,
- for the assignment of the delay detonators to the drilled holes,
- for checking that the holes are free and
- for the preparation and careful insertion of the detonator cartridge.

and the time taken for the following actions is dependent on the length of the hole

- for any clearance of rock debris from the hole that may be necessary and
- for inserting and tamping the remaining charges.

Time estimate. For an experienced and practiced tunnel miner, who is provided with all necessary tools and equipment, the constant component of time required will be about 0.35 min per hole. The time required depending on the length of the hole is estimated at about 0.25 min with long cartridges and 0.35 min with normal cartridges per metre of hole.

- t_1 = 0.35 min per hole.
- t_{1L} = 0.25 min/m of hole length with long cartridges.
- t_{1N} = 0.35 min/m of hole length with normal cartridges.

Stemming. The next step after charge loading consists of closing the hole: stemming cartridges are used for cut and stoping holes, and umbrella plugs are normally used for contour holes.

Time estimate. The time required for umbrella plugs is about 0.15 min, and for stemming cartridges 0.25 min per hole.

- t_{2D} = 0.15 min per hole for umbrella plugs.
- t_{2B} = 0.25 min per hole for stemming cartridges.

Stemming is also often omitted in tunnelling (see also sections 5.3.7.4 and 5.3.9.3).

Connection of the detonator wires. After the loading the charges and perhaps stemming the holes, the detonator wires protruding from the hole are connected to the detonator wires of the adjacent holes – using connecting wire when necessary.

Time estimate. This requires about 0.15 min per hole.

– t_3 = 0.15 min per hole

Connection and checking of the firing circuit, detonation. The next step is to lay the extension wires and the firing line to make the completed firing circuit (connection of the firing circuit with the extension wire), the measurement of the resistance and checking agreement with the calculated resistance (and then the remediation of any defects), the clearing of the scatter area (all machines and equipment on site must be brought to a safe place), going to the safe area and finally the detonation.

Time estimate. For all these actions, 5 to 15 minutes per blast could be required depending on local conditions and the number of machines to be moved out of the danger area.

– t_0 = 5 to 15 min per blast.

Total time for one blast

Duration of the required works. Using the following symbols

– n_B number of holes,
– l_A the reference value for the round length in metres; instead of the (slightly different) drilled hole lengths, the calculation is based on round length as an approximation,
– y_B the reference value for the percentage of holes that are closed with stemming cartridges,
– y_D how many percent of the holes are closed with umbrella plugs,

then the total time required for one blast is:

$$T_A = n_B \cdot l_A \cdot t_{1L,N} + n_B \cdot t_1 + n_B \cdot y_B \cdot t_{2B} + n_B \cdot y_D \cdot t_{2D} + n_B \cdot t_3 + t_0$$

$$T_A = n_B \cdot (l_A \cdot t_{1L,N} + t_1 + y_B \cdot t_{2B} + y_D \cdot t_{2\,D} + t_3) + t_0$$

Necessary working time. The above estimates are based on the assumption that the actions are undertaken by one worker one after the other. In reality, these works are always undertaken by more than one worker, and some of the work can even be undertaken during the drilling work (see section 5.3.9.4). This includes the laying of the firing and detonator wiring. Regarding the time estimate, the most important actions are charge loading and stemming of the drilled holes. Whether and to what extent loading work can also be undertaken at the same time as drilling work depends on the size of the excavated cross-section and whether the miners responsible for the loading are available (and not engaged in drilling).

Using the following symbols

– x the reference value for the percentage of loading work that can be undertaken before the completion of drilling,
– n_A the number of workers who, sensibly and economically active, undertake the loading after the completion of drilling work,

gives the total time that is decisive for construction progress

$$T_{A,x} = \frac{n_B}{n_A} \cdot (1 - x) \cdot (l_A \cdot t_{1L,N} + t_1 + y_B \cdot t_{2B} + y_D \cdot t_{2D} + t_3) + t_0$$

5.3.9 Blasting technology aspects

Selection of explosive. The standard best practice is to select an explosive that both guarantees the success of blasting and is associated with the least risk, meaning it is the safest to handle and results in the least impairment of health.

For hard rock and in order to achieve successful blasting, explosives with the necessary power are required. This precondition is fulfilled by gelatin explosives and emulsion explosives.

Handling. Emulsion explosives are still safer to handle than gelatin explosives; even unintended drilling into explosive residues in the remains of holes from the previous round does not with great probability lead to detonation.

Health aspects. Emulsion explosives contain no explosive oils, so the ingestion of nitroglycol or nitroglycerine through the skin or the windpipe when handling such explosives cannot occur. Headaches during loading work are therefore ruled out when emulsion explosives are used.

The fumes contain – both regarding the peak concentrations of pollutants and also considering the total quantity of pollutants – only a fraction of the toxic contents (see section 5.3.2.3).

The time and cost advantage of the low concentration of toxic gases results from the reduced waiting time after a blast and that the ventilation flow can perhaps be reduced, once the concentration has been measured.

Explosives costs. Compared to gelatin explosives of central European origin, the cost of emulsion explosives is about 15 to 20 % less, but somewhat higher than gelatin explosives from the former Eastern Bloc.

Blasting success. Gelatin explosives and emulsion explosives have about the same brisance or shattering effect, but the density of emulsion explosives is about 20 % less than the density of gelatin explosives. This can have the result that in the locations where high blasting performance is demanded, like in very hard rock or with high confinement, emulsion explosives do not produce the desired blasting success, that for example large remains of drilled holes are left after blasting a cut.

This can be countered to some extent by using about 5 to 10 % more holes with emulsion explosive than with a blast using gelatin explosives, mainly in the cut and in the corners of the tunnel, leading to a reduced burden in these locations.

The desired blasting success can be achieved with minimised health and safety risks when the following apportionment of emulsion explosive – gelatin explosives:

The gelatin explosives are used in very hard rock, above all for blasting cuts, and secondly for contour blasting near the invert and for the priming cartridges of the stoping shots (and perhaps also the second cartridges). On the other hand, not too many different types of explosive are used on the same tunnel site in order to prevent mistakes.

According to the hardness and bedding of the rock, about 70 to 90 % emulsion explosives are used and gelatin explosives only for the remaining 10 to 30 %. Gelatin explosives are used in very hard rock, above all for blasting cuts, and secondly for contour blasting near the invert and for the priming cartridges of the stoping. In medium-hard rock, on the other hand, the use of gelatin explosives for the priming cartridges of cuts is normally sufficient (and perhaps also the second cartridges). It should however be noted that too many types of explosive should not be used on one site to avoid mistakes.

Selection of detonator. In Europe, it is conventional to use electrical detonation almost exclusively. Since detonators are used that are highly unsusceptible to extraneous electricity, the use of electrical detonation is preferred to shock tube detonation in tunnelling since it is safer and detonation is more secure (possible misfires can be detected by measuring the resistance).

The latest applications show that the use of electronic detonation of contour shots, which can be detonated at exactly the correct time, can achieve an improvement of profile precision and less disturbance of the surrounding rock mass, resulting in less follow-up work [124]. The preconditions for the sensible application of electronic detonation, in order that the precise detonation time is relevant at all, are the careful arrangement and precise drilling of the contour holes. A sensible compromise is the use of non-electrical detonators, which are cheaper than electronic detonators, for the cut and stoping shots and electronic detonation of the contour shots.

As a result of the long and consistently positive experience of blasters in tunnelling with electrical detonation, the use of non-electrical detonation will presumably remain restricted to exceptional cases.

Whether the use of electronic detonators pays, and whether the – simply demonstrated – extra cost of the detonators associated with the extra cost of precise drilling of the contour holes is balanced by the – more difficult to quantify – lower costs of follow-up work, may be demonstrated by future investigations of the economics of tunnelling.

Stemming. It is quite usual practice in tunnelling not to close the explosive column with stemming (saving time and money). Due to the length of the explosive columns, the cartridges near to the mouth of the hole have a similar effect to stemming for the cartridges further down the hole and guarantee the correct blast and thus the desired round length without the use of stemming.

Due to the considerable risk that explosive residues could end up in the muck pile, contour holes charged with small-diameter cartridges should always be closed with umbrella plugs (or short stemming). When contour holes are charged with detonating cord, it is not necessary to close the holes.

For cut and stoping shots, safety reasons argue for the provision of stemming cartridges; the considerable extra cost and the lack of justification from blasting technology argue against.

Loading before the completion of drilling. In the applicable national regulations, there are no provisions and no precautionary measures to be found, whether and under what conditions loading can be started before the completion of drilling. The regulators presumably assume that drilling and loading follow one after the other and there are therefore no additional danger situations.

In contrast, it is the usual practice in tunnelling, particularly in large-diameter tunnels, to start loading the drilled holes before the completion of drilling and thus save time and money. This sequence of working poses the additional danger that a hole already containing a charge is unintentionally drilled into causing the charge to detonate.

This danger can be countered – if simultaneous drilling and loading is not to be forbidden – by observing a safety distance between the active drill and the charge already loaded into the adjacent hole, and this safety distance should be at least one hole depth.

5.4 Mucking

5.4.1 General

The clearing and transport away of the material thrown out by blasting is described as mucking. Mucking can be divided in to two activities, loading and transport.

The duration of the work to clear the top heading depends on the type of rock and the round length and can amount to about 15 % (squeezing ground) up to 40 % (stable rock) of the duration of a round cycle [56]. In drill and blast tunnelling, clearance of the muck lies on the critical path, which means that each minute saved here shortens the round cycle and thus the overall construction time, also associated with cost saving. This makes thorough planning and adaptation of mucking operations to local conditions of great importance for both technical and economic reasons. Particularly the matching of the output of loading and transport machinery plays a decisive role. A suitable combination of machines should be selected for each project. The essential criteria for the selection of machinery are [285]:

– The available clearance in cross- and longitudinal section including consideration of its limitation due to supply and disposal installations.
– The transport distances.
– The gradients.
– The excavation quantities per excavation cross-section.
– The characteristics of the excavated material (grain size and shape, grading curve of the muck).

In principle, the available machinery can be categorised into pure loaders, machines capable of loading and conveyance and pure transport or conveyance machines. The first two groups are dealt with in section 5.4.2 “Loading machines”, and the transport machines in section 5.4.3 “Muck conveyance”.

The form of energy used to power the machinery has a decisive influence on the logistics of the entire tunnel drive. Diesel motors are normally used, as the higher cost of ventilating the face can easily be covered by the cheaper purchase cost of standard diesel-powered machines. Electrically driven loading and conveyance machines are therefore only found in larger tunnels in such cases where the length of the tunnel would otherwise over-proportionately increase the cost of ventilation, or else the tunnel is so small that there is no room for large ventilation ducts.

5.4.2 Loading machines

Loading can theoretically also be done by hand, but shovelling is limited to a few exceptional cases (extreme lack of space) and has practically no importance in modern tunnelling. It is therefore not described.

Requirements and types. The task of the loader is to pick up the blasted rock and load it into the transport vehicle. The material is either loaded directly into the container on the transport vehicle or through a feed and transfer device.

Loading machines can be described through four features [314]:

- Loading device (digging bucket, loading bucket, scraper boom, scraper loader, lobster-claws).
- Undercarriage (rail, tracks, wheels).
- Drive (diesel, electric, compressed air, hydraulic).
- Loading system (front, side, overhead, conveyor belt, chain scraper).

Loading machines can be categorised according to various criteria. They can work intermittently or continuously [176], although the latter mostly only work continuously because they load a conveyor belt and the actual loading is discontinuous.

This listing only differentiates the type of loading mechanism between the machine groups excavator with bucket, loaders with front bucket and special loading machines.

Excavator

Construction of an excavator. Hydraulic excavators are now usual, although dragline excavators are still used for special purposes. In tunnelling, practically only hydraulic excavators are used and these are described here. Excavators normally consist of the base machine and various attachments.

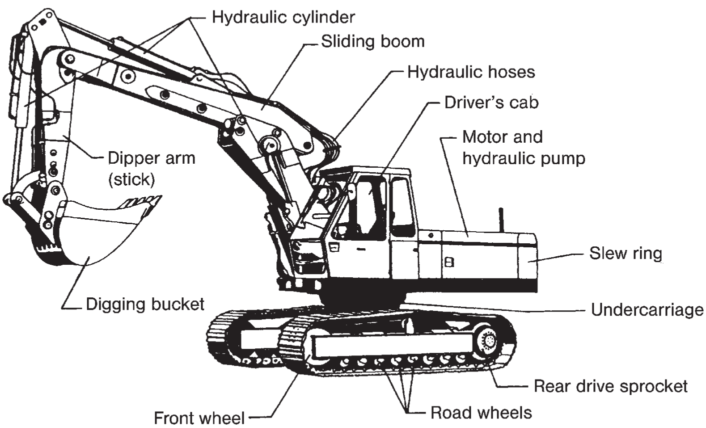
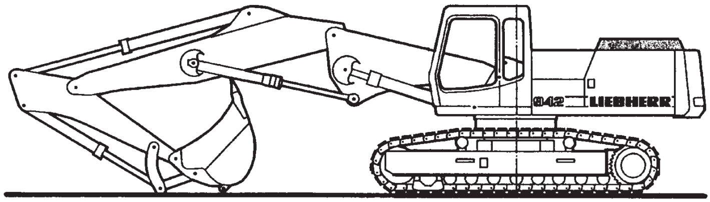


Figure 5-46 General construction of a hydraulic excavator [277]



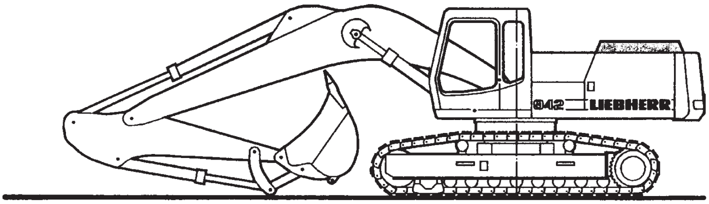


Figure 5-47 Hydraulic excavator with knuckle boom (top) and mono boom (bottom); Liebherr R 942

The base machine consists of undercarriage and the superstructure or house (Fig. 5-46). The undercarriage has, except for special constructions, runs on either tracks or wheels mounted on a stiff frame. This frame also supports the slew ring, which connects the undercarriage to the house and permits 360° rotation.

The drive motor, drive train, fuel tank of diesel machines and the driver‘s cab are mounted in the house, the construction of which is very dependent on the intended application. Next to the cab, which is placed forward at one side to give the river a good view, is the working equipment.

The working equipment can consist of lower boom section, adjustable boom section, stick (or dipper arm) and bucket or a mono boom, stick and bucket (Fig. 5-47).

Conventional hydraulic excavator. Conventional hydraulic excavators with dipper arm and bucket for deep digging (Fig. 5-47) are only used in very large tunnels, in which their great tearing force can also be useful for excavating and where there is sufficient space, for example removing the bench. In pure drill and blast tunnelling in hard rock, conventional hydraulic excavators are seldom used.

Tunnel excavator. The tunnel excavators that are mostly used can also be used for clearing muck.

Specialised excavators in tunnelling

Overhead loading excavator. The overhead loading excavator does not have its own drive but moves itself forwards or backwards using the loading bucket (Fig. 5-48). Instead of rubber tyres on the front axle it has two steel drums fitted with ribs for grip. This type of excavator was developed especially for tunnelling and has a very low construction and good stability.

It is about 15 to 20 % cheaper to purchase than a normal hydraulic excavator [176]. Overhead excavators are fitted with very large overhead buckets, which enable rapid loading of large volumes.

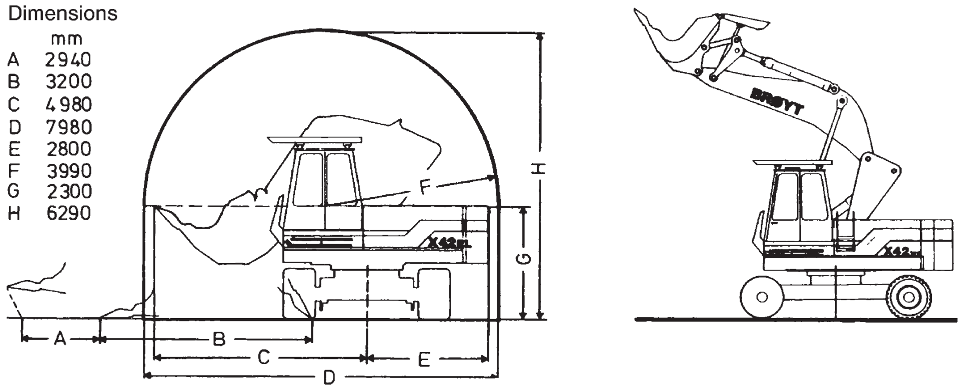


Figure 5-48 Dimensions of standard equipment and side view of the Broyt X 42 EL

Tunnel loading machines. Tunnel loading machines, which can also be called tunnel headers, muckers or conveyor loaders, are fitted with a belt or chain conveyor, which can transport the material to a hopper situated behind the machine. This enables the muck to be loaded onto a conveyor or an additional loader without the machine having to slew. In drill and blast tunnelling, the machine can be fitted with special booms, scratching arms, rippers, rotary cutting units or lobster claws (which would then be a special loading machine), but normally have an excavator boom in order to pull back the muck into the loading ramp, from where it is transported on the conveyor belt (Fig. 5-49).

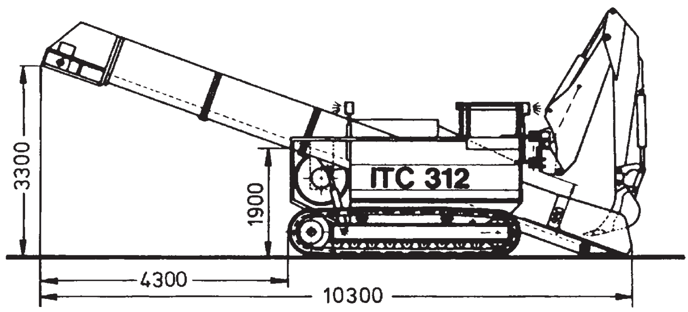


Figure 5-49 Schaeff ITC 312 tunnel loading machine [313]

The machine can travel on tyres, rails or tracks, as with the tunnel loading machines from Schaeff. Pony truck undercarriages, which are wheeled or track undercarriages with retractable rail wheels, are also available. The Schaeff tunnel loading machine is constructed similarly to a normal tunnel excavator and can perform the following movements [266] (Fig. 5-49):

1. Slewing of the entire boom about a vertical axis over the conveyor belt (2 · 55°).
2. Rotation of the boom about a horizontal axis (corresponds to the backward and forward movement of the slew bearing).
3. Slewing of the slewing arm by 2 · 55°.
4. Up and down movement of the intermediate boom about movable axis on the slewing boom.
5. Up and down movement of the bucket arm about a movable axis on the stick.
6. Digging movement of the bucket.

Movement (1.) replaces the slewing of the superstructure of a normal tunnel excavator, and movements (2.) to (6.) correspond to those of a tunnel excavator, except that the fitting of an intermediate boom (4.) provides an additional movement. The width of the feed hopper to the conveyor belt can be adapted freely to suit the tunnel width. The conveyance equipment of a Schaeff tunnel loading machine consists of a hydraulically adjustable loading ramp, a conveyance channel of wear-resistant material and a two-roller chain with drive dogs. The chains are driven by two hydraulic radial piston motors.

These machines can also be used as independent heading machines.

Loaders. Loaders have the big advantage of much better mobility than excavators. For example, tracked excavators have a maximum speed of about 5.6 km/h, while tracked loaders can reach up to 12 km/h [277]. This means that loaders can be used not only for loading but also for transporting over short distances, so wheeled or tracked loaders can be described as combined loading and transport machines.

The base machine runs on wheels or tracks, and the properties of these are similar to excavator undercarriages. Tracked vehicles are more robust and have better stability, while wheeled vehicles have more flexibility and speed.

According to the type of loading bucket movement, loaders can be

– Front loaders
  as wheeled loader (front loader with pneumatic tyres).
  as tracked loader (front loader on tracks).

– Loaders with special bucket types
  including overhead loaders, side-tipping loaders and slewing bucket loaders.

Wheeled loader. As a wheeled vehicle, the wheeled loader has very good agility. Most modern loaders have articulated steering (Fig. 5-50), which makes the machines much more manoeuvrable than loaders with rigid frames and steering axles, with the same stability. The advantages of articulated steering have been summarised in [176]:

– Better longitudinal stability and thus better driving behaviour on bumpy roads, although combined with worse transverse stability.
– Steering is possible while stationary.
– Better agility.
– The front and rear wheels run in the same track.

These advantages make the articulated wheeled loader a powerful and universally usable loading machine, which is often used in larger tunnels. The front bucket requires the performance of loading operations in a V shape, with the machine travelling forwards and backwards to approach the transport vehicle at least nearly at a right angle.

One special type of wheeled loader is the underground loader, which is adapted for working in restricted space (Fig. 5-51). The underground loader, also called a LHD or load haul dumper, has very low construction and is often used in smaller tunnels as a combined loading and transport vehicle.

Generally however the use of wheeled loaders to transport material for longer distances is not very cost-effective due to their small loading volume.

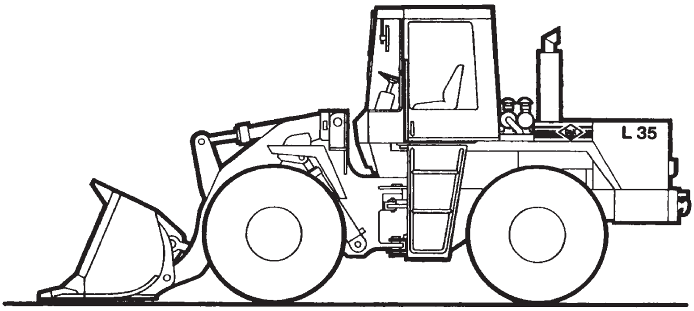


Figure 5-50 Wheeled loader: Orenstein & Koppel L35

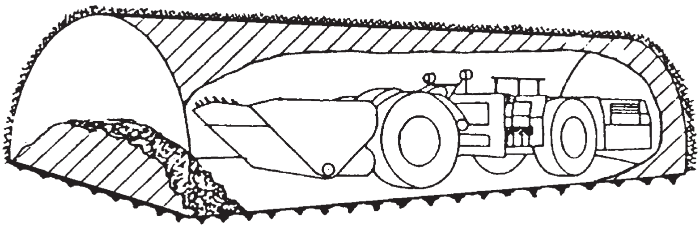


Figure 5-51 Underground loader for work in restricted sections [330]

Tracked loaders. Tracked loaders with front bucket (Fig. 5-52) are used in drill and blast tunnels with restricted space, as the tracks make it possible to turn the vehicle on the spot. This type of operation does, however, put heavy stress on the tracked undercarriage leading to increased wear and repair costs.

Another advantage of the tracked loader is the reduced danger of loosening the invert when the subsoil is soft due to the lower ground pressure of the tracks. This advantage only applies in loose ground when turning on the spot is not necessary, which does not apply in drill and blast tunnels, so tracked loaders are hardly used in drill and blast tunnels.

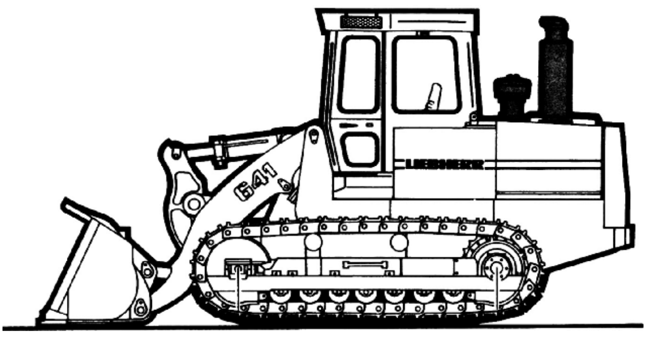


Figure 5-52 Tracked loader; Liebherr LR 641

Loaders with special bucket types. If the width of the available working area is not sufficient for the space-consuming V operation of a wheeled loader, loaders are also available with special types of bucket to operate without turning and thus require less space. All these machines have the feature that they do not have to move forwards and backwards to load and that no great slewing of the machine is necessary (Fig. 5-53).

The most common types of special loader are described below.

Overhead loaders. Overhead loaders are used in tunnels with very small cross-sections. They can be fitted with wheeled, tracked or rail undercarriages. When loading, the shovel is driven forward into the muck pile and then emptied overhead into a transport vehicle standing behind the loader or an intermediate transfer device.

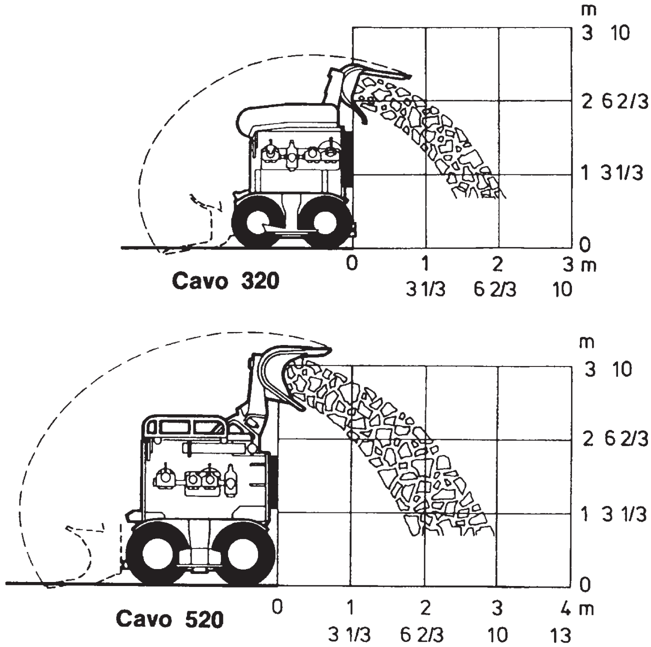


Figure 5-53 Overhead loaders; Atlas Copco Cavo 320 and Cavo 350

Side-tipping loaders. The side tipper performs a forward and backward movement next to the transport container it is loading and then discharges sideways. The longer sliding distance to tip the muck does however increase the danger of cohesive material sticking in the shovel.

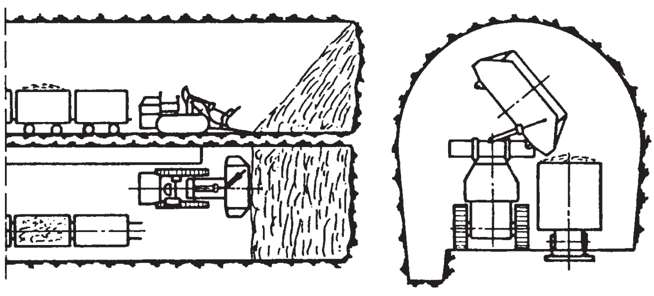


Figure 5-54 Operation of a side-tipping loader [314]

Swing loader. The loading shovel is mounted on a slewing ring to permit slewing of up to 120° to each side. Swing loaders are only made with wheels. In order to load, the shovel is driven into the muck pile like a front loader and then swung over the transport vehicle to discharge (Fig. 5-55).

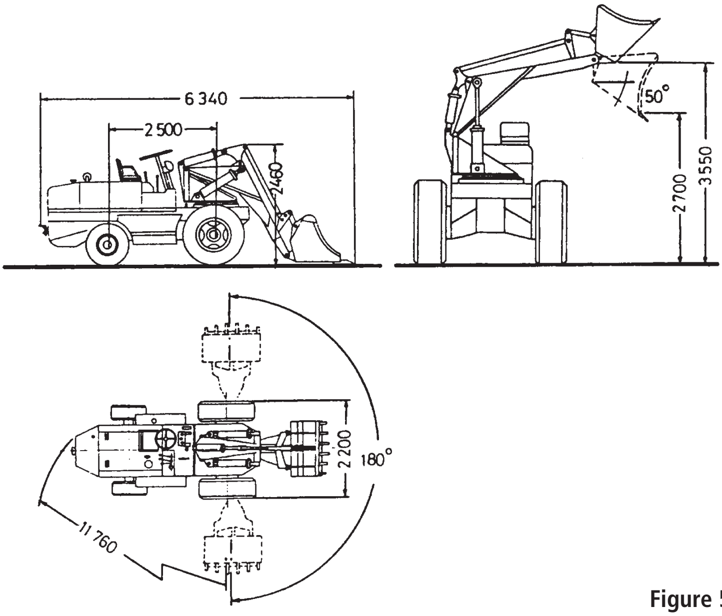


Figure 5-55 Swing loader [285]

Specialised loading machines. Many of the special loading machines for tunnelling that are often mentioned in the literature, like cutting disc loaders or ripper loaders, come from mining and only have historic interest for tunnelling. The most common special device used in tunnels today is the Häggloader from Hägglund (Fig. 5-56).

The Häggloader can be fitted with a rail-borne, tracked or wheeled undercarriage. The machine pulls back the muck with two scratching booms mounted at the middle of its central chain conveyor. The height of the conveyor can be adapted to suit the transport vehicle. The Häggloader is used in small tunnels.

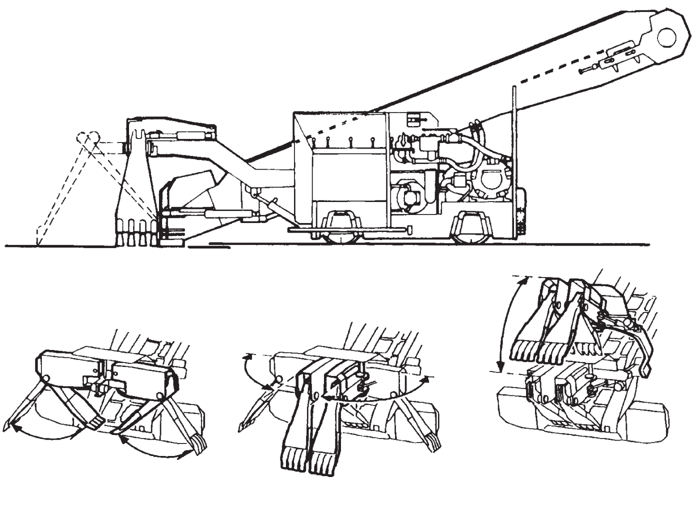


Figure 5-56 View and function of the Häggloader; Atlas Copco Häggloader

The Alpine Loader from Voest is similar to the Häggloader except an overhead loader is used. The Alpine Loader was developed for the clearance of muck in small tunnels of 15 to 35 m² cross-sectional area. The machine runs on crawler tracks and is equipped with an integrated plate or belt conveyor. The loading shovel picks up the muck and tips it backwards onto the plate band conveyor, which transfers it to the conveyor belt, which can be slewed. The conveyor belt feeds the material onto the transport vehicle.

5.4.3 Muck conveyance

Overview of transport devices

Categories of transport devices. The features, which characterise transport devices, can be summarised as

- the undercarriage (wheels, crawler tracks or conveyor belts as a special form of transport by conveyance).
- the drive (diesel, electric or hydraulic).
- the method of emptying (side or overhead tipping, rotation or underfloor emptying).

Transport devices are normally categorised according to the type of undercarriage. These are

- tracked operation.
- trackless operation.
- the special case of (sometimes partial) transport on conveyor belts.

Application criteria. The decisive selection criteria for transport devices are the gradient of the transport route, cross-section and length of the transport route in the tunnel.

A comparison of the advantages of the two transport types mainly used in tunnelling is shown in Table 5-8.

Table 5-8 Comparison of the advantages of trackless and track-borne muck transport in tunnelling

Advantages of trackless mucking	Advantages of track-borne mucking
agility of the transport vehicles	low ventilation costs
reuse of vehicles on other sites is more easily possible	low operational maintenance and energy costs
very high transport capacity in large cross-section	low manning costs for longer routes
can cope with steeper gradients	greater capacity in smaller diameter tunnels

The general rule is that rail operation is of more interest with decreasing size and increasing length of the tunnel, although rail operation is ruled out when the gradient makes rail transport physically no longer possible. This limit is a gradient of about 3 %.

In tunnels with 20 to 10 m² cross-sectional area, rail transport is used almost exclusively [314]. The reason for this is that high-capacity wheeled vehicles have to be large and do not have enough freedom of movement in smaller sections. Passing is either impossible or there is not enough safety distance. In larger sections, the advantages of trackless transport multiply. With the exception of special cases like very long tunnels or compressed air operation, tunnels of more than about 20 m² section all use trackless transport.

Transport on a conveyor belt remains restricted to special applications. Conveyor belts are mostly integrated into loading or transport machinery or serve as an independent transfer system between loading and transport vehicle.

Trackless operation. Wheeled vehicles for transport in tunnelling are specially adapted to the conditions below ground. The essential technical requirements are [285]:

- A low construction height in order to be able to work in restricted sections, to keep the loading times short and in order to be able to pass other vehicles or under obstructions like formwork units,
- Vehicles with articulated steering to enable turning manoeuvres in restricted widths,
- In some cases the provision of two cabs or a rotating cab so that turning manoeuvres are not necessary at all and
- Careful restriction of the motor for the practical speed in the tunnel and integrated exhaust cleaning in order to reduce fuel and ventilation costs.

Dump trucks are the most common transport vehicle in tunnelling (Fig. 5-57). They are available in practically all sizes up to a load capacity of about 35 t. They have a compact, low construction, good climbing ability and good manoeuvrability. As with wheeled loaders, they normally have articulated steering. The front and back parts are connected with a Cardan joint designed to rotate in all directions. This joint and the low-pressure off-road tyres give the vehicle the necessary manoeuvrability and off-road capability for travelling on the normally blasted and unpaved tunnel invert. The combination of dump truck and underground loader pays off for travel distances from about 800 m. Up to this distance, an underground loader can work alone [362].

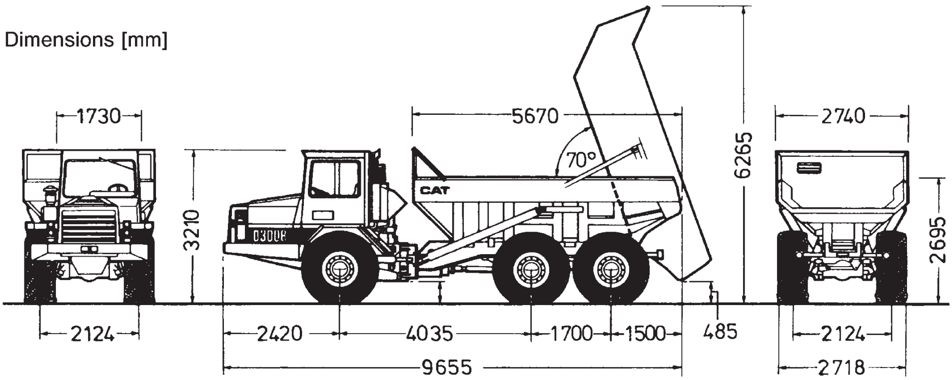


Figure 5-57 Articulated dump truck; Caterpillar D300B

Smaller transport vehicles. Small vehicles are seldom used to transport muck in drill and blast tunnelling since smaller tunnels are normally provided with rail tracks for transport. Front tippers (or dumpers) are available in various sizes, but their use is normally restricted to special cases like very small and very short tunnel sections.

Rail-borne operation

1. Transport vehicles

Locomotives. Compact mine locomotives from the mining industry are also used in tunnelling. For the sizing of a locomotive, the decisive parameters are the weight of the locomotive and the motor power. The locomotive weight determines the pulling force that can be transferred as friction between wheel and rail, and the motor power determines the possible acceleration and running speed. As a first rough estimate, the rule of thumb can be used that the weight of the locomotive should be at least 10 % of the weight of the loaded train. The motor power can be estimated from the ratio of power in kW and tare weight of the loco in tonnes. For diesel locomotives, the ratio is 5 to 7, and for battery locomotives 2 to 5 [313]. The diesel-powered tunnel locomotive type CFL 180 DCL from the company Schöma (Fig. 5-58), for example, has a working weight of 20 t and a motor power of 139 kW, so the ratio in this case is 1 to 6.95 [313].

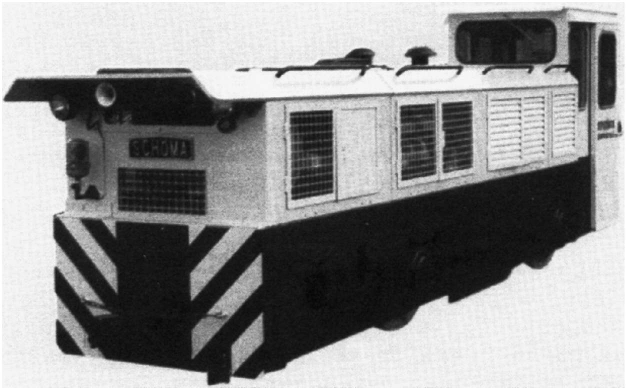


Figure 5-58 Diesel locomotive for tunnelling; LIT type Schöma CFL 180 DCLLIT

Diesel and battery-driven locomotives are mostly used in tunnelling. Compressed air locomotives have lost their market share even in mining, and locomotives with a contact wire for traction are only economical for very long tunnels due to the high cost of installation, and are also subject to much more stringent safety regulations.

Battery locomotives store the traction energy in accumulators, which have to be charged at a charging station. Two battery sets must normally be maintained for each locomotive, and the size of the accumulators should be selected so that they can be swapped at the change of shift. One special form is the combined locomotive, which is operated on battery near the tunnel face and further back with a traction wire.

Diesel locomotives have in contrast to battery locomotives an almost unlimited radius of action and robust construction with the associated low downtime. The cost of purchasing diesel locomotives is also less than that of battery locomotives complete with charging station. But the operating costs are higher than battery locos due to the increased air requirement for the combustion motors.

Transport wagons. The wagons, which serve as transport containers, are made as large as possible for economic reasons. The ratio of wagon width to track gauge must be limited to about 2.2 for stability, and the ratio of wheelbase to the tightest possible curve should not be less than 0.1 to avoid increased wear [314]. Transport wagons are available in many forms, which mainly differ in the construction of the wagon and the method of discharging (Table 5-9), with rotary unloading systems and side-tipping wagons mostly being used in tunnelling. These two types are described in more detail below.

Table 5-9 Overview of the various rail wagon types [314]

Box wagon	small, medium and large transport wagon rotating tipper container wagon
Tipping wagon	tipping wagon box self tipper one-side forced tipper one-side self-discharge
Opening bottom	longitudinal transverse
Other wagons	saddle-bottom sloping floor, one-side self-discharge roller tipper large tipper

The robust rotating tipper has like all box wagons a fixed body with no moving parts except the wheels and couplings. The muck is discharged at a rotating car dumper or tippler (Fig. 5-59). The loaded train is pulled to the rotating tippler by the locomotive and the wagons come to rest singly or in pairs on the tippler. The axis of rotation is the same as that of the freely rotating couplings so that the wagons can be turned over without uncoupling. The relatively high cost of a rotating tippler are compensated by the lower cost of the wagons with an increasing number of wagons, so that rotating tipplers work out cheaper than one-sided self-discharge wagons for large excavation quantities, which require the use of many wagons.

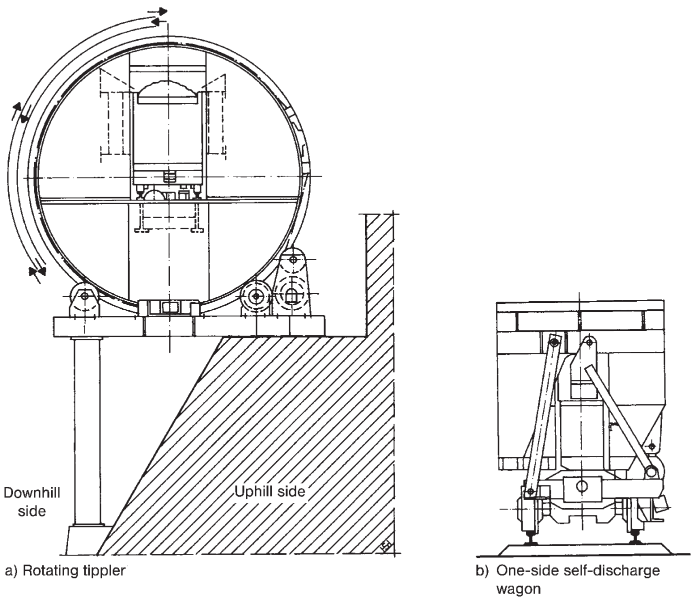


Figure 5-59 Rotating tipper with rotating tippler (left) [314] and one-side self-discharge wagon (right); Mühlhäuser "Zillergründl"

The one-side self-discharge wagon has a transport container, which can be tipped to one side. Dumping is actuated automatically as it passes an unlocking and locking mechanism mounted on a dumping bridge. The dumping bridge consists of actuating switch, the horizontal rail, the overlapping return rail and the locking mechanism. The tipper runs up the actuating switch, is unlocked and tips independently due to the eccentric centre of gravity of the transport container. The dumping of the contents alters the centre of gravity, which causes the automatic return of the tipper to the normal position. The wagon than passes the locking mechanism and is locked again. The use of single-side self-discharge wagons has the advantage that the dumping mechanism is cheaper, although the wagons themselves are more expensive and also require more maintenance with their tipping mechanism.

Bunker trains. Bunker trains were developed in mining with the intention of transporting as much muck as possible. A bunker train consists of a plate or chain conveyor, which is loaded at the front by the loading device, possibly with an intermediate loading wagon, and many wagons, which are either made without fronts or have sloping conveyor belts reaching over the front of the next wagon as a wagon floor. The material is transported further by the conveyor belt over the entire length until the end of the train. After the loading process is complete, the bunker train transports the muck out of the tunnel. Bunker trains are suitable for very long tunnels.

One special form is the shuttle bunker train, in which the bunker train is loaded at the loading device. The bunker train remains at the location transfers the muck on the conveyor belt to the shuttle train, which shuttles back and forth between the bunker wagon and the tip. The shuttle bunker train is, in contrast to the bunker train, also suitable for shorter distances.

2. Track

Only a general description of the track is given here, a more detailed discussion of the subject can be found in [313], [314].

Track system. The track system consists of the track and the track bed. The track consists of rails, the ties (sleepers) and the fixing of the rails to the ties. A track bed is often omitted in tunnels, and may be replaced by an incidentally expensive filling or packing under the track bed with suitable excavated material.

One particular problem is the continuous elongation of the track at the face, since this activity has to take lace simultaneously with many others. But the typically narrow cross-sections often only permit consecutive performance of the various individual activities, leading to an extension of the round cycles and increased costs.

Gauge. The gauge is the smallest distance between the inner surfaces of the railheads measured at right angles to the track axis. Common gauges in tunnelling are between 750 mm in smaller and 900 mm in larger cross-sections; 600 mm and 1000 mm are also used in isolated cases. It should be considered that a narrower gauge does save purchasing costs but results in less stability and lower running speeds.

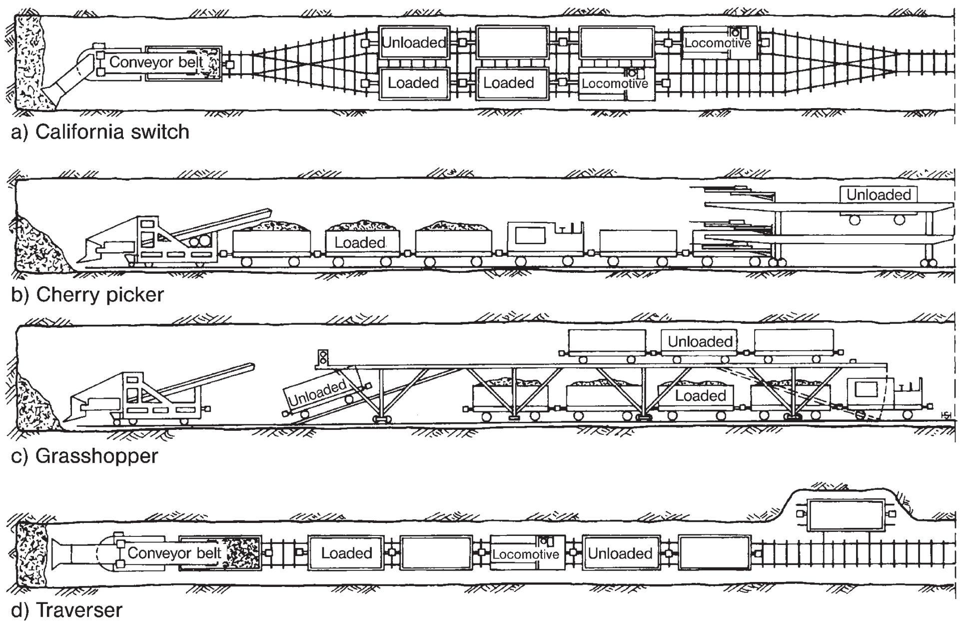
Passing tracks. Rail operation is mostly used when the cross-section is small, and the tracks are almost always laid singly to save space and cost. For systems using mucking trains, this gives the problem that the loaded trains from the face have to pass the unloaded trains coming back. If the trains can be loaded at the side or bunker trains are used, then this problem does not arise.

These follow descriptions of some usual methods of permitting the trains to pass in the tunnel [174], [313], [314].

The California switch (Fig. 5-60a) provides a two-track passing loop for an entire train. It is equipped with ramps at both sides and sits on top of the normally laid tracks. It can be moved either by sliding or on rollers.

A wagon-lifting device, also called a cherry picker (Fig. 5-60b), is a crane that lifts the empty wagons over the already full wagons. The cherry picker remains stationary and the train performs the shunting work. A similar type of wagon lifting device is the grasshopper (Fig. 5-60c), in which the empty wagons run over a wagon platform to the face.

Traversers (Fig. 5-60d), also called transfer tables, are used when extremely restricted space allows no other solution or such a solution would be uneconomic for a very short tunnel. Niches are excavated at regular spacings to permit the sideways movement of an empty wagon into the niche. The wagon is moved on a sliding table and the locomotive performs the necessary shunting. A traverser is slow to operate, demands expensive niches and is thus relatively expensive.



a) California switch

b) Cherry picker

c) Grasshopper

d) Traverser

Figure 5-60 Wagon passing methods [314]

The sliding floor is a marshalling yard on the tunnel floor, which is only suitable for larger cross-sections. It consists of three (Fig. 5-61) or more elements, which are moved singly by hydraulic jacks. The sliding floor is however very expensive and is therefore only used in special cases.

Dump. A dump is an intermediate storage at the tunnel portal. The discharge point is on a bridge over a slope, or a tipping pit is used. The muck wagons are emptied here by gravity using the discharge devices described above.

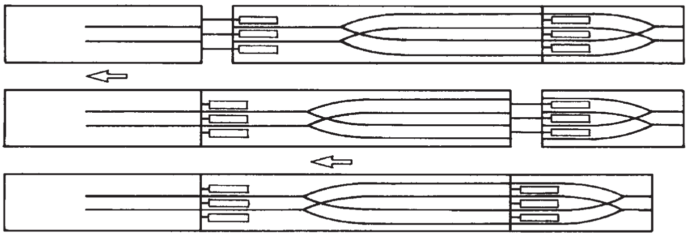


Figure 5-61 Sliding Floor, principle of operation [314]

Belt conveyance. Conveyor belts are mainly used as a handover system in tunnel mucking, which means they do not pick up the material but are used as a connecting link between loading device and transport vehicle (Fig. 5-62). They are sometimes mounted on the loading machine but are often freestanding devices. Their task is the handover of the muck to the transport vehicle with the intended reduction of the discharge height from the loader, also to provide a buffer function at the transfer between individual transport units.

When rail transport is used, the track-borne conveyor belt system is relatively common. It is used when the muck wagons cannot be pushed through a wagon passing device to the face. The loading belt is adapted to the length of the train.

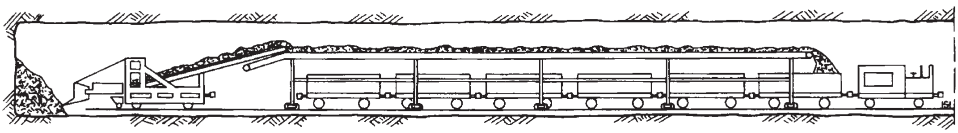


Figure 5-62 Belt loader in conventional tunnelling [313]

One further possible application of a conveyor belt is to mount it in the tunnel crown. The conveyor belt in the crown can be moved with rollers and thus fulfil the same purpose as the rail-borne belt loading device, but takes up less space, and material transfer is then decoupled from the activities in the invert. As the crown conveyor belt also functions as intermediate material storage, (bunker belt), the length is 200 to 300 m.

Determination of capacity. The capacity of loading and transport equipment in tunnelling can generally be determined in the same way as with earthworks above ground. Some works, which describe the determination of performance of construction equipment, are [143], [176], [175].

Operations research methods have formed detailed numerical models of construction machines in order to determine the output with great precision, but have failed due to the variety of influential factors on the long-term output of the machines in practice. It should be pointed out that the outputs actually achieved are very greatly influenced by the specific conditions of each project and theoretically determined values always need critical and practical evaluation.

Muck should be cleared as fast as possible for cost reasons, so optimised utilisation of machinery is the intention and the number of transport units is planned accordingly. The output for mucking therefore results in an estimation of the size and number of transport vehicles from the practically possible output of the loader less the time, during which the loader cannot work. These times include shunting or manoeuvring activities to change the transport vehicle, the bringing of the transport vehicle to the face after blasting or the extension or rails and supply cables and pipes, when this extension has to be completed before mucking. If the loading and transport devices are not ideally matched, then the loader will have unproductive waiting times and the transport capacity will determine the mucking time.

Output of loading machines. In this book, no detailed calculation of the output of loading machines will be given, but rather practically achievable ranges for long-term output in operation (Table 5-10). It should be noted that no shunting or manoeuvring activities or set-up times are included.

The outputs are given in cycles per hour, and the possible bucket or shovel filling and the bulking factor of the muck pile have to be considered for the determination of long-term output (values refer to Table 5-11 to Table 5-13).

Table 5-10 Number of cycles of usual loading machines in tunnelling [150]

Loading machine	Number of cycles in operation without forced waiting time	Remarks
Hydraulic excavator		
backhoe bucket	60 to 80	the number of cycles
shovel bucket	40 to 60	falls with larger buckets
Front loader		
wheeled loader	50 to 70	cycle number without longer travel
track loader	40 to 60	distance to the transport device

The formulae given here are based on [173]. From the possible usable load in one loading cycle

$C = F_{100} \cdot \varphi \cdot \alpha$

with

C usable load per cycle [solid m³]

F_{100} bucket volume [m³]

φ filling factor of the loading container (Table 5-12)

α bulking factor of the blasted muck (Table 5-13)

gives an output of

$Q_L = C \cdot X \cdot \eta_G$

with

Q_L long-term output of the loading machine in operation without forced waiting time [solid m³/h]

X cycles/hour [cycles/h] (Table 5-10)

η_G machine utilisation (Table 5-11) if longer waiting times occur, which reduce the time the loading machine can work, then the machine utilisation falls greatly (Table 5-11). Such waiting time can occur while the transport vehicles are changed.

Table 5-11 Utilisation η_G of the loading machines [173]

Related to	Practical maximum	Operating conditions are		
		good	medium	bad
work cycle	1.0			
operating hour	0.9	0.82	0.7	0.5
shift	0.75	0.7	0.6	0.4
week	0.7	0.65	0.5	0.35
month	0.65	0.6	0.45	0.3

Table 5-12 Filling factor φ dependent on material [173]

Material	φ
light soil	1.2
medium soil	1.0
heavy soil	0.75 to 0.9
rock, well blasted	0.55 to 0.8
rock, poorly blasted	0.25 to 0.6

Table 5-13 Bulking factors [173]

Material	α
clay, dry	0.80
clay, dense, viscous or damp	0.75
soil, dry	0.80
soil, wet	0.80
soil, with sand and gravel	0.85
soil, mixed with rock	0.77
gravel, dry	0.89
gravel, wet	0.88
loam	0.83
rock, solid, well blasted	0.67
rock and stone, broken	0.74
rock, weak or bedded	0.75

Other forms of output calculation are based on Hetzel [133] and Burkhard [49].

5.4.4 Output of transport vehicles

The estimation of the output produced by transport vehicles is performed over the duration of one cycle of loading, journey to tip, discharge, return journey and shunting or manoeuvring [217].

The practically possible output of a transport vehicle can be given as

$$Q_T = V_T \cdot \alpha \cdot \frac{60 - t_N}{t_B + t_H + t_E - t_Z + t + t_R}$$

with

Q_T long-term output of the transport vehicle [m³/h]

V_T volume of the transport container [m³]

α bulking factor (see Table 5-13)

t_N time per hour, in which the transport vehicle is not available for mucking [min]

t_B loading time [min]

t_H time for journey to tip [min]

t_E time for discharge [min]

t_Z time for return journey [min]

t_R shunting or manoeuvring time [min]

The time for loading is based on the number of cycles required by the loading machine, and the time for the journey to the tip and back is the result of the travel speed and the transport distance. For discharge and shunting or manoeuvring, the times are very different according to the properties of the transport vehicles used, although it can be generally assumed that wheeled vehicles discharge and manoeuvre more quickly and flexibly than rail-borne transport.

5.4.5 Examples of transport chains

The muck is frequently transported to an intermediate stockpile in the tunnel, from where the further transport normally takes place to a final tip outside the tunnel. In some cases, the excavated material can be suitable for reuse, for example recycled as concrete aggregate for the inner lining or the backfilling of the invert after the spraying of the bottom arch. In many European countries, the waste disposal laws demand the best possible recycling or reuse of the material.

An intermediate tip is used when the face has to be made accessible as quickly as possible or when immediate removal to outside the tunnel would require the use of too many transport units and thus be uneconomic.

Two examples of transport chains from the picking up of the muck pile to the intermediate tip near the portal are described. The subsequent treatment of the material is not specific to tunnelling and would break the bounds of this book.

The transport chain begins with the loading machine picking up the muck and ends with the discharge of the transport vehicles.

Mucking in a transport tunnel. A mucking concept is required for a rail tunnel on a high-speed line with a cross-sectional area of about 100 m² (Fig. 5-63). The tunnel is to be driven from each end and has a length of 6 km. The rock to be excavated is stable along all the length so that a high-performance drive can be carried out with round lengths of 3.2 m in the top heading. The rock is to be excavated in the steps top heading and bench left/right.

In order to clear the muck, a wheeled loader with a bucket capacity of 4.5 m³ is used in the top heading, and large dump trucks with a loading volume of 21 m³ are available for transport. It is assumed that the muck is transported directly from the face to outside the tunnel. The material is transported further from the intermediate near the tunnel portal.

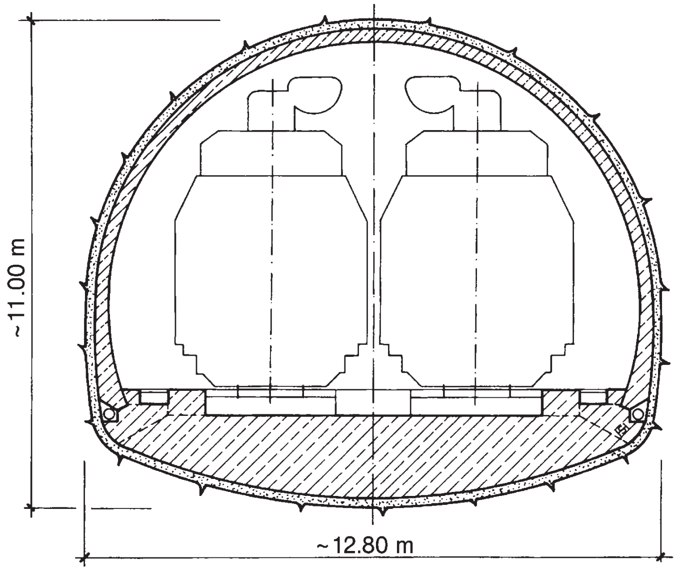


Figure 5-63 Excavation cross-section for the rail tunnel bypassing Melk, Austria, 1995 [135]

The excavated quantity to be cleared per round of top heading advance, with a top heading excavation cross-section of 55 m², a round length of 3.2 m and an estimated overbreak of about 10 % is

$$V_{\mathrm{KAL}} = A_{\mathrm{KAL}} \cdot l_{\mathrm{a}} \cdot 1.10$$

$$V_{\mathrm{KAL}} = 55 \cdot 3.2 \cdot 1.10 = 193.6 \text{ m}^3$$

with

V_{KAL} volume of muck to be cleared per round [m³]

A_{KAL} excavated cross-section of top heading [m³]

l_{a} round length [m].

The wheeled loader offers a long-term output of 40 cycles per hour, which already includes consideration of the reduced utilisation due to hindrance from the dump trucks manoeuvring. Assuming a bulking factor α of 0.67 (Table 5-13) and a filling factor φ of 0.8 (Table 5.12) for well-blasted rock, this gives a usable loading of

$C = F_{100} \cdot \varphi \cdot \alpha$

$C = 4.5 \cdot 0.8 \cdot 0.67 = 2.41\ \mathrm{m^3/cycle}$

and a long-term output of

$Q_L = C \cdot X$

$Q_L = 2.41 \cdot 40 = 96\ \mathrm{m^3/h}$

with the machine utilisation already having been considered in the possible number of cycles. A time of slightly more than two hours must be planned for mucking.

Now the number of dump trucks required is determined. The transport volume of the trucks V_T is, considering a bulking factor of $\alpha = 0.67$ and a filing of the transport container of 1.0

$V_T = V \cdot \alpha \cdot \varphi = 21 \cdot 0.67 \cdot 1.0 = 14\ \mathrm{m^3}$.

The cycle time is calculated from the average transport distance of 1.5 km. This results from half of the tunnel section length as the distance of the centre of mass:

$T_{UML} = t_B + t_H + t_E + t_Z + t_R$

with

T_{UML}	cycle time of the transport vehicle [min]
t_B	loading time [min]
t_H	time for the journey to the tip [min]
t_E	time for discharging [min]
t_Z	time for the return journey [min]
t_R	manoeuvring time [min]

The individual times can be calculated as:

time for loading

$$t_B = \frac{V_T}{Q_L} \cdot 60$$

with

t_B	time that the wheeled loader requires to load the dump truck [min]
V_T	volume of the dump truck [m³]
Q_L	long-term output of the wheeled loader [m³/h]

$$t_B = \frac{14}{96} \cdot 60 = 9\ \mathrm{min}$$

time for the journey to the tip (full) at 12 km/h

$$t_H = \frac{1}{v_{voll}} \cdot 60$$

with

t_H time for the journey from the face to the tip [min]

l transport distance [km]

v_{voll} average speed of the dump truck in loaded condition [km/h]

$$t_H = \frac{1{,}5}{12} \cdot 60 = 7{,}5 \text{ min}$$

time for discharge $t_E = 2$ min (estimated)

time for the return journey at 15 km/h

$$t_Z = \frac{1}{v_{leer}} \cdot 60$$

with

t_Z time for the journey from the tip to the face [min]

l transport distance [km]

v_{leer} average speed of the dump truck empty [km/h]

$$t_Z = \frac{1{,}5}{15} \cdot 60 = 6 \text{ min}$$

time for passing vehicles and manoeuvring at the face

$t_R = 2$ min (estimated)

The cycle time T_{UML} of a large dump truck is thus 26.5 minutes. Therefore are

$$n = \frac{T_{UML}}{t_B}$$

$$n = \frac{26{,}5}{9} = 2{,}94$$

with

n number of dump trucks required

T_{UML} time for one cycle for a dump truck [min]

t_B time for loading a dump truck [min] three dump trucks are required. For the advance past 1.5 km, the journey time is longer so that another dump truck will be required if the wheeled loader is to be fully utilised. For station 3.0 km before the tunnel breakthrough, which is planned for the middle of the tunnel, this gives a cycle time of 40 min. This requires a total of five dump trucks for a short time.

In principle, the mucking machinery for the bench advance is also planned in this way. However, the advance of the bench has less priority than the top heading while both are active simultaneously, which means that the work on the bench cannot be allowed to hold up the work in the top heading.

The following machines will be required for the muck clearance from the tunnel:

- 1 wheeled loader — mucking from top heading
- 1 wheeled loader — reserve, will be used for mucking from the bench, moving ramps
- 3 dump trucks — mucking from top heading
- 1 to 2 dump trucks — reserve, mucking from bench and later mucking from top heading.

Example: muck clearance from an investigation tunnel. In an investigation tunnel for an autobahn tunnel with a cross-sectional area of 13.2 m² and a length of 6000 m, the muck is to be transported by rail. The full face of the tunnel will be excavated and the gradient is 1.5 % uphill.

The forecast rock mass is classified as loose to brittle and the expected round lengths are about 2.0 m.

The loading machine will be an overhead loader with a shovel volume of 0.6 m³. The transport system will be a shuttle train from Hägglund, type C. The model corresponds to the principle of the bunker trains described in section 5.4.3. The overhead loader loads the first wagon, which then transfers the material to the following wagons on an integrated chain scraper conveyor.

The muck volume assuming an overbreak of 5 % is

$$V_{\mathrm{GALLERY}} = 13.2 \cdot 2.0 \cdot 1.05 = 27.7 \text{ m}^3.$$

The output of the overhead loader according to the manufacturer is 1.5 m³/min. Experience from already completed sites showed, however, that a long-term output of about 80 cycles/h is practically possible. This gives assuming a bulking factor a of 0.75 (Table 5-13) and a degree of filling φ of the shovel of 85 % (Table 5-12) a usable load per cycle of

$$C = F_{100} \cdot \varphi \cdot \alpha$$

$$C = 0.6 \cdot 0.85 \cdot 0.75 = 0.38 \text{ m}^3$$

and a long-term output of

$$Q_{\mathrm{L}} = C \cdot X$$

$$Q_{\mathrm{L}} = 80 \cdot 0.38 = 30.4 \text{ m}^3/\text{h}.$$

The bunker train must have a capacity of 30.4 m³, because it must transport all the muck in one trip. Using wagons with a capacity of 11.5 m³ gives a number of

$$n = \frac{30{,}4}{11{,}5 \cdot 0{,}67} = 3{,}9.$$

so four wagons will be required. The duration of mucking is just over one hour. The locomotive will be a battery loco with a service weight of 16 t and a power of 50 kW.

5.4.6 Further developments

After the rapid development of drilling machinery in tunnelling, the further development of mucking operations will be an important factor for economic construction progress. The adaptation of capacities and technical properties of the machinery to the conditions on site has to be performed anew for each site.

The general tendency to larger, faster and more powerful machines is unbroken [180]. The exhaust behaviour of diesel motors is being improved continuously so that machines with higher power can be used and the cost of ventilation still remains within bounds. When a tunnel contract is very long, a combination of electric loaders and diesel-powered transport vehicles is often used, in order to technically and economically optimise the ventilation requirements.

5.5 Combination of drill and blast with mechanised tunnelling processes

The advantages of pilot tunnels, which offer very good geological probing, drainage of the rock mass, ventilation and gentle excavation, has led to combinations of drill and blast with mechanised tunnelling for larger cross-sections. The following section described some possibilities.

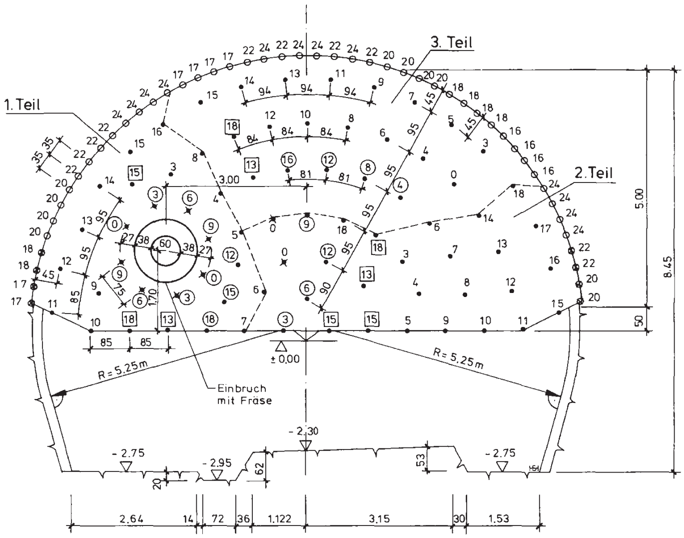
5.5.1 Combinations with roadheaders

The are three possibilities of combinations of drill and blast with the operation of a roadheader:

– Using the roadheader to mine a cut and create free surfaces.
– Mining a pilot tunnel.
– Profiling work.

Mining a cut or a pilot tunnel. In order to create free surfaces, tunnel blasters first blast a cut, which then enables the subsequent blasts to have more effect. The blasting of this cut is normally responsible for a high proportion of the vibration caused by blasting. This proportion can be considerably reduced by using a roadheader to make the cut first.

One example of this method of tunnelling is the Altstadt Tunnel in Arnsberg. With an overburden of about 20 m, the road tunnel with an excavated cross-section of about 80 m² was driven through the solid to hard limestone under the old town of Arnsberg with its partially listed buildings. Vibration had to be kept to a minimum. The blasting pattern with the location of the preliminary cut made by the roadheader is shown in Fig. 5-64.



<u>Top heading round, length=2,00 m</u>
<u>Symbols:</u>

1. Detonator O U/20 ms/3 m wire length
   □ U/30 ms/3 m wire length
   other U/125 ms/3 m wire length
2. Charge quantity ⌑ 1½ cartridges AG2∅25 = 0.45 kg
   • 2 cartridges AG2∅25 = 0.60 kg

⊗ ½ cartridge AG2∅25+1.60 m Supercord 100 = 0.32 kg
o 1,60 m Supercord 100 = 0.16 kg

<u>1st partial excavation with milled cut:</u>
44 drillholesAG 2 ∅25 = 16.20 kg
25.60 m Supercord 100 = 2.56 kg
<u>2nd partial excavation:</u>
35 drillholes AG 2 ∅25 = 16.50 kg
12,80 m Supercord 100 = 1.28 kg
<u>3rd partial excavation:</u>
44 drillholes AG 2 ∅25 = 12.00 kg
38,40 m Supercord 100 = 3.84 kg

Total explosive quantity = 52.38 kg
Total holes = 123 No.

Figure 5-64 Blasting pattern with cut made by roadheader in the Altstadt Tunnel in Arnsberg, excavated section about 80 m^2, construction time 1992 to 1995

Profiling works. If a roadheader is on site for cutting work, then it can also be used for profiling if the geological conditions are suitable. In this way the overbreak from blasting, which can sometimes be considerable, can be limited.

5.5.2 Combination with full-face machines

The advantages of a pilot tunnel drill and blast tunnels mentioned above have led to a combination with full-face machines in larger tunnels. Examples of this are the Pfänder Tunnel (1st bore) and the tunnel for the Blaubeuren bypass (Fig. 5-65)

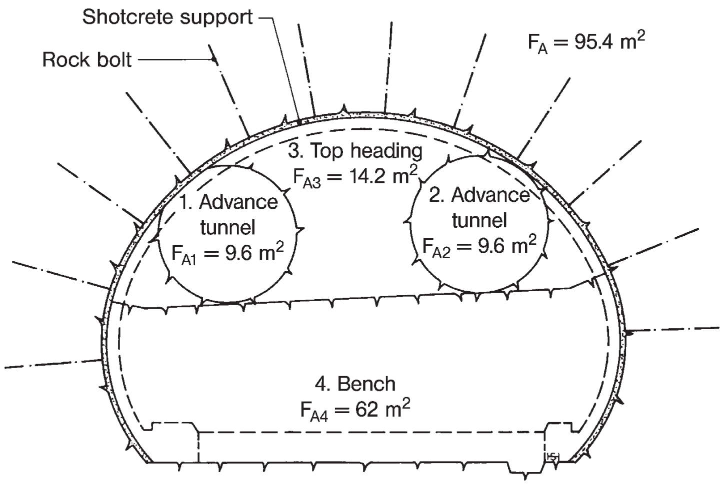


Figure 5-65 Drill and blast tunnel follows two mechanically bored advance tunnels. Tunnel for the Blaubeuren bypass, 1980 [98]

In the course of the continued development of tunnelling, a clear trend towards mechanisation of tunnelling is recognisable. The preconditions for this are that a tunnel that is economically long enough to largely write off the machine and is bored in stable rock under known and if possible homogeneous geological conditions. If on the other hand an irregularly shaped tunnel is to be driven in variable and constantly changing geological and hydrogeological conditions, then drill and blast will be preferred due to its flexibility and adaptability. The reality often shows a grey zone in the decision between these two alternatives. No clear delineation of the scope of application of these two processes is therefore possible. This leads to the situation that the most economical construction process, either drill and blast or mechanised tunnelling or a combination of both, has to be decided for each project. In many cases, both are tendered as alternatives and the final decision is only made after the bids have come in from the contractors. The parties involved in a tunnelling project have to be experienced and prepared to take risks during the design, tendering and construction phases.

5.6 Effects of blasting on the surroundings

5.6.1 Vibration

The detonation of an explosive creates a detonation front, which propagates through the ground like a wave. The most significant types of waves caused by blasting are:

- P waves or compression waves; the particles vibrate in the direction of wave propagation.
- S waves, also described as shear waves; the particles vibrate at right angles to the direction of wave propagation. S waves have lower velocities than compression waves.
- Surface waves, mostly with low velocities.

Geological and geotechnical layers damp or reduce the waves, but the damping effect of the ground is often overestimated. Reflections at discontinuities in the rock have a very minor significance at low frequencies. If the frequency of the ground wave is near that of a stratum, resonance occurs. Resonant magnifications occur much more often in such cases than is generally assumed. The theoretical description of the propagation of vibration is indeed possible, but requires precise knowledge of the spatial geometrical relationships and the geotechnical conditions and the geotechnical properties of the ground. Since however precisely these factors in addition to the oscillation intensity of the oscillation source are insufficiently known, test blasts are very appropriate and can be recommended in every case. In contrast to the very slow waves of earthquakes, in which the potential damage is proportional to the acceleration, the extent of damage from faster vibration speeds, as are caused by blasting, are proportional to the vibration velocity.

In order to determine the relationship in blasting vibration between the particle velocity at a certain location with distance D from the blast and the detonated charge per delay number (theoretically only correct for instantaneous detonation, since the interval imprecision also leads to differences), the following formula has proved useful:

$$v = k \frac{L^P}{D^q}$$

v	particle velocity [mm/s]
L	charge per detonation stage [kg]
D	distance from the blast [m]
p	factor 1/3 < p < 1/2
q	factor 2 < q < 1
k	coefficient with consideration of the soil characteristics, charge, confinement, explosive

Test blasts can be used to determine the factors *k*, *p*, *q*. The factor *p* is mostly 1/3 for a point blast, since the waves propagate in a sphere. The factor *q* was determined in tests by Langefors at 3/2, von Haller at 2. The numerous formulae actually differ from each other very slightly in the decisive frequency range of 20 to 150 Hz. In general, a large charge or a short distance from the blast lead to high vibration velocities.

The evaluation criterion in the standards of most countries [17] is the velocity vector v_R:

$$v_R = \sqrt{(v_x^2 + v_y^2 + v_z^2)}$$

In most cases, there is no objection to this. For blasting carried out as part of tunnel refurbishment inside an existing structure, a velocity vector v_R is often unrealistic regarding the effects on carriageway slabs or partition walls. The compression wave in the plane of the slab is normally of little interest, quite in contrast to the wave velocity at right angles to the concrete slab.

Fig. 5-66 shows a typical curve of blasting vibrations as a velocity vector and the individual components with a normal detonation sequence. In comparison, Fig. 5-67 shows the vibrations at the same location but with different detonation intervals with the delay numbers inside the tunnel series (20 ms 0 3 6 9 12 15 18, 40 ms 11 13 15 17, 80 ms 10 to 15, 1/2 s 3 to 5) being omitted. In addition to a slightly lower overall vibration vector, such a blast tends to much less magnification in the appropriate ground.

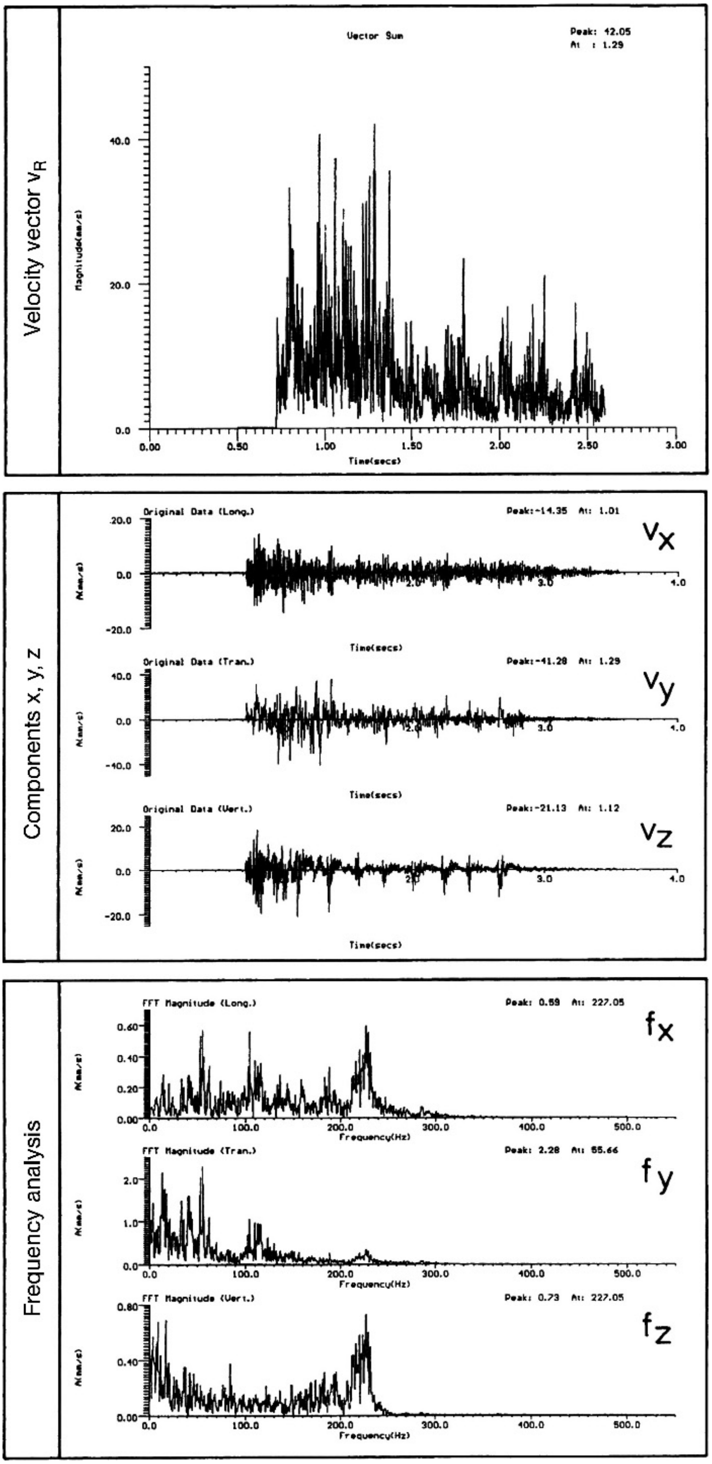


Figure 5-66 Normal detonation sequence of the tunnel series [335]

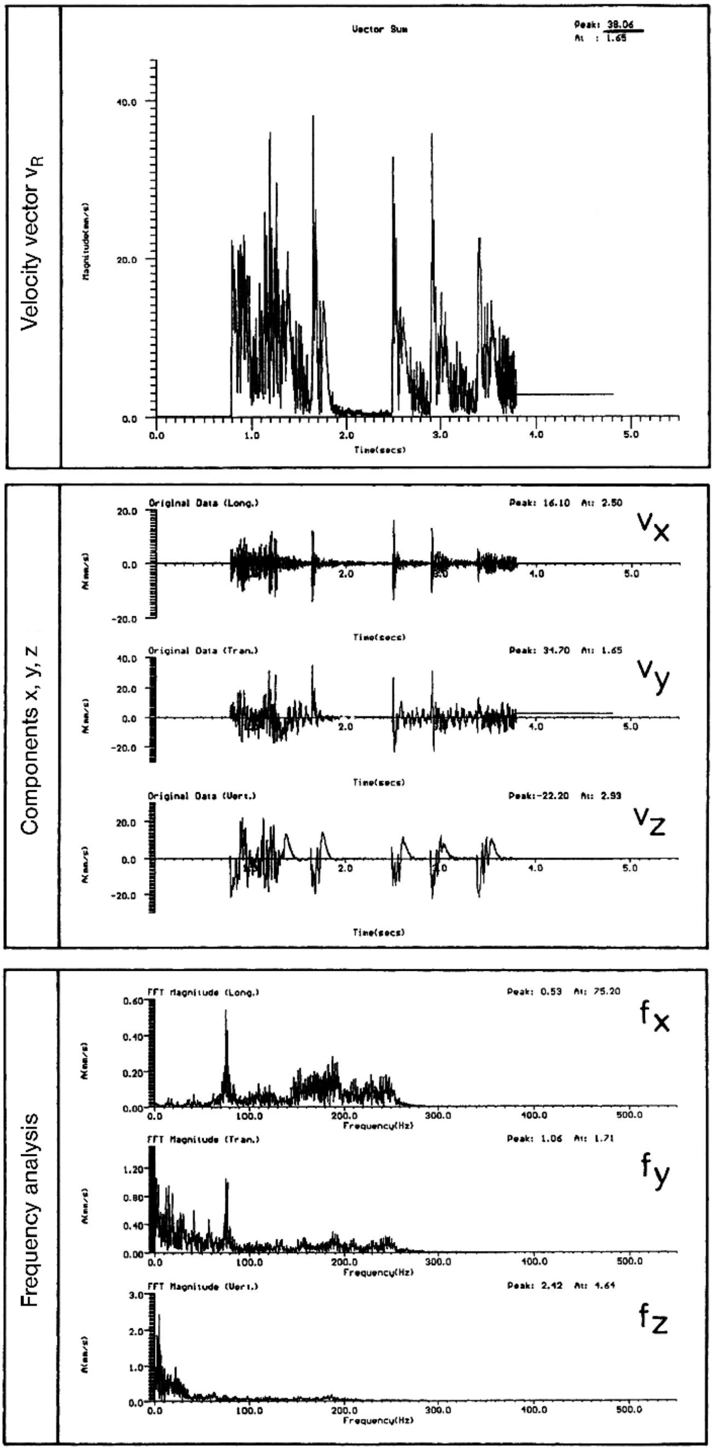


Figure 5-67 Blasting vibration with extended detonation delays [335]

Evaluation of vibration in Switzerland. The national standards regarding vibration effects on structures give guideline values for permissible vibration depending on the frequency of occurrence, the decisive frequencies and the vibration susceptibility of the relevant structure. A simple evaluation is permitted by the Swiss standard SN 640 312a, issue 4/92. Blasting vibrations scatter in relatively wide ranges due to the many influential factors, and just for this reason, a simplified evaluation procedure is very advantageous.

In general, the engineer has to evaluate the possible damaging effects of vibration according to the criteria damage to buildings, nuisance for the residents and damage to sensitive devices and facilities.

Vibration can be experienced by people as unpleasant, even very disturbing, long before any damage is caused to building. Targeted publicity to inform the local population therefore mostly leads to appreciable understanding.

Buildings and susceptible devices and facilities are normally significant for blasting vibration. The vibration that is acceptable for a building can be limited through the classification of the frequency with Table 5-14, (with blasting being classed as occasional), the susceptibility classes (Table 5-15) and the guideline values (Table 5-16).

Table 5-14 Frequency classes [336]

Frequency classes	Typical vibration sources
Occasional Events considerably less than 1,000	blasting compaction devices and vibrating rammers, if they only cause greater vibration on starting and stopping
Frequent	frequent blasting impact and vibration rammers compaction machines demolition hammers with occasional use emergency power groups, which are operated frequently
Permanent Events considerably more than 100,000	traffic permanently installed machines demolition hammers with frequent use

Table 5-15 Susceptibility classes for vibration [336]

Susceptibility classes	Buildings	Facilities and utilities
(1) Very slightly susceptible		Reinforced concrete or steel bridges. Retaining structures of reinforced concrete, concrete or massive masonry. Tunnels, caverns, shafts in solid rock or well-consolidated soil. Crane and machine foundations. Pipelines above ground.

Table 5-15 continued

Susceptibility classes	Buildings	Facilities and utilities
(2) Slightly suscep- tible	Industrial and commercial buildings in reinforced concrete or steel construction, normally without render. Silos, towers, high masonry chimneys without render or steel construction. Trussed masts. Precondition: the buildings are built according to the general rules of building and correctly maintained.	Caverns, tunnels, shafts, pipes in soil. Underground car parks. Utilities (gas, water, drainage, cables) laid below ground. Drystone walls.
(3) Normally susceptible	Residential buildings with masonry in concrete, reinforced concrete or artificial stone. Office buildings, schools, hospitals, churches with masonry or artificial stone with render. Precondition: the buildings are built according to the general rules of building and correctly maintained.	Water towers. Reservoirs. Cast iron pipes. Caverns, intermediate slabs and carriageways in tunnels. Susceptible cables.
(4) Increased susceptibility	Houses with plasterboard ceilings or hollow pot slabs. Timber-framed buildings. Newly built or renovated buildings of class (3). Historic and listed buildings.	Old lead cables. Old cast iron pipes.

Table 5-16 Guideline for permissible vibration depending on the frequency [336]

Susceptibility classes	Frequency classes	Maximum value of the velocity vector v_R (mm/s) decisive frequencies		
		< 30 Hz*	30 to 60 Hz	> 60 Hz**
(1) Very slightly susceptible	occasional frequent permanent	Guideline: up to three times the corresponding value for buildings in susceptibility class (3)		
(2) Slightly suscep- tible	occasional frequent permanent	Guideline: up to two times the corresponding value for buildings in susceptibility class (3)		
(3) Normally susceptible	occasional frequent permanent	15 6 3	20 8 4	30 12 6
(4) Increased susceptibility	occasional frequent permanent	Guideline: between the values for class (3) and half the values for class (3)		

*) Lower guideline values should be taken for frequencies below 8 Hz

**) Higher guideline values can be taken for frequencies above 150 Hz

Evaluation of vibration in Germany. In Germany, building vibration is evaluated according to DIN 4150, T.3 [73], issue 5/86. Compared to the Swiss standard, slightly higher values are permissible at medium and high frequencies, but are related to the individual components and not to the total vector (Table 5-17).

Table 5-17 Guideline values for the vibration velocity v_i for the evaluation of the effect of short-term vibration according to DIN 4150, T.3 [73]

		v_i (mm/s)			
Line	**Type of building**	**Foundation**			**Slab level of the uppermost full storey**
		Frequencies (Hz)			**All frequencies**
		<10	**10 to 50**	**50 to 100***	
1	Commercially used buildings, industrial buildings and similarly structured buildings	20	20 to 40	40 to 50	40
2	Residential buildings or similar in their construction and/or use	5	5 to 15	15 to 20	15
3	Buildings, which due to their particular susceptibility do not correspond to those in lines 1 and 2 and are particularly worthy of preservation (for example listed buildings.	3	3 to 8	8 to 10	8

*) At frequencies greater than 100 Hz, at least the values for 100 Hz may be taken.

Evaluation of vibration in Austria. In the Austrian standard ÖNORM S 9020 “Building vibrations; blasting vibrations and comparable emissions of impulse shape” [267], issue 8/86, the building types are in contrast to Germany divided into four classes (Table 5-18). In Table 5-19, guideline values ore given for the permissible resultant oscillation velocity $v_{R,max}$ for the various building classes. This is based on uniform ground conditions with a propagation velocity of c_L=500m/s. The guideline values are valid for blasting vibration from regular extraction blasting or from comparable blasting in the course of construction projects.

Table 5-18 Building classes according to ÖNORM S 9020

Building class	Building type
I	Industrial and commercial buildings Storey frames (with and without core) with load-bearing construction of steel or reinforced concrete Buildings with the walls acting as deep beams (in-situ concrete, precast) Engineered timber construction (halls etc.)
II	Residential buildings Storey frames (as with I) Buildings with the walls acting as deep beams (as with I) Buildings with in-situ concrete slabs, rising masonry of concrete blocks, bricks or other artificial building blocks with cement or lime mortar Timber buildings excepting timber frames with masonry filling
III	Building with less frame stiffness than with I and II Building with slab over basement of concrete or brick vault, in the upper storeys precast, timber joist, or brick precast element floors Timber framed buildings with masonry filling
IV	Listed buildings, which are particularly susceptible to vibration due to their construction or other condition

Table 5-19 Guideline for permissible oscillation velocity $v_{R,max}$ at the building foundation according to the ÖNORM S 9020 [267]

Building class	Guideline $v_{R,max}$ (mm/s)
I	30
II	20
III	10
IV	5

If the guideline value according to Table 5-19 is exceeded, then not only the vibration measurements at the building foundation but the dynamic properties of the subsoil are to be recorded. This is done by measuring the propagation velocity of the seismic waves at the level of the top of foundation of spread foundations. The propagation velocity of longitudinal waves is decisive. In Fig. 5-68, guideline curves are illustrated for the various building classes, which give the permissible resultant oscillation velocity $v_{R,max}$ for the various building classes as a function of the propagation velocity c_L of the longitudinal waves. If the relevant guideline curve is not exceeded by the measured vibration values, then the blasting vibration is evaluated as permissible.

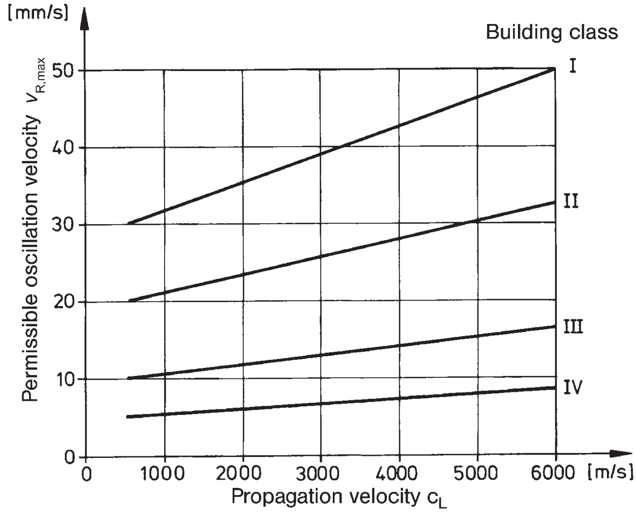


Figure 5-68 Vibration guideline curves

It is also possible to evaluate the effects of vibration through the measurement of dynamic actions occurring in the buildings or determined through calculations and compared with the permissible actions, taking into account the frequency of occurrence. This procedure is not however suitable for the evaluation of slight damage [73].

Test blasts and an appropriate measurement programme, which are undertaken on a case-by-case basis during the entire duration of tunnelling, normally enable the necessary optimisation of the blasting work while still observing the property interests of the individual citizen. Nonetheless, it is often necessary to take measures to reduce vibration.

Reduction of blasting vibration. Blasting vibration can be reduced at source. The limitation of the charge per delay number is suitable and simple as long as the total possible detonation of the manufacturer suffices. The charge per delay number can indeed be further reduced by splitting the cross-section into partial excavations, but this fairly quickly becomes uneconomic. In addition to the normal ways of adapting the blasting plan, the delay numbers, selection of explosive, division of the cross-section or also the provision of appropriate uncharged holes, another effective alternative is a mechanically bored pilot tunnel. This drastically reduces the degree of confinement for subsequent excavation, a factor with a decisive effect on vibration.

5.6.2 Composition and effects of the blasting gas emissions

General remarks. The fumes produced by blasting with their high concentrations of gases and dust are toxic and dangerous for the miners at the face and further back along the tunnel. The gases produced by blasting, particularly the "nitrous gases" or NOx, can also under some circumstances pollute the groundwater through flushing water. The danger to the groundwater from the flushing of piled muck also has to be considered at a time of increased environmental awareness.

Composition of the primary fumes. Primary fumes means the gas and dust cloud, which mixes with the air in the tunnel after a blast, but without artificial ventilation. The maximum extent is normally reached after 3 to 5 minutes [111].

Many factors have an effect on the composition of the primary fumes, and the most significant are:

- Charge density (kg explosive per m^2 of tunnel cross-section).
- Rock mass.
- Water in the rock mass.
- Type of explosive (and packaging).
- Detonation.
- Type of stemming (water stemming).

Measurements made of primary fumes seldom show agreement with the gas concentrations measured in tests in blasting chambers due to the many variable influential factors. The formation of nitrous gases seems to be subject to particularly large variations.

An extensive investigation of the composition of primary fumes in six tunnels with very different cross-sections and very variable geological conditions carried out in 1964 to 1967, showed the concentrations in the primary fumes shown in Table 5-20 [111]:

Table 5-20 Composition of primary fumes [111]

Rock type	Explo-sive	Charge density (kg/m^2)	Extent of fumes (m)	C0 (ppm)	$C0_2$ (ppm)	Nitrous gases (ppm)
Paragneiss	type A	4.9	120 to 140	400	7,000	200
Molasse sandstone	type A	3	100	700 to 1,000	4,000	350
Molasse sandstone	type A	1.1	120 to 140	80	2,000	40
Dolomite	type B	5.25	105 to 120	1 500	6,000	100
Dolomite	type B	6.3	105 to 120	2 000	8,000	100

Type A = Gelatinous ammonium nitrate explosive with nitroglycerine content.

Type B = Powder ammonium nitrate blasting agent with no content of type A.

Back-calculation from a large number of measurements made in the area behind the face, which were made since by SUVA as the fumes proceeded through the tunnel, confirm the values given in Table 5-20.

The fine dust concentration in the blasting fumes should also not be underestimated. Fig. 5-69 shows the quantity of fine dust to be expected in the primary fumes based on a large number of measurements.

These concentrations of gases and dust in the primary fumes provide the basis for the national regulations for the required thinning with the simplified assumption of an average flow velocity in the largest profile (CH 0.3 m/s, D and A 0.2 m/s) as the basic requirement for ventilation in drill and blast tunnelling with electric traction and loading machinery without combustion motors.

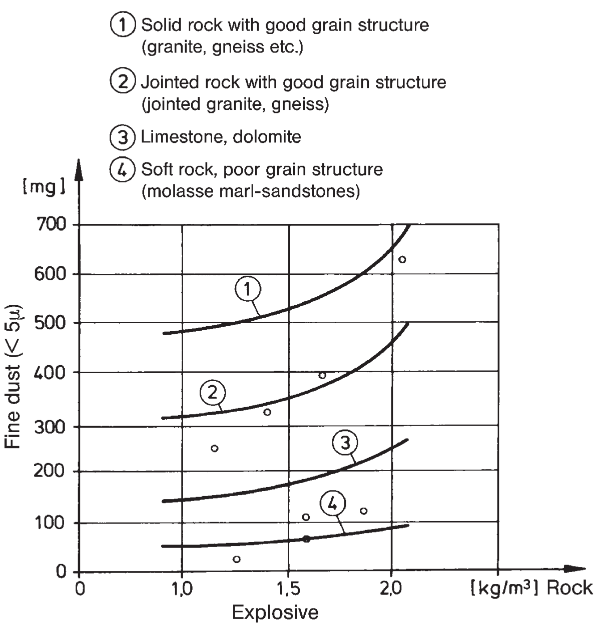


Figure 5-69 Fine dust development from various rocks

Impact on the environment

Air. Blasting gases and dust are produced suddenly. In contrast to diesel exhaust fumes, which are produced continuously and can also be thinned continuously, blasting fumes cannot be thinned to a completely uniform concentration before being discharged into free air at the portal. If suction ventilation is used, which was formerly common but today unusual, then a considerable part of the fumes remains unthinned. The arrangement of the air duct as a chimney with a high air speed is therefore demanded.

The air supply ventilation that is usual today leads to the propagation of the blasting fumes, with the concentration of gases and dust falling with increasing distance from the blasting location. The impact on the environment can be neglected unless the tunnel site is in an urban area, above all due to the very short daily exposure time. In the tunnel itself, there is no problem as long as the fresh air opening of the blowing duct is sufficiently far from the portal.

Water. The contamination of groundwater by nitrous compounds, mostly as nitrates or nitrites, may not be entirely due to blasting. Possible sources are:

- blasting gases, which adhere to the muck pile or fill the pores of the muck and are washed out by water sprayed to reduce dust.
- Diesel exhaust fumes, clinging as particles to damp surfaces or originating from diesel washers, whose cleaning fluids are not disposed of properly.
- Concrete aggregates, in rare cases also through the use of nitrites as corrosion protection products. Nitrate from anti-freeze would also be possible underground.

Muck pile or deposits from tunnel excavation. Part of the blasting fumes with their high gas concentration will remain in the pores of the muck pile. Nitrous gases converted into nitrates and nitrites will remain adhering to muck that is damp or has been wetted to reduce dust. Rain falling on the material after deposition can wash the nitrites and nitrates adhering to the particles into the groundwater or as run-off. Additional nitrite or nitrate pollution comes from diesel exhaust gases when the exhaust is fed through the piled muck, as is usual in the transport vehicles used underground.

Washing tests, which Jodl [151] performed for the Zammer Tunnel of the ÖBB, show the same results as parallel tests in Switzerland, that the nitrite concentration in waterways can be at the limit under current discharge conditions. These are individual values from very intensive washing and the thresholds were also reached in small tests on some litres. Such small tests are not however suitable to determine whether a muck pile has the condition of an inert material.

The measured nitrate concentration was a fraction of the current threshold for discharge conditions.

Overall consideration. The considerations of air, water, and muck pile show displacements of the "nitrous gas" pollution from one to the other. Good dust binding before starting to clear the muck pile, by intensively spraying the muck pile, reduces the pollution of the tunnel air by nitrous gases, but increases the adhesion of nitrites and nitrates to the muck and if the water use is very intensive can also partly wash pollutants out of the muck into the groundwater. Measures should be undertaken during construction works at critical landfill sites, which are in the groundwater. Transport vehicles should then not feed their exhaust gases through the loaded muck.

5.7 Mechanisation and Automation

5.7.1 General

Mechanisation and automation have the aims of simplifying the tasks of people or reducing exposure to harmful actions during their work, improving safety and performance [221]. The decision to introduce mechanisation is much easier when the contractor can also see an economic benefit. One of the stated aims must however always be fulfilled, or else mechanisation is simply a game.

To establish the scope of practical application of mechanisation and automation, the work activities drilling, blasting, muck clearing and support installation must be considered as an overall cycle.

Fig. 5-70 shows diagrams of the proportion of these activities for two different excavation cycles.

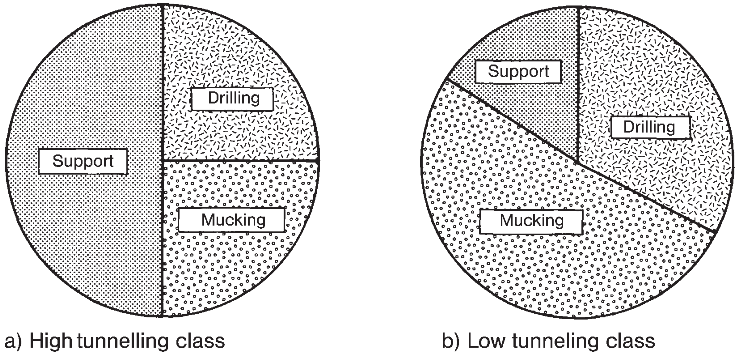


Figure 5-70 Excavation phase, proportional duration of activities for two different excavation cycles

The duration diagram of the tunnel drive in Fig. 5-70 a) shows an excavation sequence in a rock mass in a high tunnelling class. The work involved in installing temporary support is considerable compared to drilling and mucking. In contrast, the diagram in Fig. 5-70 b) shows a tunnel drive in a low tunnelling class with little support work, and which also permits longer round lengths.

Without a holistic consideration of all activities, which as explained interact with each other, the mechanisation of an individual work activity cannot lead to the intended success.

This means that it could be sensible not to increase the round length to an absolute maximum because this would increase the amount of support work, a higher tunnelling class would become applicable or the mucking capacity would be inadequate.

5.7.2 Emphasis of mechanisation

Mechanisation and automation in the 1980s developed through mechanised excavation, particularly the use of full-face machines (TBMs), shield machines and all varieties of mechanised shields.

Drill and blast tunnelling, which became much less common in some countries, like for example Switzerland, did not experience significant development in this period.

From the later eighties, longer Alpine tunnels were planned and sometimes also constructed (Brenner and Semmering in Austria, Neat in Switzerland) with some sections through very hard rock formations and great overburden with high rock pressure, leading to drill and blast tunnelling once more finding increased interest among tunnel engineers.

Drill and blast. Drilling technology has moved forwards in recent years. Hydraulically driven drills have become common on tunnel sites and have practically replaced pneumatic drills. Drilling jumbos are being fitted with ever more electronics; computer-assisted and controlled drilling machinery is often found in tunnels.

But blasting technology has also made progress, above all regarding detonation systems. Shock tube detonation has been refined and the range of electrical systems has been extended (Tunnel series).

Electronic detonation is new on the market. The detonators, equipped with capacitors and micro-chips, are all built the same. Their delay times can be regulated according to the ideas of the blasting consultant or the blaster. This controllability of detonation sequence offers only advantages for larger detonation series, and also greatly simplifies the blasting of tidy profiles.

Due to the small production quantities at present and the few blasting computers with their high development cost, electronic detonation systems are not yet competitive, but regular use on the tunnel site does seem inevitable in the near future.

Support. There have been attempts to mechanise and automate support work since the 1950s. These include installation aids for steel arches, rock bolt aids and particularly the improvement of shotcrete spraying.

The quality of shotcrete or steel fibre shotcrete used as tunnel support today still depends on the skill and capability of the construction workers. Poor working conditions and health risks can lead to a lack of motivation and thus defective performance of shotcrete works.

Correctly targeted application of mechanisation in the shotcrete application process can considerably improve not only workplace health and safety but also the quality and cost-effectiveness of the shotcrete.

Logistics. Long tunnel sections pose particular logistical problems. Drilling, blasting, mucking, support and material transport have to be optimally planned not to obstruct each other. Optimisation of blasting technology, for example of the round length, cannot be an end in itself but has to be considered as part of the overall schedule.

Drilling. The introduction of the hydraulically powered hammer drills or drifters that are usual today has led to an improvement of drilling performance by about 50 % compared to compressed air drills.

Electronics control. Hydraulic rock drills are increasingly being equipped with electronic components, which can optimally control the pressing force, rotation and hammer mechanism.

This means, for example, that the impact frequency may be reduced in favour of the pressing force or the torque according to the properties of the rock. This is most noticeable when a new hole is started automatically (Fig. 5-71).

First, rotation and flushing are switched on and the drill is advanced. Contact with the rock face leads to increasing contact pressure and the drill starts with a low pressure in the hammer mechanism for two seconds. During the next two seconds, the pressure in the hammer mechanism is increased to the maximum set value. This results in longer lifetimes for bits and drill rods and thus improved cost-effectiveness.

The control system does of course also regulate the quantity and pressure of the flushing water and can react to jamming of the drill bit.

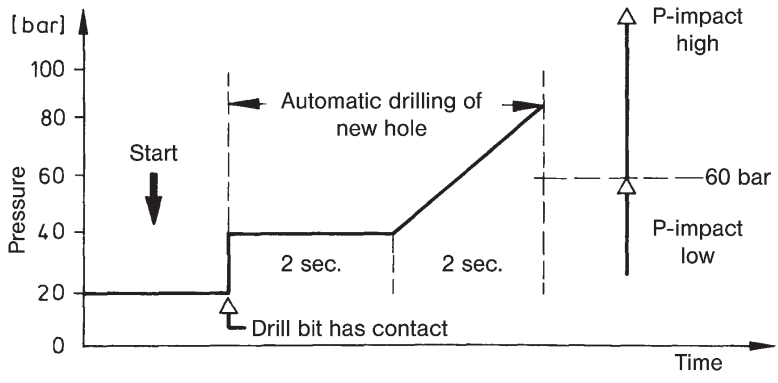


Figure 5-71 Automatic starting of a new hole [372]

5.7.3 Computer-assisted drill jumbos

Precise drilling is a precondition for the economical excavation of the rock mass by blasting [221].

The designed drilling pattern should be correctly repeated on the face. The desire for a control system capable of not only guiding the drill but also coordinating the drilling boom of the jumbo has now been fulfilled.

A computer installed on the drill jumbo enables the driller to align the setting of the drill feed with the intended drilling pattern using the screen display. A drilling pattern can be designed with CAD software in the office, stored on a transportable storage media and ready by the computer of the jumbo (Figs. 5-72 and 5-73).

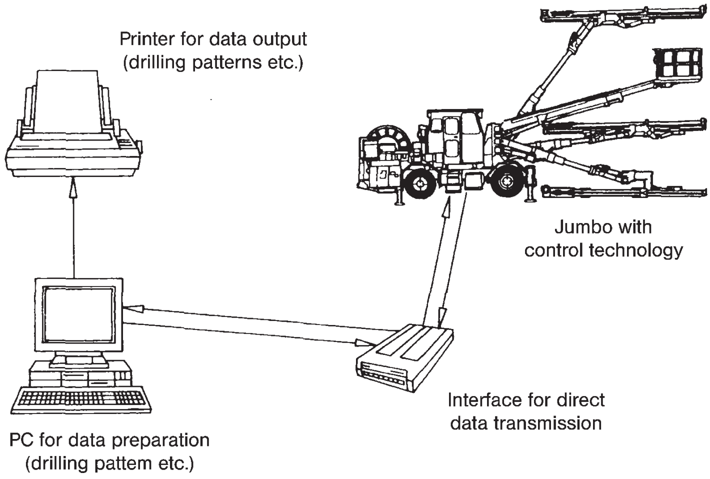


Figure 5-72 Diagram of the route from drilling pattern in the office to the drill jumbo on site, from [16]

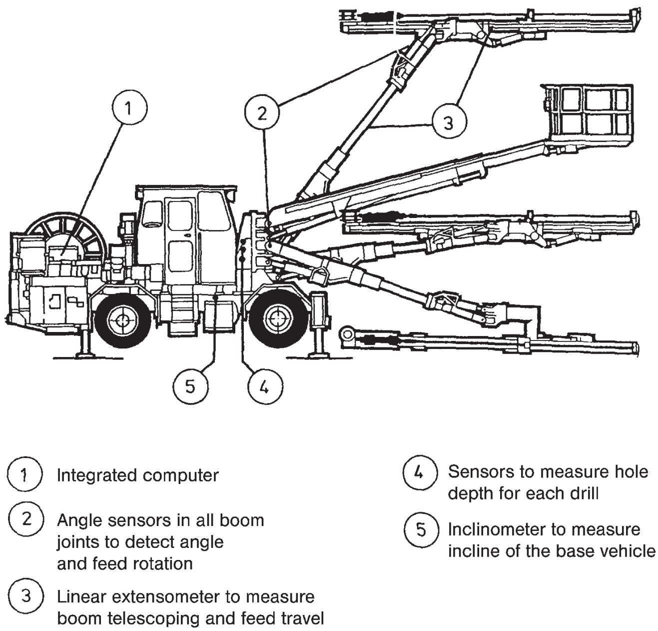


Figure 5-73 Components of the automatic control system of a high-performance mobile drill rig, from [16]

In order to set out the holes, the drill jumbo is positioned in front of the face. A laser beam sighting a target on the boom of the jumbo is sufficient to define its position in the tunnel (Fig. 5-74).

Such computer-assisted drill jumbos have great advantages. The time-consuming manual marking of the holes is no longer necessary, and the drilling pattern no longer has to be simplified in order to be able to transfer it onto the face, mostly by hand. Drilling patterns can be designed entirely to optimise blasting and put into practice.

If any corrections to the drilling pattern are found necessary during the evaluation of the success of blasting, these can be made more simply and quickly since the data of all drilling can be called up and altered.

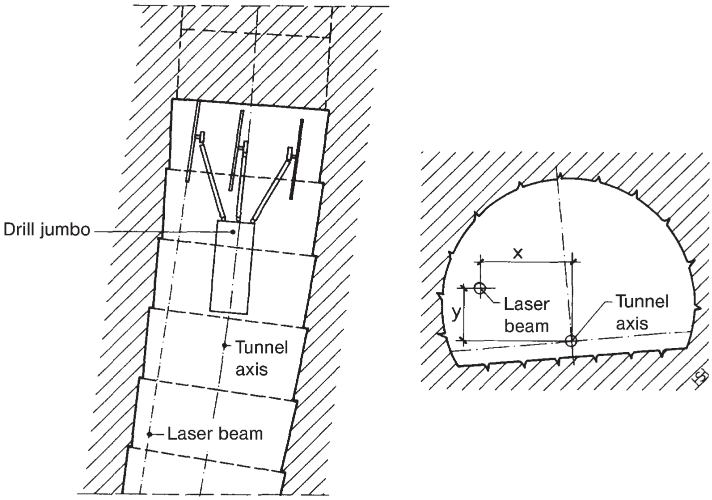


Figure 5-74
Positioning of the drill feeds with a directional laser [5]

Summary. Drilling performance has now reached about 200 cm/min, which means about 100 m of hole per jumbo boom and hour.

The optimisation of drilling work using an electronic control system also permits the matching of drill rods and bits. Drill rods with diameters of 1¼ or 1³⁄₈ inch can be reduced to 1¹⁄₈ and 1 inch and bits from 45 mm diameter to 38 to 32 mm. This increases the drilling speed.

Good drilling precision is a precondition for successful blasting. It begins with the marking of the holes, which is no longer necessary with a computer-assisted drill jumbo. Holes running off course due to excessive pressure or changing geological conditions has become less frequent. The lookout angle is also adapted to suit the round length by the electronics.

This type of drilling system is very flexible in use, with many intermediate solutions being available between fully automatic and hand-controlled drilling. The tunnel drill jumbo can thus also be used to drill holes for rock bolt.

Workplace safety is improved and the working conditions are improved because the noise level of modern drilling machinery is considerably less than former machines, and the mist of air and oil that formerly infested the tunnel when pneumatic drills were operated is now history.

The saving of data from drilling operations that is possible with a computer-assisted drill jumbo and the linking of this data with the survey data has become a cost-effective procedure.

5.7.4 Mucking and tunnel logistics

Mechanisation of muck transport is advantageous, above all in longer tunnels. The loading of the means of transport and the transport to the tip are to be considered as a whole.

The formerly often practiced method of tunnelling with numerous workplaces only demanded wagons to pass in the invert heading. As an alternative, in the St. Gotthard Rail Tunnel (built 1872 to 1882), two miniature roller conveyors were installed parallel to the main tracks, on which small containers, which were often baskets, could be run along the length of the train. The enlargement to the full profile in various stages was always transported along the already installed main transport line in the pilot tunnel.

In contrast to this, rail-borne transport in a tunnel excavated as a full face or in large part sections, as became possible with the mechanisation of drilling work and the introduction of loaders, requires measures to load a train from the end.

Tunnelling logistics includes the transport chain for the supply of the advance and the disposal of the blasted muck. In order to avoid any friction, obstructions of the main means of transport should be removed, at least in the advance area. Appropriate staging of the works between advance and backup area also helps avoid accidents.

Equipment to enable wagons to pass in the excavation area, like traversers, California switches or cherry pickers that exploit vertical space, can avoid time wasted for the organisation of wagons, which is always dead time in the overall loading process, but not completely solve the problem.

The sliding floor was invented for rail-borne transport in order to enable simple track changing in the advance area. The muck wagons still had to change tracks. A sliding floor does, however, have disadvantages when the method of working has to be changed from full face to partial face excavation.

The use of conveyor belts first made continuous working possible. The first trials in the Great St. Bernhard Tunnel, which have to be seen as pioneering work, were admittedly not very promising because the blasting could not produce small enough fragments and because the conveyor belt technology of the time was not sufficiently large and robust.

For smaller tunnels, the bunker train and the Häggloader represent real steps forward.

Further developments, particularly of conveyor belt construction with wide transport belts, have managed to overcome different transport levels (top heading advance, bench excavation). Formerly, the muck produced by blasting sometimes had to be broken due to the irregular block sizes (Great St. Bernhard). The improved drilling performance now available permits the drilling of the face with closer hole spacings. The cost of explosives becomes ever cheaper compared to wage costs, so it does not have to be so sparingly used and this in combination with the increased number of holes lead to more uniform small pieces of rubble in the muck pile. This in turn makes the processing of the excavated material easier; the wide transport belts transport the material over the working sites near the face into the loading area behind.

Horizontal division of the cross-section has also become attractive thanks to the development of bolting technology and the materials of rock bolts.

Backup systems such as have been developed for the supply to and disposal from tunnel boring machines can now also be used in drill and blast tunnelling. The trailing platform carries the conveyor belt, power cables and water supply pipework as well as the air duct cassette for the continuous extension of the ventilation duct. Overhead cranes serve to distribute the material delivered by rail to the construction works below the platform. The integrated pedestrian bridge ensures the best possible safety for the crew.

The possibilities of horizontal separation are considerable. They range from a simple conveyor belt to a small travelling workshop with hand magazine.

A backup system for a smaller tunnel is being used in the pilot tunnel of the Semmering Base Tunnel.

On the Vereina Tunnel south contract (built 1993 to 1997), an extensive system was used. The relatively long narrow-gauge rail tunnel (7,500 m) with a minimum cross-sectional area of 39 m^2 permitted a major innovation (Figs. 5.75 and 5.76). This route will be followed further in the Lötschberg and Gotthard Tunnels.

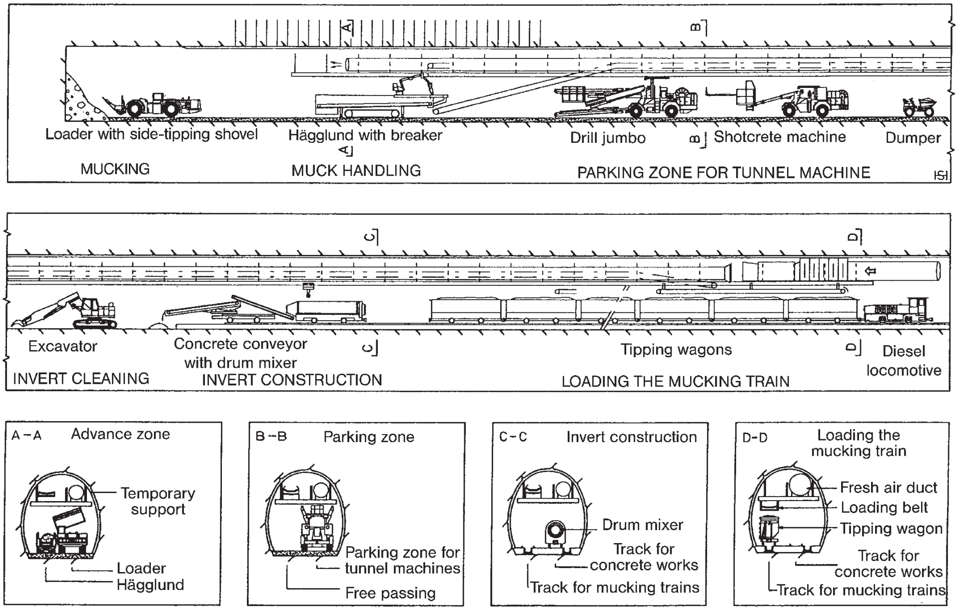


Figure 5-75 Schematic diagram of a backup system in drill and blast tunnelling, Vereina South Tunnel [10]

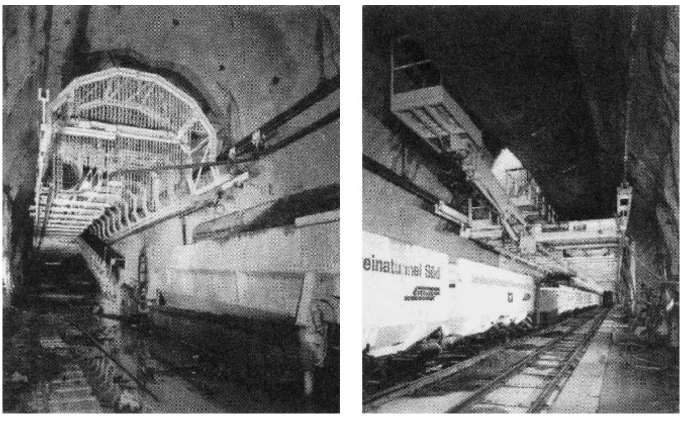


Figure 5-76 Backup system of the Vereina South Tunnel at the transfer of the excavated material (left) and the loading of the mucking train (right); ROWA AG

6 Mechanised tunnelling

6.1 General

As an alternative to the classic methods of tunnelling, tunnels can also be bored by tunnelling machines. Technical innovations in recent decades have had the result of considerably increasing the number of mechanically excavated tunnels and the scope of application.

Tunnelling machines can be equipped with mechanised cutting tools or non-mechanical means of excavation. Mechanical excavation tools go back to Brunel, who produced the first designs at the start of the 19th century. Tunnelling machines can be made without shield, with shield or with crown support or head shield [294]. An overview of the use of machines in modern tunnelling is shown in Fig. 6-1. The most important factor for categorisation is the properties of the rock mass [333].

For more detailed information about tunnelling machines, reference can be made to the books “Mechanised Shield Tunnelling” [244] and “Hard Rock Tunnel Boring Machines” [242].

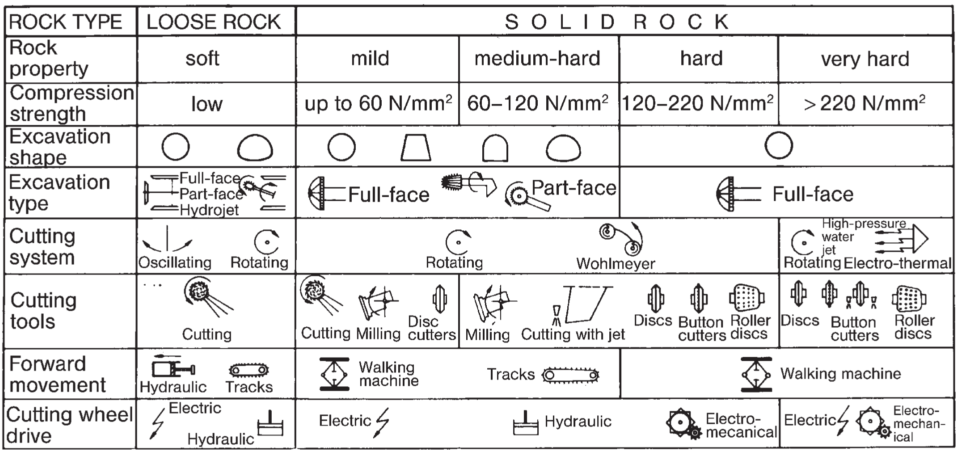


Figure 6-1 Scope of application and functional principle of tunnelling machines, based on L. Müller [260]

6.2 Categories of tunnelling machines

An overview and definition of tunnelling machines is shown in Fig. 6-2. Tunnel boring machines and partial face machines are generally used in hard rock and shielded tunnelling machines are intended for use in friable unstable rock and particularly in soil, although the scope of application overlaps.

Tunnel boring machines (TBM) can be built with or without shield. The difference to shield machines is in the construction of the excavation tools and the different method of transferring the thrust forces. A TBM normally excavates the full face with a rotating cutterhead. The face is normally stable or only supported by the steel construction of the cutterhead. Shield machines can also excavate the full face with a cutting wheel but some also excavate the face partially. The two main groups of full-face and part-face excavation are further subdivided according to the method of face support. Part-face machines are used in ground, which is temporarily stable without a shield. The excavation takes place not by boring the full face but by cutting the face with a boom-mounted cutting unit. The functional principles of tunnelling machines will be further explained in the following sections.

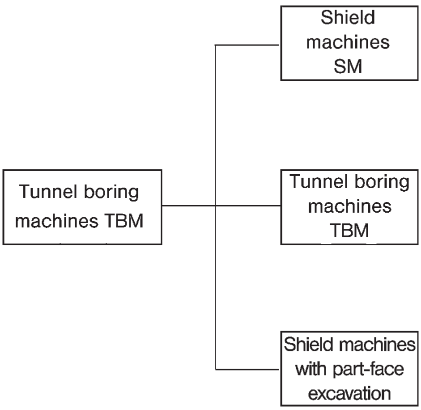


Figure 6-2 Overview of tunnelling machines (from [242], based on [60])

6.3 Shield machines

6.3.1 Categories of shield machines

An overview of the categories of shield machine is given by the German association for underground construction DAUB [60] (Fig. 6-3).

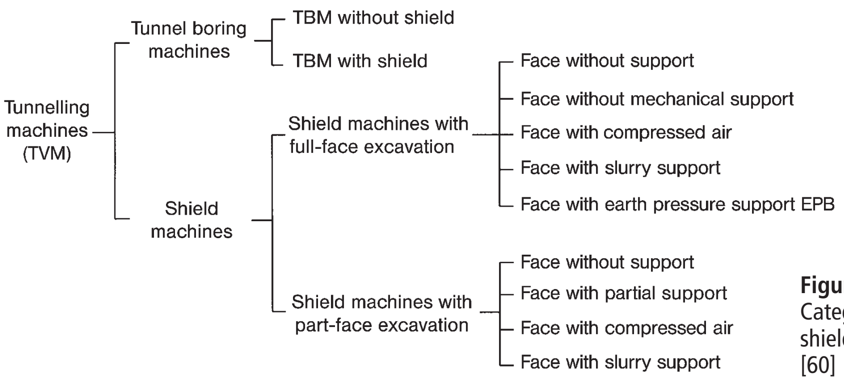


Figure 6-3 Categories of shield machine [60]

6.3.2 Basic principle, definition

General. The basic principle of shield tunnelling is that a normally cylindrical steel construction (shield) is thrust forward along the tunnel axis and simultaneously the ground is excavated. The steel construction supports the excavated cavity until the temporary support or the final lining is installed at the end of it. The shield has to resist the pressure from the surrounding ground and hold back any groundwater.

The shield is pushed forward along the tunnel axis as the excavation proceeds in order to support the excavated cavity. The necessary thrust forces are normally provided by thrust cylinders pushing against the already installed lining. This means that the tunnel lining and the tunnelling machinery have to be coordinated to suit each other in detail. Both the correct functioning of the shield and the quality of the completed tunnel depend on the compatibility of the machine and the lining system.

As support is normally installed inside the protection of the shield skin, a gap remains as the shield progresses forwards, and this has to be filled in order to minimise loosening of the ground and subsidence. A suitable means of backfilling or grouting behind the machine has to be found and the shield needs to be appropriately equipped.

The use of a shield to construct a tunnel with variable cross-sections is normally uneconomic and very difficult. Nonetheless, there are some examples of the driving of combined sections with shields, for example for underground stations. In this case, not only the shield drive is problematic, but also the final lining, as such applications are mostly constructed in water-bearing soil types. There are various ways of solving such problems. While complicated special segments were formerly used for station sections, pipe screen slabs, grouted or ground freezing screens now offer attractive alternative support methods.

Face support methods. While the cavity along the sides of the tunnel is supported by the shield, additional support measures may be required for the face according to the soil and groundwater conditions encountered. Fig. 6-4 shows five different methods of supporting the face, which are described in detail in the following sections.

These methods of supporting the face represent an advantage of shield tunnelling. It is possible to support the ground even as it is excavated, in contrast to all other methods of tunnelling.

Methods of soil excavation. In addition to the face support method, the method of excavation is another important characteristic of shield machines (Fig. 6-5).

Manual digging in “hand shields” is the simplest process and is still practiced today in exceptional cases, for short stretches under favourable geological and hydrological conditions, which means temporarily stable soil above the groundwater table.

The use of machinery is more usual. This can be full-face or part-face excavation. In partial face excavation, the face is excavated in sections by machinery like hydraulic excavators or roadheader booms (boom-in-shield) with special ripping or cutting mechanisms (Fig. 6-6), which can be positioned and controlled either by an operator or automatically.

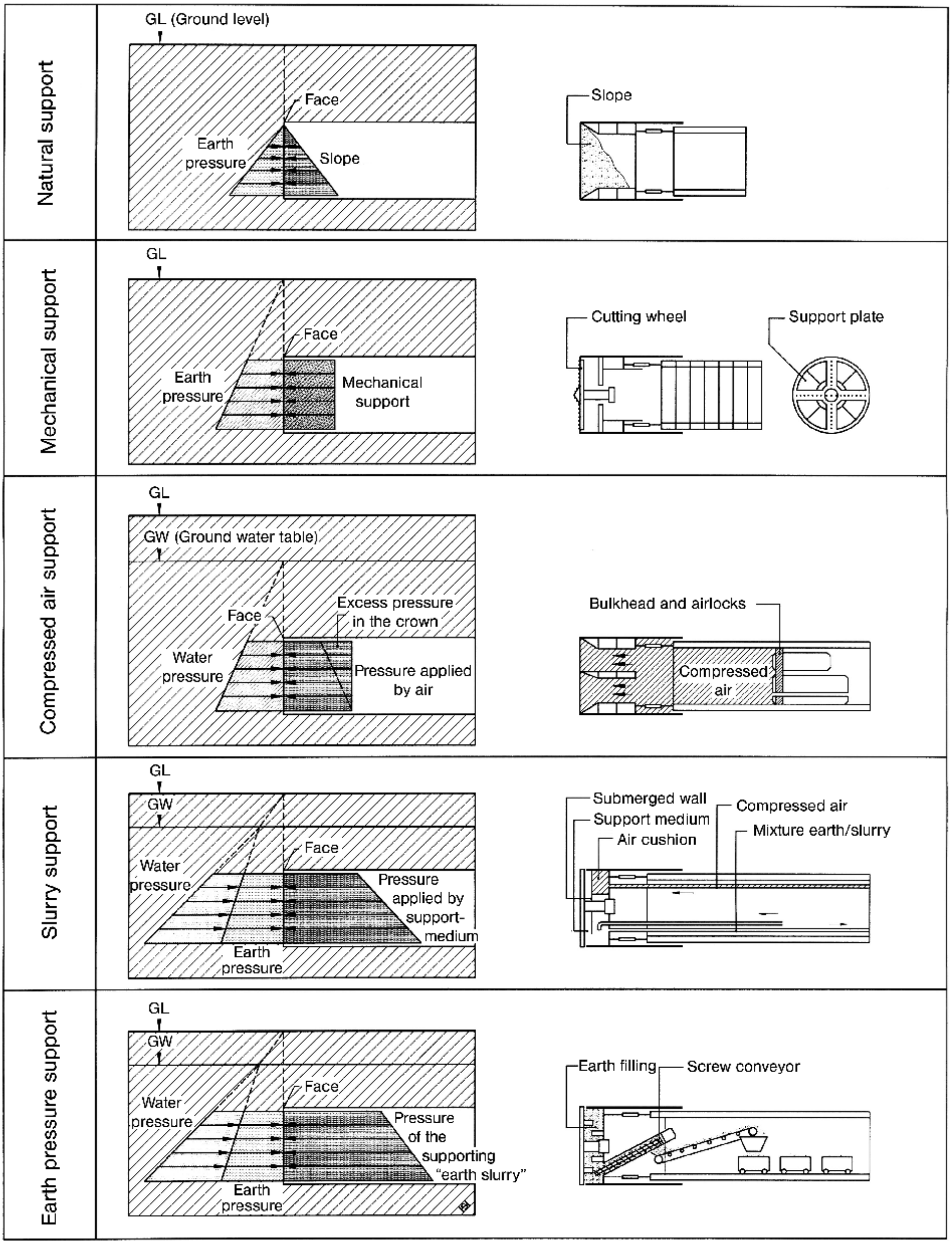
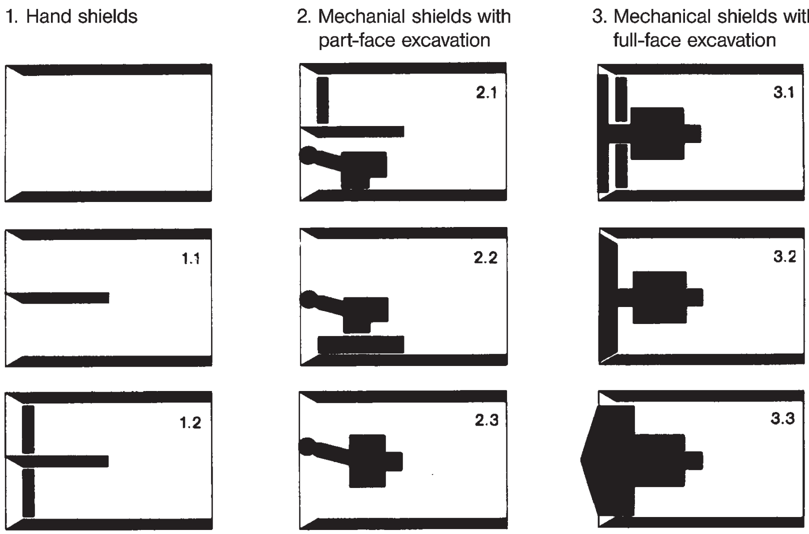


Figure 6-4 Methods of supporting the ground and holding water at the face. [331]



1.1 Working platform
1.2 Face support platform
2.1 Face support with roadheader
2.2 Mobile roadheader
2.3 Integrated roadheader
3.1 Full-face machine with cutting wheel, face support if required
3.2 Full-face machine with cutting wheel
3.3 Full-face machine with disc cutter cutterhead

Figure 6-5 Shield systems: overview of the various methods of excavation

The full face can be excavated by cutterheads, spoked cutting wheels or rim cutting wheels (perhaps with shutters) depending on the geology. Two further possibilities not shown in Fig. 6-5 are hydraulic excavation using fluid jets driven by compressed air and extrusion excavation, in which the action of the thrust cylinders on a very plastic soil forces it through shutters in the front wall of the shield into the working chamber.

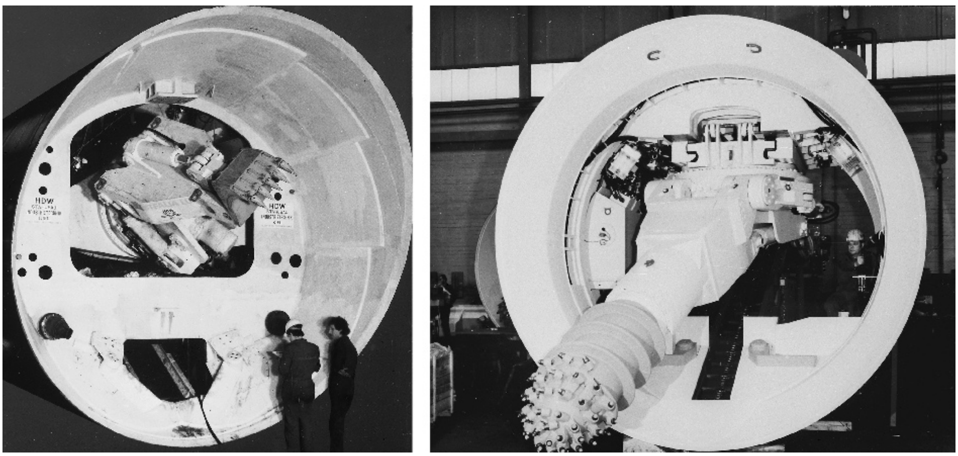
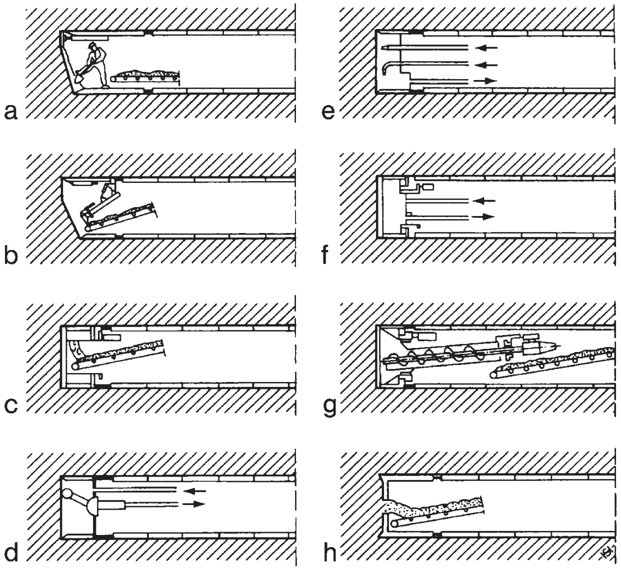


Figure 6-6 Excavator and part-face cutting unit as excavation tools for open shields; from the former Westfalia Lünen

Mucking and transport. Special transport systems are required to transport the excavated material from the face, through the shield and out of the tunnel. The appropriate system directly depends on the nature of the soil encountered and the associated methods of face support and excavation, as these influence the parameters of consistency and transport suitability of the material to be cleared. Fig. 6-7 shows the possible methods of mucking, which can basically be divided into dry removal/pumping or slurry removal.

The means of transport along the tunnel depends on the face support medium and can be slurry pumping, conveyor belts, dump trucks or rail-borne systems (mucking trains).



a Hand digging and dry mucking on conveyor belt
b Mechanical part-face excavation (here ripper boom) and dry mucking on conveyor belt
c Mechanical full-face excavation and dry mucking on a conveyor belt (the feed to the belt can be in the middle or lower part of the cutting wheel through mucking pockets)
d Mechanical part-face excavation and hydraulic mucking
e Hydraulic excavation and hydraulic mucking
f Mechanical full-face excavation and hydraulic mucking
g Mechanical full-face excavation, muck removed by screw conveyor and transferred to a conveyor belt
h Extrusion excavation and transfer of the muck onto a conveyor belt

Figure 6-7 Possible means of clearing muck from the shield

Hydraulic transport is used in combination with full-face and hydraulic excavation, and the transported muck is normally separated outside the tunnel. In large-diameter tunnels, like for the first time in the 10.60 m diameter Galleria Aurelia in Rome, economic considerations may lead to the installation of a travelling separation plant in the backup system. Hydraulic muck transport offers particular advantages in small diameter tunnels and longer tunnels.

When part-face excavation is used, the muck is loaded by the bucket of the excavator or lobster-claw loaders onto conveyor belts, chain conveyors or into a screw conveyor (Fig. 6-6). The muck is normally lifted to the middle level of the backup and loaded directly onto rail-borne or wheeled transport to be transported out of the tunnel. Conveyor belt transport direct to ground level is also possible.

Support installation. Segments, prefabricated pipes or precast elements are the normal forms of lining for shield drives [81], [174]. Due to the heavy weight of the elements, erectors or segment installation machines have been developed for mounting in shields. Fig. 6-8 shows the essential activities involved with the transport and installation of segments in the shield. Details about segments and their installation are described in section 2.8.5.

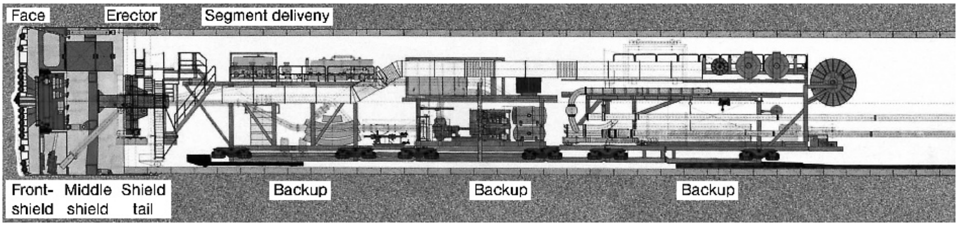


Figure 6-8 Transport and installation of segments for a shield drive in soil. (Oberbilk, 1998)

The placing of extruded concrete directly behind the shield has gained more significance in recent years. This process is also dealt with in Chapter 2. The advance of the shield and placing of the extruded concrete occur almost simultaneously. The thrust forces are normally resisted by the formwork unit, which is not normally a problem for blade shields.

Settlement behaviour. The subsidence to be expected at the surface is of great significance, particularly in built-up areas, and can lead to dangers. Settlement around the shield can have various causes (Fig. 6-9). While numbers 7 and 8 as shown in Fig. 6-9 cannot normally be influenced at reasonable cost and thus have to be regarded as unavoidable, the causes numbered 1, 2, 3, 4 can be minimised by taking appropriate measures.

Settlement causes numbers 5 and 6 can be prevented by early and sufficient grouting of the annular gap, which is created behind the shield tail between the solid lining (segment tube) and the surface of the excavated ground as the shield progresses. While this gap is grouted, an adequate shield tail seal has to be provided to prevent the grout penetrating between the lining and the shield and hindering the advance.

K. Fujita [101] gives an overview of settlement, comparing various processes.

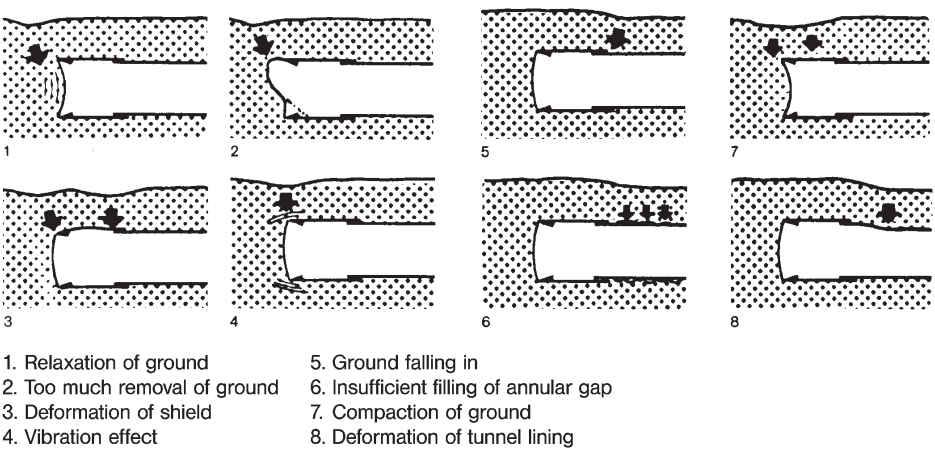


Figure 6-9 Causes of settlement from a shield drive, from W. Schenck [309]

6.3.3 Face without support

Shields, which do not provide face support (see Fig. 6-4) or only make use of a batter are also described as "*open shields*". Open shields have no closed system to resist pressure at the face to support the ground and hold back water.

The full face can be excavated by a cutting wheel, or only part of the face at any time. This type of shield can however not be used in soil below the water table without additional measures (groundwater lowering, grouting, ground freezing). If there are small quantities of seepage water or localised water lenses, open dewatering can be carried out at the face of an open shield. Extraction wells and other drainage measures are also suitable to relieve and overcome groundwater pressure.

When the cross-section is large, the face is divided and, except for shields fitted with closed cutting wheels, also accessible. The good flexibility, particularly of hand and partially mechanised shields, even enables tunnelling when the face partially or completely consists of rock, or the ground contains boulders. The low investment cost for machinery compared to other types of shield can also be economic for short lengths of tunnel.

Shield machines for partial face excavation are equipped with hydraulic excavators or cutting heads (roadheader boom or cutting machine) for excavation. Also worth mentioning are new systems with a modular system, in which an excavator of cutting boom can be mounted and changed to suit the ground conditions.

A significant advantage of partial face machines over full-face machines is their ability to cut non-circular sections, although even in this case there are developments to overcome the deficiency, which are being tried particularly in Japan.

In order to enable boring through harder rock formations or to increase the advance rate in soil, open shields are also sometimes fitted with fully mechanised excavation equipment. The full face can be bored by a cutting wheel or cutterhead (TBM) fitted with the appropriate cutting tools.

6.3.4 Face with mechanical support

Mechanical support to the face can be provided by breasting plates, which can be hinged or mounted on springs, but also by the construction of the excavation equipment itself. Such methods cannot however control any groundwater pressure (Fig. 6-4).

In full-face machines, mechanical support can be provided by an almost complete closing of the openings in the cutting wheel by spring plates integrated in the cutting wheel. The spring stiffness is designed so that the plates are pressed back as the machine advances and the excavated soil falls into the excavation chamber. In practice, this technology has not proved very successful, and the use of this type of shield is restricted to dry cohesive soils or alternating cohesive and non-cohesive soils. According to recent publicatons by DAUB, mechanical support is no longer recommended for use in shield machines with full face excavation due to their unreliable face support.

In partial face machines, a certain degree of support can be provided by dividing the cross-section, for example with platforms or shutters. Partially mechanised shields are often fitted with hydraulically operated breasting plates in certain areas and around the rim of the shield.

6.3.5 Face under compressed air

Compressed air shields are hand shields, partially or fully mechanised shields with an additional system of air locks to enable the use of the shield with compressed air below the groundwater table or even under open water.

Functional principle. The main feature of this process is that the excavation chamber of the shield is pressurised in order to keep water out of the working chamber. The compressed air balances the pressure of the hydrostatic pressure of the water. The air pressure must be greater than or equal to the water pressure acting in the shield invert (see Fig. 6-4**).** Due to the different densities of air and water, this results in positive pressure at the crown, so the air from the crown penetrates into the soil and flows to the surface (Fig. 6-10). This air must be resupplied continuously by compressors. Emergency power generators and backup compressors are required; for details, see the "Recommendations for water control with compressed air" [62]. When the tunnel is near the surface, there is a danger of the face collapsing (see "Blowouts", section 6.3.9.2), when the soil particles become unstable due to the air flow. The processes are described in detail in [140].

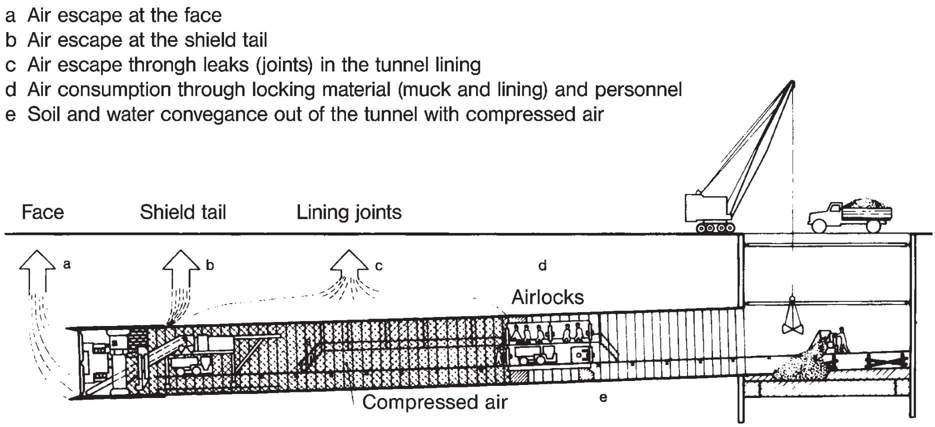


Figure 6-10 Air loss during compressed air working [168]

It is not possible to use compressed air support to resist ground pressure. The soil must therefore be temporarily stable if no additional support measures are provided.

In order to be able to enter the pressurised chamber for repair or maintenance work to the excavation machinery, man locks and material locks are required in the pressure bulkhead.

The compressed air regulations (DLV) include a requirement that man locks are not used for the locking of materials, and vice versa that material locks are not used by workers. For combined locks, the provisions for man locks apply. It is also laid down that when the operating pressure is more than 1 bar of positive pressure, a treatment chamber must be provided for compression sickness patients.

Limits to application. The limits to the compressed air process in tunnelling are essentially determined by:

– the maximum permissible positive pressure of 3.6 bar according to the compressed air regulations.
– The air and water permeability of the soil (upper threshold of water permeability $k_w \leq 10^{-4}$ m/s).
– A minimum cover above the tunnel crown (one to two times of the tunnel diameter according to soil type) to ensure blowout safety.
– Shortened effective working times under pressure due to locking time.
– Reduced performance of the crew under compressed air (and danger of compressed air sickness).
– Increased fire risk.

Support is normally installed under atmospheric pressure although in principle, it is also possible to pressurise an entire tunnel starting from the launching shaft. Various layouts of locks for compressed air working are shown in Fig. 6-11.

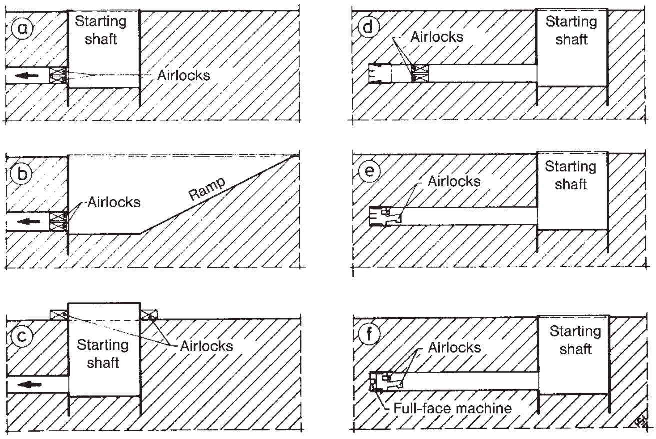


Figure 6-11 Lock layouts for compressed air working

Due to the deficits of compressed air shield tunnelling in the areas of health and safety and process technology compared to modern slurry and earth pressure shields, compressed air shields are now seldom used. Nonetheless, the regulations for working under compressed air are still very significant because even when the face is supported by slurry or earth pressure, the regulations for working under compressed air still apply when the excavation chamber has to be entered, for example to inspect the face, carry out maintenance or remove obstructions.

6.3.6 Face with slurry support

The scope of application of shields with slurry face support today covers all types of soil, even below the groundwater table. The method can also sometimes offer advantages in stable ground with groundwater.

Three lines of development of slurry shields can be distinguished, a Japanese line leading to the modern slurry shields, an English line of development which has now been abandoned and a German line that has led to the modern Hydroshield, Hydrojet and Thixshield (Fig. 6-12).

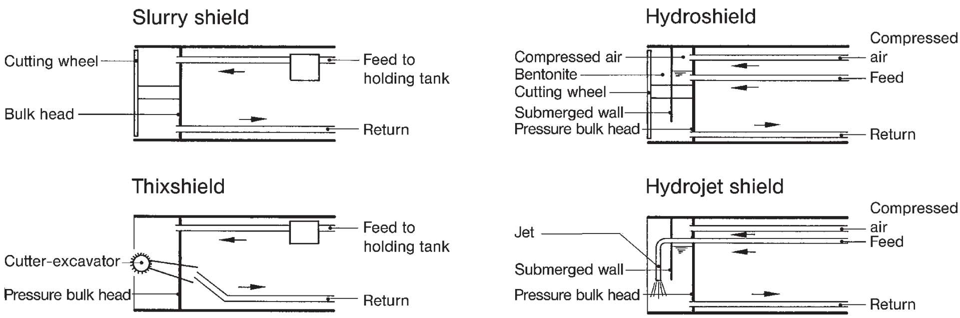


Figure 6-12 Functional principle of slurry shields

6.3.6.1 Functional principle

The face in front of a slurry shield is supported by a support medium, which depending on the soil conditions can be a suspension of water and bentonite or clay (sometimes with additives). The fluid support medium (slurry) is pumped into the closed excavation chamber against the face, where it penetrates under pressure into the soil, seals it and forms a “filter cake”. The term filter cake describes the almost impermeable layer of bentonite or clay particles, which is formed as the suspension is filtered under the flow gradients at the surface of the face. The filter cake enables the pressurised suspension in the excavation chamber to hold equilibrium against ground and water pressure. (Fig. 6-4). The support medium in slurry shields is also used as a transport medium. The soil removed by the excavation tools is mixed into the support medium in the excavation chamber. The mixture of suspension and soil is then pumped to the surface through pipework. In a separation plant, which is normally located above ground, the soil is separated from the support medium, the slurry is replenished if required and pumped back down to the face.

The essential disadvantages of this construction process are the space and energy required by the separation plant required for the operation of a slurry shield and the difficulties of disposing of the separated fine material and any bentonite suspension loaded with fine-grained material that cannot be separated. These disadvantages are particularly evident in inner-city areas. The cost-effectiveness of a slurry shield in comparison with other processes is essentially determined by the cost and difficulty of separating the transport suspension. The technical limits to its application are set by the permeability of the soil encountered.

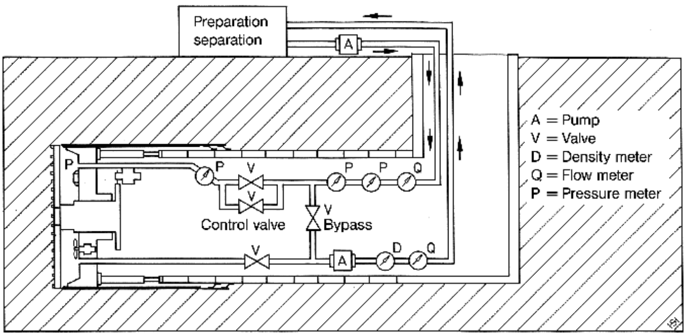
6.3.6.2 Slurry shield

The characteristic features of a slurry shield are the type of support medium used – a clay suspension is usual – the construction of the cutting wheel and the method of steering and method of controlling and checking the support pressure.

The cutting wheel of a slurry shield is flat and almost closed, which provides a certain degree of mechanical support in addition to the action of the pressurised slurry. Access to the face, for example to remove obstructions, is only possible through a few windows that are closed during normal operation. The excavation tools, in general scrapers or teeth, are fitted in two rows radially so that the machine can excavate while rotating in either direction. The soil can pass through slots in the cutting wheel parallel to the excavation tools; the width of the slots has to be suitable for the expected maximum grain size. At the same time, oversized grains that cannot be passed through the hydraulic transport circuit are held back by the slots.

The support medium is fed into the excavation chamber from the top, and the mixture of support medium and excavated soil is extracted from the bottom, passing mixing paddles intended to avoid settling and produce a homogeneous mixture for pumping.

In slurry shields, the support pressure at the face is controlled by adjusting the quantity of support medium pumped into and out of the excavation chamber (Fig. 6-12 and Fig. 6-13). The support pressure is measured by electric pore water pressure sensors in the excavation chamber and in the feed and return pipes and compared by a computer with the theoretical, calculated support pressure. The pumps and the valves in the suspension circuit are similarly controlled.



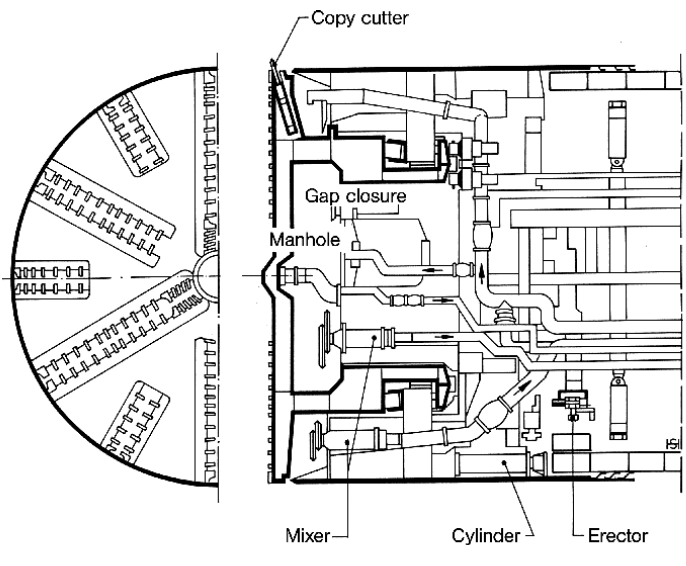


Figure 6-13 Support pressure control in a slurry shield (top) [244] and longitudinal section through a slurry shield (bottom); Mitsubishi Ltd

Since the face is not visible, its stability can only be checked to find out whether there are any local collapses by comparing the mass of the theoretical and actual volumes excavated. The actual volume excavated is determined by measuring the density of the support medium and calculating the theoretical excavated volume, taking into consideration the weight per volume, the bedding density and the void ratio. In practice, this procedure has not yet been perfected.

6.3.6.3 Thixshield

The combination of slurry support with part-face excavation is known as the "System Holzmann" Thixshield (Fig. 6-14). The telescopic excavator boom with excavator or cutting unit is mounted on a ball mounting at the centre of the pressure bulkhead and excavates the face controlled by a programme – or also manually in case an obstruction is encountered – and is also capable of cutting through soft rock bands. The range of geological conditions such a machine can cope with is similar to the Hydroshield described below. The cutting head also holds back grain sizes that would be too large for the pumping circuit at the intakes inside the cutting crown.

If it is necessary to enter the excavation space, for example to make repairs or remove obstructions, it is possible to completely or partially lower the bentonite suspension and apply compressed air to the inside of the shield. The obstructions are then removed through a lock.

Individually controllable breasting plates are provided in the excavation space, which can nearly close the shield at the front. The development and use of this system has been discontinued.

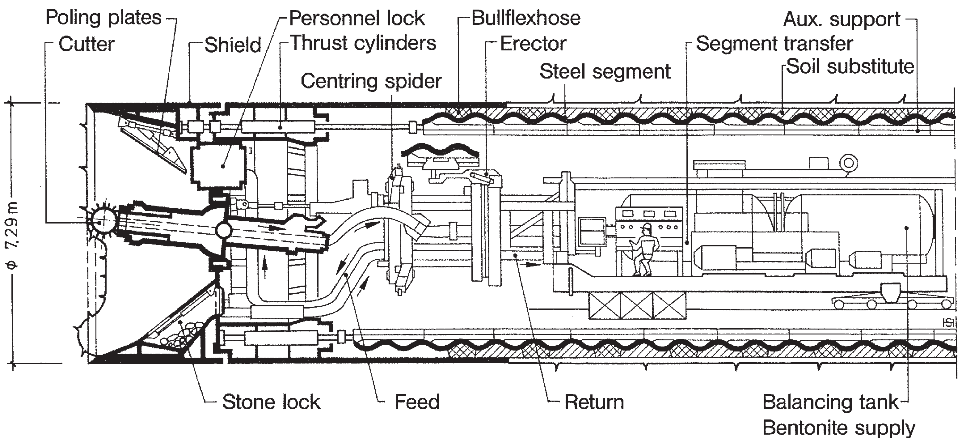


Figure 6-14 Longitudinal section through a Thixshield

6.3.6.4 Hydroshield

In contrast to Japan, the soil in Europe tends to be changeable. The basic principle of the Hydroshield is therefore more flexible regarding the scope of application in different geological conditions. Hydroshields today are suitable for almost any soil, and with additional equipment can even cope with rock formations (Fig. 6-15).

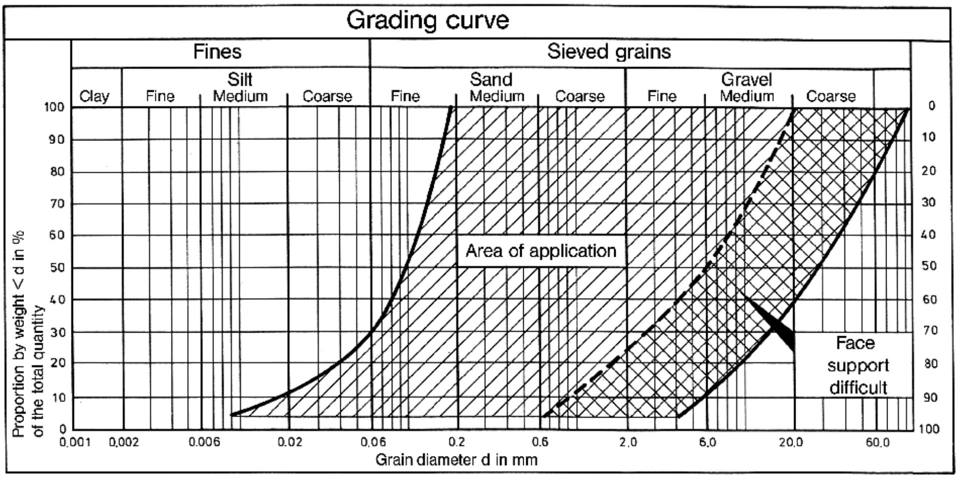


Figure 6-15 Scope of application of a Hydroshield depending on soil type [171]

All the Hydroshields on the market today are based on the original development work of Wayss & Freytag [244].

A unique construction feature is the division of the excavation chamber into two parts by a submerged wall. The support pressure to the wetted face is regulated, in contrast to the slurry shield, not by the support medium but by the air bubble in the backward part of the excavation chamber.

Another difference of this process to that of a slurry shield is the use of water-bentonite suspension, which is generally more suitable for European soils. The use of bentonite is associated with the formation of a filter cake in the face.

The essential advantage of the two-part excavation chamber with an air bubble in the back chamber is the decoupling of the control of support pressure from the quantity of suspension in the pumping circuit. Over a relatively large range of tolerance, which approximately corresponds to the volume of suspension in the back chamber, sudden losses of support fluid, for example when a fault zone is encountered, can be overcome without the collapse of support pressure at the face. It is also possible to increase or decrease the volume of suspension in circulation without any direct influence on the support pressure, in case this is necessary to improve the transport.

The air bubble and the submerged wall also permit safe access to the excavation chamber at any time through a lock provided in the crown. This makes the removal of obstructions simpler than with a slurry shield. When an obstruction has to be removed, but also for repair and maintenance to the cutting wheel, the bentonite suspension is emptied from the excavation chamber and replaced with compressed air. The filter cake continues to provide the sealing of the face and enables the face to be supported by compressed air alone, although the bentonite cake does shrink on contact with air and has to be refreshed occasionally (for example by spraying with suspension or refilling the chamber) to limit air losses.

The construction of the cutting wheel as an open star with freestanding spokes (Fig. 6-16) enables the immediate flow of the excavated material through the cutting wheel and into the excavation chamber; this also has the effect of decoupling support and excavation functions. As a result of the open cutting wheel, a screen has to be provided in front of the intake to the extract pipe in order to catch oversized grains. As a tendency, closed cutterheads have been favoured in recent years due to the higher level of working safety and better flexibility for tool arrangement. Modern Hydroshields also have a hydraulically operated stone crusher mounted in front of the screen, which breaks larger stones down to a suitable size for pumping. Material building up in front of the screen is hindered by targeted flushing jets of the suspension feed.

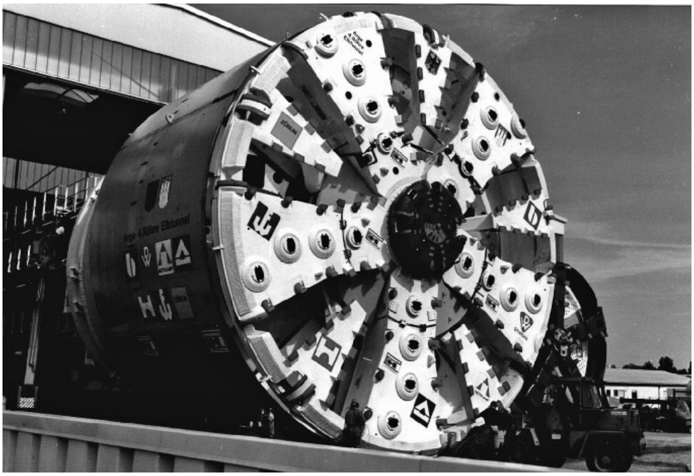


Figure 6-16 Hydroshield for the Elbe Tunnel, D = 14,12 m

6.3.6.5 Mixshield as a Hydroshield version

The concept of the Hydroshield was further developed by company Wayss & Freytag in collaboration with Herrenknecht GmbH to enable the conversion of the method of face support. The various operating modes permit a wide scope of application according to the manufacturer.

The design of this shield enables rebuilding for operation with slurry support, earth pressure support and with compressed air support, the latter including open operation. Equipment that is superfluous is blanked with shutters during operation. Most of the tunnelling shields described as Mixshields are however never converted during the tunnel drive and are only operated in the Hydroshield version. The shield for the Stadtbahn Duisburg-Duissern is shown here as an example (Fig. 6-17).

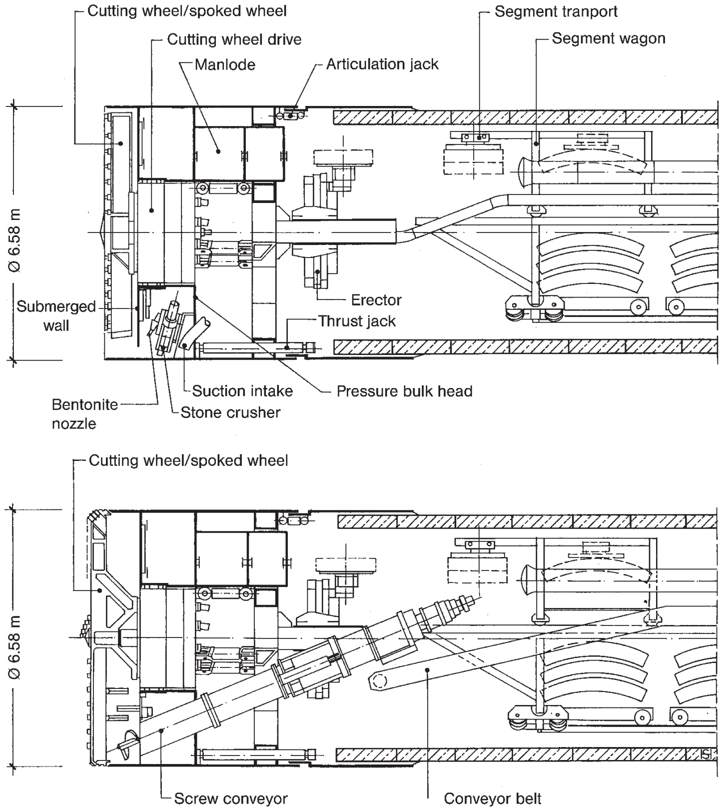


Figure 6-17 Mixshield for operation with slurry or earth pressure support, Stadtbahn Duisburg-Duissern 1994 (Herrenknecht AG)

6.3.6.6 Hydrojetshield

The Hydrojetshield (see Fig. 6-12) is another development of Wayss & Freytag AG, which was patented in 1979.

Instead of mechanical excavation with a cutting wheel, the soil is removed by directed fluid jets into the excavation chamber. The omission of a central drive block in a small-diameter shield permits manual breaking of obstructions and their removal, which makes the slurry shield with hydraulic excavation a flexible solution. The geological range of application of this type of shield is similar to that of a conventional Hydroshield. Consolidated soil and soil with a certain cohesion do however limit the scope of application. Cohesive soils only permit slow advance rates for the same excavation jet pressure.

6.3.6.7 Hydraulic soil transport

The removal of the excavated soil as a suspension in a fluid pumped by a centrifugal pump is the most elegant and space-saving method of overcoming the spatial and pressure difference [244]. The support medium also undertakes the role of transport medium. After the soil has been separated from the loaded suspension in a separating plant, it is pumped back down the circuit to the shield (Fig. 6-18).

The material excavated by the cutting wheel settles at a rate depending on its grain size in the suspension, which rotates slowly with the cutting wheel.

If the intake to the pumped circuit is mounted at high level, rapidly sinking large grains can be lifted back by pockets or mechanical paddles on the cutting wheel. The placing of the intake at the lowest point makes such aids unnecessary.

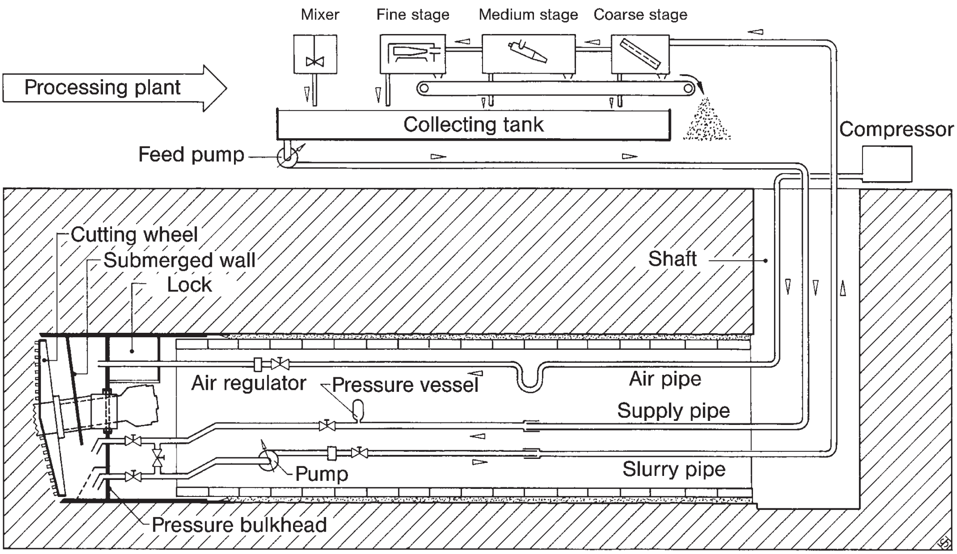


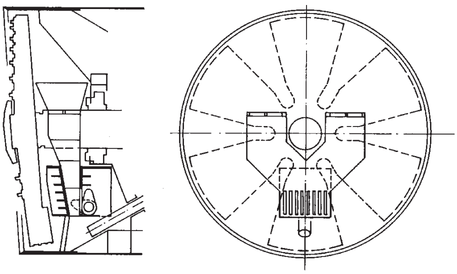
Figure 6-18 Scheme of the circuit of support and transport medium in a Hydroshield [244]

Breaker. An intake screen protects the pump and the pumped circuit from grain sizes, which could cause a blockage. The large grains gathering on the screen can be removed by hand if the quantity is small. Otherwise, they are broken down to pumpable size by a breaker in the excavation chamber. The crusher works in the suspension. In order to avoid pressure fluctuations at the face, rapidly running machines cannot be used. Various types of breaker are usual according to their installation location (Fig. 6-19).

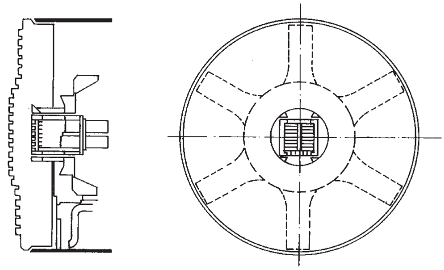
While the jaw breaker and the box breaker are supplied by the cutting wheel, the grab breaker requires no feed. The gaping mouth also collects larger stones (Fig. 6-20).

Stone traps or breakers fitted in the pipeline are not effective because the stretch of pipe in the shield with its valves and fittings cannot be made with a larger diameter than the pipeline in the tunnel, as a constant transport speed has to be maintained.

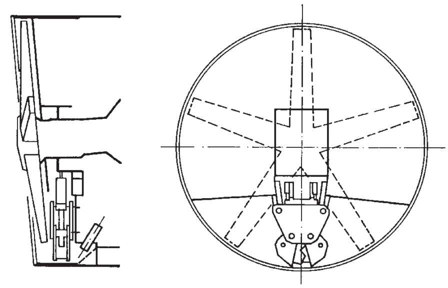
Material flow in the excavation chamber to avoid sticking. The design of the cutting wheel and the excavation chamber for the hydraulic transport from the face to the intake of the suction nozzle, which can often be slewed, requires much experience. The slow flow speed of the fluid in the large cross-section does not result in forced transport. Soil can pile up in dead corners and cohesive soil can also adhere to the surfaces of the cutting wheel or in front of the screen. Targeted flushing jets can be used to improve the situation. Breakers working in the suspension in front of the screen also mechanically support the cleaning effect.



a) Impact crusher



b) Box-type crusher



c) Jaw crusher

Figure 6-19 Various types of breaker [237]

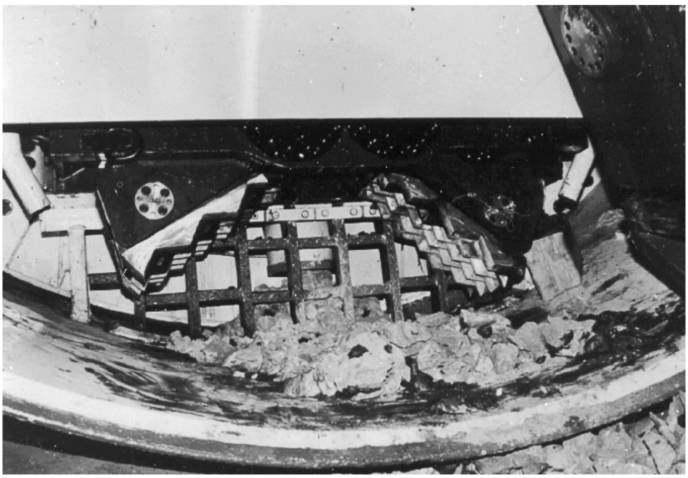


Figure 6-20 Grab breaker in front of the intake screen [237]

6.3.6.8 Soil separation in shield operation with hydraulic transport

When a Hydroshield is operated, it is important to ensure constant high quality of the bentonite suspension with removal of the partially dispersed soil particles as completely as possible before reuse. The rheological properties of the suspension should not be worsened by the separation process. This is technically complicated and expensive, particularly in fine-grained soils. The term separation describes the separation of the transported soil from the transport medium (suspension).

While larger lumps of soil and coarse-grained fractions up to medium gravel can be separated from the loaded suspension with mechanical vibrating screens, more elaborate equipment is required to remove sandy and fine fractions that have passed the sieve. Technically, it is possible to completely separate the soil from the carrier suspension, but the high purchase and operating costs of separating plants make this impractical. The machinery should therefore be planned according to commercial criteria, with the intention of finding a balance between the cost of separating the soil and transporting the suspension still loaded with fines to a special tip. The locally applicable landfill regulations and disposal practicalities have to be taken into account.

Fig. 6-21 shows an example of a separation plant integrated in the mucking circuit of a Hydroshield.

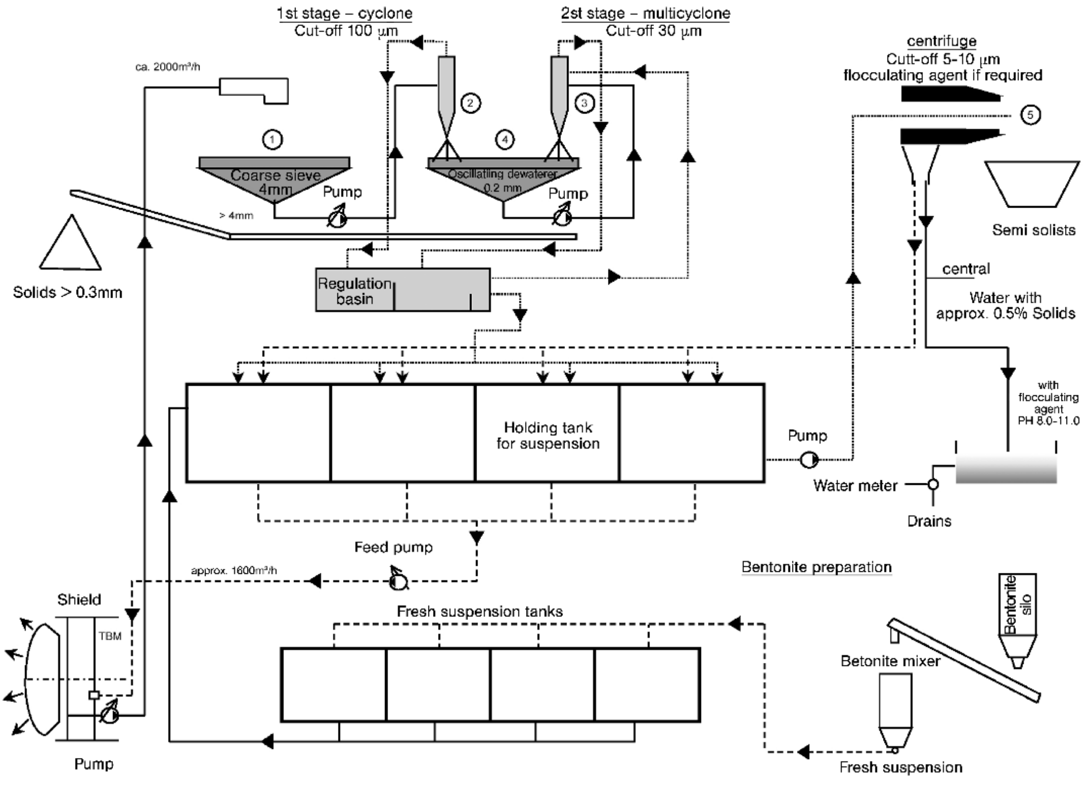


Figure 6-21 Functional diagram of a separation plant

The mixture of soil and suspension that is pumped out of the tunnel, which typically has a density of 1.2 to 1.3 t/m^3, first flows over a coarse sieve (1) and then passes through two separation stages consisting of hydro- and multicyclones (2),(3). A cyclone stage consists of a number of individual cyclones (Fig. 6-23 and 6.24). The solid content separated out in the cyclone leaves the cyclone at the lower end and passes to an oscillating dewaterer. (4). The suspension stream with the remaining content of fines leaves the cyclone at the upper end, and this flow from the first cyclone stage then passes into a regulating basin and is then fed together with the liquid that has passed the oscillating dewaterer into the second cyclone stage (3). The second cyclone stage and the regulating basin form a closed circuit. After separation, sufficient sand has now been removed from the suspension and it contains little silt and flows into a holding basin and is pumped from there back down to the shield. If required, additional centrifuges (5) can be connected to the storage basin to separate the fine silt fraction. The properties of the suspension in the storage basin are regularly monitored. If required, the suspension is refreshed by the addition of new suspension from the bentonite preparation plant or completely exchanged with fresh suspension.

The cut-off defines the results of separation in the individual separation stages. Fig. 6-22 shows examples of achievable cut-offs for a widely graded grain mixture.

The functional principle and degree of efficiency of the individual components of a separation plant are described below based on the layout shown in Fig. 6-21.

Coarse sieve (1). This separates the gravel fraction and the non-dispersed clumps of spoil down to a diameter of 4 mm. The dimensioning of the sieve surface, the form and the shaking frequency should be designed to prevent the blocking of the openings by sticky clay chips. The surface of the sieve should be made in many steps. Under good conditions, 40 to 50 % of the solids can be separated from the flow in the circuit here.

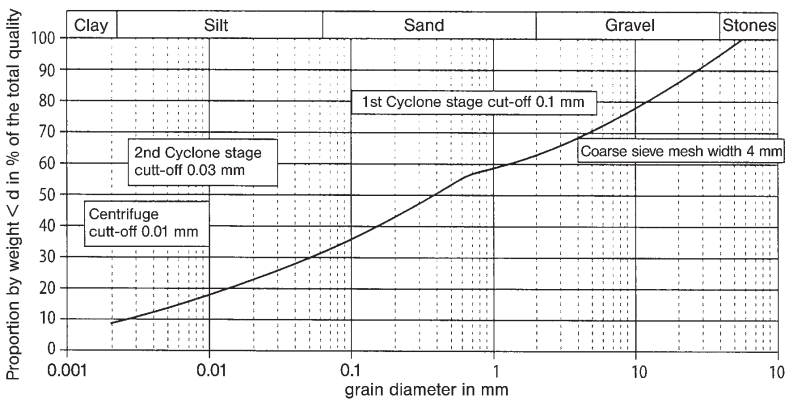


Figure 6-22 Diagram of possible cut-offs for a widely graded soil

First cyclone stage (hydrocyclone) (2). The soil fractions, which pass the coarse sieve, are fed to the individual cyclones (Fig. 6-23). In the upper cylindrical part of the cyclone, the suspension for separation is fed in at high speed at a tangent; it than moves on upward spiral tracks to the lower nozzle of the conical lower part of the cyclone. The centrifugal forces are dependent on grain size, which results in a separation of the suspension. While the larger and thus heavier fractions leave the cyclone through the lower connection, the suspension with enriched fines content still has sufficient kinetic energy and climbs inside the cyclone in a swirling flow in the opposite direction. The cleansed suspension then leaves the cyclone through the upper connection and can be reused or in two-stage systems fed into the second stage.

The cut-off of hydrocyclones is about 0.1 mm (Fig. 6-22).

Second cyclone stage (multicyclone) (3). The second cyclone stage consists of multicyclones with a smaller diameter that hydrocyclones and thus higher boost pressure and higher radial acceleration (Fig. 6-24). The cut-off can in this way be reduced to the coarse silt range (0.03 mm). The throughput of a multicyclone is smaller due to its smaller diameter, so a much larger number of cyclones is required compared to hydrocyclones.

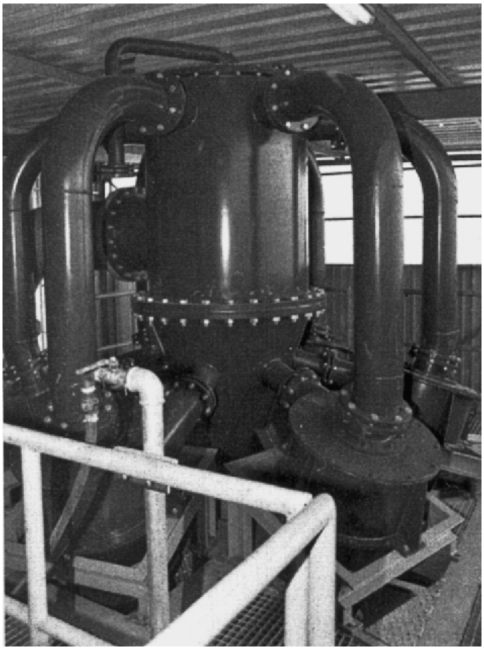


Figure 6-23 Hydrocyclone, Heinenoord Tunnel.

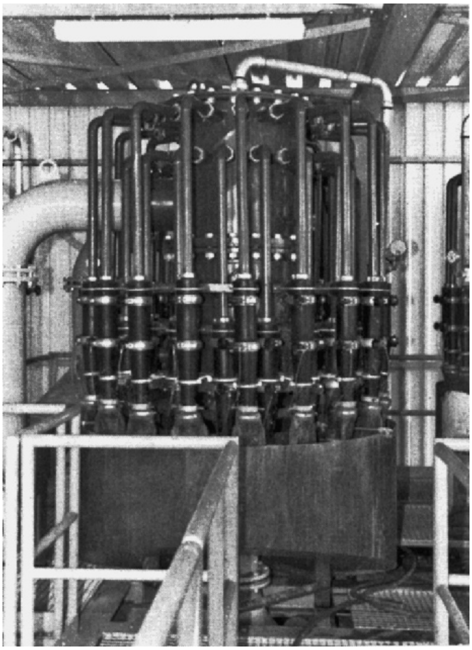


Figure 6-24 Multicyclone, Heinenoord Tunnel

Oscillating dewaterer (4). The lower outlet (precipitated material) of the two cyclone stages is dewatered in a common oscillating dewaterer. This device consists of oscillating split sieve mats, which throw out the forming cake of solid at the upper edge. The soil can be dewatered down to the fine sand limit. Increasing content of fine and very fine grains quickly blocks the soil filter as it forms and the dewaterer overflows. If there is no coarse grain structure above the approximately 0.2 mm wide slots, then the sieve mat cannot retain any solid material and the soil remains in the circuit [244].

Centrifuges (5). The performance of the centrifuges determines the capacity of the separating plant in very fine-grained soil.

The centrifuges have the task of separating as much solid material as possible from the overflow of the cyclones. If the centrifuges are undersized, either the advance rate has to be reduced or the overloaded suspension has to be expensively transported from the regulating or holding basins in skip wagons and disposed of. The separation performance of centrifuges is usually about 0.005 mm, but in practice a centrifuge should be set so that the bentonite (0.005 to 0.01 mm) can be returned to the holding tank and thus remain in the transport circuit [244].

Sieve band presses. Parallel to the centrifuges, the material retained in the hydrocyclones can be thickened with sieve band presses. Continuously operated band filters heave proved successful in practice.

6.3.7 Face with earth pressure support

The earth pressure shield or EPB was developed in Japan in the 1970s as a further development of the blind shields used in cohesive soil with pronounced plastic properties (Fig. 6-25).

A blind shield does not mechanically excavate the soil but exploits the viscosity of the material to be excavated by squeezing the soil through an opening, which is adjustable with a shutter, in the front pressure wall of the shield.

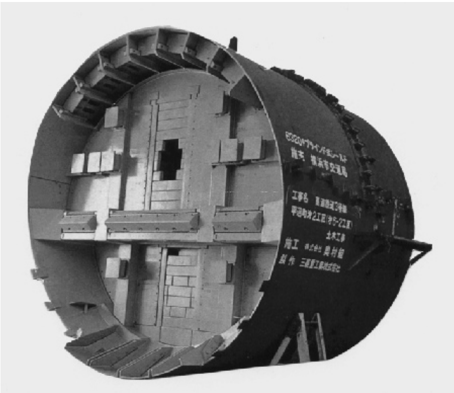


Figure 6-25 Blind shield with two chambers, external diameter 6.32 m; Mitsubishi

6.3.7.1 Functional principle

When a shield machine is used in unstable soil containing water, instability of the face must be avoided by applying a support pressure. An earth pressure shield, in contrast to other shield tunnelling processes, does not need an introduced support medium (compressed air, suspension) because the soil excavated by the cutting wheel acts as support medium (Fig. 6-26).

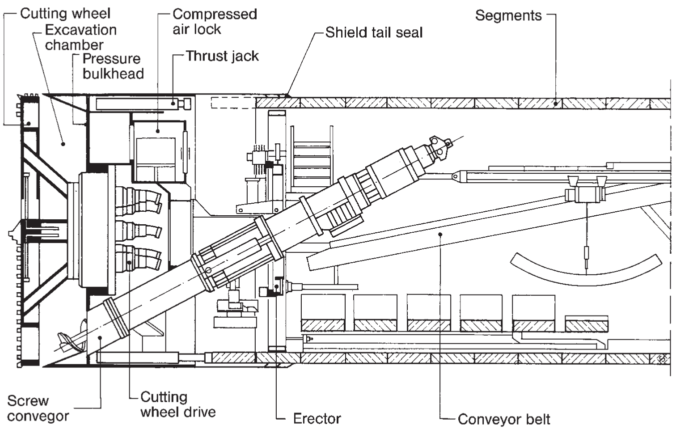


Figure 6-26 Section of through earth pressure shield; Metro Taipei, Tamshui-Linie, contract C201 A, 1992; Herrenknecht AG

Soil is cut from the face by the tools on the cutting wheel and does not fall into the excavation chamber as with a slurry shield but is squeezed through the openings in the cutting wheel into the excavation chamber where it mixes with the already remoulded earth mud.

The thrust from the thrust cylinders is transferred to the earth slurry through the pressure bulkhead and prevents uncontrolled penetration of the earth from the face into the excavation chamber. Equilibrium is reached when the earth mud in the excavation chamber can no longer be compacted by the prevailing ground and water pressure. If the support pressure of the earth mud is increased above this equilibrium state, it causes further compaction of the earth mud in the excavation chamber and the soil in front of the face and can even cause heaving of the ground surface in front of the shield. If the earth mud pressure falls too low, the soil in front of the face can penetrate into the excavation chamber and cause subsidence at the surface.

The material is removed from the excavation chamber by a screw conveyor. The operation of the screw conveyor must be adjusted to suit the thrust of the shield in order to avoid even short-term reduction of the earth pressure in the excavation chamber and thus subsidence at the surface. Further transport out of the tunnel can either be as a solid (transport conveyor, rail, trucks) or also as hydraulic transport with the addition of a liquid and the provision of a slurry pump.

6.3.7.2 Scope of application and soil conditioning process

In order that the soil cut from the face can be used as support medium, it has to fulfil the following requirements.

- Good plastic deformability.
- Pasty consistency.
- Low internal friction.
- Low water permeability.
- Good elasticity by being compressible.

Good plastic deformation properties and a pasty consistency ensure that the support pressure is as evenly distributed across the face as possible, a continuous flow of material to the screw conveyor inlet and that blockages are avoided in areas with little pressure gradient.

Fig. 6-27 shows a flow diagram for the flowing movement of the soil into the excavation chamber and through the screw conveyor. The diagram shows flow lines and the pressure lines at right angles (equipotential lines). In the crown, the pressure gradient is less, resulting in an increased risk of sticking.

The internal friction of the soil should be as low as possible to minimise wear and energy consumption. A low permeability is necessary to transfer the material to the transport conveyor at the outlet of the screw conveyor without an air lock. Good compressibility simplifies the control of support pressure by regulating the screw conveyor.

If the natural soil does not have the required properties, then it has to be conditioned. The conditioning process is selected to suit the type of soil encountered and is thus dependent on the parameters grading curve, water content w (%), liquid limit w_L (%), plasticity index (Ip) and liquidity index (Ic) [248]. These parameters can be influenced by the addition of:

- Water.
- Bentonite, clay or polymer suspensions.
- Tenside or polymer foams.

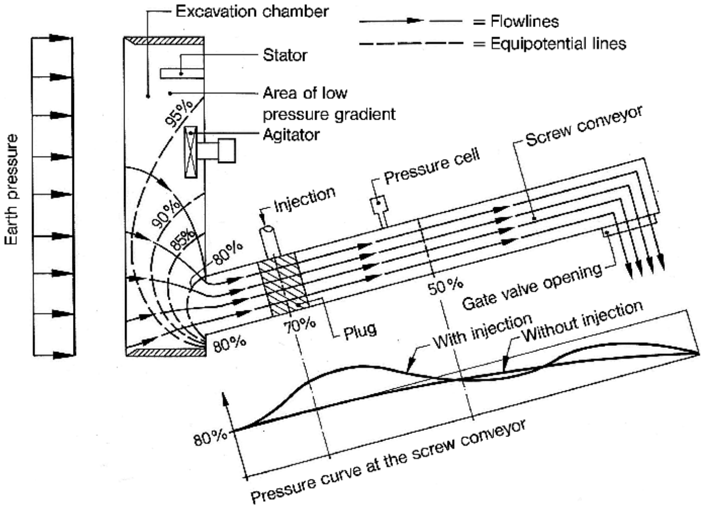


Figure 6-27 Flow diagram for the flow movement of the soil into the excavation chamber [171], [248]

The selection and planning of a conditioning process has to determine the estimated percentage addition of the additive. The initial condition of the soil should if possible remain unaltered; the consistency of the soil has to ensure further transport to the landfill site without too many operational problems and at reasonable cost. When a high percentage of conditioner is added, the separation problems are transferred to the landfill site. The conditioning of the soil should ideally take place directly as the soil is excavated at the face in order to hinder sticking of the closed cutting wheel with small openings.

Optimal conditions for the use of an earth pressure shield are offered by clayey-silty and silty-sandy soils. Depending on the initial state of the soil to be excavated, no water may have to be added or very little. The provision of agitators and kneading tools in the excavation chamber means that even highly cohesive soils can be remoulded to the required consistency by their mechanical action.

With increasing sand content, the addition of water alone is no longer effective. There is a reduction of internal friction and a danger that the remoulded earth slurry separates. The increased water permeability also makes sealing of the screw conveyor difficult. The absence of fines content has to be supplemented with the addition of clay or bentonite suspension. Pore water can be bound by the swelling properties of the suspension and the excavated material is converted into a plastic earth slurry with good flowing properties and reduced permeability. Good results have been obtained through the addition of polymer suspensions with good water-absorbent properties [246].

The entire application spectrum of earth pressure shields is shown in Fig. 6-28. In zone I, above a widely graded borderline (grading curve 1) with a fines content of 30 %, there is practically no limit to the scope of application. These are mostly impermeable soils with a consistency determined by their water content. It is normally possible to work without support pressure due to the solid consistency (Ic > 1), high cohesion and low water permeability of the soil. If however it is necessary to provide face support, then the consistency of the soil should be soft (Ic = 0.4 to 0.75). Suitable conditioners would be, depending on the mineralogical composition of the soil, water, low-viscosity suspensions (bentonite, polymer) or also foams.

Zone	Preconditions	Conditioning
I	Ic support medium = 0.4...0.75	Water Clay and polymer suspension Tenside foams
II	k < 10E–05m/s; waterpressure < 2 bar	Clay and polymer suspensions Polymer foams
III	k < 10E–04m/s; no ground water pressure	High-density slurries High-molecular polymer suspension: Polymer foams

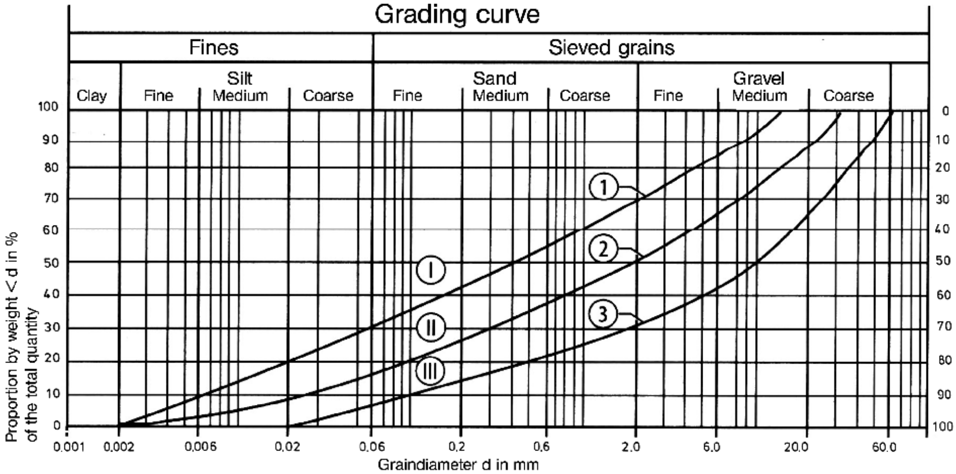


Figure 6-28 Scope of application of earth pressure shields depending on the soil properties [246], [248]

Below the borderline, in zones II and III, the water permeability and the internal friction of the soils increase strongly. The limit of practical application is set by the permeability coefficient k and the prevailing groundwater pressure. For practical application, the water permeability coefficient should not be less than $k = 10^{-5}$ m/s with a pressure of maximum 2 bar [171]. In Fig. 6-28, this zone II is above grading curve 2.

In zone III between grading curves 2 and 3, earth pressure shields should no longer be used under groundwater pressure. Below grading curve 3, the permeability of the soil is too high and the use of conditioning agents is ineffective as they flow away without hindrance and it is no longer possible to achieve a support pressure.

The diameter and number of stones also has to be limited. In contrast to Hydroshields, no stone crushers can be provided in the excavation chamber and the screw conveyor is thus easily damaged. The only suitable conditioning agents in zone III are clay suspensions (high density slurry) or polymer foams. If the content of conditioning agent exceeds 40 to 45 % of the excavated volume, then the consistency is normally liquid and material transport is no longer possible on a conveyor belt and has to be a hydraulic system.

6.3.7.3 Use of foam with earth pressure shields

Soil conditioning with foam has proved successful in earth pressure shield tunnelling and offers advantages compared to conditioning with conditioning agents in suspension form. Of particular benefit for this process is the displacement of pore water at the face combined with simultaneous, if only temporary, sealing [248], [249]. It has also been shown that fine air bubbles considerably reduce the permeability and internal friction of sandy soils. Nonetheless, there are numerous critical situations that can rapidly bring a conventional earth pressure shield to the limits of the process. The cause normally derives from the control of support pressure. Conventional regulation of the support pressure by adjusting the screw conveyor rotation speed and the thrust cylinder force is particularly problematic in coarse-grained soils. Due to the high bulk modulus, slight changes of volume can cause considerable variation of the support pressure in the excavation chamber [245].

The required compressibility of the support medium can be greatly increased by injecting foam. The latest development tendencies for earth pressure shields have the aim of using the system for producing and injecting foam to actively control the support pressure (similarly to the compressed air cushion of a slurry shield) (Fig. 6-29), with the screw conveyor only being considered a secondary element. The foam is injected both through the cutting wheel and also through the stators mounted on the pressure bulkhead. In front of the actual injection points are the pressure sensors for the monitoring of the injection pressure p_i. The quantity injected is calculated by a PC dependent on soil conditions, advance rate and measured support pressure p_{ST} , and dosed in regulated quantities.

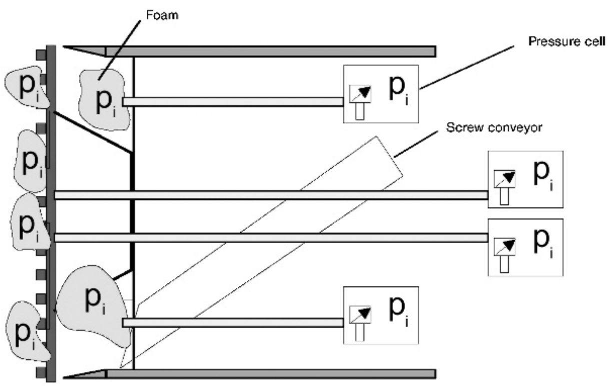


Figure 6-29 Foam injection in an earth pressure shield [245]

Considering the process, active support pressure control leads to the following improvements:

- The soil is kept in constant motion by the permanently flowing stream of foam and pressed towards the screw conveyor (Fig. 6-27).
- The support pressure at the pressure bulkhead approximately corresponds to that acting on the face and can be actively controlled through the foam injection.
- The bulk modulus of the soil is drastically reduced by soil conditioning with foam. The support medium then behaves like a spring, so that volume fluctuations in the excavation chamber cause much smaller fluctuations of support pressure.

6.3.8 Blade tunnelling and blade shields

According to DAUB, the blade shield represents a special form of tunnelling machine. The shield skin is split into blades, which can be advanced individually or "walked". The soil can be excavated by a roadheader, cutting wheel or excavator. One advantage of the blade shield is that it can drive non-circular profiles, for example a horseshoe profile.

Blade tunnelling. Blade tunnelling resulted from the basic idea of developing a mobile steel formwork unit that could fulfil the requirements for temporary support. The final lining is constructed of in-situ concrete within the protection of the blade tail (Fig. 6-30). The basic element is the blade skin, which is supported by two or three steel arches. The advancing blades are 12 to 18 cm wide and are guided in yokes. The advance is normally in steps of 50 to 100 cm driven by hydraulic cylinders. Then the last arch is removed and erected at the face. For the construction of the formwork, it is important that the weight of the fresh concrete and the ground pressure act on the formwork. Blade tunnelling is possible with all usual profiles types, also for top headings.

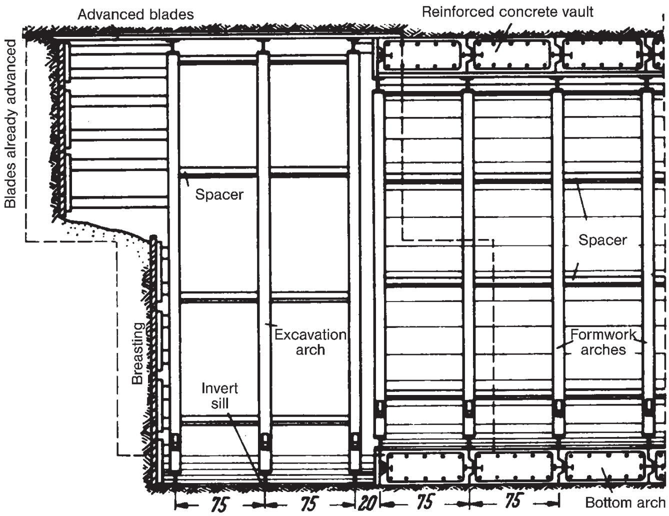


Figure 6-30 Principle of blade tunnelling [251]

Blade shields. The first blade shields were constructed by Westfalia Lünen according to the blade tunnelling principle. These made use of the advantages of advancing blades without having to push against a rigid segment ring to advance further [42], [295]. While in conventional shield tunnelling the shield is pushed forward as a unit against an abutment behind (the segment lining, which has to be designed for this loading case), the blades of a blade shield are supported on a support frame and one, two or a group are advanced hydraulically (Figs. 6-31 and 6.32). The shield thrust is resisted by friction between the surrounding rock mass and the remaining blades and perhaps a spur at the back for additional assistance if required. When all the blades have been advanced, the support frame is brought forward with the follower ring.

Simultaneously, the face is excavated in the protection of the shield skin and a lining is installed in the rear part. Intermediate platforms may be provided to mount a hydraulic excavator or other excavation machine, for example a roadheader. The advance and backup parts are mostly articulated to permit driving round curves. A chain conveyor, which reaches to the back of the backup, delivers the excavated material to rail-borne or trackless transport vehicles. Rubber belt conveyors can also be used for transport within the backup.

Blade shields are generally more difficult to steer than closed shields [19], [20], and there is a greater risk of settlement, which can only be kept within limits with extreme care and long experience.

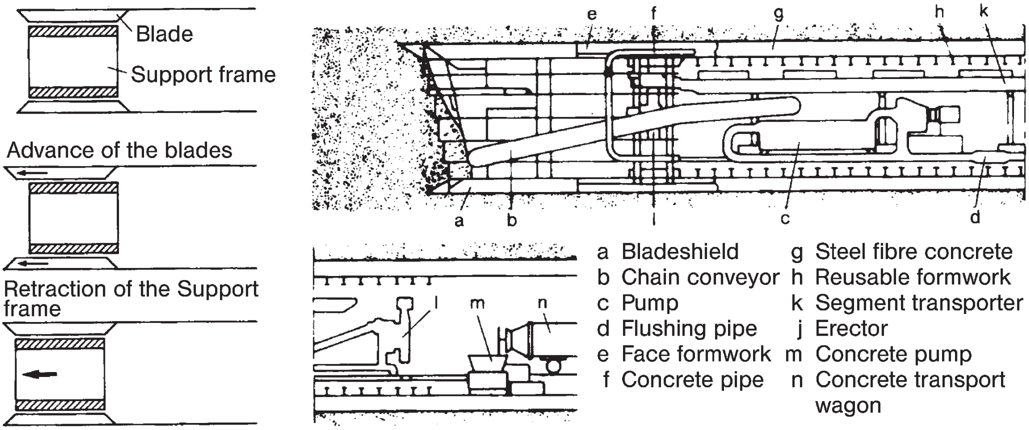


Figure 6-31 Functional principle of a blade shield [19] and blade advance tunnelling with lining of extruded concrete; inner formwork struck and moved forward. Hamburg-Harburg main sewer [198]

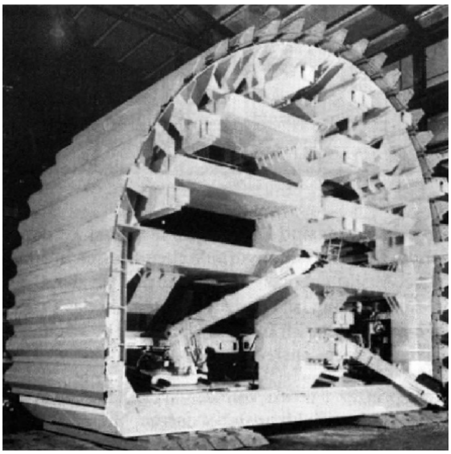


Figure 6-32 Blade shield

Caution is required with the use of a blade shield and compressed air, because the air can escape through the blade joints [197].

Blade shields can be equipped for various methods of lining:

1. Blade shield with lining of in-situ concrete and precast elements: installation of precast elements in the invert and reinforced in-situ concrete in the top heading and sides [347].

2. Blade shield with lining of pumped steel fibre concrete: simplifies the subsequent installation of the lining, as the formwork does not have to be opened to install the reinforcement. The formwork elements can be moved forward or even pulled behind the shield, the stopend is moved forward hydraulically with concreting.
3. Blade shield with support of shotcrete or steel fibre shotcrete: the shotcrete is sprayed onto exposed ground (cohesive soil) after the steel tunnel arch and the mesh have been fixed in the protection of the backup blades. There are no backup blades in the invert in order to be able to set up and adjust the support elements on the exposed ground.

Any loosening of the ground is avoided by prompt and immediate contact between shotcrete and ground. The shotcrete support thus forms an integrated load-bearing element with the surrounding ground, as in the new Austrian tunnelling method. The ring is closed immediately behind the blade shield. This process can be simplified and improved by using steel fibre shotcrete, in which case the erection of the tunnel arches and the fixing of reinforcement can be omitted.

6.3.9 The most important verification calculations

6.3.9.1 Calculation of face stability with slurry and earth pressure support

In order to ensure stability of the face, the pressure that is transferred to the face by the support medium must be in equilibrium with the water and ground pressure (Fig. 6-4).

In order to calculate the earth pressure in the in-situ soil, a plastic limit equilibrium is considered. Depending on the local conditions, linear or zone collapses could occur.

The kinematic calculation model presented below is particularly suitable for coarse-grained soil types with suspension support and has been confirmed in an extensive parameter study [4]. It is based on a failure body model [141], which assumes a linear fracture (Fig. 6-33).

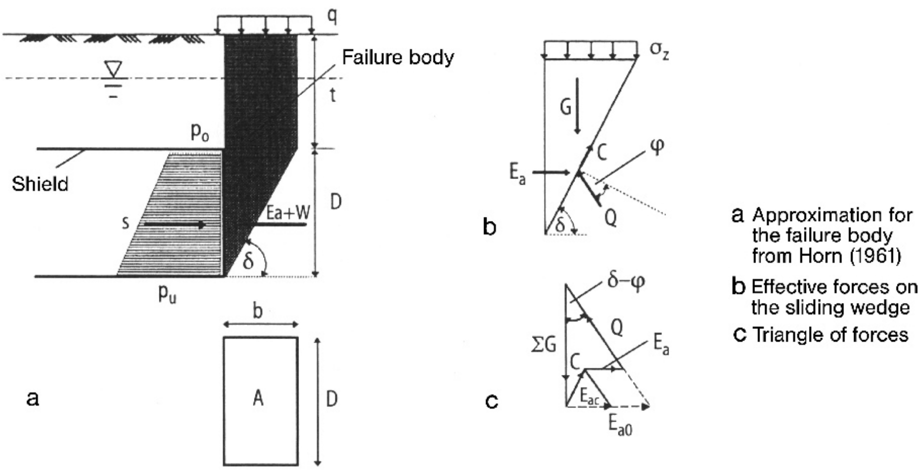


Figure 6-33 Calculation of the support pressure in the excavation chamber by investigating a sliding body: equilibrium consideration to determine the support pressure for linear kinematic failure [248]

Calculation model. In order to calculate the active earth pressure, sliding bodies are investigated, which are assumed to move into the excavation chamber along a sliding joint. This procedure is described as the "kinematic method"; it is based on the Coulomb earth pressure theory with consideration of a sliding body. The maximum resultant force of earth pressure E_a, determined from the worst case sliding joint, plus the resultant of water pressure W of the water pressure curve shown in Fig. 6-4 must correspond to the integral of the support pressure p over the area of the face (Fig. 6-33) or exceed it. The water pressure in permeable soil is assumed to equal the hydrostatic head in order to be on the safe side.

As a good approximation, the calculation of earth pressure for an earth pressure supported shield drive can be performed by considering an equilibrium state of the sliding body shown in Fig. 6-33.

The circular face area is approximated by a square with sides corresponding to the tunnel diameter D. The sliding body in front of the face consists of a sliding wedge, which is loaded vertically by a vertical prism of soil reaching up to the surface. The self-weight of the soil prism above deep tunnels ($t > 2D$) can be replaced by the reduced vertical load σ_z determined by the silo theory.

$$\sigma_z = \gamma \cdot r_0 \cdot \frac{1}{2 \cdot K_S \cdot \tan\varphi} \cdot (1 - e^{-a}) + q \cdot e^{-a}$$

$$a = \frac{2 \cdot K_S \cdot \tan\varphi * t}{r_0}$$

K_s Silo coefficient, q surcharge in kN/m^2, r_0 silo radius in m, t overburden in m, γ weight of the soil in kN/m^3 and φ friction angle of the soil in degrees.

The silo coefficient K_s can be approximately calculated from:

$$K_S = \frac{1 - \sin^2\varphi}{1 + \sin^2\varphi}$$

For the rectangular plan of the silo body with side lengths D and b (Fig. 6-33), instead of the silo radius r_0 the equivalent radius

$$r_e = \frac{D \cdot b}{D + b}$$

is used.

Fig. 6-33 shows the forces acting on the sliding body. The earth pressure resultant force can be determined graphically from the triangle of forces or calculated from the following equation:

$$E_a = E_{a0} - E_{ac}$$

with $E_{a0} = tan\ (\vartheta - \varphi) \cdot (G + \sigma_z \cdot A)$

with $E_{a0} = \dfrac{c \cdot D^2 \cdot sin\,(90° - \varphi)}{sin\,\vartheta \cdot sin\,(90° - \vartheta + \varphi)}$

A plan area of the silo body in m², *c* cohesion in kN/m², *G* weight force of the sliding wedge in kN.

In this case E_{a0} is the resultant of earth pressure without consideration of cohesion ($c = 0$). If cohesion is considered, the earth pressure resultant E_{a0} opposes the force vector E_{ac} (Fig. 6-33). When the ground is stratified, the shear parameters are determined through the thicknesses of the relevant strata. The cohesion can be taken into account in a simplified form by using an equivalent friction angle.

The sliding angle ϑ is varied until the earth pressure resultant has an extreme value. The reduction of earth pressure due to the wall shear forces *T* acting on the sides according to the "shoulder theory" [72] or the procedure of Prater [193] are not considered in the equation. Instructions about a bilinear consideration of the normal forces acting sideways are given in [72]. Three-dimensional calculation models can also be used.

With the introduction of the conversion factor $\pi/4$ for the consideration of the difference of area between the square face of the sliding wedge and the circular tunnel face, the required support force *S* is obtained from:

$$S = W + \eta\, E_a \cdot \frac{\pi}{4}$$

Where *W* is the resultant of water pressure in kN. The safety factor η is defined according to the requirements of a specific project and is laid down in the specification. For diaphragm walls, DIN 4126 gives $1.1 < \eta < 1.3$ depending on the state of loading in the immediate vicinity of the wall. For mechanised shield tunnelling, higher safety factors are often specified in practice; for example safety factors of $\eta = 1.05$ on the water pressure resultant and $\eta = 1.5$ on the earth pressure, although examples of the introduction of a uniform safety factor for both load components are also known. The required support pressure p in the excavation chamber is finally calculated from the equation:

$$S = \frac{\pi \cdot D^2}{8}\,(p_o - p_u)$$

with p_o support pressure at the shield crown and p_u support pressure in the shield invert (both in kN/m²).

6.3.9.2 Calculation of safety against breakup and blowout

The calculation procedure described in section 6.3.9.1 serves to verify horizontal equilibrium. Particularly for tunnels with shallow cover and high groundwater pressure, vertical equilibrium should also be checked to verify that the required or maximum permissible support pressure at the crown will not cause breakup (geotechnical failure in a vertical direction) in case of suspension support or a blowout in case of compressed air support.

Safety against heave is generally guaranteed when the vertical surcharge above the tunnel crown exceeds the support pressure p acting at that location (Fig. 6-34). When compressed air support is used, the support pressure normally has to be set to the support pressure required in the tunnel invert, or in case of partial lowering (suspension support) to suspension level, which results in a lack of equilibrium due to the negligible weight of the compressed air in the crown compared to the trapezoidally acting water pressure (or earth plus water pressure if there is a filter cake). Due to the positive air pressure p_L in the crown, which in this case exceeds the calculated required suspension pressure, two events could occur if cover is shallow, leading in an extreme case to a blowout:

- If the face is unsealed, air flows continuously to the surface and dries the soil while enlarging the passages.
- If there is a membrane (filter cake) in the crown, and if the soil surcharge is insufficient, there is a danger of geotechnical failure, which must develop into a blowout.

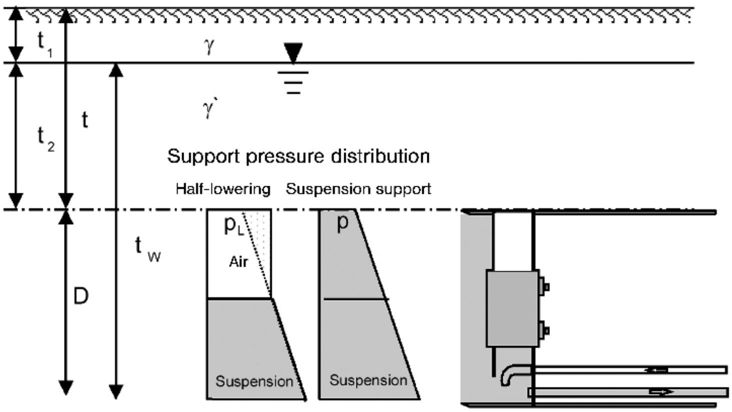


Figure 6-34 Verification of vertical equilibrium

For this reason, the suspension or air pressure acting in the crown should not exceed the surcharge acting from soil and water. The assumption of soil surcharge is however only permissible if the soil has very low permeability or sealing is guaranteed by a filter cake.

As illustrated in Fig. 6-34, safety against geotechnical failure η_{Auf} can be verified using the following relationship:

$$\eta_{Auf} = \frac{\gamma \cdot t_1 + \gamma' \cdot t_2 + \gamma_W \cdot (t_W - D)}{p_0} = 1{,}1$$

with D shield diameter in m; t_1, t_2 overburden above or below the groundwater table in m; t_W water level above invert in m; p_0 support pressure in the shield crown in kN/m^2, γ, γ', γ_W weight of the soil, the soil under hydrostatic uplift, and the water respectively in kN/m^3.

6.3.9.3 Calculation of thrust force

The calculation of the necessary thrust force is particularly important part of the design of a tunnelling shield [244]. If the thrust force is inadequate or if unforeseen resistances are encountered, this could lead in the worst case to expensive rebuilding underground and should be avoided by careful advance calculation and also by evaluating and analysing empirical data from completed projects.

Table 6-1 Friction coefficient μ between shield skin (steel) and soil type [132]

Soil type	Friction coefficient μ [-]
gravel	0.55
sand	0.45
loam, marl	0.35
silt	0.30
clay	0.20

Resistance to advance from shield skin friction forces. The radial and horizontal loading from overburden, buildings and imposed loads and the self-weight of the shield result in friction forces around the shield skin, which have to be overcome by the thrust cylinder forces. These friction forces can be reduced by tapering the shield, overcutting by the shield cutting shoe or by lubrication (e.g. bentonite).

According to the composition and soil type of the ground to be tunnelled through [132], the friction coefficients μ [-] (Table 1) differ and result in an approximate calculated friction force W_M on the shield skin:

$$W_M = \mu \cdot [2\pi \cdot r \cdot L\,(p_v - p_h) \cdot 0{,}5 - G_s]$$

with $p_{V\ ges} = p_v + p_{Beb} + p_{Verk}$

with $p_h = K_0 \times p_{v\ ges}$

D shield diameter in m; G_S self-weight of the shield in kN; K_0 earth pressure coefficient; L length of the shield skin in m; p_{Beb} vertical loading from buildings in kN/m^2, p_h, p_v, p_{vges} horizontal, vertical or total vertical loading respectively in kN/m^2; p_{verk} vertical loading from imposed load in kN/m^2.

In sandy and gravelly soils, lubrication of the shield skin with a bentonite or other clay suspension can reduce the friction coefficient μ by 0.1 to 0.2. For the calculation of the vertical surcharge, the reduced overburden depth h‘ resulting from arch or silo effect can be assumed.

Resistance to advance at the shield blade. The shield is forced through the ground with a blade or a cutting shoe. The peak resistance p_{Sch} [kN/m^2] is given depending on the soil type (Table 2) [132].

The stated peak resistances are independent of the actual overburden and other loading assumptions. The earth pressure coefficient K is normally higher than the passive earth pressure coefficient K_p

$$p_{Sch} > K_p \cdot p_{v\ ges} \quad \text{in } kN/m^2$$

According to [132], this gives for the unreduced blade resistance at the perimeter of the shield blade W_{Sch}:

$$W_{Sch} = \pi \cdot D \cdot p_{Sch} \cdot t$$

with D shield diameter in m, p_{Sch} peak resistance in kN/m^2 and t blade wall thickness in m.

When a critical value p_{Sch} is reached, a "local" ground failure occurs at the shield blade so that the shield can penetrate the ground. Intentional overcutting (cutting wheel, rock cutterhead, extending gauge cutters) can reduce or nullify the blade resistance.

Table 6-2 Peak resistance p_{Sch} of the ground according to soil type [132]

Soil type	Peak resistance p_{Sch} [kN/m^2]
soil similar to rock	12,000
gravel	7,000
sand, densely consolidated	6,000
sand, medium consolidated	4,000
sand, loosely consolidated	2,000
marl	3,000
Tertiary clay	1,000
silt, Quaternary clay	400

Resistance to advance at the face due to platforms and excavation tools. The presence of platforms, for example in hand shields, has a similar effect to shield blades and leads to resistance to the advance of the shield. The loads should be assumed to be similar to shield blades.

The pressing forces on the excavation tools to loosen the soil are dependent on the type of soil encountered.

The load assumption for resistance W_{BA} caused by the excavation tools while excavating can be determined as follows as an approximation in soil:

$$W_{BA} = A_{BA} \cdot K \cdot p_{vges}$$

with A_{BA} pressure surface of the excavation tools in m^2, K earth pressure coefficient ($K_a < K < K_p$), K_a, K_p active or passive earth pressure coefficient respectively, p_{vges} total vertical loading in kN/m^2. Different assumptions should be made for the pressing force required on cutter discs in rock.

Resistance to advance with slurry, earth pressure and compressed air support. The resistances to advance from earth and water pressures or from the resultant support pressure also have to be applied by the thrust cylinders.

As shown in Fig. 6-35, the resultant support force W_{ST} is the integral of the support pressure over the area of the face A_0.

$$W_{ST} = W_E - W_W$$

$$W_W = \frac{A_0 \cdot (p_{Wcrown} - p_{Winvert})}{2}$$

$$W_{ST} = \frac{A_0 \cdot (p_o - p_u)}{2}$$

A_0 face area in m^2;

p_o, p_u support pressure in the shield crown or the shield invert in kN/m^2;

p_{wcrown}, $p_{winvert}$ water pressure in the shield crown or invert respectively in kN/m^2;

W_E resultant earth resistance from the fracture body investigation ($W_E = E_a$) in kN;

W_{ST} resistance from face support in kN;

W_W resultant water pressure resistance in kN. When compressed air is used, the support pressure should be assumed equal over the entire face.

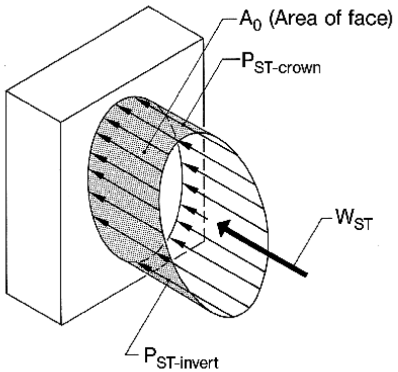


Figure 6-35 Support force W_{ST} from the integral of the support pressure over the face area

Resistance to advance from steering the shield. A shield is driven around a curve be differentiated activation or extension of the thrust cylinders. The thrust cylinders can be controlled individually in smaller shields but in larger shields, they are combined into groups.

Small radius curves cause constraint effects, which are larger the longer the shield is in relation to its diameter. Intentional overcutting, tapered construction of the shield and lubrication by injecting bentonite through the skin can reduce these effects.

The provision of an articulated joint in the middle of the shield can also reduce constraint effects.

The remaining resistances to advance from steering the shield should be derived from experience.

Summary. The thrust cylinders should be designed to provide the sum of individual resistances ΣW plus a safety margin.

$P_V = \Sigma W - safety\ margin$

$\Sigma W = W_M - W_{Sch} - W_{BA} - W_{ST}$

P_v max. thrust cylinder force in kN; W_{BA} resistances of the cutting tools in kN; W_M friction forces in kN; W_{Sch} shield blade resistance in kN; W_{ST} resistance from face support in kN.

The decisive case is the least favourable combination of individual resistances. The safety margin is a rule of thumb and covers all the forces that cannot be calculated, as listed below:

- Towing tension from backup.
- Friction force on the shield tail seal on the tunnel lining.
- Increased resistance on encountering obstacles.
- Increased skin friction and increased blade resistance passing through grouted zones.
- Increased skin friction through ground swelling pressures.
- Increased skin friction from curves and steering.

6.3.9.4 Determination of the air demand for compressed air support

For a rough estimation of the air demand for compressed air shield drives under open water and thus the installed capacity (related to the air intake), the following formula was published as long ago as 1922 by Hewett and Johannesson [134]:

$Q_L = (3{,}66\ldots7{,}32) \cdot D^2$

With D shield diameter in m und Q_l intake air quantity in m^3/min; factor 3.66 for normal water-bearing soil (e. g. medium sand), factor 7.32 for very permeable soil (e. g. gravel or sandy gravel)

This formula includes no parameters concerning pressure, ground permeability, tunnel length and leaks, but refers solely to the diameter of the tunnel. Nonetheless, it can be assumed that the parameters are included in the formula. The experience of Hewett and Johannesson still delivers usable results today when the given preconditions apply to a compressed air shield drive. This will mostly not be the case because tunnels are not always under open water but can also be below the groundwater table under a built-up area.

For a more precise and general determination of the air demand (compressor capacity) for compressed air shield tunnelling, the work of Wagner and von Schenck [310] is available (Table 6-3).

Although the theoretical relationships only apply precisely in homogeneous soil, they can be considered valid for non-uniform and stratified ground when an average value of the air permeability coefficient of the soil k_L in accordance with the prevailing permeability is taken. The precondition is, however, that an air flow can occur similar to that in homogeneous soil (Fig. 6-36).

The air permeability coefficient of the soil k_L should ideally be determined in a large-scale trial in the open, rather not just in laboratory tests. The evaluation of compressed air drives carried out in comparable soil conditions can also be helpful. A good approximate calculation is delivered by the equation $k_L \approx 70 \cdot k_W$, with k_w the water permeability coefficient of the soil according to Darcy.

This should be determined through groundwater lowering or pumping tests. It should be noted that the water permeability may be different vertically and horizontally.

Table 6-3 General determination of the air demand for compressed air shield drives from [310]

Air demand $Q_L = n \cdot c \cdot k_L \cdot A_O \cdot q_L \cdot 60 + Q_S$	
$q_L = [(\alpha + \beta_i)/\ \beta_i] \cdot [(p_T/p_a) + 1]$	$q_L = [(t_2 + D)/\ (\beta_i \cdot D)] \cdot [(1 - \alpha)/\ \beta_i] \cdot [(p_T/p_a) + 1]$
A_0	Face area in m^2
c	Correction factor to consider the influence of the spatial air flow field (for an overburden above the tunnel crown of one to two times tunnel diameter: c = 2)
D	Shield diameter in m
n	Component to consider the proportional air escape at the face, at the shield tail and through any leaks
p_a	Atmospheric pressure in kN/m^2
p_T	Excess air pressure in the tunnel in kN/m^2
q_L	pressure gradient and conversion of the compressed air quantity in the extracted air volume
Q_L	Extracted air volume in m^3/min
Q_S	Air demand required for locking in m^3/min
t_2	Overburden below the groundwater table in m
α	Factor to consider the accepted residual water quantity in the tunnel
β_i	Ratio of the overburden depth and tunnel diameter

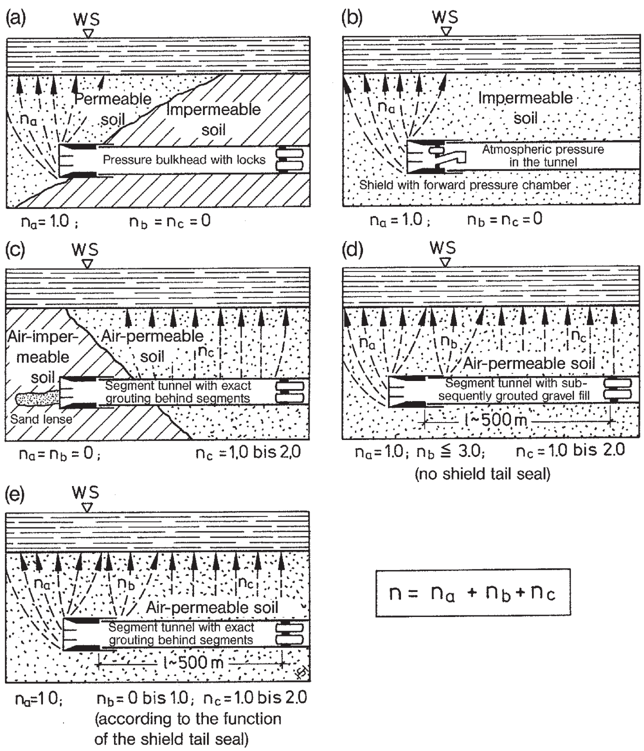


Figure 6-36 Factors for the consideration of the air escape at the face, at the shield tail and resulting from leaks (joints) in the lining [170]

6.4 Tunnel boring machines in hard rock

6.4.1 Categorisation of machines for use in hard rock

Various types of machine can be used to bore tunnels through hard rock. A systematic categorisation of hard rock tunnel boring machines is shown in Fig. 6-37 based on the classification of DAUB:

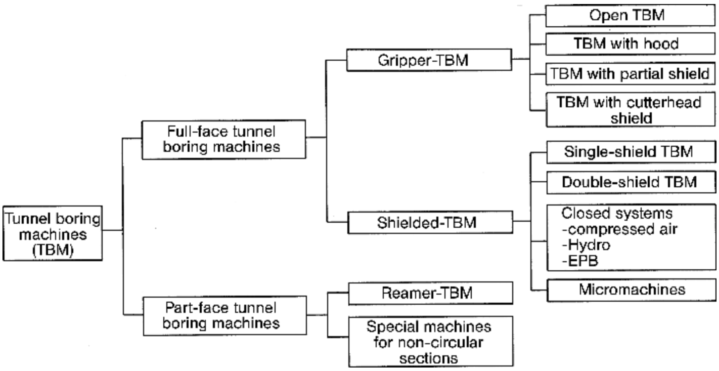


Figure 6-37 Categorisation of tunnel boring machines based on the classification of DAUB [60]

6.4.2 Basic principles

General

Modern developments in tunnel boring machines started with the use of movable cutters, a machine with rolling disc cutters already having been designed and tested in 1857. R. J. Robbins built a new generation of machine in 1956 for the Toronto tunnels, followed only ten years later by German manufacturers like Fried. Krupp GmbH (according to the Wohlmeyer principle from 1958), Mannesmann-Demag Bergwerktechnik, Wirth Maschinen- und Bohrgerätefabrik GmbH [301] and, since the start of the 1980s, Herrenknecht AG. Despite disadvantages when the geological conditions are not completely understood, development will entail increasing use of full-face machines.

For a more detailed reference work about tunnel boring machines, reference can be made to the book "Hardrock Tunnel Boring Machines" [242].

In the terminology used in Germany, the term tunnel boring machine (TBM) describes a machine intended to bore tunnels in hard rock using a circular cutterhead, which is generally fitted with disc cutters. It should be noted that the term TBM often is used in the English-speaking countries synonymously for all mechanised tunnelling machines with full face excavation, i. e. including shield machines.

The disc cutters are used to excavate the rock in front of the machine through the rotation of the cutterhead and the pressing force onto the face. They are moved forward by regripping. While the machine bores, it braces against the sides of the tunnel in order to be able to apply the optimal pressing force onto the face. The cutterhead rotates in one direction or in exceptional cases in alternating directions. The cutting and excavating tools are either fixed to the cutterhead itself or to tool carriers (brackets). The excavated material is picked up by the spokes of the cutterhead and fed through openings to a transport device (conveyor belt, chain conveyor) installed in the backup.

Gripper TBMs are suitable for use in hard rock with medium to long stand-up time. The face must be mostly stable as support is only provided to a limited extent by the cutterhead as the machine advances. When the cutterhead is withdrawn for maintenance or to change the fitted tools, there is no more face support. If support is required, this can sometimes be achieved with additional measures. The scope of application of TBMs in rock strengths of 300 MPa permit the boring of most types of hard rock.

The basic elements of a TBM are the cutterhead, the cutterhead beam with the cutterhead drive motors, the machine frame and the gripping and thrust equipment. This basic construction tows the control and backup functions on one or more backups.

There are four system groups (Fig. 6-38) [24]:

- Boring system.
- Thrust and gripping system.
- Mucking system.
- Support system.

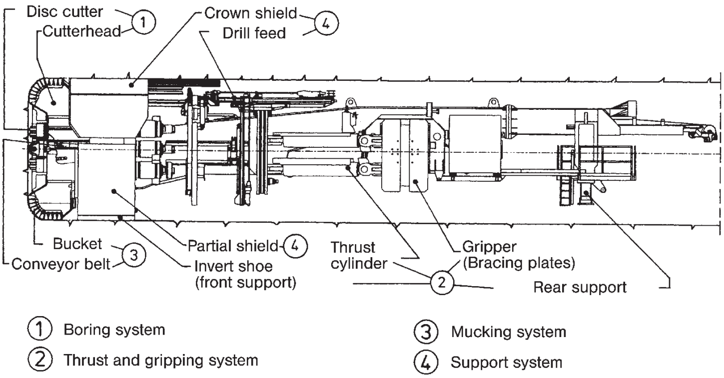


Figure 6-38 System groups of a tunnel boring machine

Fig. 6-39 shows the various machine systems of tunnel boring machines in section, which are briefly described below:

The gripper TBM, also commonly described as an open TBM, is the classic form of tunnel boring machine with a field of application mainly in solid rock with medium to high stand-up time. It can be used most economically when the prevailing rock does not regularly require support with rock bolts, steel arches or even shotcrete.

In order to be able to apply the thrust force to the cutterhead, the machine is braced against the tunnel wall with hydraulically driven bracing shoes, commonly called grippers. Over time, two different bracing systems have been developed, single bracing (Robbins) and double bracing (Wirth and Jarva).

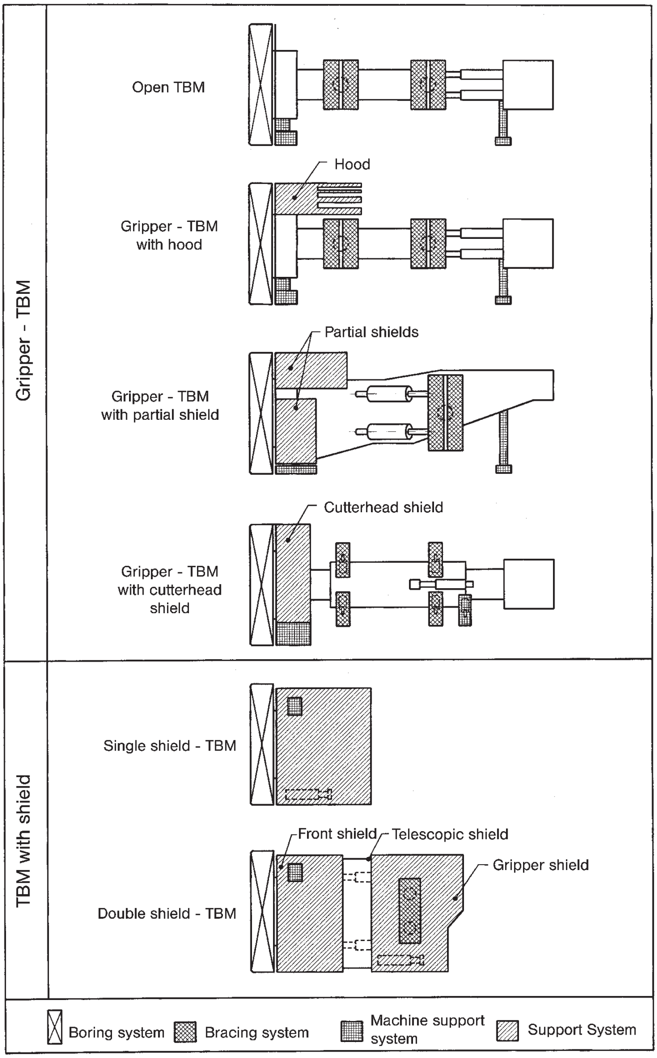


Figure 6-39 Various machine systems of tunnel boring machines in section [242]

Gripper TBMs can be further categorised into open TBM, TBM with hood, with partial shield and with cutterhead shield:

– *Open TBM.* The description open TBM applies to TBMs without any structural protection behind the cutterhead. Such machines are only found in small diameters today.
– *TBM with hood.* The construction of a TBM with hood is the same as an open TBM. If, however, isolated rock or stone falls can be expected during the drive, this type of machine additionally has structural roof elements called hoods behind the cutterhead in order to protect the crew.
– *TBM with partial shield.* Partial shields are expanding cutterhead shields with the same protection function but also serve to support the machine when it is regripped and provide a means of steering while boring. The sides of the cutterhead shield can be expanded radially into the tunnel sides.

– *TBM with cutterhead shield*. The cutterhead shield in this type of TBM serves to protect the crew near the cutterhead. When the machine is regripped, the short shield skin provides support at the front.
– *Single shield TBM.* Single shield TBMs are primarily intended for use in rock with a short stand-up time and in fractured rock. The excavation tools and muck clearance of the cutterhead are not essentially different from those of a gripper TBM. In order to temporarily support the tunnel and to protect the machine and the crew, this type of machine is fitted with a shield skin. The shield skin extends from the cutterhead over the entire machine. The lining is installed in the projection of the shield tail, with support consisting of reinforced concrete segments having become established as the most common system. According to the geology and intended use of the tunnel, the segments are either used as the final lining (single-pass lining) or as temporary support later supplemented by an in-situ concrete inner lining (two-pass lining).

In contrast to a gripper TBM, a single shield TBM is pushed forward against the already installed lining with thrust cylinders.

– *Double shield or telescopic shield TBM.* The double shield or telescopic shield TBM is a variant of the shield TBM. It is capable like the shield TBM of tunnelling through fractured rock with a short stand-up time, but has the following differences compared to the single shield TBM:

The double shield TBM is built in two parts, the front shield and the gripper or main shield. These two parts are connected to each other with a telescoping joint controlled by hydraulic cylinders. The machine can either use the bracing mechanism of the gripper shield to sufficiently brace itself against the tunnel sides or if the rock worsens, thrust its advance against the already installed lining. The front shield can be advanced without affecting the gripper shield, so almost continuous boring is possible almost independent of the installation of the lining. This system is illustrated in Fig. 6-40.

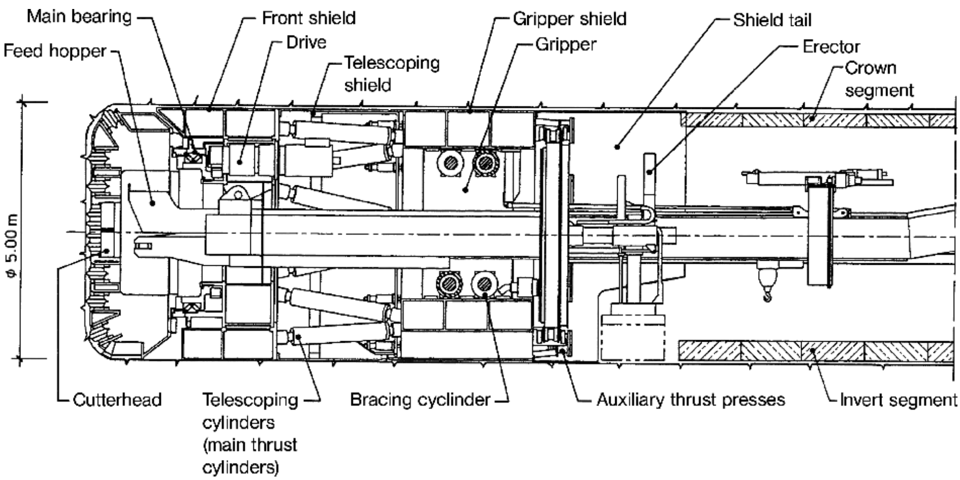


Figure 6-40 System diagram of a double shield TBM [242]

The double shield does however have considerable disadvantages compared to a single shield. When it is used in fractured rock with a high strength, the back shield can become blocked by material penetrating into the telescopic joint, which is then sometimes falsely described as the TBM getting stuck, although blocking and getting stuck are the result of different causes and should actually be differentiated.

The apparent advantage of rapid advance of a double shield TBM only applies if a single-pass segment lining is used, which requires an installation time of about 30 to 40 minutes per ring. If two-pass lining is used, the ring installation time of about 10 to 15 minutes no longer justifies the higher purchase price and the increased time required for repairs.

– *Closed systems.* Closed TBM systems are combined system solutions for work below the groundwater table, with the groundwater being held back by compressed air or face support according to the Hydroshield or EPB principle. These systems have been used both for working in solid rock and in soft ground [244].
– *Micromachines.* These can also be equipped for boring hard rock. The design of the cutterhead is based on the same or similar principles to those of larger machines. Micromachines are fitted with a shield.
– *Enlargement machines.* Enlargement TBMs, which are also called reaming machines, are a special version of the gripper TBM used for tunnels with diameters of more than 8.00 m. A pilot tunnel has to be completed on the centreline of the tunnel for the entire distance first and the reaming machine then enlarges the profile to the final diameter. This reaming can take place in one or more stages. The machine is braced in the pilot tunnel in front of the cutterhead of the enlargement stage. Fig. 6-41 shows an enlargement TBM from the company Wirth.

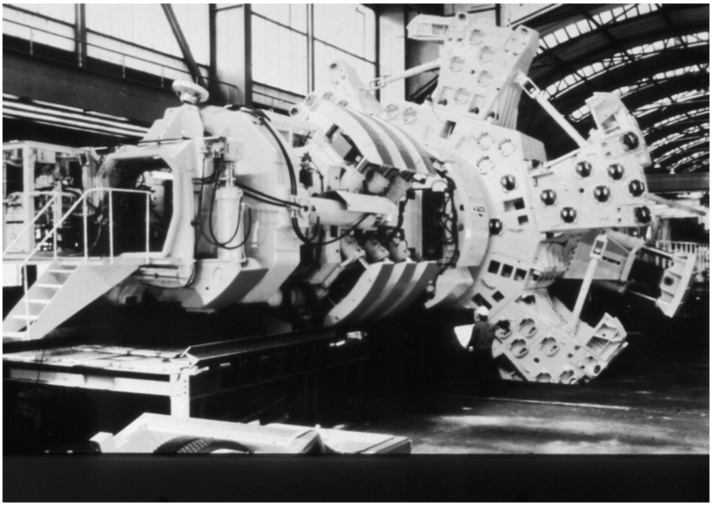


Figure 6-41 Tunnel reaming machine; Wirth [38]

– *Special machines for non-circular sections.* This category covers all machines, which excavate part of the face at a time and are thus capable of producing non-circular profiles. Some examples of special machines developed by various companies are the Mobile Miner (Robbins), the Continuous Miner (Wirth), the Mini-Fullfacer (Atlas Copco) and Japanese developments.

6.4.3 Boring system

The boring system is the most important subsystem of a TBM in determining its performance. It essentially consists of the excavation tools, normally rolling disc cutters (Fig. 6-43), often simply referred to as discs, which are mounted on a cutterhead (Fig. 6-42).

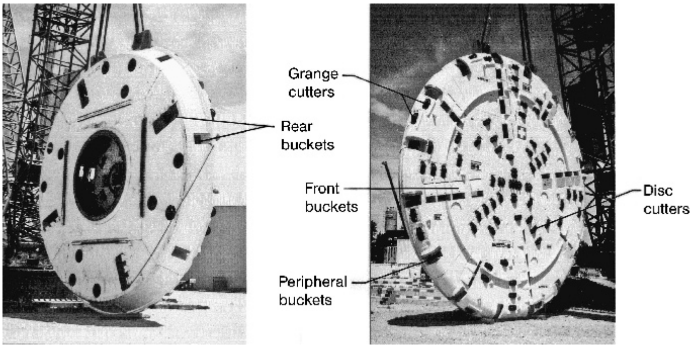


Figure 6-42 Cutterhead of the TBM Zürich – Thalwil with 16 gauge buckets, 24 front buckets with adjustable flaps and 6 rear buckets [242]

The discs are placed so that they can cover the entire face on concentric tracks. The spacing of the cutter tracks and the diameter of the discs are selected according to the type of rock and the possible cuttability. This determines the possible size of the excavated rock chips.

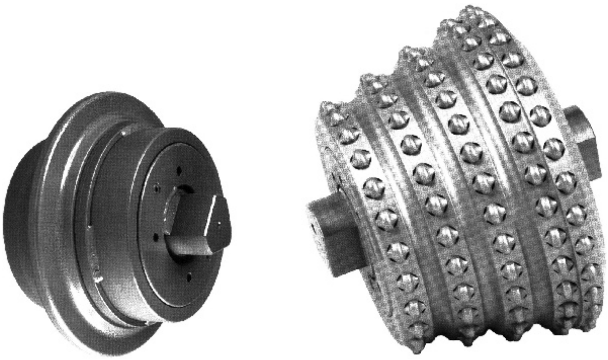


Figure 6-43 Various cutter types for a TBM: roller disc cutter, button cutter (Palmieri) [242]

The rotating cutterhead presses the discs against the face with high pressure, and the discs roll around the tracks in the face. At the cutting edge of the disc, the applied pressure exceeds the compression strength of the rock and grinds it locally, which permits the cutting edge of the disc to penetrate into the rock until applied pressure and rock strength are in equilibrium. This is described as net penetration and the disc cutter creates high localised stress (splitting tension), which leads to the spalling of long flat chips of rock (Fig. 6-44).

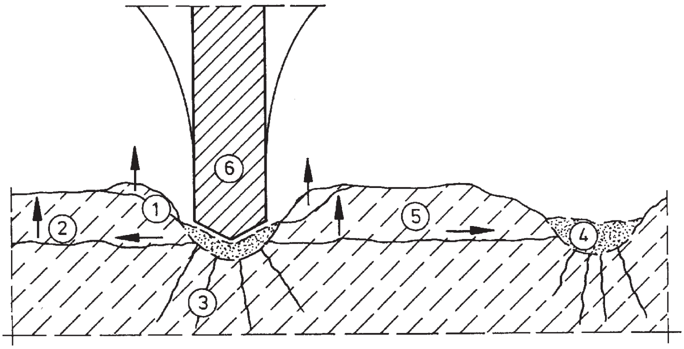


Figure 6-44 Diagram of the cutting process, based on [46], with a disc of constant width

1. Splitting due to tension fracture
2. Shear or splitting tension fracture
3. Radial crack formation under the disc
4. Material flowing out of the disc track
5. Typical shape of a larger chip
6. Disc cutter with almost constant thickness in the cutting area

Each disc only affects a limited area of the rock and this excavation process can thus be regarded as having comparatively little impact on the surrounding rock mass.

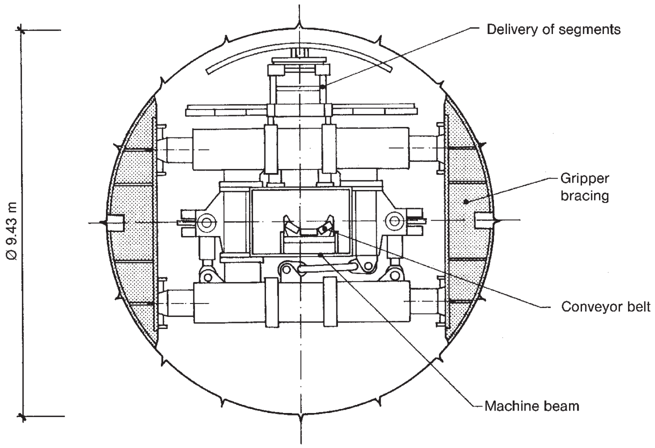
6.4.4 Thrust and bracing system

The thrust and bracing system also influences the performance of a TBM. It is responsible for the application of pressure onto the face and the progress of boring. The cutterhead with its drive unit is driven forward with the required pressure by hydraulic cylinders. The length of the pistons of the thrust cylinders limits the maximum stroke length. Modern TBMs can bore a stroke of up to 2.0 m.

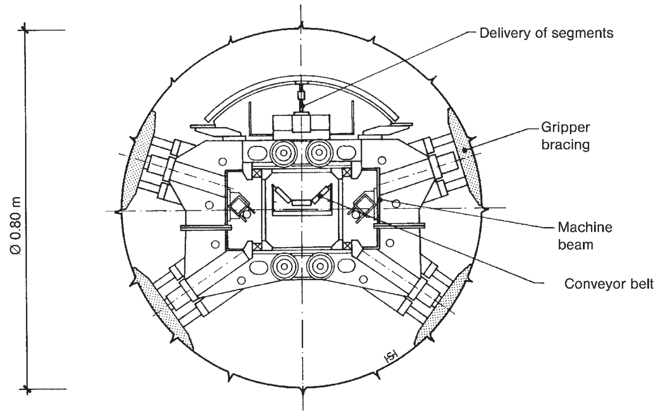
Figure 6-45 Grippers with thrust cylinders, Gripper TBM S-167 (Herrenknecht AG) Lötschberg Base Tunnel, Steg section [242]

The thrust system places a limit on the thrust force that can be provided and also has to transfer the moment resulting from the rotation of the cutterhead. The applied bracing forces are not limited by the forces exerted by the machinery, which can be increased, but by the natural characteristics of the rock. It is possible to apply a greater bracing force through the grippers than can be resisted by the sides of the tunnel.

The curved bracing shoes (Fig. 6-45) are called grippers. These are curved to suit the excavation profile and are expanded against the sides of the tunnel to provide bracing. When the boring of a stroke is complete, the rotation of the cutterhead is stopped so that the machine can be moved forward or regripped using the bracing system. A gripper TBM is stabilised during regripping by the rear support and the shield surfaces next to the cutterhead, which are expanded radially into the side of the tunnel. There are two different types of bracing; either the grippers work horizontally or diagonally in an X shape (Fig. 6-46). Horizontal bracing is suitable for zones where the rock mass tends to break in the crown, and diagonal bracing can better resist the sinking of heavier machines.



a)



b)

Figure 6-46 Gripper bracing [242]
a) horizontal sideways bracing
b) cross bracing

While the machine is moved, the hydraulic cylinders of the grippers are retracted and the grippers are then expanded against the tunnel sides with the necessary pressure in the new position. This requires the sides of the tunnel to be free, which is only the case in stable rock. The process of boring a stroke and regripping is illustrated in Fig. 6-47, through the example of a Wirth machine. Machines from other manufacturers work with a similar sequence of movements.

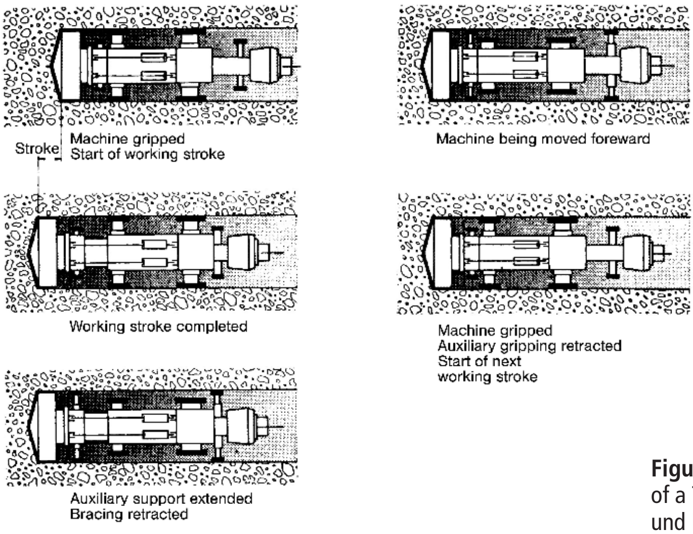


Figure 6-47 Working sequence of a TBM from Wirth Maschinen- und Bohrgerätefabrik GmbH

For shielded TBMs, not the rock strength but the segment lining is decisive, as this type of machine is not supported radially against the sides of the tunnel but axially against the already installed lining tube. There are, however, systems combining elements of each system.

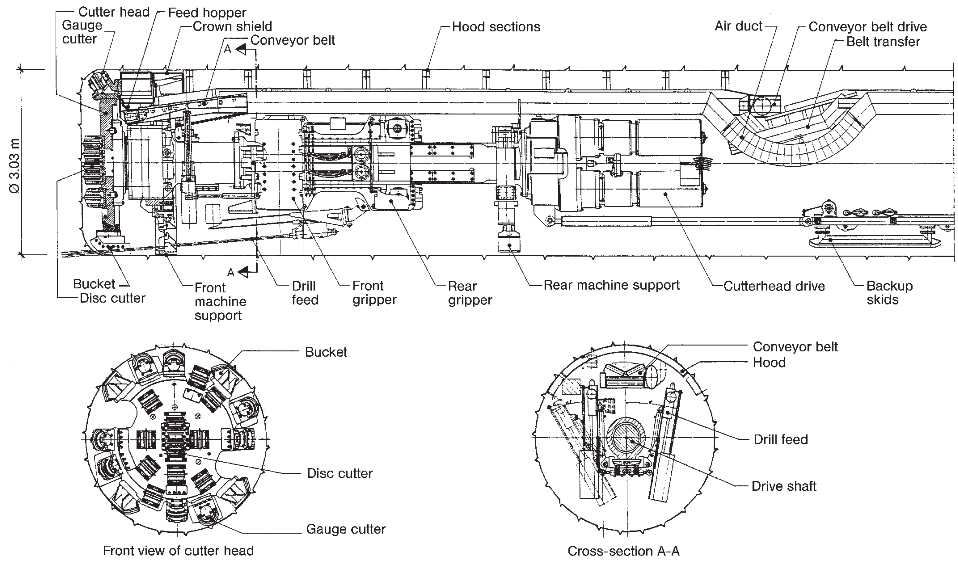


Figure 6-48 Gripper TBM S-96 (Herrenknecht) with high-level conveyor belt, Kompakt-TBM 3000 [242]

Mucking system. The rock chips are picked up at the face by buckets, which in a TBM are mostly slots at the perimeter of the cutterhead, and loaded down transfer chutes onto a conveyor belt. In order to guarantee the transport of the excavated material along the tunnel, a system should be chosen with sufficient capacity and which does not obstruct the supply of support materials to the TBM. According to the local conditions, this could be a rail-based or a conveyor belt system. The use of large dump trucks is also possible. Fig. 6-48 shows an example of the layout of the mucking system for a gripper TBM.

In order to ensure continuous transport along the conveyor belt, band cassettes are used, which can store up to 600 m of belt. This means that the TBM can bore up to 300 m of tunnel without interruption as the transport belt extends automatically and only the tunnel belt idlers and the support work have to be added behind the TBM, the tail pulley travelling with the backup (Fig. 6-49). Only when the stored belt is used up must the transport belt be extended. This is done by cutting the tunnel belt, retracting the belt into the cassette and adding a new length of belt by vulcanising. A technical innovation was the belt storage at the Sörenberg Tunnel with a 25 m high belt storage tower at the portal due to the very restricted space available.

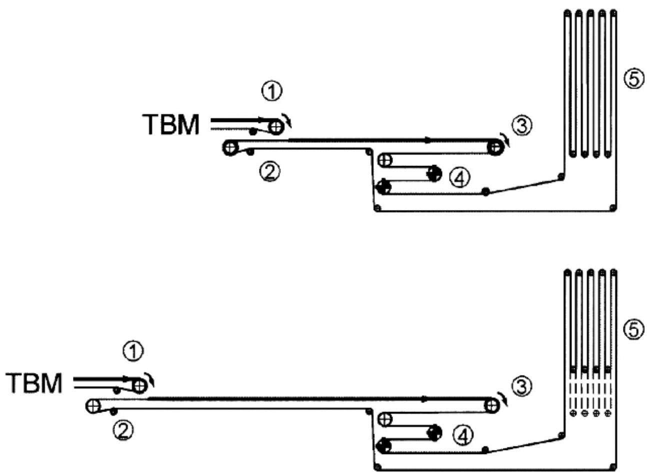


Figure 6-49 Belt storage with travelling tail pulley, example Sörenberg Tunnel [130]

1. Material discharge of backup belt
2. Tail pulley of the tunnel belt travelling with the backup
3. Drive station
4. Belt storage tower
5. Material discharge from tunnel belt

Problems can occur both with the buckets on the cutterhead and with conveyor belts through blockage by overlarge rock pieces and sticking of fine-grained and cohesive muck. Excessive water ingress at the face can also obstruct muck transport if the muck no longer has the character of broken rock but becomes muddy slurry, which would make a sludge pump or a screw conveyor necessary, which would make the normal operation of a TBM impossible. In this case, short fault zones can be overcome by ground improvement measures such as grouting or even ground freezing, but for longer stretches, the entire tunnel drive would have to be differently planned. Constant adaptation is not possible.

6.4.5 Support system

The materials and methods of excavation support in TBM tunnelling have not essentially changed from those used in drill and blast or roadheader tunnelling.

Even the terms used with the use of a gripper TBM have remained the same with the terms head protection and light to heavy support. The excavation support normally has a final or at least partially final character. In contrast to drill and blast tunnelling, TBM boring is said to be kind to the rock mass, although the high bracing forces on the tunnel sides sometimes give a different impression.

One essential difference is that the high intended advance rate demands much earlier load-bearing capacity than with drill and blast. With the daily advances achieved even by medium and large diameters, neither normal grouted anchors nor shotcrete achieve the required load-bearing effect soon enough. With the primary intention being rapid advance rates, support systems are therefore required, which can also be used with a gripper TBM, analogously to segments with a shielded TBM. Support elements can no longer be materials, they have to be used as construction elements. For daily advance rates, which can be 20 to 40 m even in medium to large diameter tunnels, neither normal grouted anchors nor shotcrete achieve the required load-bearing action early enough. Since the primary aim is rapid advance rates, support systems are therefore required, which can also be used with gripper TBMs, analogously to the segment linings used with shield TBMs. Support elements should not longer be materials, but should be installed as construction elements. But shotcrete should not be dispensed with, considering its excellent properties like good bonding with the rock mass and impressive ductility. The necessary supplementation of the excavation support as a whole can be performed in the backup area, normally however at least 30 m behind the cutterhead.

The use of tunnel boring machines in slightly friable rock reduces with increasing diameter. Support measures in small-diameter tunnels can only be relocated to the backup area behind the TBM. This means that tunnelling though fault zones with poor geology, when the stand-up time of the rock is shorter than the advance time, is always a problem case. In large-diameter machines, drill feeds can be provided in the area behind the cutterhead for the installation of rock bolts or spiles. The installation of support arches is also possible and shotcrete spraying behind the cutterhead has already often been tried out. In order to support fault zones, advance measures like spiles, piles, grouting or even ground freezing can be installed above and in front of the cutterhead to stabilise the rock mass sufficiently to enable continued boring with the TBM.

The advance rate can be greatly reduced or even stopped depending on the extent of the necessary support measures. The application of shotcrete in the machine area directly behind the cutterhead has still not yet been satisfactorily solved, as particularly the rebound leads to problems. This demands further development work, both regarding shotcrete technology and the machine. One exception in this case is the reamer TBM, which makes the entire tunnel section available for support work directly behind the cutterhead and thus already permits the successful use of shotcrete.

The Swiss Baumeisterverband (Master Builders Association) [328] has also described the reduction of advance rate depending on the support measures (Fig. 6-50).

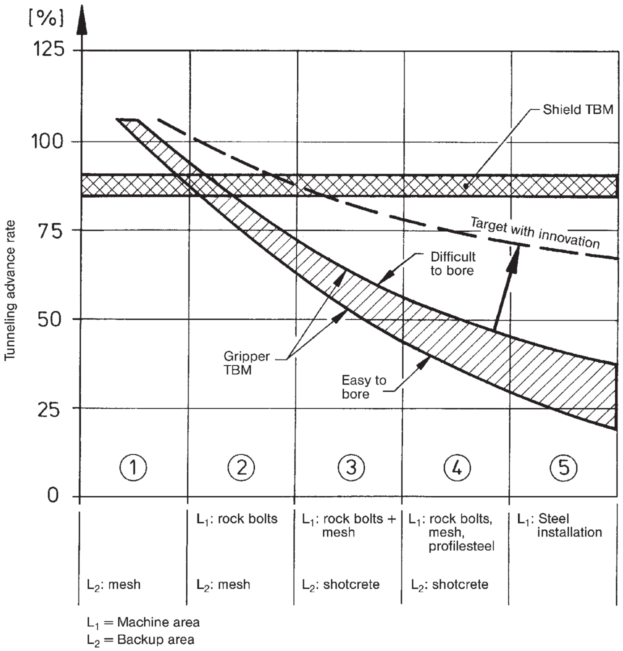


Figure 6-50 Graph of the advance rates of gripper TBMs with conventional support measures and a shield TBM [328]

Fig. 6-50 shows clearly how the advance rate falls greatly with conventional support measures, and the aim of increased performance through the use of innovative support measures. The advance rate of a shield TBM, however, maintains the same level in different rock mass classes.

If the rock mass is only slightly friable, meaning that only isolated or small rock falls have to be expected, it can be sufficient to support the crown with a crown shield or hood. Such a shield protects the crew working immediately behind the cutterhead against danger from falling rock and stones. When a gripper TBM is fitted with a short cutterhead shield, this ends behind the cutterhead, so that the already described support measures have to be installed more or less directly behind the cutterhead, then follows the backup.

The currently most usual description of the individual sections that have to be supported can be found in the Swiss SIA standard 198. According to this standard, support measures have to be defined for the various working areas of a TBM before the start of the tunnel and depending on the excavation classes. These working areas behind the cutterhead are defined by the standard as follows:

– L1 machine area.
– L2 backup area.
– L3 rearward section 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 support work is carried out depending on the requirements of the project and the constraints of the machine type (Fig. 6-51).

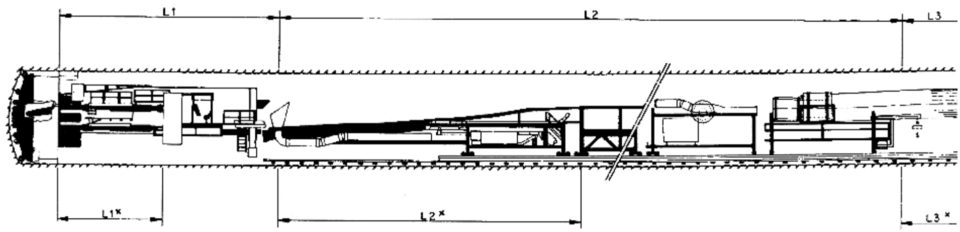


Figure 6-51 Working areas and working zones with a TBM [242]

The standard equipment of a large gripper TBM currently includes the provision of a ring erector and drills for rock bolts behind the expanding cutterhead shield. The installation of mesh or nets for head protection has until now been carried out by hand in the unsecured machine area between the expanding cutterhead shield and the gripper bracing, designated L1* according to SIA 198.

In order to improve this situation and improve the safety of the miners, extensive changes were made to this basic concept for the tunnelling work in Gotthard and Lötschberg (Fig. 6-52).

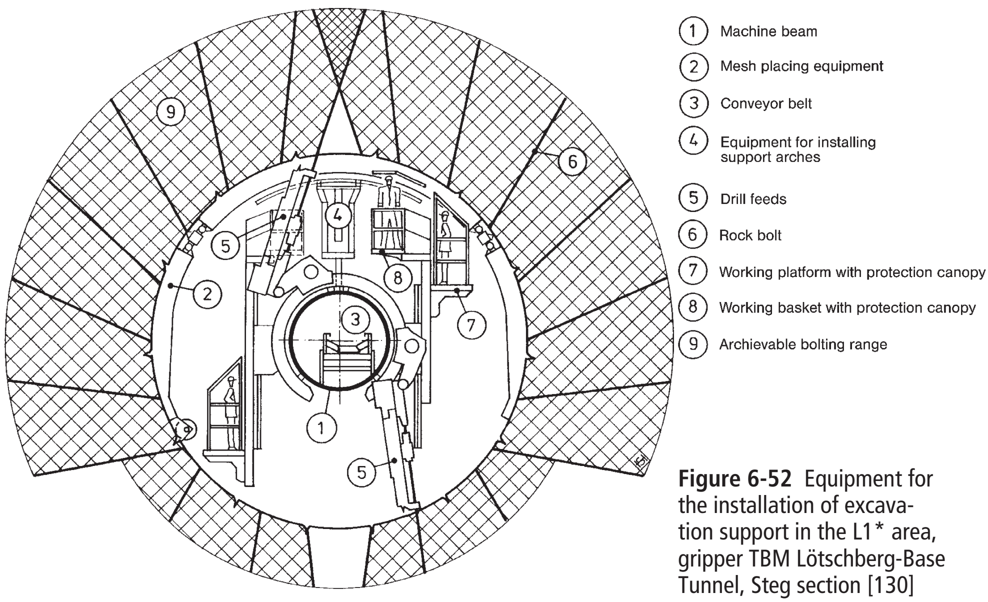


Figure 6-52 Equipment for the installation of excavation support in the L1* area, gripper TBM Lötschberg-Base Tunnel, Steg section [130]

The operation of drills and the setting of the rock bolts is carried out in this case from working baskets with protective roofs or from working platforms, which are situated under the finger shield of the expanding cutterhead shield. The head protection is partially installed mechanically with a net setting device (Fig. 6-53). This bearing construction is extended backwards during the drilling for the rock bolts and gives the working area free for drilling work. The net setting device is loaded in this backward position. Either reinforcing mesh or nets can be used. Once the net setting device has been loaded and the drills have been withdrawn from the working space, the net setting device can be travelled to the installation position. The head and sides are folded against the tunnel sides and the head protection is fixed.

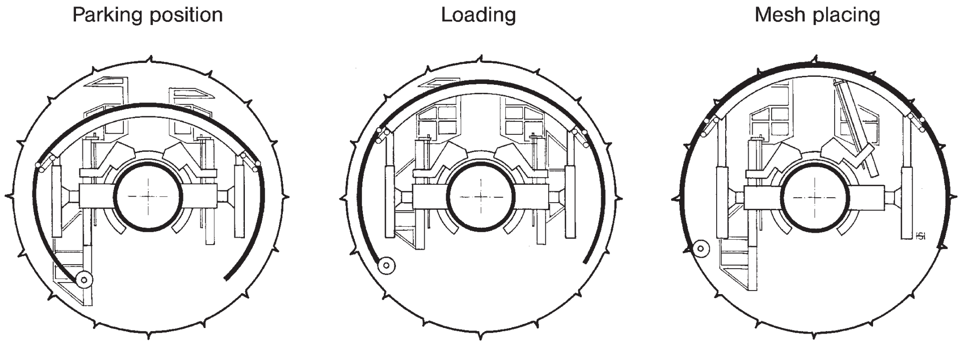


Figure 6-53 Net installation with net setting device, gripper TBM Lötschberg Base Tunnel, Steg section [130]

In recent gripper TBM tunnels, one invert segment has been used. This segment provides temporary and permanent support and can be pre-fitted with fixings for the rails for the backup and transport along the tunnel and thus enable rapid track laying. Drainage pipes can also be cast into the segment. The final lining of the rest of the tunnel profile is than placed in the classic manner using a mobile formwork unit.

In contrast to this, the shield of a shielded TBM provides temporary support to the rock mass all round. The shield skin starts directly behind the gauge cutters and continues to include the area where the segments are installed. Reinforced precast segments are normally used for the lining. Each segment is installed with the erector in the correct position these provide immediate support. Where high pressures from squeezing rock are to be expected, invert segments up to 90 cm thick have been installed in extreme cases. A TBM with shield can be equipped for compressed air, Hydro or EPB support and is than capable of working below the groundwater table. However, this leads to particular problems in hard rock, as current experience is mostly only in soil.

The back end of the shield, the shield tail, extends past the last segment ring and supports the rock mass until the annular gap has been filled. The shield tail of hard rock shield TBMs is described as the tail shield. The stowing or filling of the annular gap limits any loosening of the rock mass slight, and a bond is created between the rock mass and the lining. In order that the advance can continue without obstruction or longer interruptions, the backup has to house all equipment needed for the prompt installation of the lining.

6.4.6 Ventilation

Mechanised boring of rock with the partial grinding of the rock under the disc cutters produces a lot of dust. At the same time, the mechanical energy used for the destruction of the rock is also converted into a considerable amount of heat. In order to ensure good air conditions at the face – which is a precondition for the appropriate working performance – the dust has to be collected and removed and the large amount of liberated heat has to be removed by suitable measures. Further details of ventilation for TBM tunnels can be found in Chapter 8.

6.4.7 The use of slurry and earth pressure shields in hard rock formations

Slurry and earth pressure shield machines were originally designed for boring through soil but are increasingly being used in solid rock or mixed face conditions. This term describes heterogeneous ground conditions with both solid rock and soil being encountered in the face. The tunnelling machinery has to provide active face support, but at the same time also include essential elements for boring hard rock. Convertible tunnelling machines (for example Mixshield, Kombishield) have to be able to work in closed mode (Fig. 6-54) in unstable ground, which often leads to difficulties with the process.

Operating mode: slurry shield. A Hydroshield made by Herrenknecht was used as long ago as 1989 for the "Stadtbahn Mülheim" crossing under the River Ruhr. This originally had a hard rock cutterhead in soil and then worked with a bentonite-supported face (closed mode) and later operated in sandstone with a half-full working chamber and hydraulic mucking circuit (open mode). The experience and the problems encountered, particularly concerning increased wear and the low degree of utilisation of the tunnelling equipment, were described in detail in [133a]. The cost of such a tunnelling method can be considerably increased by the conditioning agents, which are often necessary with slurry shields, and interrupted operation due to sticking of the cutting wheel.

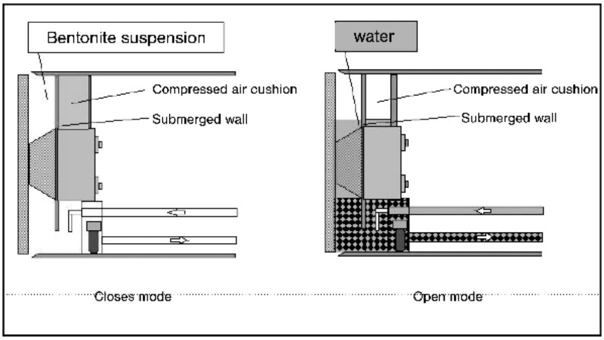


Figure 6-54 Operating modes of slurry shields [247]

Operating mode: earth pressure shield. Due to the process technology, which at first glance is simpler, and the wide range of applications, the use of EPB shields is constantly increasing, particularly outside Germany but also in Germany. Fig. 6-55 gives an overview of the various operating modes of an earth pressure shield. It seems possible to use this machine in almost any ground. No detailed description of the various operating modes and areas of application is given here, but the reader can refer to [244] and [242]. A differentiation of the following modes has become established internationally:

- *Open mode.* Shield tunnelling in open mode is possible where the face is temporarily stable and water ingress does not cause any hydrogeological or process-related problems. In abrasive hard rock, it is highly advantageous to operate the machine in open mode in order to reduce the wear on the excavation tools and steel construction due to excessive exposure. The process problems, which can occur during operation in mixed face conditions, are described in more detail under point 5.
- *Closed mode.* The purpose of closed mode is to ensure a stable face and low surface subsidence in unstable geological formations and in the presence of groundwater. This is done by pressurising the soil or support medium in the excavation chamber to achieve a state of equilibrium or balance against the prevailing earth and water pressure.

– *Transition mode.* The term transition mode describes an operating state with a nearly full but not pressurised excavation chamber. The face needs to be largely stable. The intention is to hold back water and be able to change rapidly from open to closed mode in case the geological conditions suddenly worsen. The excavation chamber is partially or fully filled with soil and/or water. A support pressure of only 0.2 or 0.3 bar is often sufficient, for example in jointed rock, to greatly limit the water inflow. It has proved especially advantageous that the technology permits the application of compressed air in the excavation chamber during a stoppage.

As with slurry shields, sticking can also be a problem with earth pressure shields and conditioners are also used to improve the advance rate.

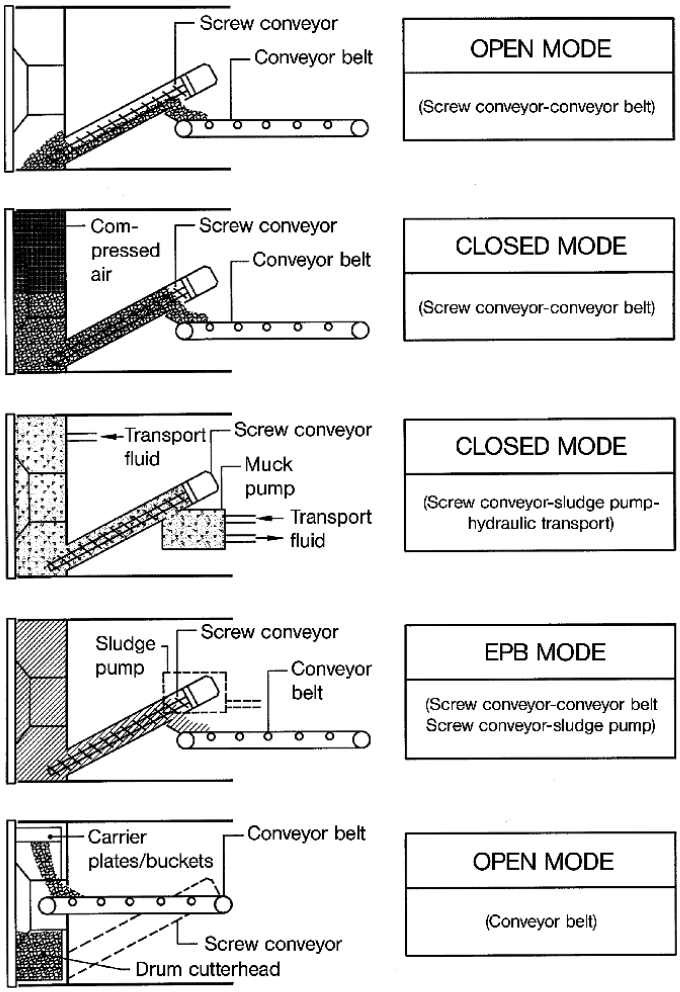


Figure 6-55 Operating modes of earth pressure shields[247]

Operating parameters. Operation in hard rock and soil requires fundamentally different processes and can also lead to conflicts of objectives in the setting of the optimal operating parameters.

Machines operating in soil are normally operated with high penetration rates but relatively low revolution speeds. For hard rock cutterheads, the penetration of the cutting tools is an essential wear factor, depending on the rock strength and the structure and texture of the rock mass. The penetration rate is normally limited in order to optimise the utilisation and reduce the thrust forces. In order to ensure an adequate advance rate, limitation of the penetration leads to an increase of the required cutterhead revolution speed. The secondary wear to the excavation tools and the steel construction of the cutterhead depends according to experience on the distance travelled and thus increases with increasing revolutions and reducing penetration. Detailed suggestions for optimising the operating parameters in hard rock are contained in [247].

6.5 Special processes: combinations of TBM drives with shotcrete tunnelling

Under certain project-specific conditions, combinations of mechanised and conventional tunnelling methods can be favourable from the point of view of rock mechanics and economics.

The following section therefore gives an overview of construction processes, which combine TBM and shotcrete methods of tunnelling.

6.5.1 Areas of application

Combined processes can be used both in deep and shallow tunnels. The idea also offers the possibility of producing non-circular cross-sections. In general, a smaller pilot tunnel inside the final cross-section is first bored with a TBM and the overall cross-section is then enlarged using shotcrete support. Fig. 6-56 illustrates this method of working through the example of the Amberg autobahn tunnel. The Pfänder Tunnel on the same autobahn was constructed similarly [221].

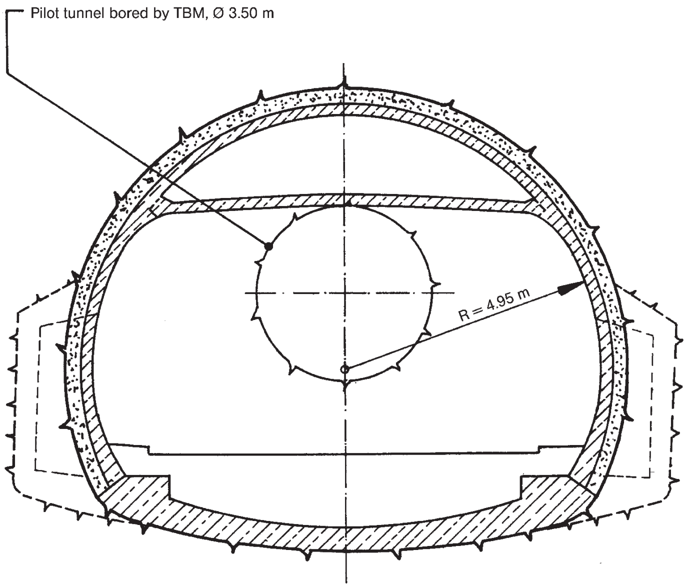


Figure 6-56 Overall cross-section with the location of the investigation tunnel, Amberg autobahn tunnel (Austria) [119]

The combination of an economic TBM drive with flexible conventional tunnelling gives the following essential advantages:

– Early discovery of the geological and hydrological ground conditions and thus reduction of the uncertainties and risks for the subsequent enlargement.
– An available tunnel boring machine can be used.
– Reduction of the environmental impact due to the reduced amount of blasting, less noise nuisance for the inhabitants and less vibration when the tunnel is enlarged by blasting.
– Improved ventilation and drainage conditions for the drill and blast works.
– Overall reduction of the cost risk.

6.5.2 Construction possibilities

Depending on the conditions affecting the entire project, the possible construction methods of combined processes can be differentiated as follows:

– Probe and investigation tunnels completed in advance provide almost complete geological investigation of the ground on the alignment and are preferred for deep tunnels. They are normally tendered as a separate project before the construction of the main tunnel and can be used along the entire route or for sections. Information gained about ground conditions can be integrated into the design and tendering of the main tunnel.
– Pilot tunnels, which are driven in advance and are generally situated inside the later tunnel cross-section, are included in the contract for the main tunnel as they are considered construction process measures and integrated into the construction process for the main tunnel. They also offer the advantage of a pilot tunnel.
– Enlargements of the cross-section, which are carried out for various purposes along a mechanically bored tunnel, for example for the construction of stations, niches and points or machine halls.

Investigation tunnels are mainly used for the detailed investigation of the geology, hydrology, gas occurrence and rock properties. They are mostly constructed as a separate contract before the awarding of the main contract in order to provide a better basis for the design and tendering of the subsequent main tunnel. The information gained from the advance investigation is integrated into the contract documents for the main tunnel, although the geological risk is still borne fully by the client.

Advanced investigation of the rock mass is especially advantageous when the overburden is deep, as for example is often the case in the Alps, because the construction of a deep tunnel poses considerable risks with the selection of a method of tunnelling. Good examples of this are the investigation tunnels for the NEAT Base Tunnels at Gotthard [381] and Lötschberg [353]. But particularly difficult conditions for tunnelling can also make the boring of an investigation tunnel necessary. Examples for this are the Seikan Tunnel [99] below the sea or the Freudenstein Tunnel shown in Fig. 6-57, constructed in leached gypsum Keuper susceptible to swelling on contact with water [137].

In longer tunnels, access tunnels to provide intermediate starting points can also be used as investigation tunnels. Examples of this were the intermediate starting points for the Irlahüll and Euerwang Tunnels on the new high speed line Nuremberg-Ingolstadt [164] or also the NEAT Base Tunnels Gotthard and Lötschberg [381], [353].

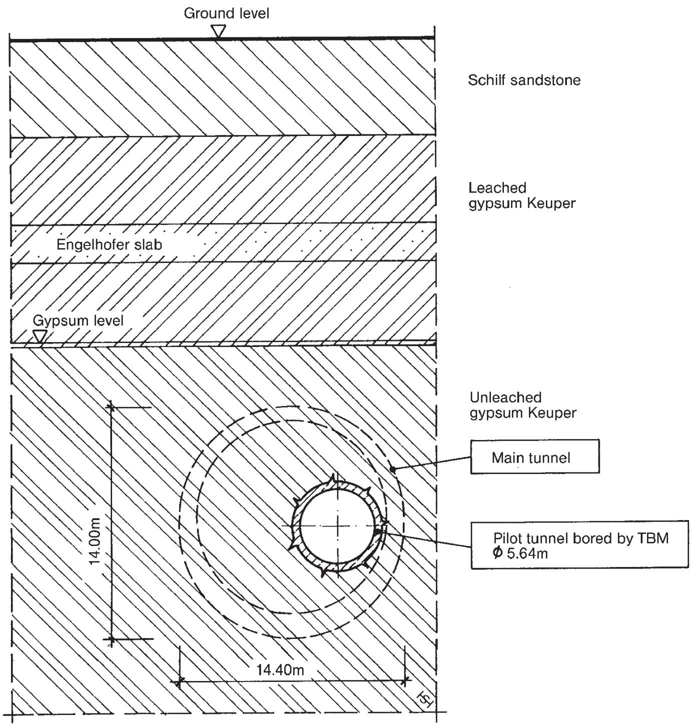


Figure 6-57 Investigation tunnel in difficult geological conditions, Freudenstein Tunnel [137]

The construction of investigation tunnels maybe along part sections with difficult geological conditions, particularly with water ingress, or it may be along the entire length of the tunnel. The investigation tunnel is generally inside the later tunnel cross-section, but can also be outside. The selection of the section to bore mechanically also depends on the available tunnel boring machine and is typically between 8 and 14 m² (bored diameter 3.2 to 4.2 m).

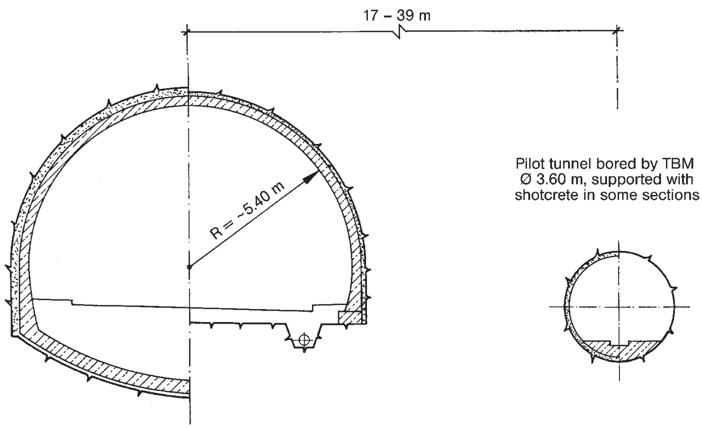


Figure 6-58 Investigation tunnel outside the final cross-section, Tunnel de Raimeux [125]

A good example of the decision to bore the investigation tunnel outside the final cross-section is the Tunnel de Raimeux (Fig. 6-58). The tunnel was bored by reusing a Robbins 123-133 gripper TBM that was already 20 years old at the time, bored diameter

3.65 m, completed in advance of the main tunnel and then connected to the main tunnel with cross passages. In this way, the investigation tunnel could be used during the construction of the main tunnel for water drainage and material transport. After the opening of the road tunnel, it remains in use as part of the overall tunnel safety plan and can, for example, be used for purposes of maintenance or for rescue in emergencies. The TBM boring of the investigation tunnel could be used to investigate several karst cavities and their hydrological behaviour.

Pilot tunnels generally fulfil the same aims as investigation tunnels, the main difference being that they are constructed as part of the main contract. Pilot tunnels are constructed simultaneously with the main tunnel and mostly serve operational purposes. They can be used, for example, to drain large flows of groundwater as the main tunnel proceeds to improve the water drainage. The rock mass is dewatered and any high water pressures can be relieved.

Due to the size, dimensions and geology, the final cross-section is normally constructed in drill and blast with shotcrete support; the enlargement of a pilot tunnel to the main tunnel requires less drilling and less blasting. The pilot tunnel can then be regarded as a large cut and removes the confinement of the face [221]. The use of a pilot tunnel is particularly advantageous for shallow tunnels, as is the case for tunnelling below buildings with little cover. The Uznaberg Tunnel [252] is an example for transport tunnels driven in drill and blast under densely populated residential areas, industrial zones or historic buildings.

At the same time, these mechanically bored sections also offer the advantages of an investigation tunnel. In addition to the opportunity to investigate the ground, a pilot tunnel completed in advance can also be used for effective ventilation of the main tunnel drive. For example at the Milchbuck Tunnel, Zürich, a pilot tunnel was bored with a Robbins TBM (diameter 3.20 m) in order to ensure efficient ventilation during the main tunnel excavated by a roadheader (Fig. 6-59).

Figure 6-59 Pilot tunnel with ventilation duct to ensure efficient construction ventilation, Milchbuck Tunnel, Zürich

Above all in drill and blast tunnelling, efficient tunnel ventilation can be of economic advantage since the activities drilling, blasting and mucking create a great quantity of gases and dust, which pollute the damp and often warm air in the tunnel and endanger the health of the workers to a high degree. This applies particularly in longer tunnels, where the sizing of temporary ventilation plant with a high number of diesel-powered vehicles is often at the limits of feasibility.

Enlargements for stations, points or machine halls. The enlargement of an already bored tunnel to provide an enlarged profile along a certain length is a particular challenge for the construction process. Such enlargements may be required for the construction of stations, sidings and points in rail tunnels or for emergency stopping bays in road tunnels. Some examples of the enlargement of the cross-section of mechanically bored tunnels are the junction structure in the Zürich – Thalwil rail tunnel [39], the two-track crossing station in the Vereina Tunnel and the stopping bays in the Sachseln road tunnel [65].

The procedure for the construction of a station enlargement is shown in Fig. 6-60 and Fig. 6-61, in this example for the underground railway in Mülheim.

Figure 6-60 Enlargement of the segment tube to station cross-section, Mülheim underground [140]

In Mülheim, the tunnel bored by a Hydroshield with a hard rock cutterhead, diameter 6.90 m and cross-sectional area 37.5 m², was enlarged to a 73 m² station cross-section after the completion of the mechanised tunnel drive. The segment lining was partially removed and the section was enlarged by drilling and blasting with shotcrete support.

The use of shotcrete support methods in combination with a TBM tunnel is also advantageous for the construction of crossings and junction structures due to the adaptability of the method to the local geological conditions and variability of cross-sectional area and shape. This was also shown during the construction of the Channel Tunnel, where the service tunnel bored by a Howden shield TBM (dia. 5.76 m) had to be enlarged along a length of 65 m for a crossover. The procedure used for this enlargement is illustrated in Fig. 6-62.

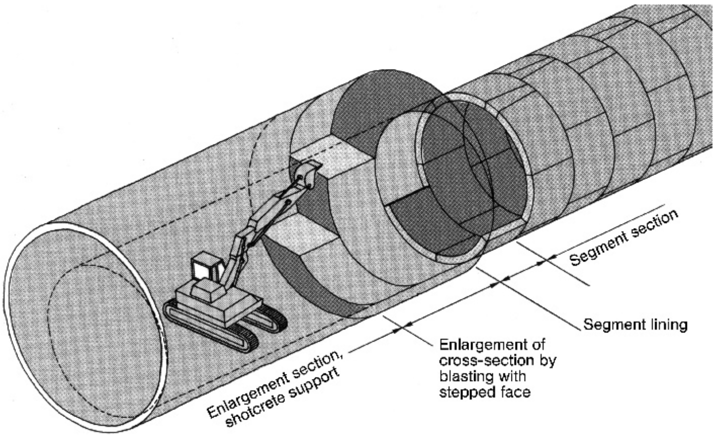


Figure 6-61 Sequence of station enlargement, Mülheim underground [6]

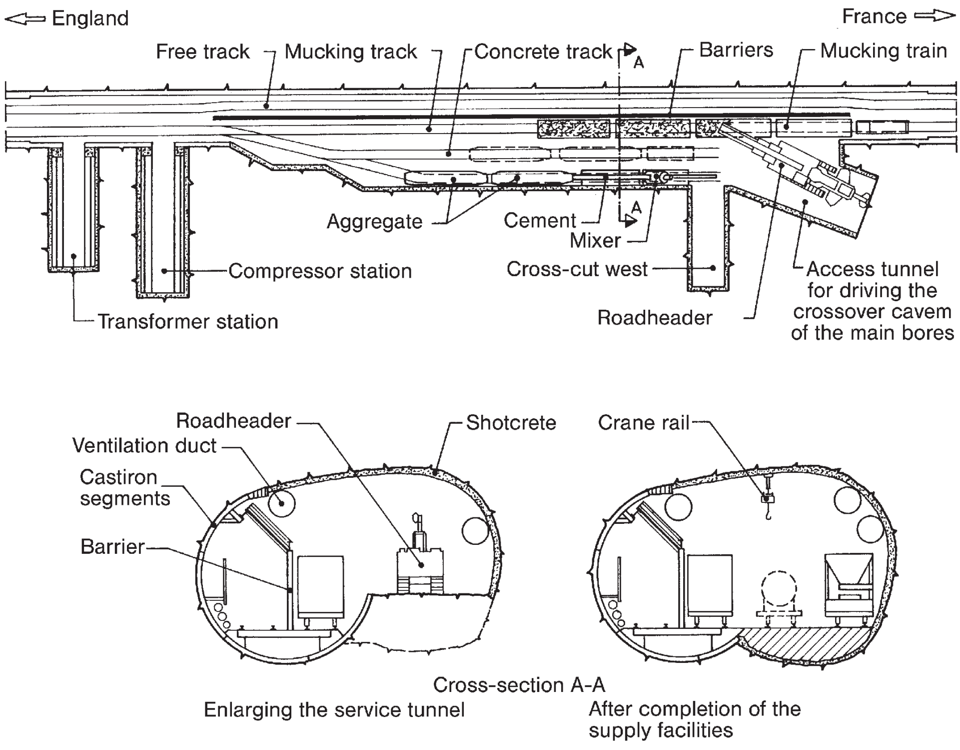


Figure 6-62 Enlargement of the service tunnel in the Channel Tunnel [156]

The support at the location of the enlargement consisted of cast iron bolted segments, which were removed at one side in the course of enlargement. The shotcrete support of the enlarged section was connected to the cast iron segments with a shoe in the crown, and the connection in the invert was supported by an invert anchor. The actual enlargement was started by hand and continued with a small roadheader, with the section being divided into top heading – bench – invert. After the completion of the enlargement, the adit necessary for access to the crossover cavern between the running tunnels could be constructed, The enlarged cross-section provided room for the necessary siding for construction trains loaded with aggregates, cement and muck.

6.5.3 Example

Piora fault investigation tunnel. The alignment of the 57 km long Gotthard Base Tunnel was intended to pass through the Piora fault, a geological fault zone. The available geological knowledge could not rule out that the Piora fault could extend downwards to the main tunnel in the form of a floating wedge of sugar-grain dolomite, also under high water pressure. In order to avoid delays to the boring of the base tunnel, it was necessary to obtain detailed information about the thickness and structure of the Piora fault. For this reason, a 5,552 m long investigation tunnel was started northwards from Faido.

Investigation concept. The construction works on the investigation tunnel began in 1993 using a Wirth TB III-450 gripper TBM (dia. 5.0 m). In March 1996, the drive stopped at chainage 5,552 m, about 350 m above the level of the Base Tunnel and about 50 m short of the Piora fault. Finally two headings were driven from the investigation tunnel and connected to each other (Fig. 6-63).

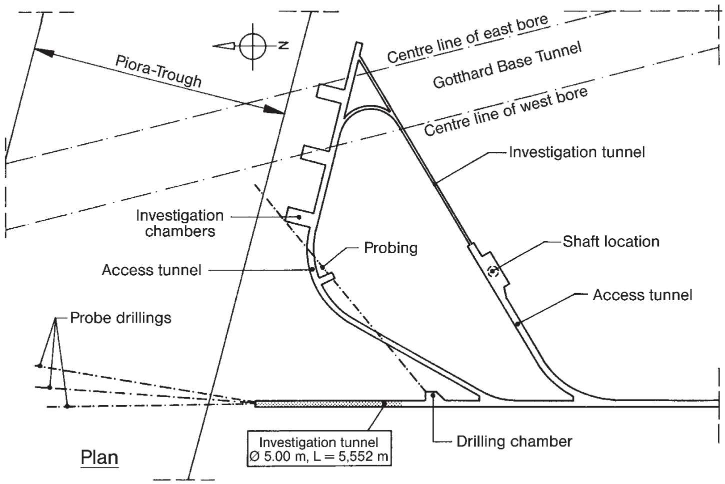


Figure 6-63 Piora fault investigation tunnel, Gotthard Base Tunnel, overview of probing system [381]

The front heading provided access to the investigation chambers. It was intended to drive a heading with a diameter of 4 to 5 m from one of these chambers through the Piora fault with the aim of testing the machinery, materials and process planned for use in the Base Tunnel to investigate the boring of the Piora fault at Base Tunnel level. The back heading served primarily as access to a shaft to be sunk down to the level of the bores of the Gotthard Base Tunnel in case the results of the probe drilling should be inadequate. From the bottom of the shaft, additional probing would be carried out from a further system of headings. The actual investigation works took place from August 1997 to March 1998. In this period, the rock mass was probed by several probe drillings at investigation tunnel and Base Tunnel level from a niche in the investigation tunnel.

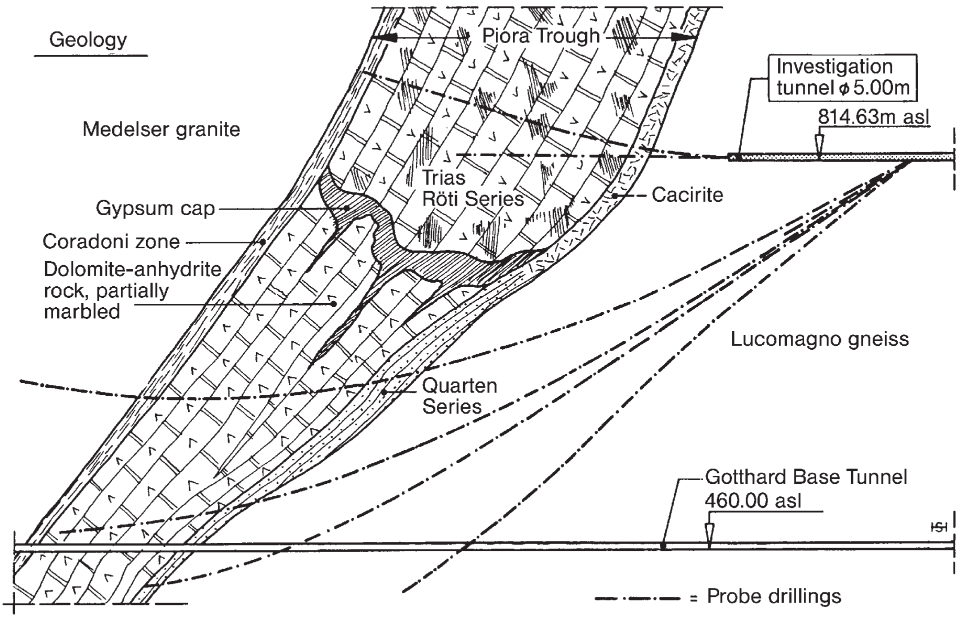


Figure 6-64 Piora fault probing system, Gotthard Base Tunnel, geological longitudinal section [100]

Results of the preliminary investigation. The results of the probe drilling, which altogether investigated a 200 m wide corridor around the future Gotthard Base Tunnel, showed that the Piora fault at Base Tunnel level consists of dolomite and anhydrite and the rock is solid and dry (Fig. 6-64). The thickness of the zone is between 125 and 155 m. At the level of the investigation tunnel, the probe drillings had discovered sugar-grained dolomite with a water pressure of over 100 bar. The difference between these two zones can be explained due to the reaching of the base of the karst water circulation system about 250 m above the level of the Base Tunnel. A gypsum cap closes or stops all weak points in the rock mass body. This leads to complete sealing of the dolomite-anhydrite below it against the water circulation at higher level [380].

The conclusion of the investigations was that boring through the Piora zone would be feasible using normal methods of tunnelling. As another result of the knowledge gained was that the sinking of the shaft with further probing was not considered necessary.

Kandertal investigation tunnel. In order to obtain information for the project and tender documents for the new 34.6 km long Lötschberg Base Tunnel, the 9.5 km long Kandertal investigation tunnel was bored between 1994 and 1997 (Fig. 6-65). This was used to investigate the geological and hydrogeological conditions and the occurrence of gas in the project section Frutigen (north portal) to Kandertal.

In order to save construction costs for the Base Tunnel, only the east bore is being constructed initially from the north portal in Frutigen to the service station in Mitholz. This is possible because the Kandertal investigation tunnel runs parallel to it and can function as a rescue and safety tunnel in the operation phase. South of Mitholz, two tunnel bores are intended, although the railway equipment in the first section of the west tunnel is not planned for the first phase. Details of the use of the single-track cross-section of the Lötschberg Base Tunnel are not given here.

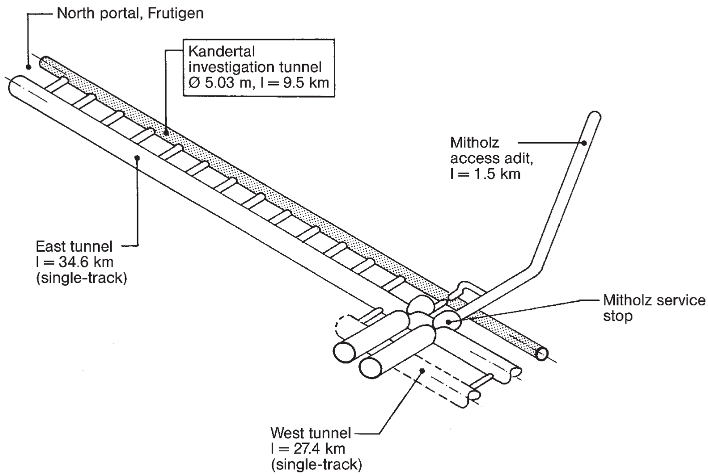


Figure 6-65 Kandertal investigation tunnel, Lötschberg Base Tunnel [34]

The alignment of the Kandertal investigation tunnel runs from the north portal at Frutigen about 30 m east of the Base Tunnel bore in the western flank of the Kandertal valley, where it connects after about 7.2 km to the side of the Mitholz adit, which provides ventilation. The end of the investigation tunnel was placed after 9.5 km at the transition from the Wildhorndecke into the flysch of the Doldenhorndecke. From this zone at the height of the Kandersteg, the geological forecast to the south is considered adequate.

Tunnelling concept. The investigation tunnel was bored with a new Robbins MB 1610-279 gripper TBM (dia. 5,03 m) (Fig. 6-66) with an overcutting mechanism of 20 cm. The advance rate for the cross-section of about 20 m^2 varied between 2 and 45 m/d. The average advance rate achieved was 19.5 m/d. During the boring of the tunnel, the contact pressure, advance rate and also the data from the various probe drillings were recorded electronically, which was useful for the evaluation of the geology encountered.

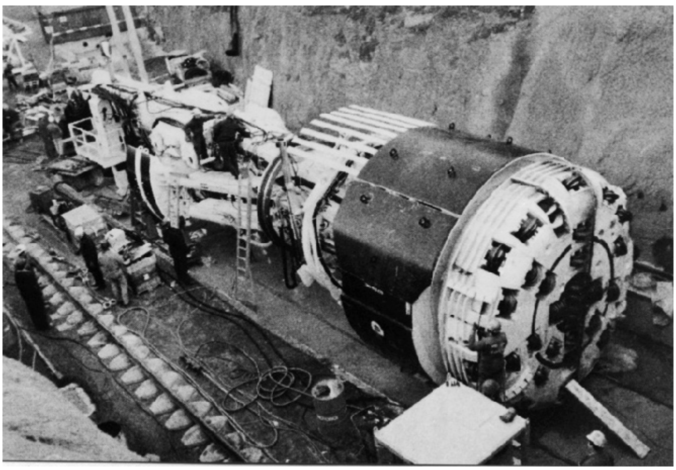


Figure 6-66 Gripper TBM MB 1610-279 (Robbins) in the starting cut, Kandertal investigation tunnel, dia. 5.03 m [352]

Depending on the stability of the rock mass, it can be necessary to undertake support works directly behind the cutterhead. This could be the installation of rock bolts, crown arches or additional steel arches. The steel fibre shotcrete support about 8 cm thick was sprayed about 40 m behind the cutterhead. Finally, about 55 to 65 m behind the face, the invert segments were laid [352]. The 1.6 m long invert segment weighing 5 t with integrated drainage were placed on the cleaned invert and poured with mortar (Fig. 6-67). Where additional probing was considered necessary, it was also possible to perform probe drillings using independent drills mounted directly behind the cutterhead to drill up to 80 m into the surrounding rock mass.

Figure 6-67 Invert segment installation in the Kandertal investigation tunnel [44]

Results of the preliminary investigation. For the first 4 km, the Kandertal investigation tunnel crosses the Taveyannaz series and the flysch, which forms the base of the following Wildhorn decke (nappe). The flat base overlap of this decke lies slightly above level of the tunnel and then up towards the brow of the Doldenhorn decke a little under the Base Tunnel. The geological findings are shown in Fig. 6-67 and 6.68.

Water appears mainly in the form of dripping water or as springs along open fissures. No large karst systems or water-bearing valley joints were found. In summary, the annual quantity of water at 15 l/s is considered very low for the approximately 9.5 km tunnel length.

Natural gas was observed from many anchor drill holes, particularly in the schists of the Wildhorn decke. No actual gas blowouts were recorded.

The information gained showed that the choice of the alignment for the Base Tunnel could be kept. Further, the data, supported by mechanical measurements on rock samples, provided the information that the section of the Kandertal investigation tunnel with its “soft” rock could be tunnelled through either with a TBM drive or by drilling and blasting. Both tunnelling methods were included in the tender documents.

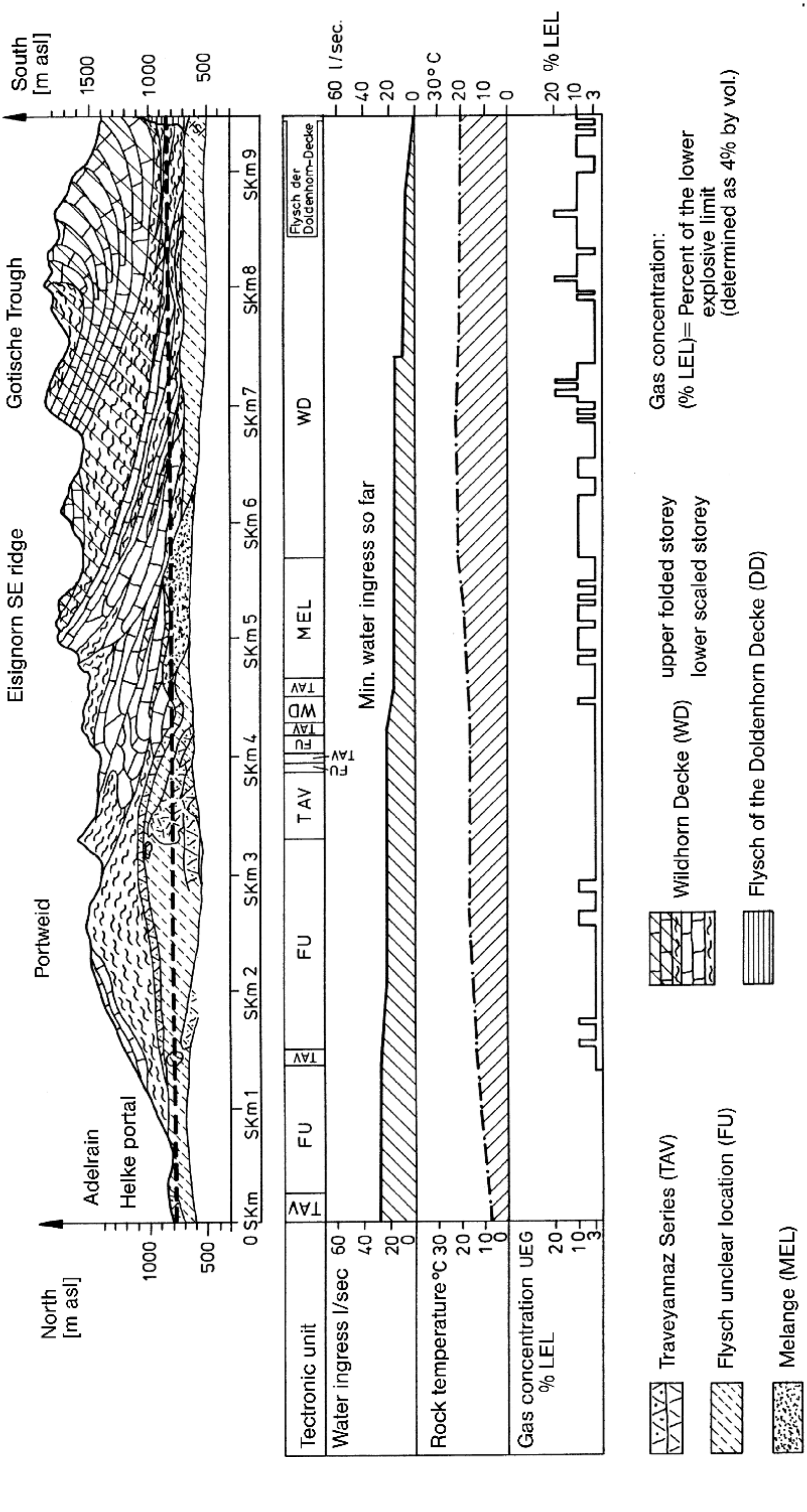


Figure 6-68 Kandertal investigation tunnel – geological findings [352]

Uznaberg pilot tunnel. The Uznaberg tunnel with its two 923 m and 937 m long double-track bores is the longest tunnel in the Wagen–Eschenbach–Schmerikon by-pass project (Zurich–Chur). The tunnel has a maximum overburden of 50 m and passes under a heavily built-up area. Due to the very shallow cover, in some places only 12 m, only low-vibration excavation was possible. In addition to a pure excavation by blasting, another variant was tendered – a TBM pilot heading followed by enlargement to the final cross-section by blasting. The client finally decided for the variant with a TBM pilot tunnel (Fig. 6-69), because this possibility was not much more expensive and was advantageous considering the sensitivity of the built-up area.

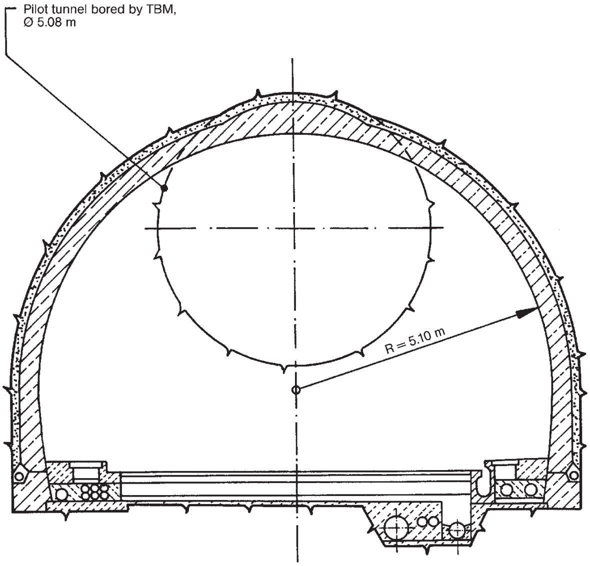
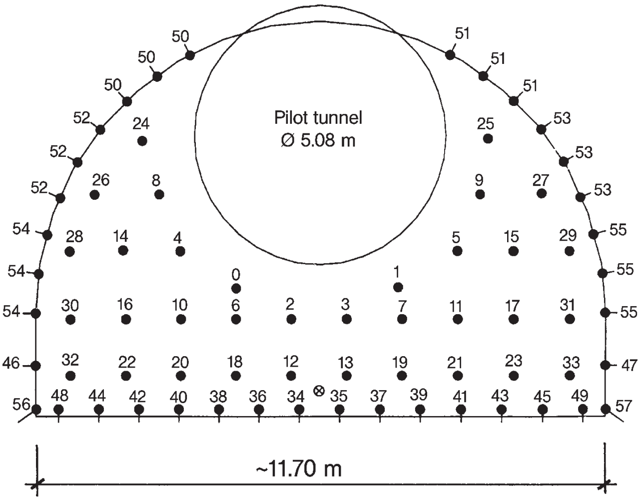


Figure 6-69 Standard cross-section with location of the pilot tunnel, Uznaberg Tunnel [252]

Geological and hydrogeological conditions. The geology of the Uznaberg range of hills consists predominantly of grey, fine to medium grained sandstones of the lower fresh water molasse. Under these lie marl sandstones and marlstones. The hydrogeological conditions showed that because of the partially jointed sandstones, little water was to be expected along the seams, which could have penetrated into the excavated cavity.

Tunnelling concept. Both approx. 800 m pilot headings were driven with a Robbins MK 15 gripper TBM (dia. 5.08 m) in the crown area of the final section of the relevant tunnel bore.

The pilot heading of the west bore was driven from north to south in 50 working days with an average daily advance of 15.7 m. After the TBM had been withdrawn, the probe heading for the east bore was started. Due to optimisation of the working processes, the excavation of the second pilot heading was completed in November 1999 after only 28 days with an average daily advance of 27.5 m. After the breakthrough, the TBM was dismantled so that the enlargement works by drilling and blasting could begin. The blasting pattern and a view of the tunnelling work are shown in Fig. 6-70. The final enlargement to the full size of 80 m² was performed in both tunnels at the same time in full face working, with the enlargement of the east bore being about 100 m behind the west bore.



a)

b)

Figure 6-70 Enlargement of the pilot tunnel to the standard profile, Uznaberg Tunnel [252]
a) Blasting pattern
b) Drilling work in progress

Enlargement at the Nidelbad branching structure, Zürich – Thalwil Tunnel. The Nidelbad underground junction is located close to Thalwil at the end of the two-track main tunnel Zurich–Thalwil. At the junction, two single-lane bores separate without crossing in the direction of Thalwil. The separation is staggered in two branching structures approximately 155 m long, 22.40 m wide and 12 m high. The construction of the caverns with a complete cross-section of 280 m^2 was carried out by drilling and blasting using shotcrete. The 1,376 m long tunnel that splits off from the branching structure in the direction of Thalwil at first climbs and then crosses over the double-track tunnel in a curve and drops down to re-join the 703 m long tunnel bore (in another enlargement) in the opposite direction to create a double-track tunnel to the Thalwil portal.

Enlargement concept. The two-track main tunnel was driven by a Herrenknecht S-139 shielded TBM (Ø 12.28 m) to the contract section border near Thalwil. There, the dismantling of the TBM began at the start of 2000 simultaneously with the start of conventional excavation of the single-track tunnel towards Thalwil and the actual enlargement of the branching structures. Fig. 6-71 shows the plan of the branching structure in the Zurich direction with the position of the cross-tunnel 1 for the drill and blast excavation in the Thalwil direction. It also shows the cross-tunnels 2 and 3, which were necessary for the enlargement of the segment tubes.

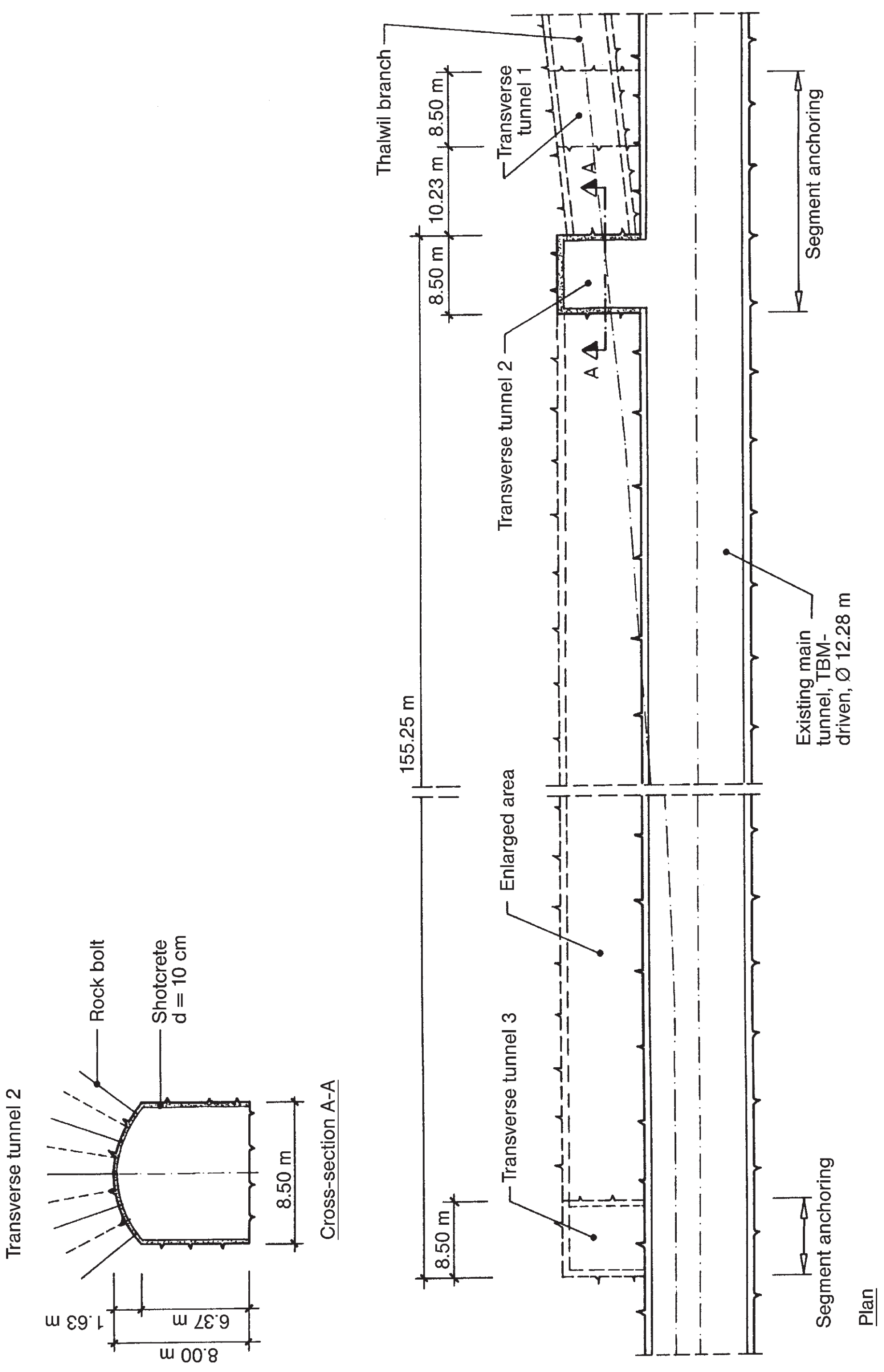


Figure 6-71 Plan of the branching structure in the Zurich direction [280]

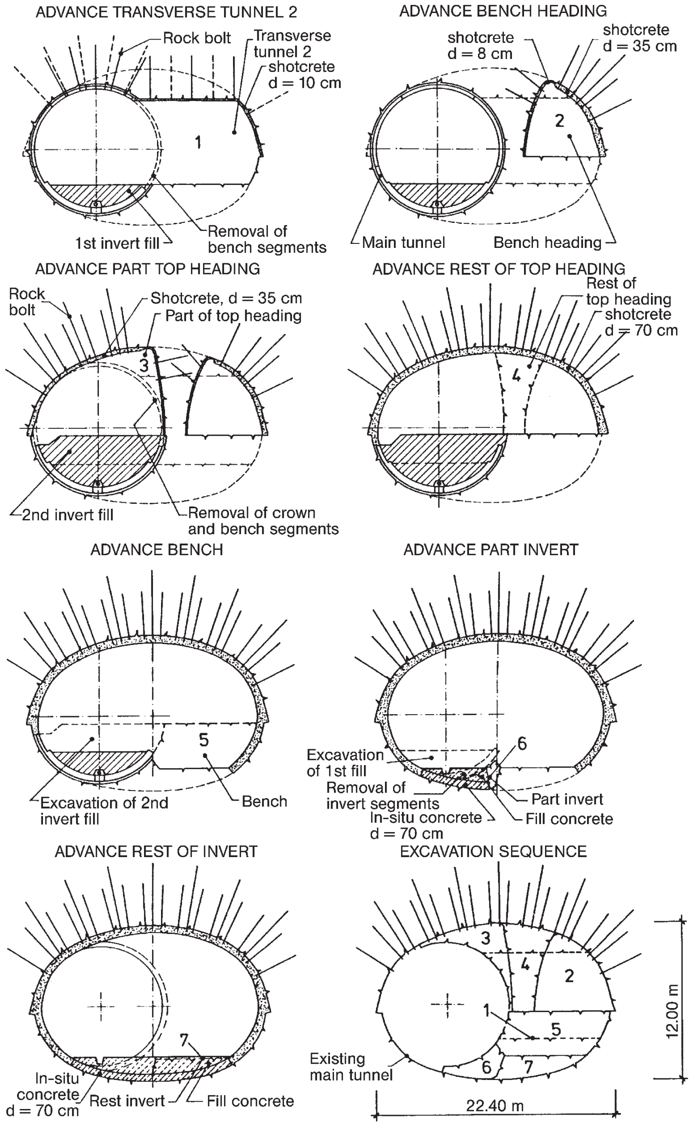


Figure 6-72 Sequence of works to enlarge the bored cross-section from 118 m² to 280 m² [280]

The construction sequence used for the enlargement for the branching structure is shown in Fig. 6-72. First, coarse sand was brought in to fill the invert of the main tunnel in the area of the later cross-tunnel and the roof segments were reinforced with anchors. Then the left side segments were broken out and the invert segments were cut over the entire width of the later cross-tunnel. After the partial removal of the lining, excavation of cross tunnel 2 could begin.

In the next step, as the start of the actual enlargement, a side heading was excavated parallel to the segment tube to cross tunnel 3.

The next step was the further filling of the invert of the main tunnel with coarse sand and the removal in sections of the side and roof segments in the main tunnel, in order to be able to first excavate the partial top heading in the roof area and then the remainder of the top heading.

All partial cross sections were excavated by drill and blast with a uniform round length of 3.00 to 3.50 m. The support in the top heading consisted of a reinforced shotcrete vault, d = 70 cm, with thickened feet.

After the removal of the second invert fill in the main tunnel area, the rest of the bench was excavated. The round length was here also 3.00 to 3.50 m and reinforced shotcrete was applied for support, d = 70 cm.

The remaining fill and then the invert segments were removed from the invert area. Then the invert was excavated in two stages. Cast in-situ concrete was poured to construct an invert vault, d = 70 cm, for support. The concreting pour length was 10 m with breaking out a maximum of 20 m ahead. Finally concrete backfill was poured.

6.6 Roadheaders (TSM) and tunnel excavators

6.6.1 Basic principle of a roadheader

Roadheaders are partial face machines, which excavated the face at a point and excavate the overall profile in multiple strokes. They normally integrate tools for cutting the rock, picking up the muck and transferring it to a device behind the machine, which makes them suitable for working in restricted space. The roadheader is therefore a multi-functional machine, which can undertake the following work in shotcrete tunnelling:

– Excavation of the face.
– Collection of the excavated material.
– Conveyance of the material behind the machine.

Construction. Roadheaders consist of a cutting boom, loading mechanism and conveyance equipment, and can normally move independently on tracks (Fig. 6-73). There may also be additional installations (see 6.6.6).

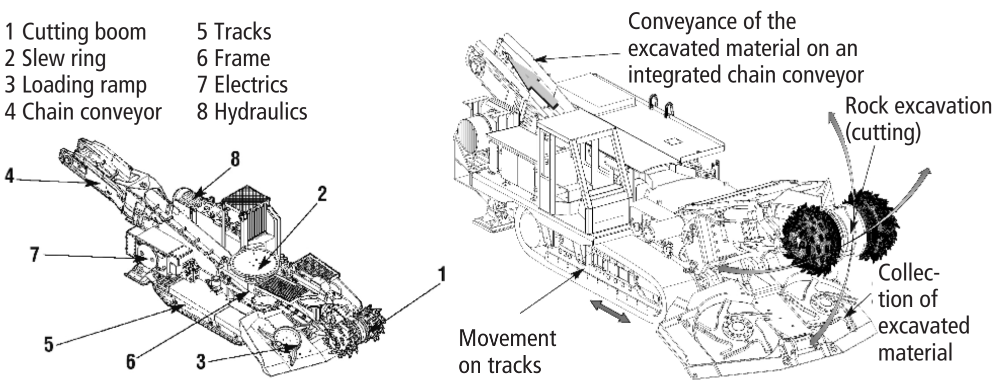


Figure 6-73 System diagram and functionality of a roadheader from Voest Alpine [365]

The equipment to excavate the face consists of a cutting head fitted with picks and a boom. The cutting boom can be slewed and is normally also telescopic in order to be able to attack the entire face.

Machines are differentiated according to their weight and installed drive power. Weights from 20 to over 100 t and installed drive powers for the cutter motor of 50 to 400 kW are usual. The following Table 6-4 shows a categorisation of common roadheaders in four size classes.

Table 6-4 Classification of roadheaders according to weight and cutting head power [178]

Weight class	Weight range	Cutting head power	Shield machines with part-face excavation			
			Normal cutting range		Extended cutting range	
	[t]	[kW]	Cross-section [m²]	Max. σ_c [Mpa]	Cross-section [m²]	Max. σ_c [Mpa]
light	8–40	50–170	≈ 25	60–80	≈ 40	20 – 40
medium.........	40–70	160–230	≈ 30	80–100	≈ 60	40 – 60
heavy	70–100	250–300	≈ 40	100–120	≈ 70	50 – 70
extra heavy ...	> 100	350–400	≈ 45	120–140	≈ 80	80 – 110

General applications. Their suitability for the advance excavation of a top heading round predestines roadheaders for use in shotcrete tunnelling, both with partial face and full face excavation. Roadheaders have the advantage of being able to work well in smaller cross-sections with restricted space in a continuous and cyclical sequence in shotcrete tunnelling. They can also be used in larger cross-sections.

Since the mucking and conveyance equipment is integrated in the machine, it can take up a large part of the available tunnel cross-section. More power can be applied by a heavier machine. The powerful cutting boom can cope with considerably higher rock strengths than a tunnel excavator, which is often the economic alternative in weaker rock.

Another advantage is the reduction of vibration compared to drill and blast and the cutting of the rock without too much disturbance of the surrounding rock mass. Roadheaders can also excavate non-circular cross-sections.

Roadheaders mostly reach their economic limits because tool wear increases over-proportionately with increasing rock strength and in highly abrasive rock. Economic application is therefore normally approximately limited according to [178] to a maximum 120 N/mm² uniaxial compression strength of the rock to be excavated (Fig. 6-74).

New roadheaders are, however, already able to cope with higher strength rock depending on the abrasiveness and other rock properties, according to information from the manufacturers [365].

Since roadheaders can be used very flexibly compared to tunnel boring machines and are also cheaper to purchase, they are just as suitable for projects with prompt starting dates and shorter construction times.

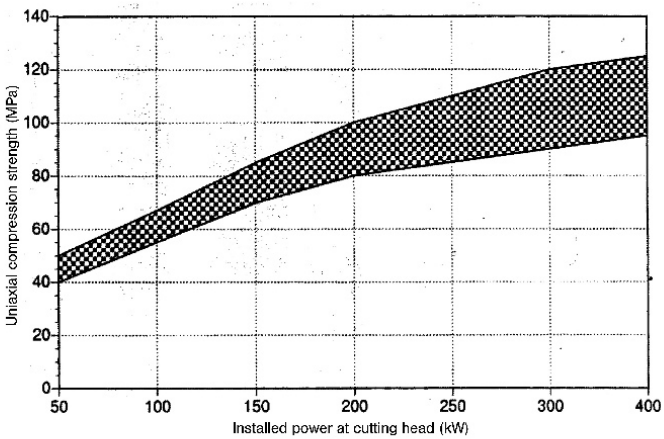
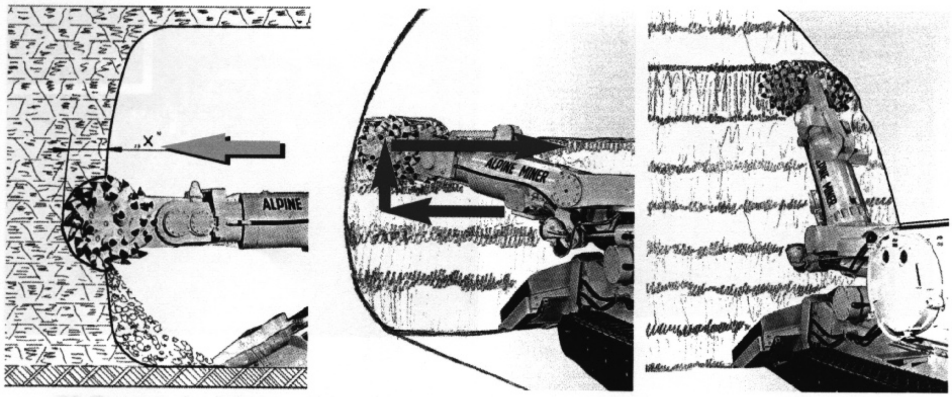


Figure 6-74 Limits to the economic application of modern roadheaders [178]

6.6.2 Rock excavation by a roadheader

In order to excavate the rock, the cutter boom first cuts into the rock face (sumping). Then the boom is slewed and perhaps the machine is also moved to mill the rock face. Finally, the perimeter of the round is profiled with the cutter boom (Fig. 6-75).



1. Penetration 2. Milling 3. Profiling

Figure 6-75 Sequence of rock excavation by a roadheader [365]

One essential characteristic of roadheaders is the arrangement of the cutting head. The first group comprises roadheaders with the cutting head rotating parallel to the boom (Fig. 6-76, left). The longitudinal cutting head works by ripping one side along its entire length, and works with a comparatively slow advance rate. The second group has the cutting head rotating at right angles to the boom axis (Fig. 6-76, right). The transverse cutting head mills the face with only a part of its length but achieves faster advance rates.

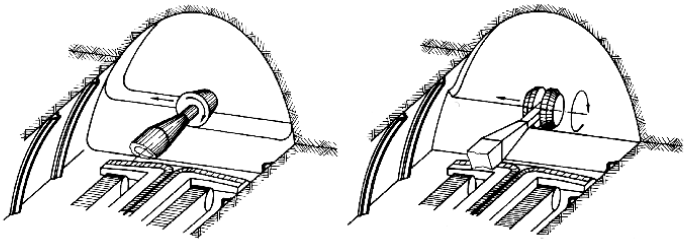


Figure 6-76 Illustration of the cutting of a longitudinal cutting head (left) and a transverse cutting head (right) [254]

Roadheader with longitudinal cutting head. This type has the cutting head rotating about the longitudinal axis of the boom. Experience has shown that machines with a longitudinal cutting head have to be heavier for the same installed power of the cutting head drive due to the reaction forces that have to be resisted by the machine. They can, however, profile the sides of the round and normally show less tool wear in softer rock than transverse cutting heads.

According to the direction of rotation, the cutting can be “progressive” or “degressive” (Fig. 6-77).

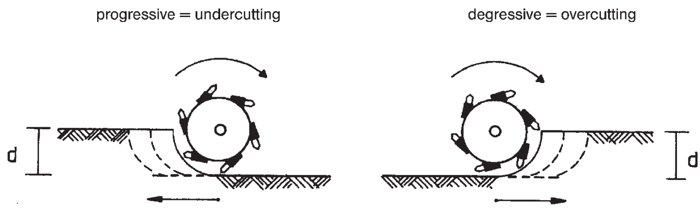


Figure 6-77 Principles of progressive and degressive cutting by a roadheader [88]

In lower strength rock, a longitudinal cutting head can both undercut (progressive) or overcut (degressive) to excavate the rock mass. The cutting direction starting from a central cut therefore runs up/down, diagonally or right/left. In stronger rock, only progressive cutting is possible, so the cutting arm has to perform an extra unproductive movement, which reduces the excavation performance.

Roadheader with transverse cutting head. In comparison, the machine with a transversely mounted cutting head is easier to operate because most of the reaction forces from cutting are resisted by its self weight. However, smoothing of the tunnel sides requires a rather laborious method of working [256]. It is, however, better at applying the weight of the machine to the face for milling. The essential force transfer from cutting with a transverse cutting head is along the machine, which creates a new range of application for roadheaders in stronger rock.

According to the movement of the boom and the direction of rotation of the cutting head, transverse cutting heads can work from above or from below. These principles are illustrated in Fig. 6-78.

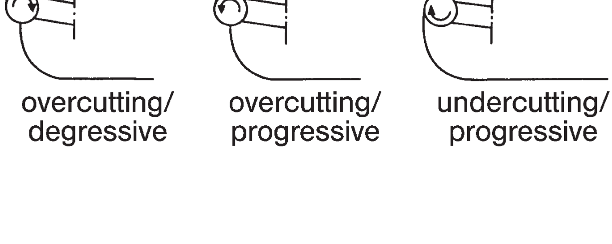


Figure 6-78 Cutting principle of a roadheader with transverse cutting head [285]

Cutter head construction. The successful and economic use of a roadheader depends greatly on the selection of the correct machine (see Fig. 6-88) and type of cutting head. The tools fitted in the cutting head and the type of picks are also important. These decisions depend greatly on the geology encountered and the mineralogical composition of the rock, and the ideal is to test the tools to be used before starting work and make use of a simulation programme for optimisation. In this way, successful excavation with cost-effective pick wear can be planned before the works begin.

The correct fitting of tools in the cutter head can involve, for example, variation of the number of picks, their spacing, arrangement and the tilting and turning angle. Fig. 6-79 shows an example how much an optimised cutting head (right) can differ from the starting situation (left).

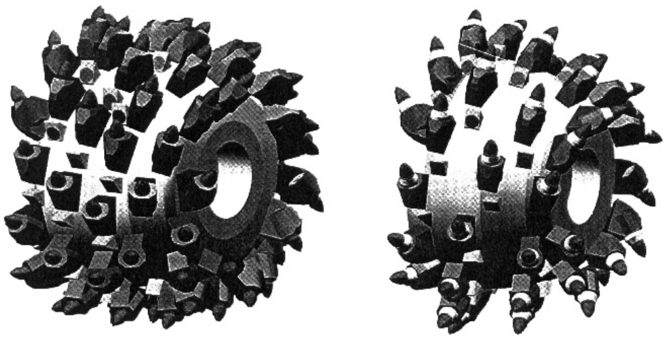


Figure 6-79 Development stages in the computer-assisted optimisation of the arrangement of picks on the cutting head of a roadheader (left: before optimisation; right: after optimisation) [178]

Just as important is the selection of the right construction and material for the picks. Fig. 6-80 shows the range of variation of modern round-shafted picks. These are rotationally symmetrical and cut into the rock in the direction of their axis. The material used is predominantly tungsten carbide of tungsten carbide-cobalt compounds.

The picks and their material have to be matched to the cutting head and the prevailing rock in order to keep wear within acceptable limits. This is particularly difficult in very abrasive rock, which cause heavy wear (mostly due to high quartz content).

In some cases additional measures like internal flushing of the cutting head for cooling and to reduce dust can be sensible.

Loading and conveyance. In order to pick up the material excavated from the face, roadheaders are normally fitted with an apron. According to the type of material to be loaded, the apron can be fitted with gathering discs or stars, gathering arms, chain loaders or lobster claw loaders. These tools pick up the muck from the face and transfer it to the conveyor. While gathering discs or stars are normally used to load dry and homogenous material, lobster claw loaders are also suitable for material with more moisture content and in small or large pieces [233].

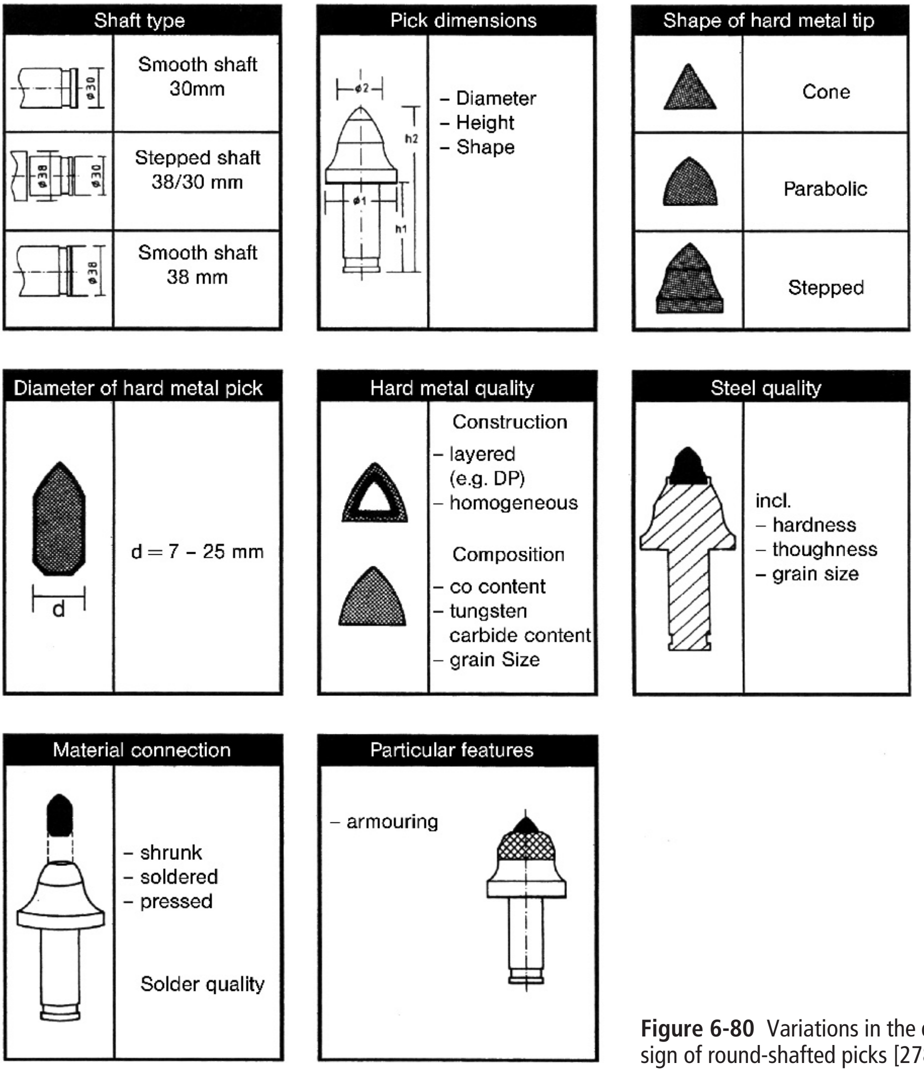


Figure 6-80 Variations in the design of round-shafted picks [278]

Since roadheaders are normally used in restricted space, the muck has to be conveyed behind the machine. This is normally a conveyor belt, which can work on the chain conveyor or scraper conveyor principle. These convey the muck from the apron and either convey it through the middle of the machine or on an external conveyor to discharge behind the machine, where it is transferred to the next link in the transport chain (for example a dump truck). The conveyors are susceptible to wear and are thus expensive components of a roadheader. They should at least be sufficiently large and easy to maintain.

Fig. 6-81 shows a roadheader with lobster claw loaders and a central chain conveyor.

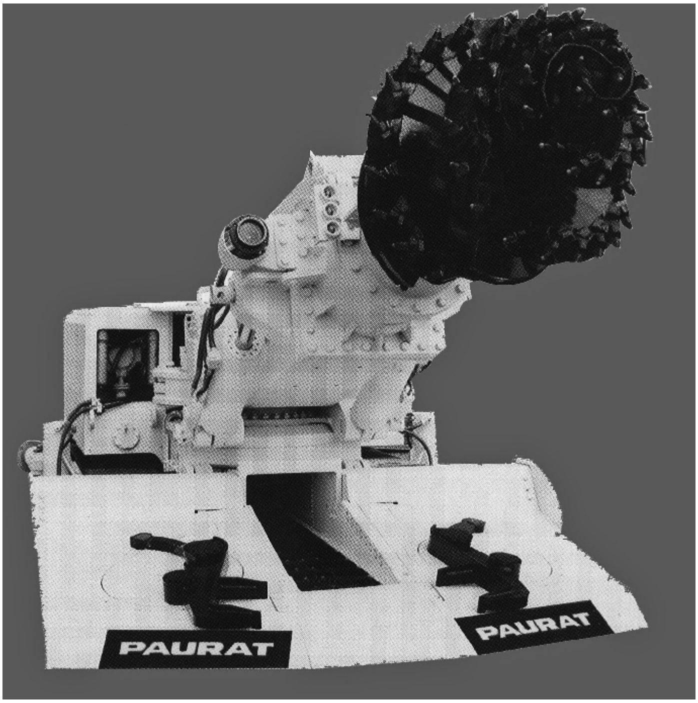


Figure 6-81 Longitudinal cutting head roadheader from Wirth/Paurat with lobster claw loaders and a central chain conveyor [378]

6.6.3 Ventilation and dust control with a roadheader

Ventilation and dust control are much more of a problem with a roadheader than with a full face machine. The excessive production of dust containing quartz during excavation can be a decisive point for the feasibility of excavation by a roadheader. More details about this subject can be found in Chapter 8. It is possible to reduce the problem if as already mentioned the cutting head has an internal flushing water system. Alternatively, a dust curtain effect can be erected through appropriate planning of the ventilation system.

6.6.4 Profile and directional control of roadheaders

It is possible to monitor and automate the control of a roadheader with surveying instruments and computer assistance. Developments in the automatic control of roadheaders serve to accelerate the working sequence, can increase the utilisation of the machine and can have a positive effect on overbreak and working conditions.

Starting from a precise position, the computer restricts the movement of the cutting head to the cross-section to be driven (semi-automatic) or can even control the entire excavation process (fully automatic). The machine operator can see the position of the cutting head in the configured design cross-section on a display and can switch to manual control at any time. The position of the machine is determined by a laser or CCD camera system, but the best precision is offered by theodolite surveying of targets fixed on the machine.

The control system must also record the movements that take place and transmit the data to the computer. The use of such systems can according to [157] determine the position of the machine within 1 cm under ideal conditions.

Further information about current control systems can be found in Chapter 6 of the second volume.

6.6.5 Construction sequence using a roadheader

The method of operating a roadheader can differ greatly depending on the size of the cross-section. In smaller sections (Fig. 6-82), like for example underground railway tunnels, the construction site is purely linear. If the section to be driven is larger, there are ways of organising operations to decouple the works and carry out different actions in parallel.

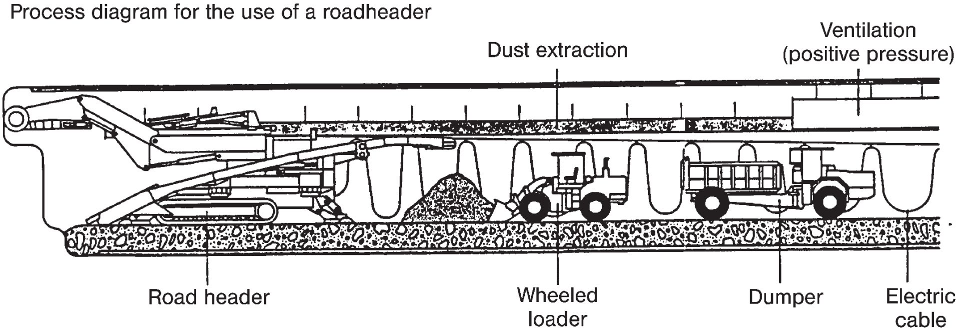


Figure 6-82 Process diagram for the use of a roadheader in smaller sections [375]

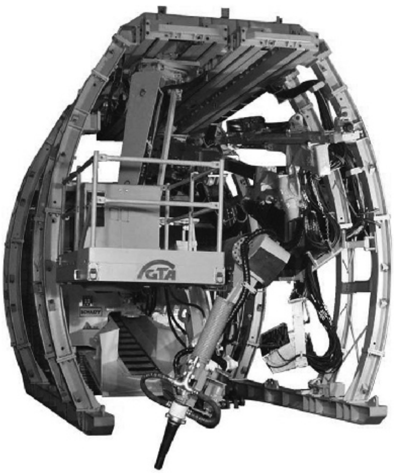


Figure 6-83 Tunnel advance portal TVP from GTA-Maschinensysteme [113]

Figure 6-84 GTA Tunnel advance portal in use on the underground railway contract Line 306, Bochum; Beton- und Monierbau [29]

Special construction processes can be used when the roadheader is operated in combination with backup equipment like conveyor belts or suspension conveyors. Also worth mentioning is the use of a "second working level" to simplify and accelerate the construction process. The space above the roadheader is used for a platform or railway track with, for example, drills, shotcrete robots or working platforms. This results in advantages of easier working, improved safety, better performance and the possibility of parallel working. The disadvantage is the further restriction of the working space for the roadheader, so that such measures do not result in improvement on every tunnel project. This can be the case if the space is very restricted, in case the space for example is required in hard rock for a heavy roadheader, which often fills the available space.

6.6.6 Additional equipment and variations of roadheaders

Additional equipment. In addition to the normal assemblies of a roadheader, numerous attachments have been tested and sometimes used, for example:

- Additional excavator booms to support mucking,
- Integrated drill feeds for the installation of rock bolts or spiles,
- Support aids, like for example arch erectors,
- Shotcrete robots,
- Water spraying to combat dust.

As these attachments also affect the roadheader tunnelling system and can even be obstructive, their application has to be considered for each project.

Variations. Hydraulic cutter attachments (Fig. 6-85 and Fig. 6-86) can be mentioned as variations of the roadheader principle, as they exploit the same excavation principle. These are mounted on a tunnel excavator and ideal for profiling work, for example in combination with drill and blast tunnelling. When they are used for the entire excavation, their performance is considerably worse than a comparable roadheader.

Figure 6-85 Hydraulic cutter attachment after use

Figure 6-86 Hydraulic cutter attachment (Eickhoff) on a tunnel excavator (Liebherr) [89, 184]

Schaeff Tunnel excavators also have some of the advantages of a roadheader, including those discussed in section 6.6.11 (Fig. 6-92).

Another possible application of the roadheader principle is the use of a cutting boom in combination with an open shield in unstable soil conditions. Fig. 6-87 shows the MHSM1 partial face or boom-in-shield machine (Herrenknecht AG), which has a shield diameter of 3.98 m and is driving a new water main for the Munich water utility [128]. The application of the roadheader principle permits tunnelling through isolated hard rock parties. The circular cavity is lined in the protection of the shield skin, normally with segments.

Figure 6-87 Open shield from Herrenknecht with partial face excavation by a roadheader boom (photo: Bilfinger Berger AG)

6.6.7 Criteria for the selection of a roadheader

The various alternative designs of a roadheader and the external influences on their suitability are shown in the flow chart in Fig. 6-88.

6.6.8 Comparison of partial face and full face machines

Roadheaders have the following advantages over full-face machines:

– More adaptable to changing geological conditions.
– Staged advance with advanced face may be possible.
– More adaptable for changing cross-sections, thus more reusable and less direct compulsion by the specification of a contract.
– The face is accessible.
– Support work in the machine area is simple. The machine can be equipped with support aids.
– Combinations are possible with all conventional and modern processes.
– Causes considerably less vibration than drill and blast and full-face machines, although not completely without vibration.
– Quicker mobilisation time, simpler transport to the site and assembly on the site.
– Less investment cost and easier to reuse.

Roadheaders have the following disadvantages over full-face machines:

– The limit to economic application lies at a rock strength of about 100 to 120 MN/m^2 given favourable bedding and machine construction.
– Tool wear in abrasive rock is more problematic.
– Dust development is considerable and the available dedusting equipment is inadequate.

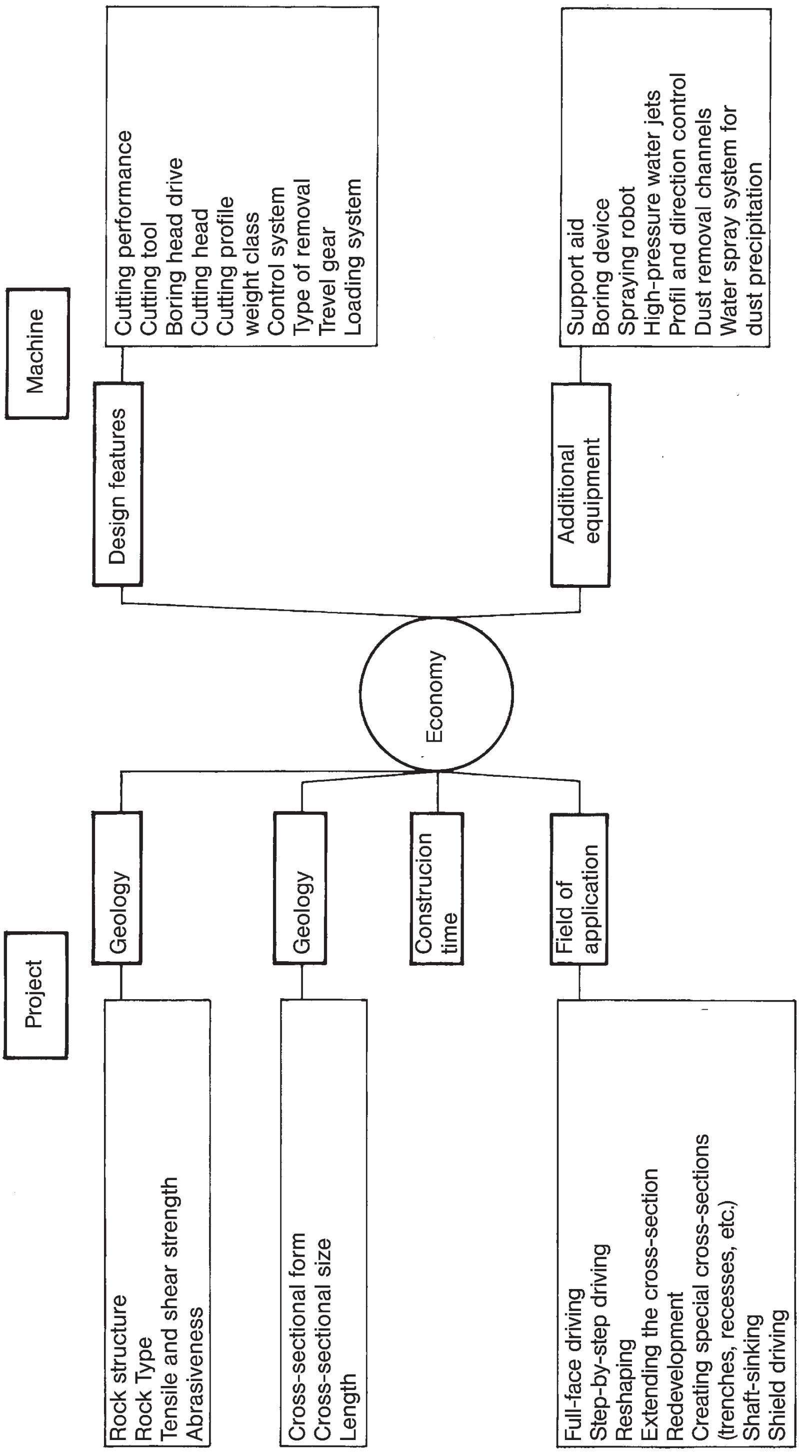


Figure 6-88 Criteria for the selection of a roadheader [233]

6.6.9 Combination of full face and partial face machines

The Mini-Fullfacer represents a combination of full face and partial face machine (Fig. 6-89). In use, there were problems with lack of space for support work in the machine area. The machine has not become established on the market.

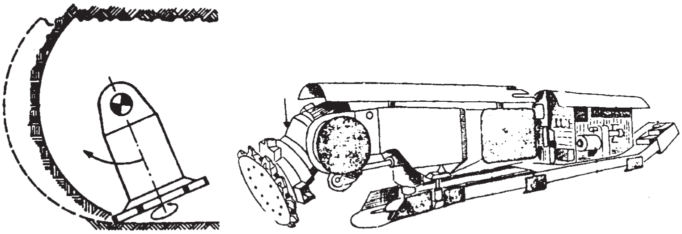


Figure 6-89 Mini-Fullfacer from Atlas Copco GmbH

6.6.10 Contour cutting process

Simultaneous contour cutting and core extraction can be achieved if one or more cutting units are moved while mounted on a portal corresponding to the tunnel outline to produce a contour slot about 1 m deep (Fig. 6-90). At the same time, a machine such as a breaker mounted under the portal can break out the core. The muck pile is gathered by a loading device and transported away. Another possibility is the vertical separation of working areas, for example the contour is cut to a certain depth in the left-hand side of the section while the core is being removed and transported away on the right-hand side where the slot has already been cut. Then the same actions are carried out the other way round. The core with two free sides could also be blasted.

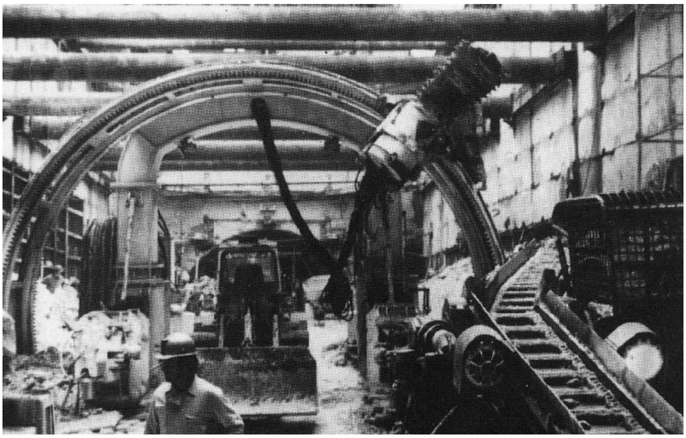


Figure 6-90 Contour cutting process with peripheral saw

6.6.11 Tunnel excavators

Application conditions. Another method of excavating a tunnel is the use of a tunnel excavator (Fig. 6-91). In comparison to road headers, tunnel excavators are more mobile, but they can seldom economically excavate rock with strength of more than 50 N/mm². They are therefore mostly used in soft and soil. Excavators are also used in combination with drill and blast in order to clear the blasted muck pile. Tunnel excavators can only be used when the tunnel cross-section is large enough (about 20 m²).

Figure 6-91 Liebherr 932 tunnel excavator [184]

Construction features. The general construction of a hydraulic excavator is dealt with in section 5.4.2. The tunnel excavator is a form of hydraulic excavator adapted for work in tunnels.

Boom, stick and counterweight are normally shortened in order to provide more manoeuvrability for use underground. Tunnel excavators almost always have heavy-duty tracked undercarriages, giving the following advantages over wheels [354]:

– Low specific ground pressure due to the larger bearing area.
– High stability.
– Good off-road capability and good climbing.
– Good power connection between tracks and ground.
– Robust construction with less maintenance.

Excavators developed especially for tunnelling have to fulfil the following requirements [285]:

– Sideways tipping of the working equipment by 2 · 45°.
– Long reach.
– Location of the hydraulics in the bucket stick in order to avoid damage.
– Full slewing of the superstructure in a tunnel profile of 5.20 m crown to working level and 5.50 m width (corresponds to a single-track underground railway profile with support).
– Improved view out of the drive‘s cabin for reversing.
– Provision of a dozer blade on the undercarriage to push material together.
– Possible fitting of a hydraulic hammer attachment.

As a tunnel excavator essentially works without travelling, it has five degrees of freedom without moving. These five degrees of freedom are the slewing of the superstructure on the slew ring (1), the lifting and lowering of the boom (2), the movement back and forth of the bucket stick (3), the tipping of the bucket (4) and the slewing of the tipping arm (5).

Special forms. The system of a tunnel heading and loading machine combines the muck clearance principle and the compact construction of a roadheader with the better digging performance of an excavator bucket in less solid ground (Fig. 6-92). This machine has an excavator boom mounted on the front of the machine in place of the cutting boom of a roadheader. The working method is similar to that of a roadheader so the machine offers particular advantages in restricted space. The excavator bucket can also be fitted with a hydraulic hammer or a cutting unit in short sections of harder rock (see section 6.6.6) as an addition to or replacement of the excavator boom.

Figure 6-92 Schaeff tunnel heading and loading machine based on the ITC 312 [369]

This machine is also used in drill and blast tunnelling for mucking only. More details of this are described in section 5.4.2.

6.7 Checking the tunnelling machine for suitability and acceptance based on a risk analysis

The tunnelling machine is the core of a construction process and is a tailor-made unit, and should be capable of overcoming all “problems” and obstructions that appear in front of the cutterhead. Conversions and adaptations underground are technically scarcely feasible, and in any case associated with considerable extra cost and delay.

Numerous practical examples however illustrate the difficulty of designing an optimally suitable machine. Long stoppages or the construction of special measures, like for example the sinking of a shaft to change the sealing system or to relieve the blocking of a cutterhead, are not at all seldom.

This situation demands a completely new strategy, for which the individual factors machine, construction process and human have to be optimally suited to each other. Partial aspects of this strategy have been applied in Hamburg and Moscow and form the basis for the holistic strategy presented here for containing risks in mechanised tunnelling, with the experience gained in Hamburg and Moscow already having been taken into account.

6.7.1 Strategy to contain risk

The strategy to contain risk (Fig. 6-93) features the following basic elements:

– Basic design for the sections to be tunnelled with the machine.
– Analysis of obstructions and breakdowns to be carried out for the shield sections in the planning phase, with the obstructions being derived from the local conditions of the project and particularly the risks for the machine and process resulting from geological conditions being recorded and covered.
– Machine specification for the construction of the machine, in which the essential requirements for the machine are assigned process and mechanical solutions.
– “Acceptance” of the shield machine (production phase of the machine).
– Shield handbook for the construction phase.
– Data checks and functional tests.
– Service stations, which enable defined maintenance stops for the machine.

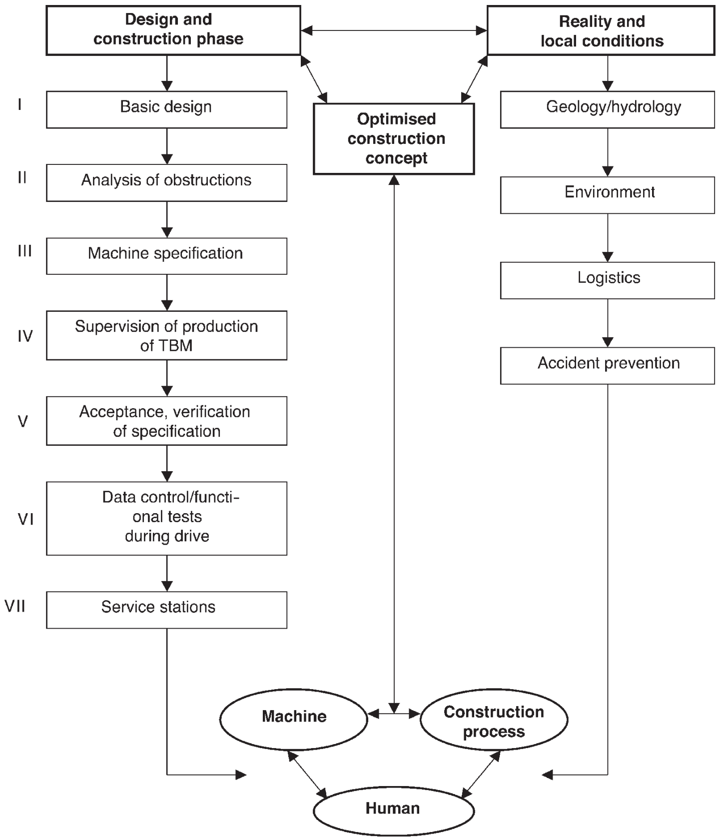


Figure 6-93 Strategy to contain risk on shield drives

6.7.2 Basic design

The aim of the basic design is the production of a reference design (Fig. 6-94), which is intended to represent a design that is feasible for the client and capable of being constructed within the given construction period, and which is also associated with a restricted risk profile for the client. The basis for this is produced in the design phase including consideration of alternative solutions, the risks attached to which are evaluated under the criteria of construction time, cost and feasibility and are then processed into a detailed reference design. This detailed design forms the basis for the requirements for the construction process and the shield machine in the tender documents.

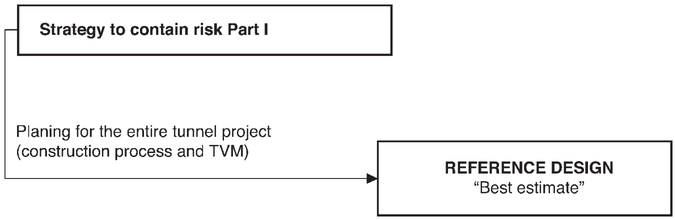


Figure 6-94 Strategy to contain risk on shield drives, Part I: Production of basic design

6.7.3 Analysis of obstructions

The analysis of obstructions (Fig. 6-95) is intended to reveal all foreseeable geological, process-technical and machine-technical obstructions and stoppages as well as obstructions and difficulties affecting the drive as a result of the surroundings. This analysis must be performed before the building of the machine if it is to be useful during the construction phase. In practice, this is often the other way round: the analysis only starts after the occurrence of a problem, too late for mechanical alterations.

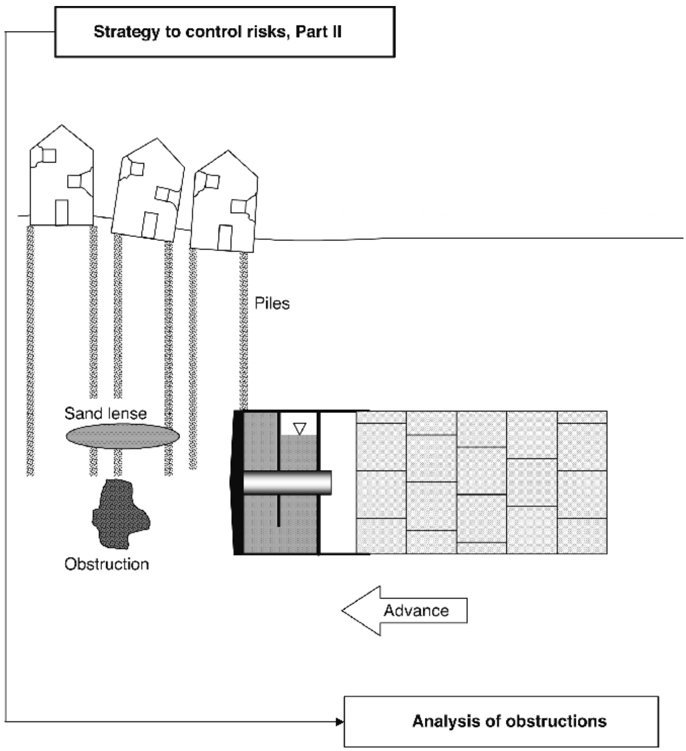


Figure 6-95 Strategy to contain risk on shield drives, Part II: Analysis of obstructions

Working from the question "what could go wrong in this tunnel?", the following points should be included in the analysis:

Geology/Hydrology:

– Are there obstructions? How can these be broken by the cutterhead or will they block the cutterhead?
– Are wear or sticking expected to occur?
– Is there quicksand, which could lead to problems at the face?

Environment:

– How much subsidence can the buildings on the surface accept?
– Will the machine bore through foundations?
– Does the alignment have any tight curves, which demand special solutions?

Machine technology:

– Which mechanical parts could break down due, for example, to wear or overloading? Can this situation be indicated to the driver through instrumentation? How are these machine parts to be changed?

6.7.4 Machine specification

The aim of the machine specification (Fig. 6-96) is to recognise early during the design phase the various operating states, that is tunnelling states, states leading to inspections of the face and possible obstructions and breakdowns, which could occur in the course of the shield drive, to list them and produce appropriate solutions to be directly implemented in the design of the machine and the planning of the construction process. Only in this way can damage to the machine, breakdowns of machine components and damage, for example to buildings, be prevented.

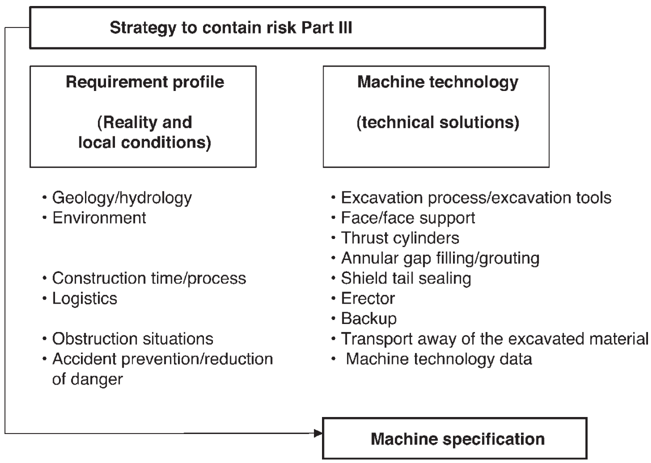


Figure 6-96 Strategy to contain risk on shield drives, Part III: Production of "Machine specification"

The specification is thus a requirement profile for the detailing and building of the machine with the associated solution.

The requirement profile must include the mechanical and technical requirements resulting from geology, hydrology, external conditions, scheduling and logistical requirements, stoppage situations as well as accident prevention and precautions against dangers.

The report to the requirement profile should provide answers, with details of the technical and mechanical solutions. In addition to the basic design and equipment of the shield machine, for example for excavation and face support, special mechanical equipment to overcome obstructions is particularly to be described, for example stone crushers to break boulders, centre cutters to reduce the torque and improve the material flow, advance grouting to stabilise the face and control and monitoring equipment for example for tool wear and other mechanical tunnelling data.

In addition, accident prevention measures are to be detailed in an emergency management plan.

6.7.5 Acceptance of the TBM

Constant supervision of the entire production process by an independent party serves to check that the mechanical and technical solutions stated in the specification are implemented. This includes functional checks, and at the end comes the acceptance of the machine (Fig. 6-97). The final acceptance protocol is to be regarded as a guarantee declaration, which ensures the client of the optimal construction of the machine.

How can problem-free tunnel advance be guaranteed during the construction phase?

The strategy intends as a control element the shield handbook with continuous data control and supervision of the entire tunnelling process with the shield machine.

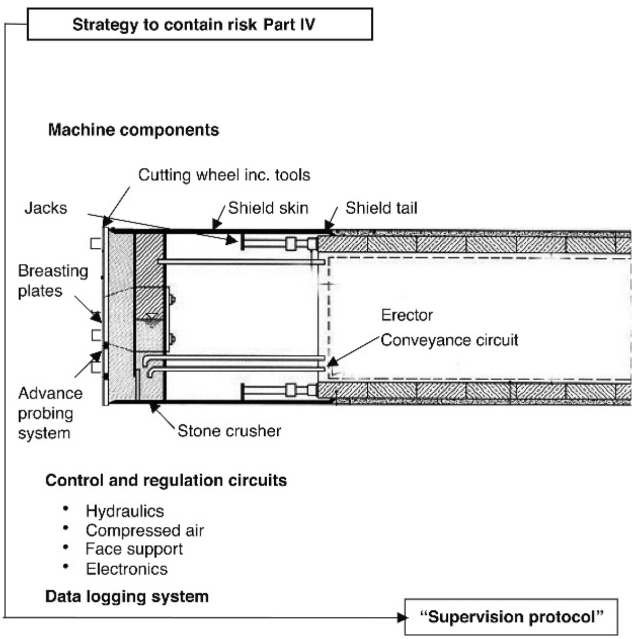


Figure 6-97 Strategy to contain risk on shield drives, Part IV: Production of "Acceptance protocol"

6.7.6 Shield handbook

The aim of the shield handbook (Fig. 6-98) is to encourage the tunnelling crew, that is the tunnel site manager, shift engineers, shield drivers, mechanics, electricians and ring installers to recognise critical situations early and overcome them. The human factor is thus attuned to the ground and the machine.

The following must be described:

– The functioning of the shield machine including its components.
– Working procedures for tunnelling and ring installation.
– Operating states during the advance of the shield, for example slurry support, compressed air support.
– Control and monitoring functions during the advance of the shield, for example face, thrust cylinder forces, annular gap grouting, subsidence, tool wear or oil pressure checks.
– Documentation of the advance.
– Tunnelling beneath works of art (historic buildings etc.), that is measure to reduce settlement.

The obstructions and safety risks identified at the design phase should also be regulated:

– Breakdowns and stoppages, for example the failure of machine components.
– Safety risks as part of the risk analysis for health and safety through emergency measures.

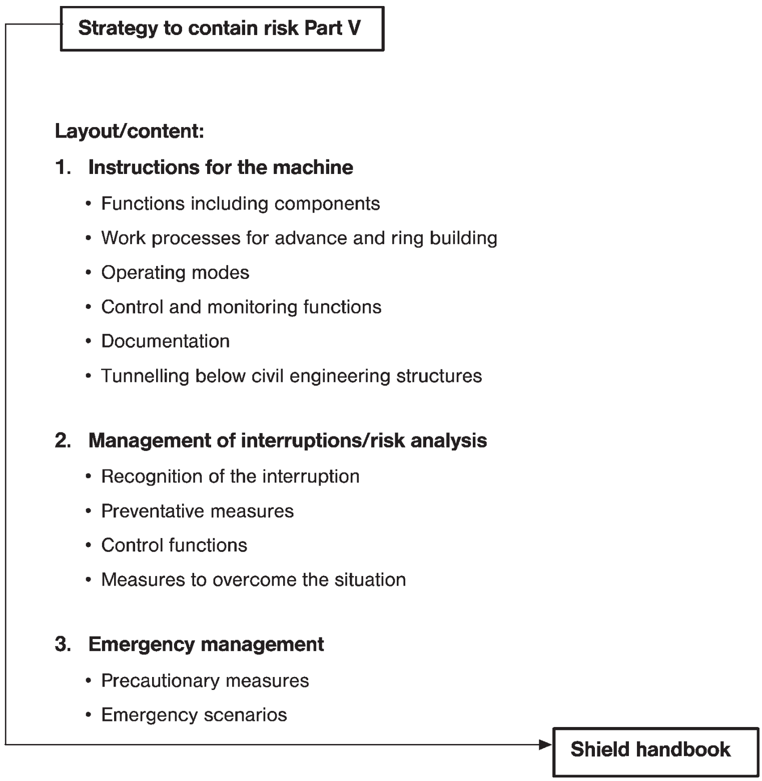


Figure 6-98 Strategy to contain risk on shield drives, Part V: Production of "Shield handbook"

Preventative, control and supervision measures as well as measures to avoid, overcome and remedy obstructions are also regulated in the shield handbook.

The "shield handbook" is thus the requirement profile for the shield drive, that is the construction phase, including addition precautions for the shield machine and additional measures for the drive such as ground improvements in case of unstable ground conditions in order to ensure a problem-free tunnel drive.

6.7.7 Data checks, functional tests

Data checks and functional tests (Fig. 6-99) serve to continuously check the functionality of the machine during tunnelling work. Warning equipment should notify the shield driver promptly of the failure of machine components or errors in the control system, for example pressure loss in the sealing system for face support or thrust force.

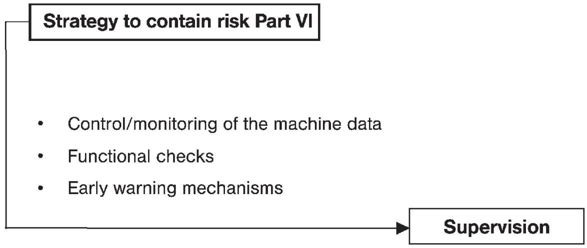


Figure 6-99 Strategy to contain risk on shield drives, Part VI: Mechanical and process "monitoring" during the drive

The successful implementation of the strategy is described below in more detail and illustrated through the examples of the Elbe Tunnel and the Lefortovo Tunnel.

6.7.8 Implementation of the strategy through the example of the Elbe Tunnel and the Lefortovo Tunnel

The construction of major tunnelling projects like the fourth bore of the Elbe Tunnel in Hamburg (Fig. 6-100) or the Lefortovo Tunnel in Moscow (Fig. 6-102) always represent engineering challenges. The completion of both these projects far exceeded the technological limits that applied until recently. This applies particularly to the cross-section of 14.20 m diameter, at the time the largest ever, providing room for up to three autobahn lanes.

Project description of fourth bore of the Elbe Tunnel. In order to increase the capacity of the western Elbe crossing, which then consisted of three bores, a further tunnel bore was constructed between 1995 and 2003 for two lanes and a hard shoulder.

The extremely heterogeneous geology in the alignment consisted of the most varied geological strata of loose to densely bedded sand, mud, riverwash marl with water-filled sand lenses, pebbles, mica silt and inclusions of stones and blocks up to 2 m diameter (see Fig. 6-100). In addition, the shallow overburden (minimum 7 m, maximum 13 m), water pressure of up to 6 bar in the invert and the gigantic diameter of the machine made this tunnel meant that this project was entering previously uncharted territory.

The total length of the shield section was 2,561 m. The shield machine crossed the River Elbe from south to north along a length of 950 m with no restriction to shipping. The tunnel finally passed under the steep north bank. The distance between the foundations of existing buildings and the shield tunnelling machine was sometimes only 9.50 m.

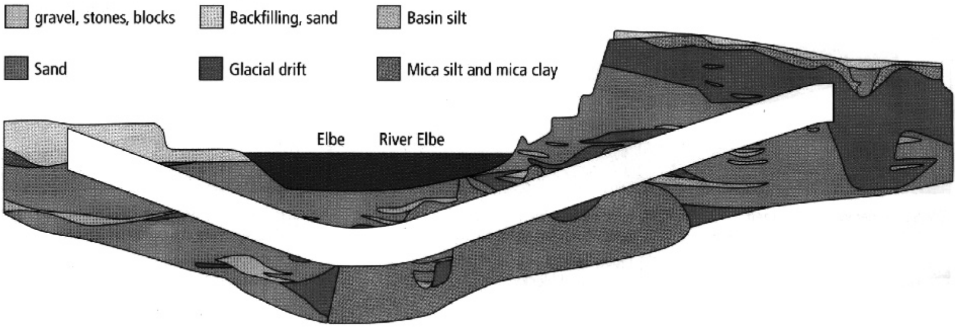


Figure 6-100 Fourth bore of the Elbe Tunnel, Hamburg – vertical alignment [129]

The requirement profile for the geology and surroundings of the fourth bore of the Elbe Tunnel was defined as follows:

1. *Requirement profile, geology*
   - Very inhomogeneous ground (loose to densely bedded sands, gravels, clays, mica silt and riverwash marl).
   - Organic soil areas (mud, peat).
   - Inclusions, for example in the riverwash marl.
   - Sand lenses, also under water pressure.
   - Pebble fields.
   - Large blocks (erratic boulders up to 2 m dia.).
2. *Requirement profile, surroundings*
   - Excavation diameter 14.20 m.
   - Crossing below the river with minimum cover of 7 m, which meant that repair works in the excavation chamber would be complicated by the danger of blowout.
   - Water pressure in the shield invert up to 6 bar.
   - Very heterogeneous ground.
   - Buildings susceptible to settlement.
   - Tunnelling below buildings at a distance of 9 m.
   - Possible artificial obstructions (piers, grouted bodies, large pipes).

The following mechanical solutions were worked out for the specification and implemented during manufacture (see also Fig. 6-16):

- Boulder problem: jaw breaker for 1.2 m dia. boulders.
- Sticking: active centre cutter 3.0 m diameter.
- Repair of stonebreaker: gate in submerged wall.
- Tool changing: accessible spokes of the cutting wheel for tool changing under full support pressure.

- Intervention under full support pressure: three double chamber locks for diver sorties in combination with “shield stations”, which serve as service stations.
- Heterogeneous geology, boundaries of strata, obstructions: seismic advance probing.
- Stabilisation of water-filled sand lenses and increase of surcharge for sealing wall production: drilling channels for perimeter drilling for surrounding protective grouting.
- Necessary longer repairs to the cutting wheel: 100 % closable cutting wheel for the production of a sealing wall.
- Correct, circular installation of the last ring: ring former.

These features adapted the machine to meet the risks entailed in the project.

In addition to the mechanical and technical aspects, responsibilities were clearly defined in order to achieve communication in compliance with the contract for the entire construction phase. In addition, bill items specially intended for the overcoming of interruptions served to delineate and distribute risk.

Fig. 6-101 shows the duties and responsibilities of the various parties involved in the process of implementing the fourth bore of the Elbe Tunnel project. These include, in addition to the classic division between client and contractor, the manufacturer of the shield and particularly the supervisory engineers for shield technology appointed for this project, whose essential task was to check whether the machine specification was appropriate and covered all mechanical, process and safety aspects. The manufacture of the machine was supervised for conformity with the requirements of the machine specification. With the completion of the machine, the client received an acceptance protocol for the machine.

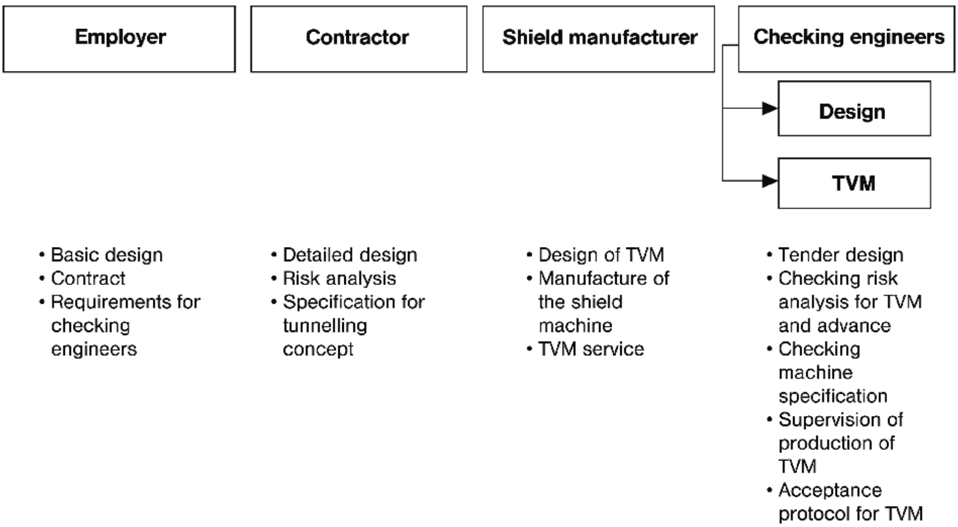


Figure 6-101 Fourth bore of the Elbe Tunnel: Duties and responsibilities of the various parties

Project description of Lefortovo Tunnel. The Lefortovo Tunnel is situated in the east of Moscow and is intended to close the gap in the third motorway ring around the city. The geology consists mostly of sand, silt, clay and thin layers of limestone under a water pressure of 3 to 3.5 bar (Fig. 6-102). The basic concept of the Moscow project essentially corresponds to that for Hamburg, except the construction of the cutting wheel is completely different due to the different ground conditions (Fig. 6-103). This was the mechanical solution to the requirement profile, which can be characterised as follows:

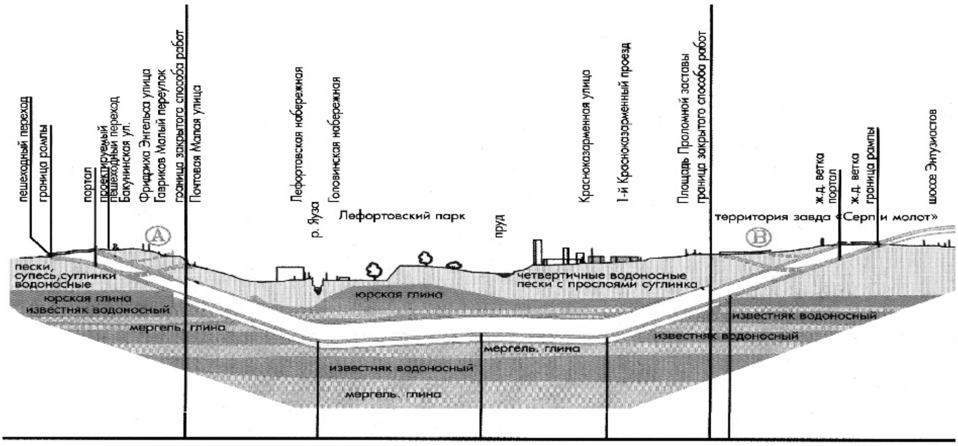


Figure 6-102 Lefortovo Tunnel, Moscow – vertical alignment [284]

1. Requirement profile, geology

- Heterogeneous ground with loosely bedded sands, clays and silts over jointed limestone and dolomite with soft clay layers with some relatively hard limestone.
- Mixed face conditions (soil/solid rock).
- Groundwater under pressure.
- Heavy jointing with a danger of block formation.

2. Requirement profile, surroundings

- Excavation diameter 14.20 m.
- Buildings susceptible to settlement.

The equipment of the machine was adapted to suit the project requirements:

- Block problem: jaw breakers, right-left rotation of the cutting wheel.
- Sticking problem: active centre cutter, rotors and stators, flushing nozzles for the breaker, centre of the machine and excavation chamber.
- Excavation problem: disc cutters and soft ground tools.
- Repairs to cutting wheel: breasting plates.
- Stabilisation of crown: hood grouting.
- Wear problem: wear measurement system, endoscopes.
- Swelling problem: copy cutters.

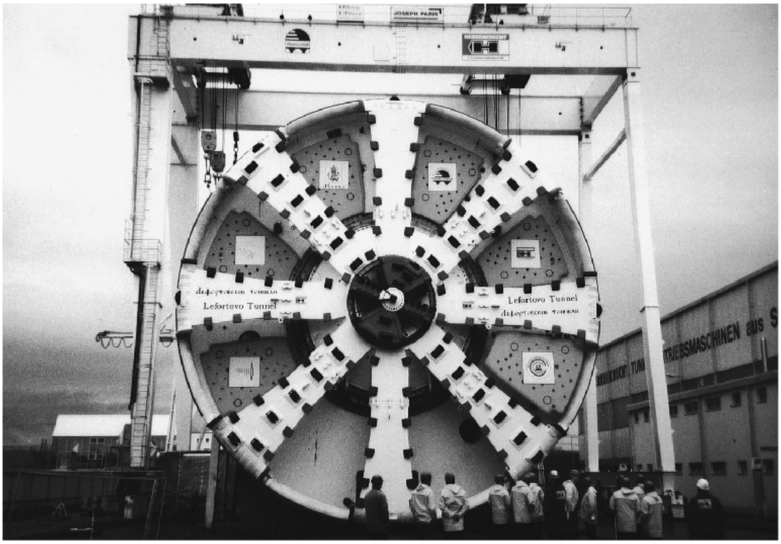


Figure 6-103 Shield machine for the Lefortovo Tunnel, Moscow [212]

Fig. 6-104 shows, analogously to the Hamburg project, the different tasks and duties of the parties involved in the Lefortovo Tunnel, with the difference that in this case the basic design was carried out by the contractor and the risk analysis by the shield manufacturer. These preconditions were not provided by the client. Instead, there was a project description with detailed characterisation of the ground conditions, and a requirement catalogue for the tunnel and the machine. The client, the government of Moscow, was represented by an administrative organisation set up specially for this project, the "OOO-Organiser", which also appointed the supervisory engineers for the design and the production of the shield machine.

Employer	Contractor	Shield manufacturer	Checking engineers
• Contract • Requirements for checking engineers, TVM	• Detailed design • Requirements for the shield manufacturer through Vinci • Tunnelling works	• Design of TVM • Risk analysis for TVM • Manufacture of the shield machine • TVM service	• Planning and design of TVM • Risk analysis for TVM and advance • TVM specification • Production of TVM • Acceptance protocol for TVM

Figure 6-104 Lefortovo Tunnel: Tasks and duties of the individual parties for TBM and tunnelling process

The same principle was applied for the design and construction of the separation plant (Fig. 6-105).

For the Moscow machine, continuous monitoring was undertaken of the production state of the machine, with an official acceptance at the end.

Employer	Contractor	Shield manufacturer	Checking engineers
• Contract • Requirements for checking engineers, separating plant	• Requirements for the contractor through Vinci • Operation of separating plant	• Design • Manufacture • Service	• Planning and design of separating plant • Risk analysis for separating plant • Specification for separating plant • Production of separating plant • Acceptance protocol for separating plant

Figure 6-105 Lefortovo Tunnel: Tasks and duties of the individual parties for the separation plant

The supervisory engineer checked and accepted all technical details including the functionality, which had to be demonstrated in an elaborate functional test of the machine, for example for:

– Cutting wheel functions, turning and closing of the cutting wheel, extension of the copy cutters.
– Breaking of blocks of rock.
– Extension of the breasting plates.
– Transport and installation of a segment as a simulation.
– Installation and testing of drills.
– Demonstration of the functioning of data monitoring including control functions in case of an interruption with the consequences, for example switching off motors, warning systems.

The extensive data checks are collected in the machine in a data management system and are capable of optimised monitoring and external checking of the functioning of the shield machine and the entire tunnelling process as it proceeds.

The tasks exercised by the supervisory engineer for the Lefortovo Tunnel project are summarised in Fig. 6-106, which can be seen as a recommended basis for future projects with similar complexity and comparable degree of difficulty.

Only after the machine had been technically accepted by the supervisory engineer at the works of the manufacturer was it delivered to Moscow.

Checking	Supervision	Recommendations	Acceptance protocol
• TVM planning and functions • Planning and functions of separating plant • Machine specification for TVM • Functionality of TVM and separating plant	• Production phase of the TVM • Production phase of the separation plant • Functional tests of the TVM and separating plant	• Structure of specification • Structure of shield handbook	• Functional testing of TVM and separating • Final acceptance of TVM and separating plant

Figure 6-106 Lefortovo Tunnel: Main tasks of the supervisory engineer

6.7.9 Recommendations for the future

The tunnelling works in Hamburg were completed without major technical problems, as the machine concept was ideally suited to the requirements. The same applies for the Lefortovo Tunnel project. Both projects are good examples optimal design and adaptation of the machine as well as supervision.

For the future, it is to be hoped that the strategy or rather the philosophy described here will also be put into practice on other projects. The intensive examination of the project, the production of mechanical and process requirements for the machine based on a machine specification and the acceptance of the machine, as well as the shield handbook for use during the construction phase, are an essential contribution to the minimisation of risk during a shield tunnelling project.

Underground works in the future should still continue to happen solely underground as a collaboration between the factors ground – machine – human.

7 The driving of small cross-sections

7.1 General

The construction of small cross-sections for utilities without trenching, called trenchless or no-dig construction [77], has achieved great importance today. Developments in Germany and Japan have led to the situation that it is now technically possible in suitable ground conditions to install underground drains and supply pipes in the exact position under inner cities with diameters from 45 mm and distances of up to 1800 m using various processes [342]. The process that was previously before the introduction of suitable horizontal boring equipment was to drive an oversize underground heading, but this is now only be used for special cases. Some applications of small cross-sections are water mains, drains and sewers, gas, district heating and cables.

Trenchless construction is dealt with separately according to DIN EN 12889 as manned or man entry and unmanned or non-man entry processes. Unmanned processes are described as "Process without the presence of personnel at the face during the drive" [77]. An overview of processes in current use is given in Fig. 7-1.

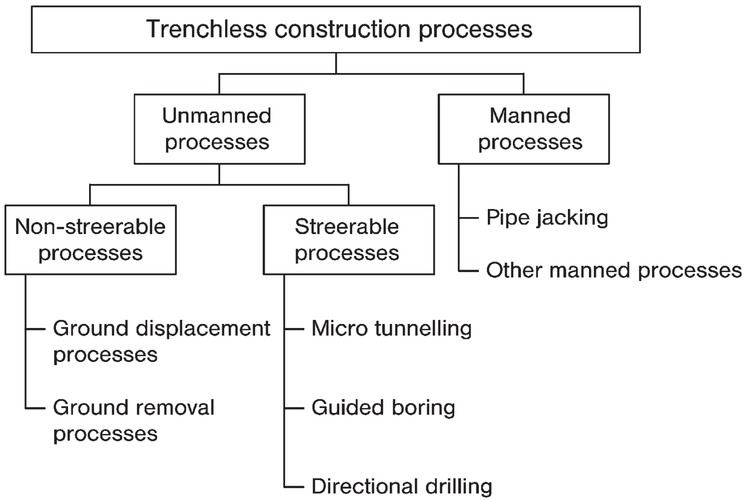


Figure 7-1 Categorisation of trenchless construction processes in DIN EN 12889

7.2 Manned processes

7.2.1 General

Whether it is possible to work at the face of a trenchless construction process depends on the diameter of the pipe to be installed. In the DWA A-125, the following minimum internal diameters are laid down for personnel access during the drive:

As a rule: internal diameter ≥ 1200 mm

– For driven distances ≤ 80 m or in case the section is accessed through a working pipe (internal diameter 1200 mm) of at least 2 m length: internal diameter ≥ 1000 mm

The regulations of the Tiefbau-Berufsgenossenschaft (the German accident insurer for civil engineering) also apply to trenchless construction below ground. These prescribe in the regulation BGV C 22 (formerly VBG 37) the following minimum dimensions for the working space in tunnels, headings and jacked pipes:

For distances of less than 50 m:

– Circular cross-section: 0.80 m diameter.
– Rectangular cross-section: 0.80 m height, 0.60 m width.

For distances from 50 m but less than 100 m:

– Circular cross-section: 1.00 m diameter.
– Rectangular cross-section: 1.00 m height, 0.60 m width.

For distances of 100 m and more:

– Circular cross-section: 1.20 m diameter.
– Rectangular cross-section: 1.20 m height, 0.60 m width.

The clear opening is defined as the free cross-section not obstructed by installations. If this opening is not available, then personnel access is only possible under special restrictions, even for maintenance or repair work.

Construction processes for the creation of underground cavities have undergone significant development in all areas of underground construction. For cross-sections above a size, in which a man can work, there are no special methods of tunnelling for the installation of pipes, so the same tunnelling methods can be used for the installation of supply and drainage pipes as for transport tunnels. All the tunnelling processes described in the previous chapters can in principle also be used for the accessible construction of smaller diameters.

Drill and blast, shield and mechanised tunnelling have however already been described in detail, so only pipe jacking is dealt with here in more detail.

7.2.2 Pipe jacking

General

In pipe jacking, special pipes are jacked forward through the ground from a thrust shaft (entry shaft, launching shaft) using hydraulic cylinders. The excavation of the ground at the face (mechanical or manual) and the removal of the excavated muck through the jacked pipe occur simultaneously (Fig. 7-2). The pipe arrives at a reception shaft (exit shaft, target shaft).

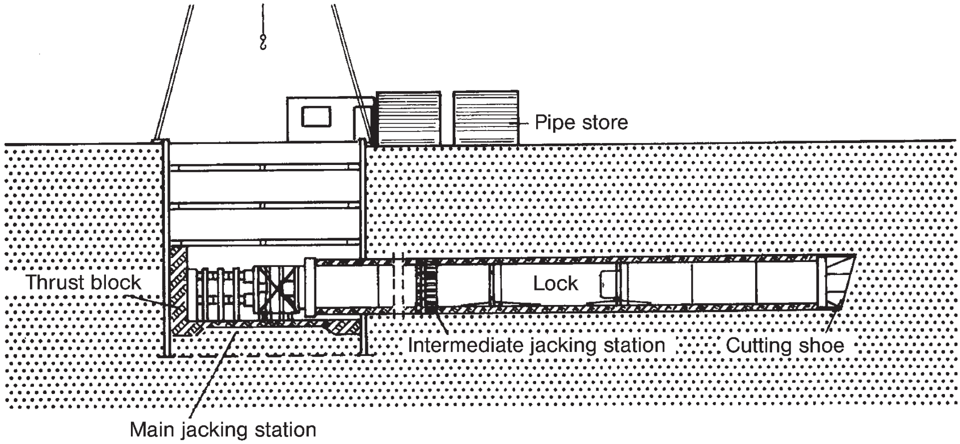


Figure 7-2 Principle of pipe jacking

The technical success of an underground pipe jack depends greatly on the collaboration of the individual parts of the mechanised system: steering shield, intermediate jacking station and main jacking station should all be matched in their dimensions, forces and advance rates. The equipment for excavation, loading and mucking is normally integrated into this system. Pipe jacking offers the following advantages and disadvantages:

Advantages:

- Long jacking distances up to about 15 km.
- Good target precision because steering is possible.
- Alignments with horizontal and vertical curves are possible.
- Single-pass and two pass construction is possible: i. e. direct installation of the product pipe or two-step installation of a casing pipe followed by the product pipe.
- Can be used in almost any ground conditions even under groundwater pressure.
- High degree of mechanisation.

Disadvantages:

- Relatively large drive and reception shafts are required.
- It may be necessary to construct a thrust wall in the drive shaft.
- Limitation of curve radius depending on the nominal diameter.
- Special pipes are required to bear the high edge stresses.
- The longer the pipe jack, the slower it is since more intermediate jacking stations are needed.
- Large diameter pipes are difficult and expensive to transport; it may even be necessary to produce the pipes in site.

Steering shield. One essential component of steerable pipe jacking is the steering head or shield, which is positioned at the front of a jacked pipe string (Fig. 7-3). This has the following purposes:

a) to cut through the ground so that the following pipe string can be jacked forward with a minimum degree of subsidence and as little skin friction as possible;

b) to support the opened cavity against ground pressure until the jacked pipes can finally resist all loads and forces;

c) to support the face against collapsing ground;

d) to steer the pipe run along the intended horizontal and vertical alignment and comply with the permissible deviations.

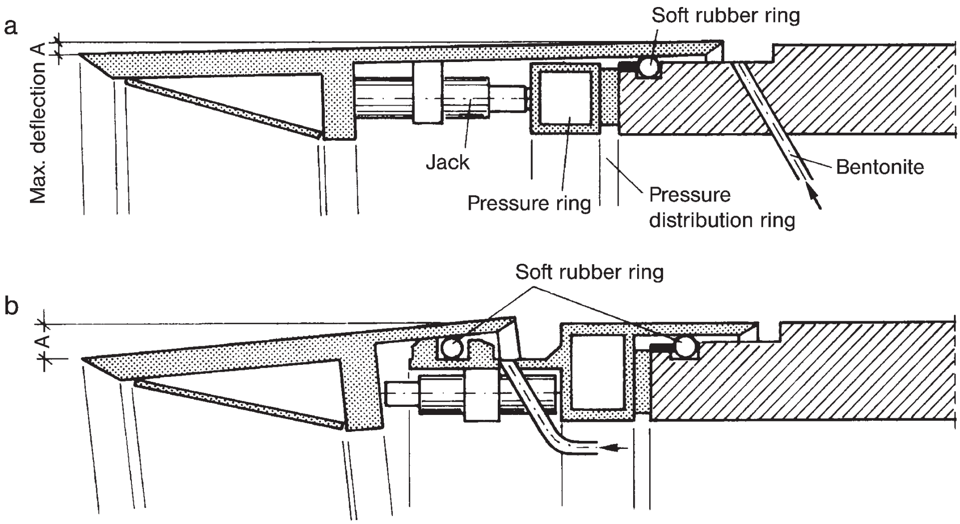


Figure 7-3 Construction of a one-piece (a) and two-piece (b) steering shield [311]

The steering shield is normally similar to a shield in tunnelling, although there are numerous construction types depending on the method of excavation, support of ground pressure and support of water pressure. A steering shield is driven forward and steered by a number of hydraulic cylinders located around the perimeter, which push backwards against the pipe string. It is possible to drive round curves by asymmetrical application of the cylinders.

Scope of application. At the moment, circular cross-sections are used with scarcely any exception [342]. The limits to the scope of application are essentially set by the pipe diameter, weight and the length to be driven.

The minimum pipe diameter required results from the accessibility requirements of the cross-section.

The maximum possible pipe diameter is about 5.4 m outside diameter for pipes that are delivered to the site and the maximum permissible weight is about 50 t [118]. Pipes made on site are only limited by the available lifting capacity to about 100 t [18]. When pipes are made on site, good dimensional accuracy and smooth external faces are important.

Long distances worsen the cost-effectiveness of the process, as the increasing number of intermediate jacking stations increases the cost and slows the advance rate [18].

Pipe jacking is possible in dry and also wet ground. As the process can be controlled, curved vertical profiles are also possible. 2000 mm diameter reinforced concrete pipes have been installed with curves of only 95 m radius. [361]. According to [341], the radius of curves should not be less then the values given by the formula:

$$R = \left(\frac{L^2{}_{\mathrm{Rohr}}}{16 \cdot \ddot{u}} - \frac{1}{2} \cdot D_A \right) \cdot \frac{1}{1000}$$

where:

$L^2{}_{\mathrm{Rohr}}$ = the pipe length [mm],

ü = the overcut [mm],

D_A = the outside diameter of the pipe and

R = the curve radius

Working sequence. Pipe jacking is carried out from a thrust shaft, which houses the jacking equipment and is provided with a thrust bearing constructed at the back (see Fig. 7-2). The first pipe is equipped with a shield to reduce the resistance to penetration. The shield is normally capable of being steered. The jacking itself is performed with a jacking machine installed in the thrust shaft, with the force being transferred to the pipes through a steel pressure distribution ring. Intermediate jacking stations can be provided to enable staged jacking of sections of a pipe run, which makes total jacking lengths of over 1,000 m possible without intermediate shafts [131]. The working sequence is illustrated in Fig. 7-4.

The forces required depend on the soil type, pipe diameter, pipe material, pipe length, vertical alignment, any necessary steering corrections and ground surcharge. In order to reduce the resistance to jacking, which is the sum of leading edge resistance and skin friction, bentonite suspension is injected between the pipe and the surrounding ground (see Fig. 7-3). This lubrication reduces the skin friction by about 50%.

In water-bearing ground, the possible solutions are groundwater lowering, open dewatering, slurry shields, earth pressure shields or working under compressed air. Compressed air locks can be installed in the thrust shaft or in the pipe itself.

Excavation and mucking. The excavation of the ground is increasingly mechanised today, with full face and partial face both being used. The muck can be removed without rails, on rails, on a conveyor belt or hydraulically (Fig. 7-5). The following three examples illustrate the use of tunnel boring machines in pipe jacking:

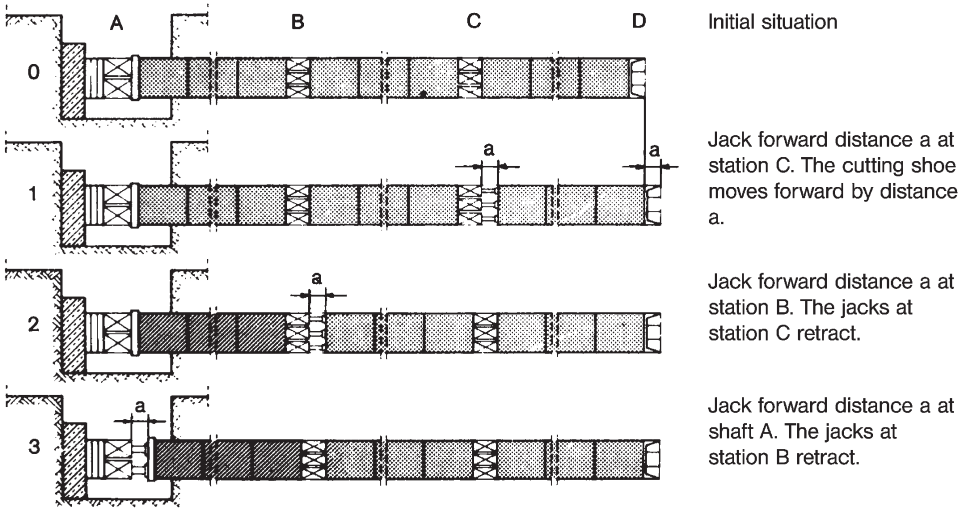


Figure 7-4 Staged jacking with main and intermediate jacking points

For the construction of 1,400 and 2,000 mm dia. collector sewers for the city of Aachen, the tunnelling system shown in Fig. 7-5, top, was used. The ground is excavated by a hydraulic excavator arm, which can be individually controlled according to the conditions at the face. The muck is fed through the shield opening onto a conveyor, which is constructed as a bunker belt with a capacity corresponding to that of the muck trucks pulled by hydraulic winches. The advance rate was an average of 7 m/d, with a peak of 11 m in very difficult and heterogeneous ground.

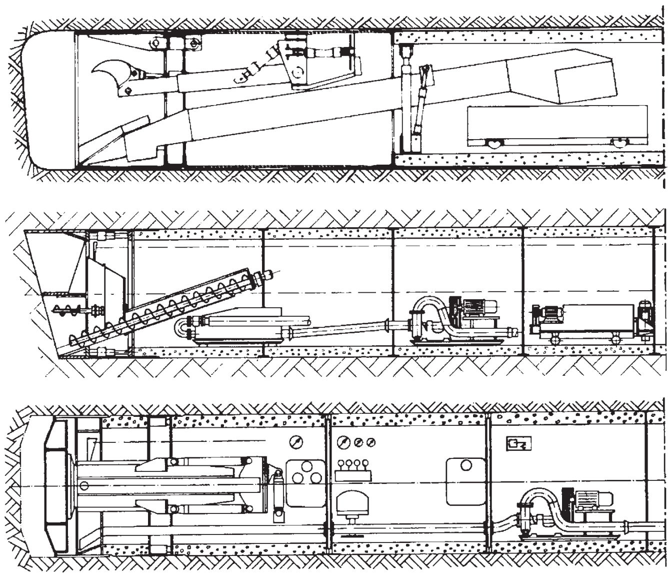


Figure 7-5 Excavation and mucking in pipe jacking. Top: with excavator boom and conveyor belt. Middle: with excavation auger and hydraulic pipeline transport. Bottom: with full-face tunnel boring machine and hydraulic pipeline transport

For the construction of a 1,300 mm dia. collector sewer for the city of Hanover, the groundwater was lowered using gravity wells to below the level of the pipe invert (Fig. 7-5, middle). An auger excavated the soil and transported it to a transport skip, with further transport being hydraulic. The excavation auger is mounted like a Cardan shaft parallel to the pipe axis, which makes it possible to reach every point of the face. The average advance rate was over 17 m/shift, with a peak rate of 26.5 m/shift.

For the construction of a 2200 dia. flood relief tunnel, the face was excavated by a full-face machine (with compressed air support in a stretch of terrace sands). The ground consisted of sedimentary rocks interrupted by a zone of water-saturated terrace sands. The partial functions of the tunnelling system interact with each other as follows: the shield is a composite construction of steel and reinforced concrete, an exceptional construction for a tunnel boring machine (Fig. 7-5, bottom). It consists of the drive unit with cutting wheel and the supply equipment, which is mounted in the reinforced concrete pipe behind the shield. The drive unit can be moved along the shield. While boring solid rock, the machine supports itself from the ground. The cutting wheel is rotated and pressed against the prevailing ground by the drive unit, which is pushed out of the shield by hydraulic cylinders. In this way, the full-face machine can bore a stroke of 50 cm. Then the drive unit is retracted and the entire pipe run is jacked forward.

Pipes for pipe jacking can be made of reinforced concrete, steel, stoneware, concrete, steel fibre concrete, fibre cement, ductile cast iron, glass fibre-reinforced plastic (GRP), polymer concrete, PE or PVC according to DWA A 125. The most commonly used are reinforced concrete or steel fibre concrete jacking pipes according to DIN EN 1916 as well as stoneware jacking pipes for smaller diameters. When used for sewer construction, these can be protected internally by a corrosion protection layer, e. g. by a HDPE inliner or by using polymer concrete.. The length of the pipe depends on the nominal diameter and the material, with lengths between 2.5 and 5.5 m being usual. Instructions for the determination of the loads and section forces and moments including many pipe construction details can be found, for example, in [48, 61, 311 and 342].

7.3 Unmanned processes

7.3.1 General

This section deals with the jacking of smaller non-accessible sections to install utility and sewer pipes in inner cities. These pipes were formerly laid almost exclusively by trenching: a trench was excavated, and the pipes or cables were laid in the protection of a batter or a trench support system; then the trench was backfilled. If the ground was water-bearing, this also entailed dewatering measures, the effects of which on adjacent buildings and vegetation are well known.

Considering these disadvantages and the growing environmental awareness of the population, it is now the case that trenchless methods of pipe laying offer a realistic alternative. Economic influences are also significant since the costs of construction works also include the ancillary costs resulting from environmental impact or its reduction. These include:

- Noise, vibration and emission nuisance caused by construction works and traffic diversions.
- The effects of dewatering measures on neighbouring buildings and vegetation.
- Energy consumption, turnover and working time lost due to traffic diversions.
- Safety risks for the local population.

Unmanned trenchless construction methods can be divided into steerable (guided, directional) and non-steerable processes.

7.3.2 Non-steerable processes, or with limited control of direction

These processes can be categorised into ground displacement and ground removal processes.

Table 7-1 Process and functional principle of non-directional processes, or with limited control of direction

Ground displacement processes		**Ground removal processes**	
Impact moling	Driving of an independently advancing impact mole, displacing the soil; the product pipe can be pulled simultaneously or subsequently.	Pipe ramming/ jacking with open end	Ramming of a casing pipe open at the end with simultaneous or subsequent soil removal.
Pipe ramming/ jacking with closed end	Ramming of a casing pipe closed at the end from the launching shaft, displacing the soil.	Auger boring (also with limited control)	Driving of a drill stem by jacking with simultaneous excavation of the soil at the face by a cutting head; continuous muck clearance by an auger.
Guided boring with reamer	Ramming of a rigid casing pipe, displacing the soil. The pipe is installed subsequently by pulling or jacking behind a reamer.	Hammer drilling	The drill bit is driven by a hammer drill and the soil is removed mechanically, hydraulically or with compressed air.
Pipe bursting	An existing pipe is burst from inside and displaced together with the soil by a reamer, which follows the pipe alignment.	Pipe ramming with expansion cone	Displacement of the soil by ramming a rigid casing pipe. The product pipe is subsequently pulled in behind a rotating expansion cone.
Pipe pulling	An existing pipe run is removed by pulling or jacking and simultaneously replaced with a new one.		

The limits to the scope of application result from the pipe diameters that can be driven, the comparatively shorter jacking distances and less precision as well as the limitations of geological and hydrogeological conditions.

Ground displacement processes

General. In displacement processes, a circular cavity is formed by a displacement head, mole or drive shoe, which is driven into the ground statically or dynamically. The soil in this case is not excavated but displaced around the hole. After the cavity has been created, casing pipes or product pipes are pulled or pushed in immediately.

Displacement processes are relatively simple to operate and can achieve a relatively fast advance rate with little cost and effort. This process has not become established for drain and sewer pipes because the maximum practical diameter is normally only 200 mm, or 300 mm in case a reamer is used, with distances of 25 m [18]. This effectively limits the application of this method to house connections, cable ducts and cables [379].

Machines that work on this principle are the impact mole, horizontal rammers with closed-end pipe, auger boring and pipe eating and pulling.

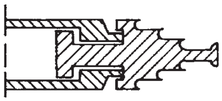
Impact mole.

The impact mole, also called a piercing tool, is in principle a torpedo-shaped pile, which is driven through the ground by hammering.

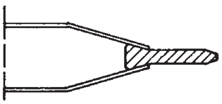
Instead of the hammer is an impact piston, which reciprocates inside a housing (Fig. 7-7). The piston is driven by compressed air at a pressure of 6 to 7 bar, impacts a sliding head (chisel head), which opens the necessary cavity in the soil to pull the cylindrical housing through. Some examples of different head types are shown in Fig. 7-6). The impact of the percussion piston is converted into fracture energy and displacement work. The reaction impact caused by the acceleration of the piston in the housing has to be resisted by skin friction. In this way, a hole with compacted and stable sides is created. The product pipe can be pulled into this hole simultaneously with the moling or after completion.

Impact moles can be used in dry or damp soil in soil class L according to DIN 18319 to produce straight holes with a diameter of up to 200 mm. The precondition is compressible soil with loose consolidation. Although this device can split stones and pebbles without difficulty, the soil to be penetrated should ideally be uniform and free of obstructions, as inhomogeneous conditions increase the risk of deviation. Ideal soils for impact moling are mixed-grained soils with low cohesive content and a maximum grain size up to 60 mm. In contrast, densely consolidated single-grained soils can only be further compacted and thus displaced by destruction of the grain particles, which explains why impact moles are no longer successful in single-grained sands with a grain size of 0.05 to 0.2 mm diameter and dense consolidation.

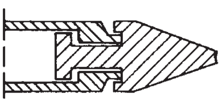
Another important aspect that has to be considered for impact moling is the shrinkage of the opened hole. The more cohesive the soil is, the greater is the degree of shrinkage, in this case the reduction of the hole diameter behind the impact mole. This process is of great significance when the casing or product pipe is pulled through, as the hole diameter can shrink by up to 5 to 15 % in some types of soil.



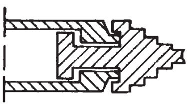
1. Blunt head (Terra AG für Tiefbautechnik): for sand and gravel soils with cohesive content and in soil types with boulders.
   Directional stability: good
   Speed: slow



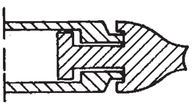
2. Staged head (Terra AG): for sand and gravel soil with little skin friction but high tip resistance.
   Directional stability: good
   Speed: good



3. Conical head (Terra AG): for homogeneous, fine-grained soil types without boulders.
   Directional stability: good
   Speed: good



4. Conical head with chisel tip (H. J. Essig): for all soil types up to soft rock.
   Directional stability: poor
   Speed: very good



5. Staged head with cutting edges (Tracto-Technik): in cohesive soil, the steps fill up resulting in high speed; if a stone is encountered, it is destroyed by the axial cutting blades.
   Directional stability: very good
   Speed: good

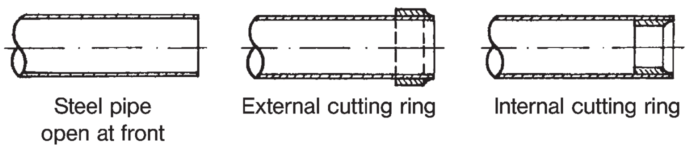


Figure 7-6 Piercing heads for impact moles

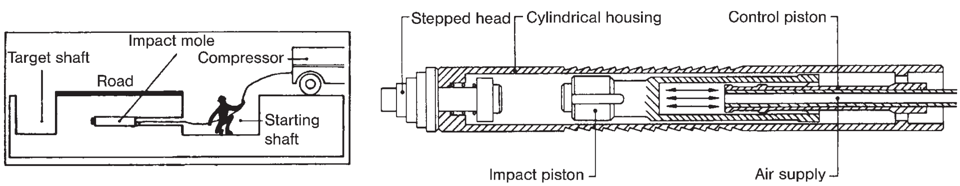


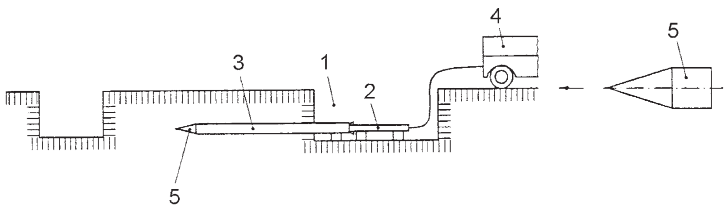
Figure 7-7 Impact mole, principle and section through the machine (Grundomat)

Although the distance of impact moling is theoretically unlimited, practical application is limited to 25 m [18]. The reason is the worsening precision with increasing distance, according to information from manufacturers about 1 to 2 % of the distance, as the device cannot be steered externally. The only way at the moment of influencing the directional stability is offered by different and exchangeable types of head. Depending on the type of soil, blunt, stepped or conical heads can be used. Advance rates depend on soil type and consolidation density and are between 3 and 30 m/h (normal rate 10 to 20 m/h) according to information from the manufacturers.

The impact mole is started and directed from a launching pit. For larger holes of more than 150 mm diameter, and in cases like drains where high precision is important, a pilot hole can be made with a smaller diameter (about 130 mm). A steel wire is draw through the pilot hole and the direction and the position of the hole are checked. Finally, a larger head can be run through the pilot hole.

As there is no direct method of controlling an impact mole, longer drives have to be corrected by digging intermediate shafts along the route, in which the impact mole can be aligned again [149].

Horizontal ramming/jacking with closed pipe. A closed steel pipe (casing or product pipe) is driven by ramming or jacking. As the leading end of this pipe is closed by a cone or a blunt head, which has the effect of displacing the soil (Fig. 7-8).



1 Starting shaft
2 Horizontal ram
3 Rammed pipe with closed tip
4 Compressor
5 Detail: closed front pipe

Figure 7-8 Horizontal ramming/jacking with closed pipe [77]

This process can be used to drive steel pipes, normally with a diameter of 150 mm, through all types of soil that are suitable for displacement. The scope of application is wider compared to an impact mole and also includes artificially backfilled or instable soils like quicksand, clay soils and soils below the groundwater table, as horizontal ramming does not have to resist the rebound force through skin friction. The average advance rate with this process is between 0.5 and 5 m/h depending on the soil type and consolidation. A rule of thumb for the necessary minimum cover is 12 times the diameter of the rammed pipe.

In order to be able to apply the short-stroke impact of the horizontal ram more effectively to the steel tip, particularly longer pipes are prestressed with 15 to 100 kN. This is normally provided by two steel cables each with 15 to 30 kN tensile force. The abutment to resist these forces must be reliably supported. Due to the relatively high loading on the pipes, this process cannot be used with plastic pipes.

Guided boring with reamer. In this process, the soil is displaced by static jacking of a rod fitted with a drive shoe. The machinery available on the market is hydraulically or pneumatically driven. The actual pipe run is installed by pulling or jacking behind a reamer (Fig. 7-9). Since this techno logy demands relatively high jacking forces, the drive shaft must provide the appropriate thrust bearing to resist the forces.

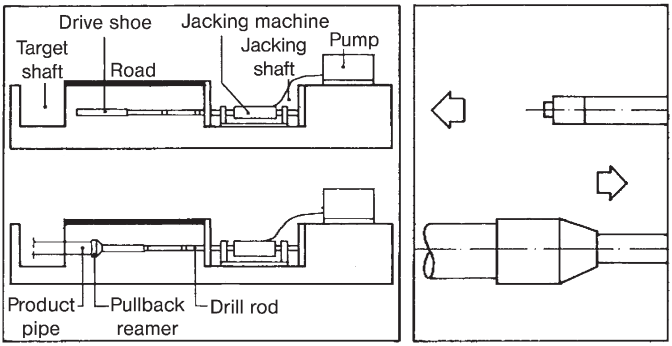


Figure 7-9 Horizontal guided boring with reamer. Principle of jacking the drill rod (top) and pulling back (bottom)

This process enables pipe jacking in soil suitable for displacement in the soil classes LNE/LNW/LBO/LBM 1 and 2 according to DIN 18319. In principle, any water-bearing soil types and also plastic clays and loams are suitable for this process, in contrast to impact moling. Difficulties occur with larger inclusions, as it is only possible to destroy an obstruction with static pressure and high jacking forces can be required. Horizontal guided boring machinery is designed for a compression force of up to 500 kN at 550 bar for distances of 30 to 60 m. The main application is distances between 15 and 30 m. The advance rate is influenced by the capacity of the hydraulic pump and the soil type and consolidation; it averages 1 to 40 m/h.

This process can be used to install casings and product pipes of steel, PVC, PE and stoneware in diameters from 70 to 150 mm. The minimum cover depends on the diameter of the reamer head and the distance and thus also the applied jacking forces. It should not be less than 1 m.

The diagram in Fig. 7-9 shows the functional principle of a horizontal guided boring system. After the setting up and bracing of the jacking machine in the thrust shaft lined with heavy timbering, a push rod fitted with a drive shoe is jacked into the soil. This fixes the position of the future pipe run. After it has reached the target shaft, the drive shoe is exchanged for a conical back reamer with the appropriate diameter for the product pipe. This has the appropriate fitting to connect to the rod and is pulled back through the hole by the rod.

Pipe bursting and pipe pulling. These processes are used to replace existing pipe runs.

In the pipe eating process, the existing pipes are broken out from inside by a compressed air or hydraulic reamer, which is pulled or pushed through the pipe. This displaces the surrounding soil. The new pipe is pulled in behind the reamer (Fig. 7-10).

In the pipe pulling process, the existing pipes are removed by pulling or pushing and simultaneously replaced by the new pipe (Fig. 7-11).

(To a certain extent) steerable ground displacement processes. Ground displacement processes are now also being offered with steering capabilities. There are two different basic systems.

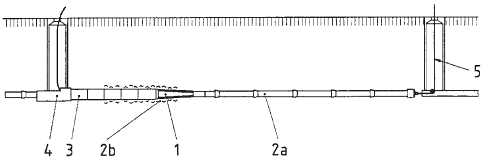


Figure 7-10 Pipe eating process [77]

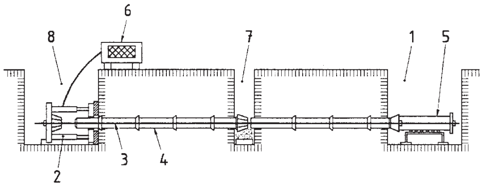


Figure 7-11 Pipe pulling process [77]

In the first system (Fig. 7-12), steering is provided by the rotation of a directional boring tool with an inclined leading face. If rotation is continuous, the inclined steering surface of the directional boring tool produces linear advance. If rotation is stopped, the advance curves depending on the orientation of the steering surface. The location and direction are recorded by a locator above ground, which receives a signal from a sonde installed in the directional boring tool. This is also called a walkover system (see also Fig. 7-13). The drill string, which bores a 100 mm pilot bore, is pushed by dynamic impacts from the drive shaft.

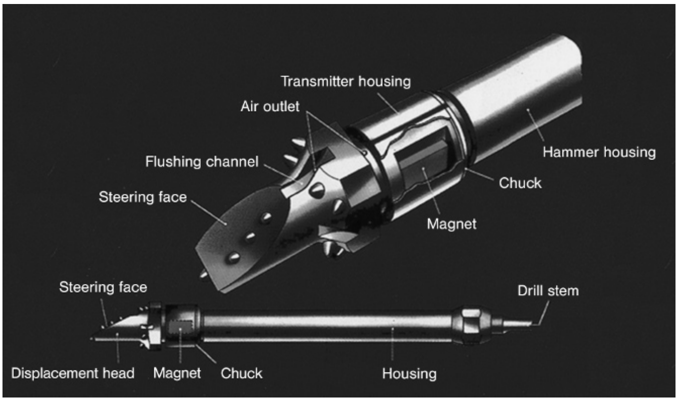


Figure 7-12 Principle of a steerable impact mole [342]

The second system is based on an adjustable displacement head mounted in the housing, which can be pointed by turning the air supply pipe, which is torsionally stiff. The direction is checked by receiving signals from a sonde in the cutting shoe using a locator on the surface (Fig. 7-13):

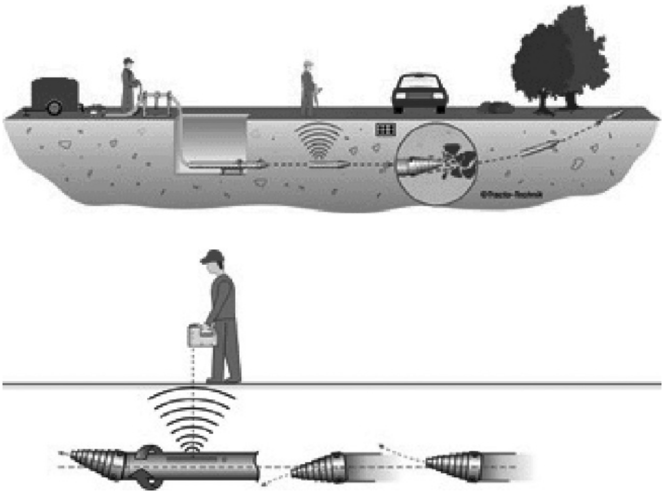


Figure 7-13 Principle of a steerable impact mole; "GRUNDOSTEER" system, Tracto Technik GmbH

Soil removal processes

General. If the diameter to be bored is greater than 200 mm or if the ground conditions are unsuitable for a soil displacement process, then a soil removal process is used. The soil can be excavated mechanically by a cutting head and removed by an auger flight in the casing, or by flushing. Auger muck removal is well suited for the transport of non-cohesive soils, but in very wet or cohesive soil becomes difficult or impossible since the soil sticks to the flights and can cause a blockage. When very wet ground has to be bored through, there can be difficulties with dewatering the soil and mud in the shaft, which often interrupt the work.

Horizontal pipe ramming/jacking with open end. In this process, a steel pipe open at the leading end is driven into the ground by a horizontal ram or jacks and the plug of soil at the front is removed from the pipe at reasonable intervals with compressed water, compressed air or an auger. In order to reduce skin friction, the open pipe to be driven can be fitted with an internal or external cutting ring. Some examples of the detailing of the steel leading edge are shown in Fig. 7-14.

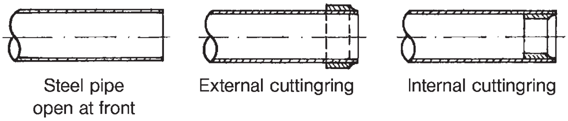


Figure 7-14 Steel pipe leading edges for horizontal ramming with open pipe [148]

With regards to the scope of application, the details given for horizontal ramming with closed pipe still apply. Due to the fact that the soil is removed instead of displaced, however, this system can be used with pipe diameters of up to 2000 mm nominal diameter and in exceptional cases to 3500. The cover can also be less than is feasible for ramming a closed pipe.

In contrast to the process with a closed pipe, if a certain resistance is reached the horizontal ram can be driven backwards and thus separated from the pipe. After the plug of muck in the pipe has been cleared, the drive is restarted. If the resistance of the ground is high, a pilot hole can be opened with an impact mole in order to be able to assist the horizontal ram with a tension cable.

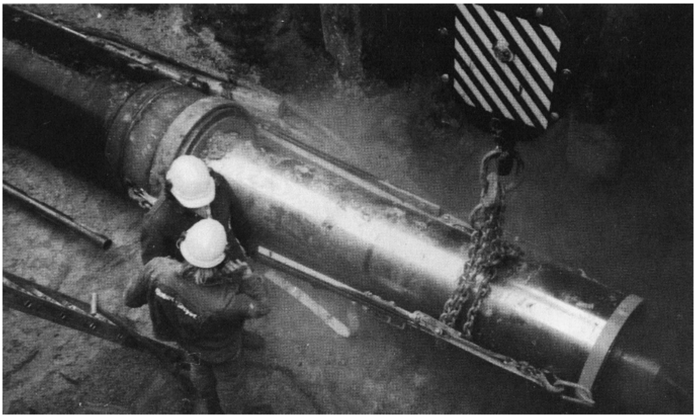


Figure 7-15 GRUNDORAM Taurus pipe ramming machine; Tracto Technik

Pipe jacking with auger. In this process, the muck excavated by the cutting head is pulled back into the drive shaft by an auger and generally lifted out by a crane. The driven pipes (casing or product pipes) are simultaneously jacked along the alignment. This process is particularly suitable for short runs since relatively little machinery is required. The process is illustrated in Fig. 7-16.

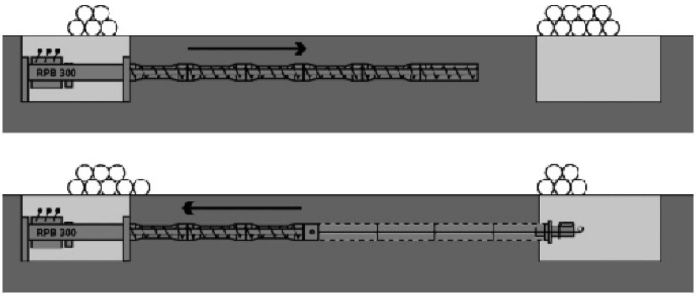


Figure 7-16 Principle of pipe jacking with auger; Lancier Cable GmbH

Working sequence:

1. Alignment with optical targeting device.
2. The pilot pipe and the casing with internal auger are jacked forward, with the muck being removed by the auger.
3. After reaching the reception shaft, the pilot pipe is replaced by a pull-back cutting head.
4. The product pipe is connected to the cutting head with a steel cable and clamping piece and pulled in as the casing is withdrawn.

For larger product pipes, a scraper head is used for reaming.

The geological range of application of pipe jacking is shown in Fig. 7-17. In cohesive soils, water may have to be added; gravel soils for longer distances require a bentonite feed in the auger in order to reduce friction in the auger. In general, all soil classes from L according to DIN 18319 to rock can be driven (FD, FZ 1 to FD, FZ 4).

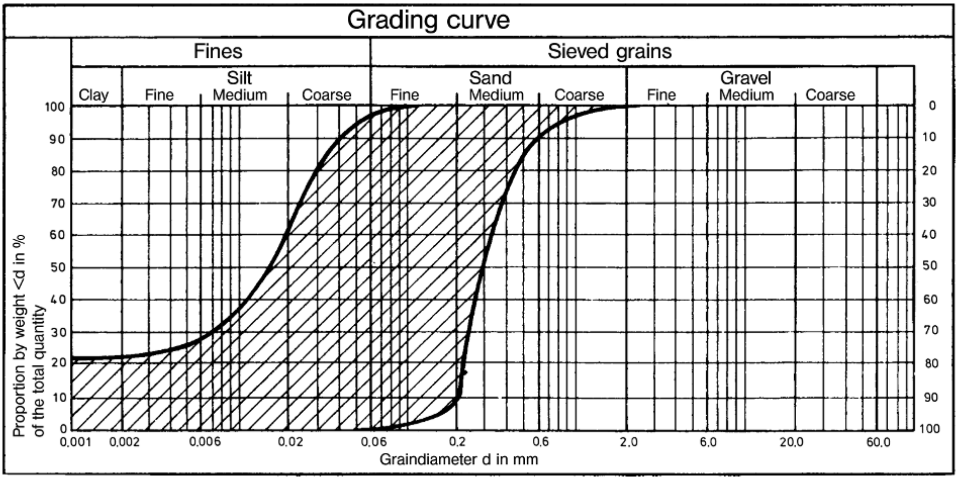


Figure 7-17 Range of application of auger boring machines. [199]

Pipe jacking with auger is only possible to a limited extent in groundwater, in which case the pipe run is pressurised with compressed air to hold back the water at the face. Pipe diameters of up to 1600 mm are possible, but applications are normally restricted to 1300 mm diameter and 100 m distance. The practical limit of maximum distance with this process is determined by the torsion moment on the auger. In conventional systems, this comes from the soil conveyance and the cutting head drive, which is driven from the drive shaft. If the excavation mechanism has its own drive, then distances of 150 m are possible according to information from the manufacturers.

Pipe jacking with auger can also be directionally controlled [77]. The direction is checked with a laser beam and changes of direction are performed using a hydraulically articulated steering head [18]. Fig. 7-18 shows a schematic drawing of a guided auger boring machine.

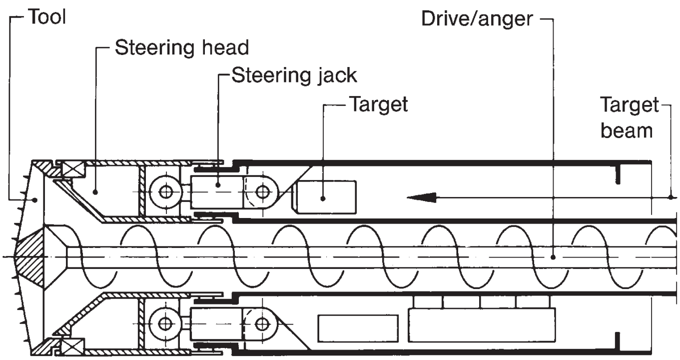


Figure 7-18 System drawing of a guided boring machine for pipe jacking

A method of correcting the direction is shown in Fig. 7-19.

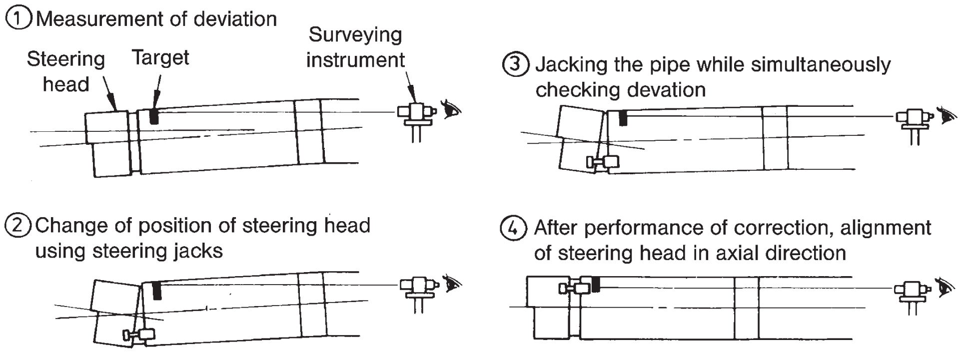


Figure 7-19 Procedure to correct the direction of steerable or partially steerable auger boring

Hammer boring. In this case the cutting head is driven into the ground under rotation and percussive impacts by a pneumatic hammer installed in the pipe. The excavated material is then removed mechanically, hydraulically or with compressed air (Fig. 7-20). Hammer boring is used in loose ground of little stability with hard inclusions of stones and blocks, up to rock of class F according to DIN EN 18319 with strengths of up to 200 N/mm². The bored diameter is normally less than 1100 mm, the distance less than 100 m and the average advance rate 6 to 15 m/h [342].

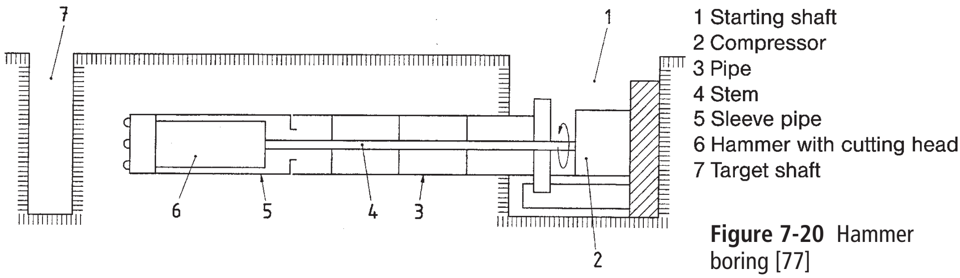


Figure 7-20 Hammer boring [77]

The cutting heads can be centrally or eccentrically mounted.

Pipe ramming with expansion cone. In this process, the soil is displaced by the jacking of a pilot string. The new pipe is pulled in behind a rotating expansion cone [77].

This process essentially corresponds to a version of the pilot boring process without the steering capability.

7.3.3 Guided processes

Guided processes (Table 7-2) offer considerably more development opportunities compared to unguided processes and thus have a much greater significance.

Table 7-2 Process and functional principle of the guided processes (according to DIN EN 12889)

Guided processes	
Microtunnelling	Driving of a casing by jacking with simultaneous full-face excavation by a cutting wheel with earth-pressure or slurry-pressure face support; continuous hydraulic muck removal.
Pilot boring	Steered driving of a pilot casing by soil displacement or removal; subsequent jacking of a product pipe run with the hole being enlarged by a reamer, simultaneously pushing out the pilot casing.
Directional drilling	A pilot bore is drilled with flexible drill string. The bore is enlarged by reaming until the necessary diameter is achieved and the pipes are pulled or jacked in.

Guided processes are used today in all fields of utility pipe construction. In the case of sewer, surface water or mixed drains, the product pipes are jacked in directly, but in the case of district heating, gas water supply or electric cables, the driven pipes act as a casing for the actual product pipe or cable. Guided pipe jacking is used in cohesive and non-cohesive soils, loose and solid rock.

Microtunnelling is generally used to drive pipes with internal diameters up to 1 m, with the tunnel boring machine being remotely steered from a control position [77]. The continued progress of technical development has also made it possible to jack larger pipes.

In microtunnelling, the pipes are jacked with simultaneous excavation of the face, which is supported mechanically, by slurry or by earth pressure, by a cutting wheel. The pipes are jacked from a jacking station in the launching shaft and if required also from intermediate jacking stations along the route. Fig. 7-21 shows the principle of operation of a microtunnelling site for a machine with hydraulic mucking.

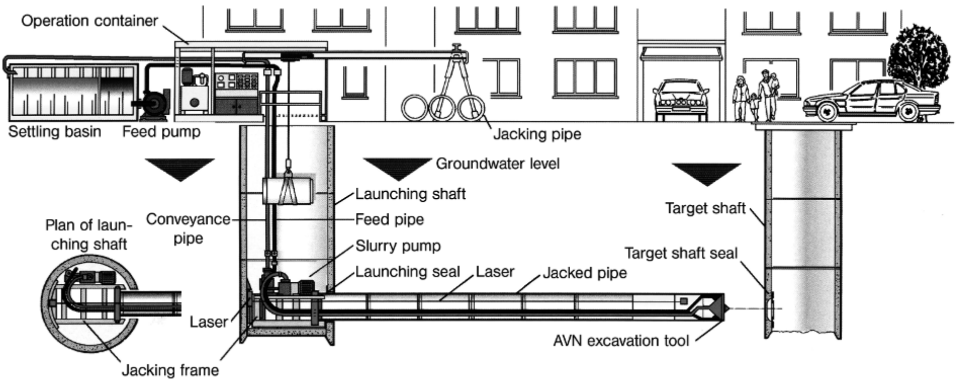
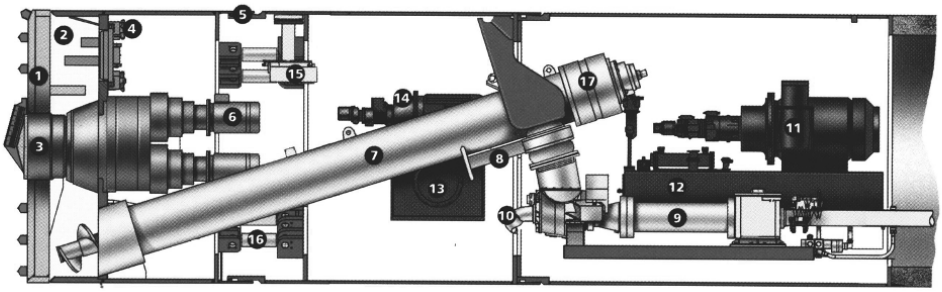
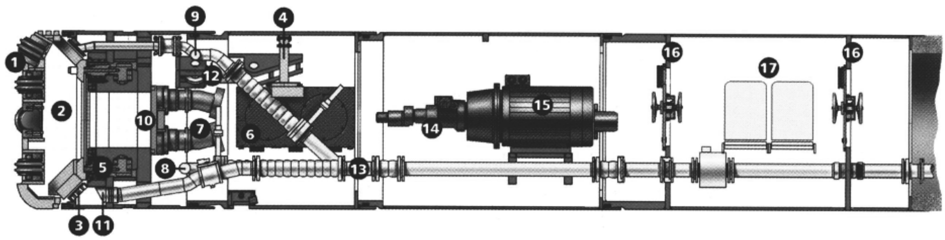


Figure 7-21 Principle of a microtunnelling site with hydraulic muck clearance; Herrenknecht AG

The muck is removed continuously by a screw conveyor, hydraulic, pneumatic or mechanical (conveyor belt etc.) mucking. Microtunnelling machines are generally based on the same systems as are used in full-scale tunnel boring machines and have been further developed for the specific conditions of microtunnelling [234]. Fig. 7-22 top shows a section through a microtunnelling machine with earth-supported face and hydraulic mucking; Fig. 7-22 bottom shows a machine with slurry-supported face.



1. Excavation tools
2. Excavation chamber
3. Cutting wheel drive
4. Excavation chamber hatch
5. Shield articulation
6. Cutting wheel drive
7. Screw conveyor
8. Screw slider
9. Muck pump
10. Conveyance pipe
11. Muck pump motor
12. Muck pump oil tank
13. Oil tank machine
14. Drive unit machine
15. ELS target
16. Steering jack
17. Screw drive



1. Excavation tools
2. Excavation chamber
3. Breaker
4. Target
5. Main bearing
6. Switch cabinet
7. Drive motors
8. Distribution annular gap nozzles
9. Distribution cutting wheel nozzles
10. Access hatch
11. Suction inlets
12. Steering jack
13. Bypass
14. Drive pump
15. Drive motor
16. Lock wall
17. Man lock

Figure 7-22 Microtunnel boring machines: top) earth pressure support, bottom) slurry support; Herrenknecht AG

The control of direction and position and the steering are performed similarly to the already described pipe jacking processes by aiming a laser beam at a target plate mounted on the cutting wheel and steering cylinders mounted in the backup of the shield.

The scope of application covers distances of up to 500 m and pipe diameters of more than 200 mm. With regard to geology, the appropriate cutting wheel can bore through all soil classes L, loose rock LBM/LBO 1, stone classes to S 4, solid rock classes F and FZ/FD 1 to 3 according to DIN 18319, even in the groundwater. The geological spectrum of application in combination with the appropriate cutting wheel is shown in Fig. 7-23.

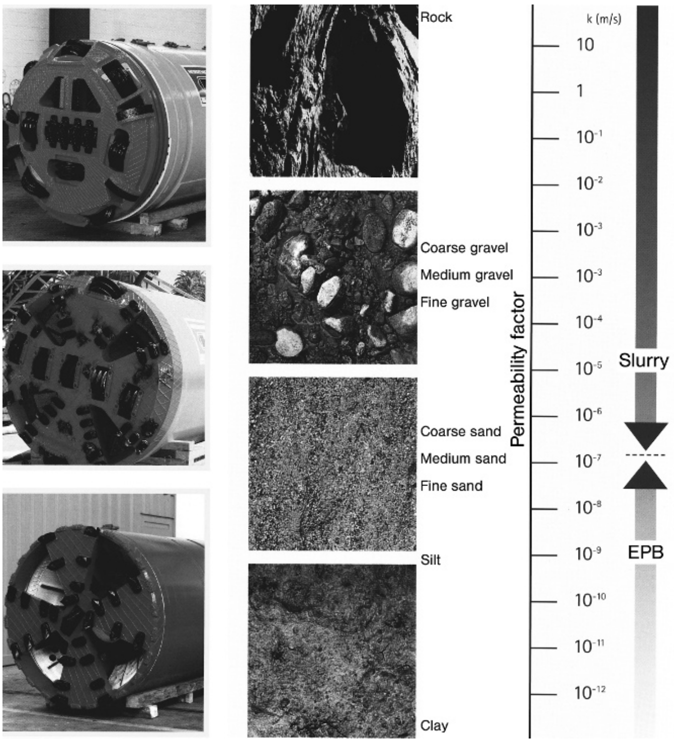


Figure 7-23 Geological scope of application and design of cutting wheels of microtunnelling machines; Herrenknecht AG

The use of precast rings for the thrust and reception shafts, which are installed like caissons as system and regulating shafts, and precast thrust bearing blocks reduce the setting up time and enable changing of the direction of the drive (Fig. 7-21).

The integration of all necessary site facilities including an overhead crane rail and a diesel power generation unit in a container has now become standard in Germany. These are placed directly above the drive shaft and enable rapid setting up, independence from external energy sources and working in any weather. Microtunnelling systems installed in containers enable quick conversion from screw conveyor to hydraulic mucking and even the changing of the cutting wheel, permitting flexible reaction to changing ground and groundwater conditions [227].

Pilot boring. The pilot boring process (Fig. 7-24) is a multi-stage process, the development of which only became possible with the ability to steer the ground displacement process.

The pilot casing is jacked according to the soil displacement principle with the direction and position being checked with a theodolite and a target mounted at the end of the pilot casing. The steering head is controlled as described in section 7.3.2 with a rotating steering head. Other methods of installing the pilot casing are also possible. Pipe diameters between 150 and 800 mm and distances up to 80 m are practical.

Regarding the geological and hydrogeological scope of application of the process, the statements already made about soil displacement processes without guidance still apply. The main application of the system shown here is drain connections.

The sequence of operations according to DIN EN 12889 is shown in Fig. 7-24. In the first phase, a steerable rigid pilot casing is driven. In the next phase, this bore is enlarged and the product pipes are jacked in with the soil being displaced or removed.

1 Jacking shaft
2 Pilot casing head
3 Pilot casing
4 Jack
5 Target shaft

6 Reamer
7 Casing
8 Auger

9 Product pipe

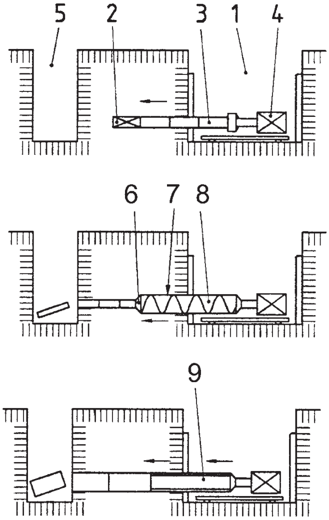


Figure 7-24 Pilot boring [77]

Directional drilling. In the directional drilling method, also described as horizontal flushed drilling or horizontal directional drilling (HDD) according to DWA A 125, the soil is excavated at the drill bit by bentonite suspension under high pressure.

Mobile drilling rigs are normally used. First, a pilot bore is drilled with a steerable drill bit and a flexible drill stem. This can be undertaken from the surface, which makes the sinking of a thrust shaft unnecessary. The bore is enlarged by back reaming until the required diameter has been reached and the product pipe can be drawn in. The principle is shown in Fig. 7-25.

Location and guidance are undertaken from the surface with locator and transmitter sonde in the drill tool. Steering of the drill bit is made possible by the provision of an inclined cutting face in combination with the flushing jets (Fig. 7-25).

1 Power unit
2 Drill bit
3 Flushing

1 Power unit
2 Reamer
3 Drill stem
4 Product pipe

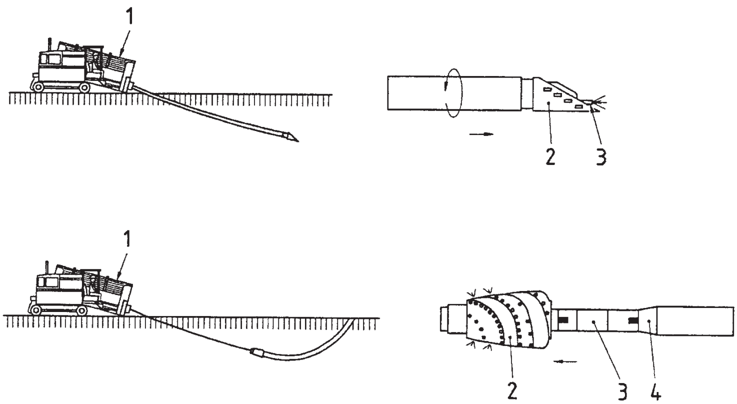


Figure 7-25 Directional drilling [77]

The scope of application includes all soils except rounded gravels without any cohesive content. No additional measures are necessary for working below the groundwater table.

7.4 Shafts and jacking stations

7.4.1 Thrust shaft

The thrust shaft is generally only a temporary measure, which has fulfilled its purpose after the completion of the pipe jack and is backfilled. Nonetheless, it has a very important role to play since the care taken in planning and excavating it to a great degree determine the success of the pipe jack and also the cost of the jacking operation. The shaft can be:

- Single thrust shaft (jacking in one direction).
- Double thrust shaft (jacking in two directions, mostly opposite).

The design and construction of the shaft are mainly determined by the ground conditions, but also any necessary auxiliary measures like compressed air operation or groundwater lowering. The sides are formed with sheet steel piles, although support beams or open and closed caissons are also conceivable. The shaft can also be reused later for operational purpose or as a siphon head.

The size of the thrust shaft is determined by the space required for the positioning of the product pipes, the dimensions of the jacking unit and the equipment required to remove the excavated muck, supply energy, for drainage and access for the workers. Further important elements of a thrust shaft are the thrust wall and the wall to be broken through by the pipe.

The thrust wall has the purpose of transferring the entire jacking force into the ground. It is either constructed of parallel steel beams or concrete, either cast on site or precast elements. The use of steel beams or precast elements has the advantage that the thrust wall can be installed quickly, bear load immediately and be quickly removed for reuse after the completion of the work. The decisive disadvantage of a thrust wall formed of steel beams is that it is difficult for the beams to bear equally onto the ground. Reinforced concrete thrust walls on the other hand can be cast against the ground or against the sheet pile wall and are then in full contact, resulting in very good pressure distribution. In order to avoid or at least reduce the deflection of the thrust wall, it can be sensible to improve the ground by grouting with cement mortar. This type of soil consolidation is still useful after the thrust wall has started to deflect in order to provide stabilisation.

The wall to be broken through at the front of the thrust shaft also requires special construction. The simplest method is to cut the profile of the cutting shoe out of the sheet steel piles forming the sides of the thrust shaft and jack the cutting shoe and the product pipes through the resulting hole. This solution is indeed simple but technologically inadequate. The sheet steel pile wall at the front is no longer complete and this is difficult to restore. It is also difficult to avoid collapses of the face as the cutting shoe is driven. It is much more satisfactory to construct a reinforced concrete aperture wall behind the forward sheet pile wall. This aperture wall has an opening with the exact outside diameter of the cutting shoe and the product pipes plus the required tolerance. If there is groundwater pressure, special waterproofing prevents water ingress along the outer face of the pipe.

7.4.2 Reception shaft

The reception shaft is situated at the end of the jacking stretch. It generally serves to recover the arriving cutting shoe or shield. In some cases, it can also be sensible to make use of the shaft for later operational purposes, for example the construction of an inspection manhole.

7.4.3 Main jacking station

The main jacking station (Fig. 7-2) is installed in the thrust shaft and serves to propel the individual pipes or the pipe run, depending on the excavation process at the face, into the ground. Depending on the necessary thrust force, two, four, six, eight or in exceptional cases even more hydraulic jacks are used.

In order to introduce the jacking forces into the pipes, a steel thrust ring is normally used, which has to be stiff enough to ensure even distribution of pressure to the face of the pipe. The thrust ring can slide in a guide frame or on the appropriately profiled invert of the thrust shaft.

7.4.4 Intermediate jacking stations

Various methods can be used to reduce skin friction (for example smooth outer surfaces of the pipes, bentonite suspension as a support medium and lubricant), but even when all these have been exploited, there will come a point as the distance increases where the resistance from the ground exceeds the thrust force that can be applied to the pipe (Fig. 7-2). The achievable distance varies, depending on the pipe diameter, pipe surface, ground, jacking equipment and measures to reduce friction, between 40 and 100 m, although shorter and longer distances are also possible.

In order to nonetheless jack longer distances from one thrust pit, intermediate jacking stations are used, also called interjack stations. The function of intermediate stations is based on splitting the distance that has to be driven into shorter sections (Fig. 7-4 and Fig. 7-26). This means that the skin friction only has to be overcome in the section being jacked at the time, with the stationary section being unaffected.

Intermediate jacking stations consist of a number of short-stroke jacks, which combine to apply the necessary force to drive the section of pipes in front of them. In order to ensure uniform design of the product pipes, the thrust forces applied at the main jacking station and all intermediate stations should be as similar as possible, but only utilised about 70 to 80%. Since the first intermediate jacking station has to overcome not only the skin friction but also the face resistance, the first section should be shorter than the following sections. After the completion of the pipe jack, the intermediate stations are removed and the gaps are jacked together or subsequently concreted.

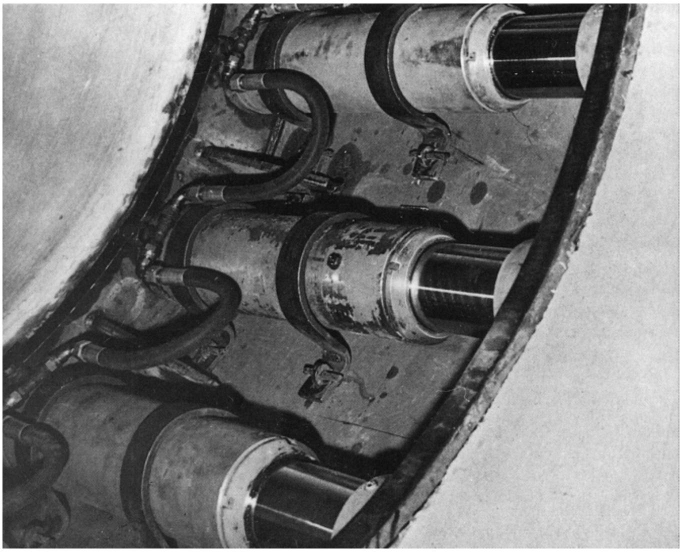


Figure 7-26 Intermediate jacking station, diameter 2.0 m, sewer in Bad Godesberg

7.5 Support, product pipe

The materials used for support or product pipes have to resist numerous loadings as the pipe is jacked and fulfil the serviceability requirements. The exposure of the material is particularly severe for sewer pipes, so the more stringent requirements have to be taken into account. Support for accessible cross-sections can use the materials and working methods described in Chapter 2. The future will certainly see more use of new materials like steel fibre concrete, fibre concrete, synthetic concrete and plastics.

7.5.1 Loading during pipe jacking

From the location in the ground. In general, it is safe to assume that sewer pipes will have to be laid in almost all types of ground and at all depths, which means that the scope of application has to include dry ground, under water and in ground below the groundwater.

From the installation process. The effects of construction on the pipes can act on the overall system and on individual elements like pipes or segments. The actions that particularly have to be considered are those from transport and installation, from jacking, working under compressed air, from grouting and the results of blasting and compaction.

For sewer pipes, the use of prefabricated pipes is almost universal, so these are not produced on site but delivered. Transport to the site and intermediate storage demand that the pipes, shaft sections or segments also have been designed for the loading case “transport and storage”.

The effects of installation are however still problematic. The loading often acts in a way that permits localised peaks of stress. In order to avoid overloading, it is recommended to use equipment developed especially for the installation in order to produce more uniform loading.

In all processes for the installation of sewer pipes, the jacking of the pipes is the most serious factor. The pipe has to transfer compression loadings resulting from the applied thrust force (peak values of 30 to 40 MN) and also extra loading resulting from eccentric loading to produce deviations. The concrete grade and the wall thicknesses have to be adequate to resist these loadings. In order to keep extreme peaks within bounds, the resultant average compression stress from the thrust force should not exceed a maximum of 9 N/mm², since steering movements can double the stress when the joints are closed and triple the average compression stress when the joints are open to the pipe axis. In order to ensure uniform pressure distribution, the thrust forces are applied to the jacked pipe through a stiff steel pressure distribution ring [115], [311].

7.5.2 Loading in operation

Loading from internal water pressure. The water flowing through a drainpipe exerts hydrostatic and hydrodynamic pressure on the outer lining. The magnitude of this loading is determined by the hydraulic situation, in other words whether the drain is free-flowing or under pressure.

The characteristic of a free-flowing drainpipe is that it is only partially full and the water typically does not reach the crown. The water level in this case is under external atmospheric pressure and no positive pressure can occur. For the structural design of a free-flowing pipe, however, a blocked condition also has to be considered as a loading case (highest possible water level in the manholes, for example when a gate valve is closed).

In contrast to a free-flowing pipe, a pressurised pipe is constantly full and the medium is transported under pressure. There are two basic types of pressure pipes, gravity and pumped. In the former, pressure results from the natural fall between the start and end of the run, whereas the pressure in the latter is artificially produced by pumping.

Pipes and lining materials have to be designed to resist the relevant permissible pressure according to DIN EN 764. For thin-walled pipes, the case of buckling also has to be verified by calculation.

Prestressed concrete can also be used for linings, with the prestress being applied externally be grouting pressure. In this case, the use of steel fibre concrete offers considerable advantages due to the transfer of tension loading, which can be guaranteed, and the better resistance against dynamic loading.

Thermal action. Due to the improved living standards in recent years, most households have washing machines and dishwashers, which result in a lot of warm water entering drains. Together with the normal residential waste water and drainage from industrial plants, which can also be hot, the temperature in a sewer can be high. Temperature differences are particularly significant for materials such as polyethylene and polypropylene, as they have fixed longitudinal joints and high coefficients of thermal expansion.

In addition to the change of the length of the pipe run, waste water at high temperature can also endanger bituminous pipe joints or protective coatings. The sudden arrival of hot waste water can produce extra stresses in the pipe material, which can be a particular problem if the pipe is partially full. This can be countered by providing a cooling basin in the run or by adding cold water. In order to avoid this danger, local byelaws often state that water must not be discharged hotter than 35° C.

Mechanical action. Various solids are transported in drains, like textiles, solid metal, sand and gravel. These lead to mechanical damage to pipes (wear), which considerably shortens their service life, and to deposits.

The transport of water-solids mixtures in pipes results, depending on the material of the pipes, in more or less abrasive wear due to the grinding action of the solids contained in the water. This effect increases with flow velocity and is thus particularly significant in steep sections. In order to avoid wear effects, it is recommended [30] to keep flow velocities below 6 m/s and select hard material with a smooth inside surface. Fig. 7-27 illustrates the wear properties of some pipe materials; the wear of asbestos cement pipes (no longer in use) is quite exceptional while all other materials show relatively similar characteristics. The increase after a certain number of wear cycles that is typical for stoneware is due to the fact that the hard glaze has been worn through and the actual stoneware is being attacked.

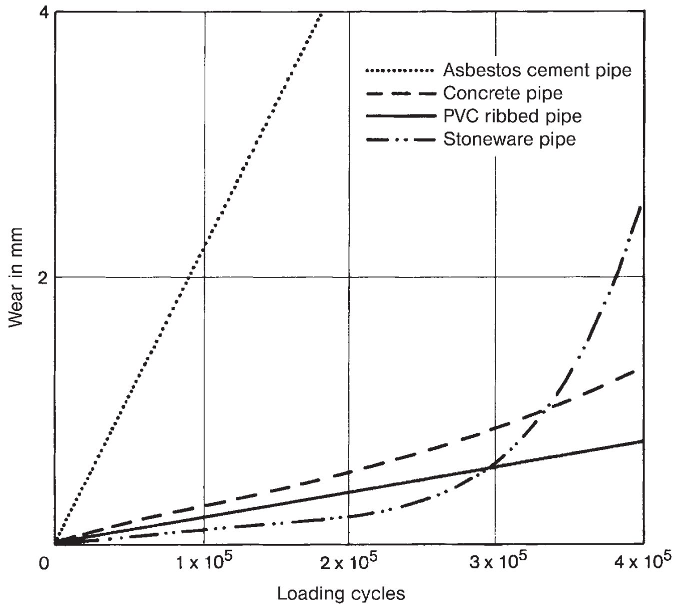


Figure 7-27 Absolute wear values for pipes made of various materials [30]

The mechanical action of very fast-flowing water without solid content is due to the cavitation effect (lat. cavitare = hollow out): if water flows at a few metres per second parallel to a boundary surface, then the geometric alteration of this surface causes separation of the flow and thus local low-pressure zones. If this static pressure is less than the vapour pressure, then bubbles full of water vapour are formed. If these then enter a zone where the static pressure is again above the vapour pressure – this is mostly the case shortly after the location where the bubbles are formed – then the vapour in the bubbles condenses and they collapse, or implode. The pressure and shock waves caused by these implosions create localised pitting and hollows in the surface of the material. Cavitation damage depends on the flow velocity, the geometrical form of the flow cross-section and the properties of the material. The preferred locations where this occurs are: impact surfaces in shafts, sharp pipe bends and sections with a flow velocity of more than 8 m/s. Concrete, due to its internal structure, is only resistant to cavitation to a certain degree, unless special requirements have been specified for the technical properties of the concrete [366]. If steel fibre concrete or plastics are used, then their properties can resist cavitation.

At higher flow velocities than the recommended value for drains of 8 m/s, impact surfaces that could favour cavitation must be avoided by constructive measures. Painting or coating with plastic emulsions or synthetic mortar in shafts can mitigate cavitation action [47].

In the course of the service life of a sewer pipe, solids transported in the water can be deposited. This process is described as sedimentation and only occurs in the parts of the pipe section that carry water. In contrast to this is encrustation, which affects the entire cross-section and results for example in rusting of steel pipes or separation of the lime content of concrete pipes. Sedimentation and encrustation have a serious effect on the flow properties due both to the reduction of the cross-section and their very rough surfaces. Particularly in a pressure pipe, this can result in the flow velocity and transport quantity no longer meeting the design values. In order to avoid such impairments, appropriate materials must be selected for the pipes and regular inspection and cleaning must be undertaken. Plastic coatings, bitumen paints and cement mortar linings have proved successful. The negative effect of cleaning depends on the process used and has to be resisted by the pipe materials [340].

Chemical actions. Chemical actions have an even more significant influence on the selection of suitable inner lining materials for sewer pipes than mechanical actions. Chemical actions on sewer pipes include the effect of aggressive water, biogenic sulphuric acid corrosion and rusting of metals. Not only the properties of the waste water can have an effect but also

- Sewer atmosphere.
- Deposits on the pipe walls.
- Lining material.
- Building physical parameters like temperature, fluid transport, ground, groundwater and the properties of the external walls.

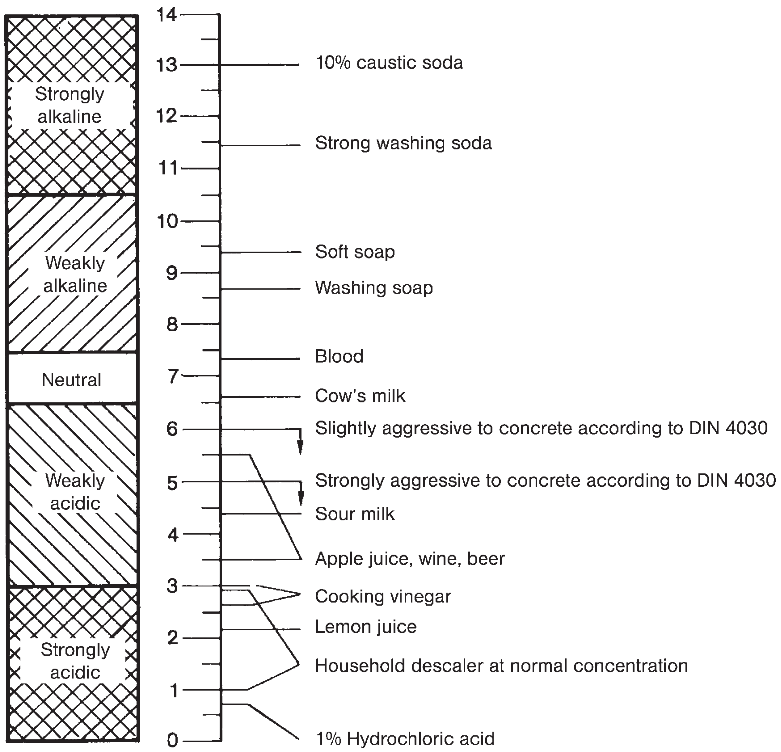


Figure 7-28
Water properties: ph value of various waste water contents [94]

A sewer pipe can be exposed to attack from aggressive water both from the inside and from the outside. An essential factor for the evaluation of the aggressiveness of water in drains or in groundwater is the pH value (Fig. 7-28 and Table 7-3). Sewer pipes have to be resistant against the effect of aggressive water. Considering the numerous different pipe and lining materials available on the market and the range of mixture and concentration variation of the aggressive substances in the foul water, pipe manufacturers generally produce tables, which show which chemicals the material can resist.

Table 7-3 Water properties: limit values for the evaluation of the degree of attack of predominantly natural water compositions according to DIN 4030 [148]

Investigation	Aggressiveness		
	weakly aggressive	strongly aggressive	very strongly aggressive
pH value...	6.5.........5.5	5.5.........4.5	below 4.5
Lime-dissolving carbonic acid (CO_2)........mg/l	15.........30	30.........60	over 60
Ammonium (NH_4^+)..................................mg/l	15.........30	30.........60	over 60
Magnesium (Mg^2). ....mg/l	100.........300	300.......1 500	over 1 500
Sulphate (SO_4^{2-})….................................mg/l	200.........600	800.......3 000	over 3 000

Biogenic sulphuric acid corrosion of concrete is the subject of a research complex, so reference can be made to this report [147]. For specialised literature, D. K. B. Thistlethwayte [355] is especially recommended.

The corrosion of metals is, with the exception of immediate attack by concentrated acids or alkalis, the result of an electro-chemical process, which can be caused by environmental factors like the presence of oxygen, dampness, temperature and other contaminations. Due to these media, galvanic currents can be caused at the surface of a metal, which can cause corrosion. When the resulting products of the reaction are not removed or consumed in some way, the metal continues to corrode [93, 255]. Pipes laid below the ground can be attacked both by the surrounding ground and by the medium flowing through.

Biological actions. In addition to mechanical and chemical actions on sewer pipes, biological actions can also be a problem. Lichens, fungi or algae can grow on damp concrete, mortar or asbestos cement surfaces, and a whole range of flora can live in seawater [160]. In addition, sewer pipes are particularly affected by bacteria and bacillae. Certain types of bacillus oxidise sulphur compounds to sulphuric acid, which can destroy pipes made of concrete or asbestos cement [147]. Many organisms are remarkably adaptable and can flourish without difficulty at a pH range from 2 to 10. They do, however, have a relatively narrow temperature interval, in which they can flourish. This interval can be in the range between 4 and 80° C.

Chemicals are often used as countermeasures in order to kill organisms. According to their scope of application, these can be described as germicide, algaecide, herbicide, fungicide, bactericide, biocide, microbiocide or slimicide. Against seawater flora, chlorine has become established as a successful solution.

A further problem is roots growing from outside into drain pipes. Fig. 7-29 shows an image from a CCTV survey of a pipe run. The roots growing into the pipe can be seen clearly. In surface water drains, such roots can form streamers metres long in a pipe, and in foul water drains they tend to branch heavily and form stoppers. In both cases, they can occupy the cross-section to such an extent that adequate water flow becomes impossible.

The exact explanation for the intentional penetration of roots into pipes has yet not been explained, but roots seem to follow the gradients of compaction density in the soil and thus grow particularly towards the less compacted zones around the socket connections in pipe runs. Once these are full, the root cannot retreat and starts to grow into the pipe [349].

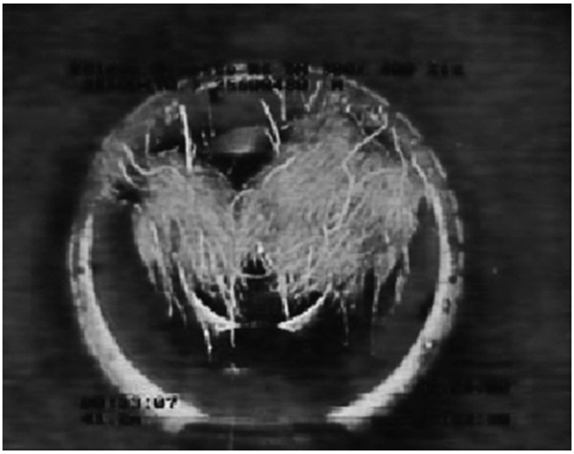


Figure 7-29 Image of roots growing in a pipe from a CCTV survey [349]

It can also occur that rats are present in sewer pipes and cause damage by gnawing on solid materials like plastic linings. Rats can be combated with appropriate contact poisons.

7.5.3 Insertion of the product pipe

Product pipes can be pushed or pulled into an existing casing or into a stabilised non-accessible cavity. These procedures are standard practice and are used to install utility pipes underground with the specialised equipment available on the German market. The installation of non-corroding pipes (stoneware, plastics) into protective steel casings is described through an example using technology based on a Drespa patent: after the hole has been bored, a steel guidance head is fitted to the sharp end of the first pipe and held in place by a steel cable passing through the pipe. In the pipe invert and fixed to the guidance head runs simultaneously a sheet metal strip, which serves as a guide for the pipe run to be inserted in order to avoid misalignment of the socket joints. The conventional product pipes are successively assembled in the thrust shaft and jacked forwards, with the bracing cable keeping the multi-part chain of the pipe run relatively stiff. Finally, the remaining cavities are filled with a cement-based cavity backfill (in German: Dämmer). This process demands exact boring that meets the target, as correction of the location of the stoneware pipe run is scarcely possible.

Jacking of product pipes. Standardised steel or reinforced concrete pipes are used almost exclusively in pipe jacking. In Germany, stoneware or GRP pipes with nominal diameters of up to 600 mm are also jacked.

Construction of an extruded product pipe. No process has yet become established for the in-situ production of product pipes with non-accessible cross-sections, although it would be possible to exploit methods already used for the construction of larger, accessible cross-sections. Examples are the construction of extruded steel fibre concrete pipes for the Hamburg-Harburg Nord sewer collector, contract 2 [31], and several refurbishment processes (Centriline, Relining and Instuform processes) for corroded pipes [239].

8 Ventilation during the construction phase

8.1 General

Workplaces on tunnel sites are exposed to numerous nuisances and sources of health risk. In addition to the hard physical work, which is still sometimes required today despite the use of modern machinery, the miners are normally exposed to wet conditions, high but sometimes also very low temperatures, noise, dust, exhaust fumes particularly carbon dioxide from the diesel motors of tunnelling machinery, and an increased risk of accidents. Petrol engines are not generally allowed in tunnelling. In order to reduce the pollution caused by dangerous gases and dust, tunnel engineers use ventilation systems.

Sufficient fresh air must be available at all workplaces during all underground works. The oxygen concentration of the air is reduced by breathing, combustion, and the addition of other gases and has to be supplemented. The concentrations of dangerous gases and dust must be thinned to below the permissible values at the workplace (MAK values, Table 8-1).

The applicable regulation in Germany is the BGV C22 (occupational accident prevention regulations) of the body responsible for accident insurance "Berufsgenossenschaft der Bauwirtschaft" (BG BAU) [357]:

§ 40 Ventilation

(1) Workplaces and access routes below ground must be ventilated so that

1. the oxygen concentration is at least 19 % by volume at every workplace,
2. the permissible concentration of dangerous substances in the respiratory air is not exceeded,
3. no explosive gas can occur in a dangerous quantity
   and
4. the average velocity of the air flow does not fall below 0.2 m/s or exceed 6.0 m/s.

If natural ventilation is used, then the oxygen concentration of the respiratory air must be monitored by instruments to measure oxygen with a threshold alarm function.

(2) If the conditions according to (1) cannot be met by natural ventilation, then artificial ventilation must be provided.

(3) If working processes or combustion motors are used, which liberate dangerous substances into the respiratory air, then artificial ventilation must be provided.

(4) Artificial ventilation has to comply with the following extra requirements in addition to (1):

1. An additional 2.0 m³/min of fresh air must be supplied for each employee and at least 4.0 m³/min of fresh air for each kW of diesel motor power in use; air escaping from compressed air machinery and tools may not be considered in this calculation.
2. In branched and crossing tunnels, the airflow must be directed by self-closing doors. If there is heavy vehicle traffic, double doors must be provided to act as an air lock.

(5) In headings and pipe jacking operations up to 5 m² cross-section, the average air speed must be at least 0.10 m/s, not the value given in (1) No. 4.

(6) Dust must be settled or extracted as near as possible to its origin.

(7) The maintenance of the conditions according to (1) Nos. 2 to 4 and (4) No. 1 is to be monitored by instruments if necessary. A log of the measurements is to be kept.

Similar threshold values to the BG BAU are also required by the SUVA (Switzerland) and AUVA (Austria).

An unacceptable concentration of harmful substances in the air has been reached when the value given in the list of maximum permissible values (threshold limit values of airborne contaminants) is exceeded.

Table 8-1 Critical substances and their permissible maximum concentrations according to TRGS 900 [358]

Gas or dust	Threshold
diesel motor emissions (fraction reaching the lung alveoli)	0.3 mg/m³
carbon monoxide CO	30 ppm (35 mg/m³)
carbon dioxide CO_2	5,000 ppm (9 100 mg/m³)
nitrous gases, NOx	
NO	25 ppm (30 mg/m³}
NO_2	5 ppm (9.5 mg/m³)
aldehydes	
formaldehyde	0.5 ppm (0.62 mg/m³)
acrylaldehyde	0.1 ppm (0.25 mg/m³)
sulphur dioxide SO_2	2 ppm (5 mg/m³)
dust with quartz content (fraction reaching the lung alveoli)	0.15 mg/m³

The concentration of substances in air for breathing is given in ppm (parts per million):

1 ppm = 1 ml/m³; 1 % by volume = 10,000 ppm

The permissible workplace concentration is the concentration of a working material as dust, vapour or suspended particles in the air at the workplace, which according to the current state of knowledge generally does not impair health even in case of repeated and long-term exposure, normally defined as eight hours of exposure per day, but observing a maximum of 40 working hours per week (42 hours in four-shift operation). Table 8-1 gives an overview of the critical substances and their maximum permissible concentrations.

If the required conditions cannot be met, then the workplace must be ventilated.

Swiss regulations SUVA have required the use of particle filters for diesel-powered vehicles in order to eliminate carcinogenic sooty particles since summer 2000 or 2001.

The following are excepted from this general requirement:

- Machines such as drilling jumbos with auxiliary diesel motors, which only use the diesel motor to travel.
- Machines whose motors are used for less than two operating hours per day.
- Machines with a power output of less than 50 kW.

Harmful dust, mainly quartz dust, is a by-product of drilling, blasting, muck clearance, transport, shotcrete spraying and tunnelling machinery. Dust must be settled as near as possible to its point of origin or extracted. Reference should be made to the BG regulation 219 “Dealing with mineral dust” of the BG BAU (the accident insurance body of the German construction industry).

A significant danger potential can also be caused by the natural gas that is often encountered, which occurs more often than is generally assumed. This is mostly methane or less frequently higher hydrocarbons with a somewhat lower lower explosion limit (LEL), or combined with hydrogen sulphide; carbon dioxide and radon are less frequently encountered but can be significant in isolated cases. Detailed information about the danger resulting from the occurrence of natural gas can be found in “Tunnelbau im Sprengvortrieb”, Springer 1997 [221].

8.2 Ventilation systems

8.2.1 Natural ventilation

Natural ventilation makes use of the air flow resulting from the temperature difference between outside and inside air, which results from the difference in height or sun direction (solar radiation). It is sufficient for sections less than 200 m long without diesel vehicles, as long as it can act for a sufficiently long time, although sections as short as this are also now artificially ventilated in order to avoid ventilation times. Natural ventilation can be an interesting solution for investigation or ventilation tunnels completed in advance or in combination with artificial ventilation.

8.2.2 Positive pressure ventilation

This ventilation system can also described as supply ventilation. A fan takes in fresh air and feeds it through a duct to the workplaces below ground. Gases and air containing dust flow through the tunnel section to the outside (Fig. 8-1).

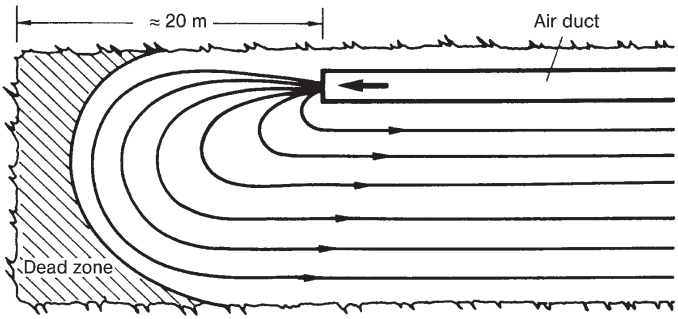


Figure 8-1 Positive pressure ventilation

The end of the ventilation duct should be as near to the face as possible, as the effect of vortexing and thorough mixing of the air mass reduces with increasing distance from the end of the duct.

While the workplace is directly supplied with fresh air, the contaminated air flows through the entire tunnel and has to be monitored to ensure permissible concentrations are being maintained. This process is used when no quartz dust is expected and vehicles with combustion motors are used. The ducts should be regularly checked for leaks. When a single-stage reamer boring machine is used with the pilot hole already broken through, the air should be fed so that the driver‘s position is always in the fresh air flow.

8.2.3 Extraction ventilation

The contaminated air is extracted in at the workplace and blown out of the tunnel through the air duct, being replaced by fresh air flowing into the tunnel (Fig. 8-2). The radius of action in the space to be ventilated is small because most of the air flow is not reached. The air ducts have to be made stiffer than with positive pressure ventilation. Contaminated air is extracted immediately. However, the effect of extraction nearer to the face is limited and since a mixture of fresh and contaminated air is extracted, large air volumes have to be transported.

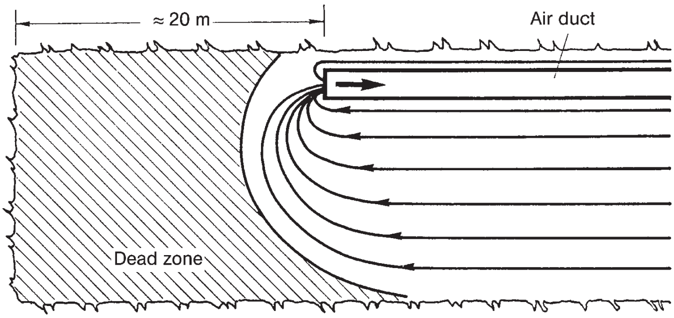


Figure 8-2 Extraction ventilation

Extraction ventilation should be preferred to positive pressure ventilation when the latter would be expected to lead to unacceptable conditions in the tunnel behind the face. The opposite effect, that extraction ventilation feeds exhaust gases and contamination from other working areas to the face, cannot be denied but only indicates that fumes from diesel operation should be reduced as much as possible.

When quartz dust is produced by drilling or blasting, extraction ventilation should be preferred.

8.2.4 Reversible ventilation

Reversible fans enable the switching of ventilation operation from positive pressure to extraction. After blasting, the resulting cloud (rock dust and blasting fumes) is immediately extracted to provide fresh air when the miners arrive back at the face. During the remaining working time, the ventilation feeds air to the workplace. Due to the increasing danger of dust settling in long air ducts, this process can only be recommended for short or medium-length air ducts. Extraction proceeds slowly unless additional fans are provided in the air ducting. Reversible (or also non-reversible) ventilation can also be used to combat quartz and when machinery powered by combustion motors is used.

8.2.5 Combined ventilation

Extraction ventilation can only be used practically in combination with a supplementary positive pressure air feed, which feeds clean air from the tunnel and blows against to the face (Fig. 8-3).

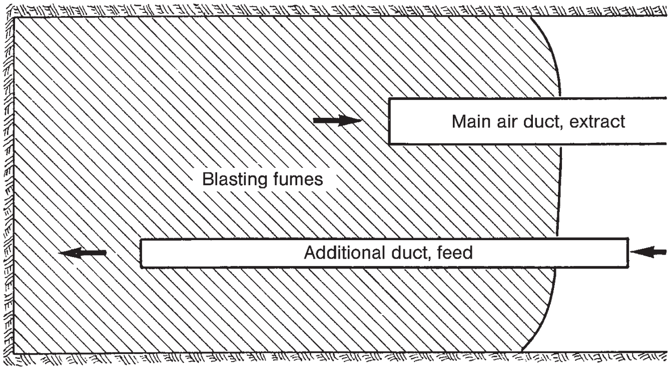


Figure 8-3 Combined ventilation

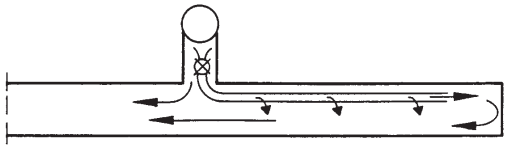
This ensures not only mixing of fresh air into the otherwise immobile zone at the face, but also displaces the contaminated air into the main ventilation and into the flow of extracted air.

Important applications for combined ventilation are drill and blast tunnelling, also full-face tunnel boring machines and roadheaders. When explosives are used, the resulting poisonous fumes contaminate the air for breathing in addition to dust and diesel emissions, which has to be dealt with by an appropriate ventilation system. When tunnel boring machines are used, combined ventilation can be used to assist dust suppression. (Details see Section 8.5).

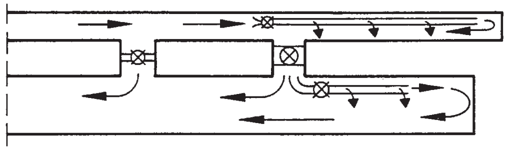
8.2.6 Recirculation systems

For longer tunnels, recirculation systems should be used whenever possible, either entirely or in partial areas. These are especially characterised by air ducting without losses. Large quantities of air can be mixed extremely economically. Fig. 8-4 shows various sorts of recirculation systems, some combined with air duct ventilation.

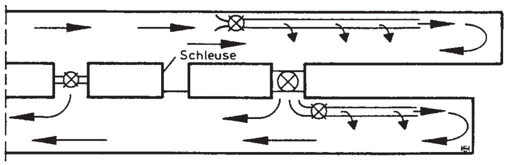
Recirculation systems can be extremely advantageous, for example in drill and blast tunnelling, when the schedule demands continuous working.



Recirculation system with shaft or adit



Recirculation system in tunnel with side tunnels: Ventilation through cross-cuts, side tunnel supplies fresh air to tunnel (Gotthard north road tunnel)



Recirculation system: two parallel tunnels with ventilation through the cross-cuts (Seelisberg Tunnel North)

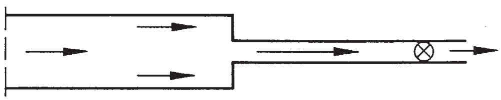


Figure 8-4 Recirculation systems, some combined with sections fed by an air duct for advances outside the actual circulation (the arrows show the qualitative, not the quantitative air flow)

8.3 Materials

8.3.1 Fans

The necessary air movement is propelled artificially by fans. Fans used on construction sits are either of the centrifugal (radial) or axial type. Centrifugal fans can achieve pressure differences of up to p = 0.35 bar (0.035 N/mm²). Axial fans arranged as contra-rotating high-pressure fans can achieve up to 0.1 bar.

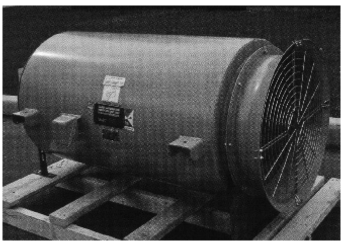


Figure 8-5 Axial fan from the company ABC Canada [1]

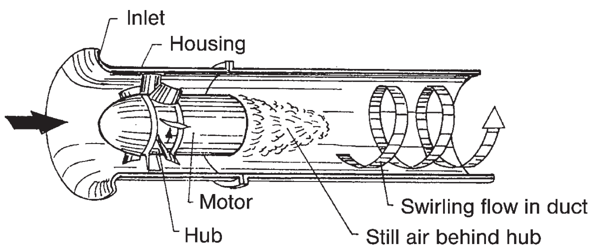


Figure 8-6 Construction of an axial fan without guide vane, with pipe connection on the pressure side [183]

Axial-flow fans (Fig. 8-5) are mounted as an extension of the air duct. They are smaller and more powerful than radial fans and are thus used more often in tunnelling. Axial fans are seldom used with dedusting systems because of the pressures required.

In order to reduce the noise output, fans can be combined with silencers.

8.3.2 Air ducts

Air is transported in pipes (air ducts) of 300 to 3500 mm diameter.

The plastic air ducts used in modern tunnelling are of the following types [327]:

- **Folding ducts** consist of a plastic tube with an end ring at each end of the duct. This enables them to be folded up for storage. They generally have diameters of 300 to 3500 mm and are used for positive pressure ventilation.
- **Hooped ducts** are additionally provided with reinforcing hoops every 0.5 to 2.0 m, which are intended to prevent folding up after the fans are switched off.
- **Spiral ducts** also have a continuous reinforcement spiral, in contrast to folding ducting. This provides improved stability and means that they can even be used for extraction with negative pressure. The unfavourable aerodynamic properties of the waved inner surface restricts their use to short sections up to about 150 m. Available diameters are 250 to 1,800 mm.
- **Shaft ducts** have at least two tension wires as a connection between the end rings in order to support the hanging tension when they are used vertically.
- **Swirl ducts** are system elements, which swirl the air locally along a length of 2 to 5 m to combat dust or gas at the location.

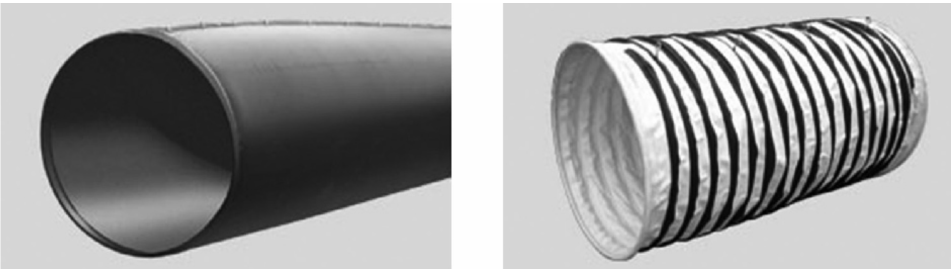


Figure 8-7 Folding duct (left) and spiral duct (right) [269]

In addition, **steel ducts** made of rolled sheet steel can be used. These are mostly built into tunnel boring machines.

For the purposes of design and sizing of ventilation systems (for example according to SIA 196), ducts are categorised into duct quality classes S (very low losses, leakage about 5 mm²/m²), A (low losses, leakage about 10 mm²/m²) and B (medium losses, leakage about 20 mm²/m²) [327].

The duct diameter depends on the required air demand for tunnel excavation. It is also important that the space taken up by air ducts does not excessively restrict the structure gauge in the tunnel. According to SIA 196, a spacing at least 0.2 m should be maintained between the air ducts and the extent of any passing vehicles (Fig. 8-8).

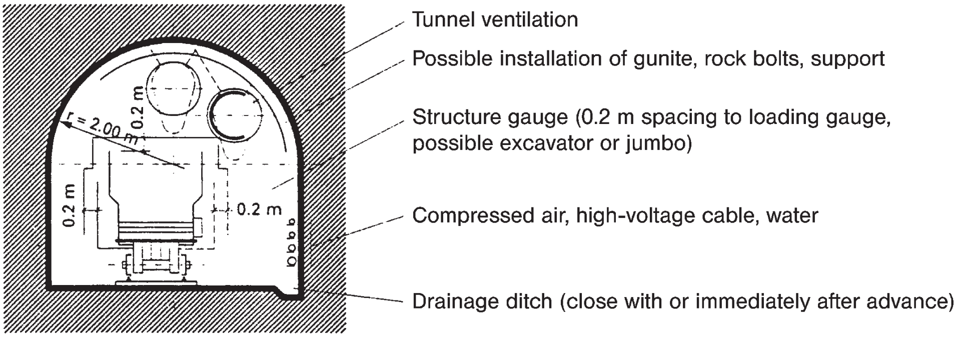


Figure 8-8 Minimum profile for drill and blast tunnelling

8.3.3 Dedusters

If the dust concentration resulting from construction operations is too high to be lowered below the permissible values by thinning alone, then dedusters have to be used to remove dust from the air. These are also called dedusters and can be divided according to their function into

- dry dedusters
- wet dedusters

Dry dedusters make use of various filter technologies like hose filters, bag filters or vane filters to remove the dust from the air. Fig. 8-9 shows the function of a dry deduster. These have proved successful in cases of high dust production (for example for dedusting TBMs).

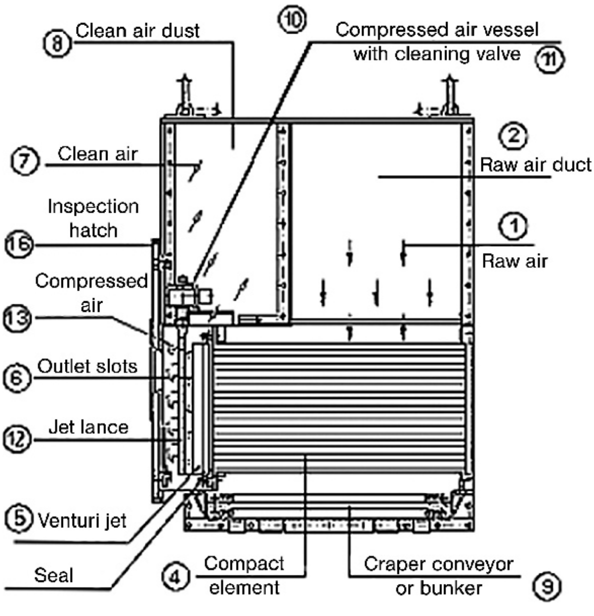


Figure 8-9 Dry deduster from the company CFT [54]

When the dust is damp or for dust produced by shotcrete spraying, however, dry dedusters reach their limits and wet dedusters are more suitable in such cases. Low-, medium- and high-pressure systems use tiny water particles, which adhere to the dust grains and can thus separate the grains from the air flow. The “hoeko-vent” system for example works on the following principle (Fig. 8-10).

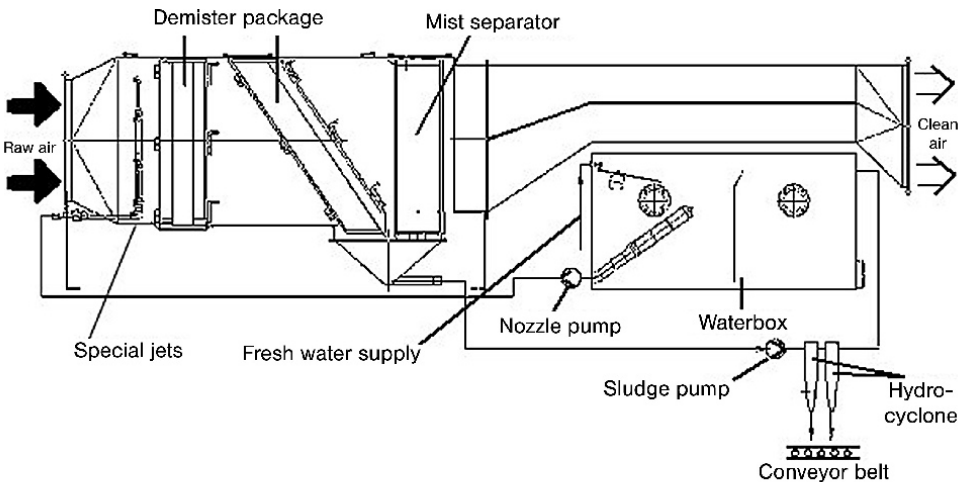


Figure 8-10 Wet deduster from the company CFT [54]

Dust should be extracted as near as possible to its origin. When the quantity of dust is large, it is important that the dust does not settle in the air ducts, as this can increase the weight and lead to failure of the hangers. In order to reduce sedimentation in extract ducts, the air speed should not be less that about 18 to 20 m/s.

For the preliminary sizing of dedusters, the SIA 196 [327] (see Chapter 8.4) provides rule of thumb values.

8.4 Design and cost

For the design and sizing of ventilation systems, the German civil engineering accident insurer recommends the procedure from the Swiss standard SIA 196 [327]. The provisions of BGV C 22 (see Chapter 8.1) should also be complied with.

The information in SIA 196 can be used for preliminary design and sizing of ventilation systems, and construction details and calculation examples are also given. A calculation according to SIA 196 is available online under http://www.tunnel-ventilation.net/en/solve1.asp.

In the next section, the design calculation possibilities of SIA 196 are presented and their theoretical background is explained.

Air duct calculation. The design of ventilation systems through calculations making use of the general pipe friction equations started early. The loss of quantity, which is unavoidable in a pipe run, is normally suggested as a linear rule of thumb value from experience. Only the introduction of the leaking pipe theory in 1978/1983 [116], [326] made it possible to perform calculations for air duct runs.

Fig. 8-11 shows a comparison air duct design according to a linear curve, which was taught at the ETH Zürich as long ago as 1917, against the use of the leaking pipe theory. The comparison shows no significant difference up to a length of the air duct of 2,000; the newer method of calculation leads to slightly more favourable values. When the duct is longer than 2,000 m, the results for the two methods differ increasingly. This phenomenon mostly occurs in practice at a ratio *L/D* (*L* = duct length, *D* = duct diameter) of 1,800-3,000. The variation results both from the linear assumption of the older calculation method and the degree of leakage, that is the state of the duct assumed for the new calculation method.

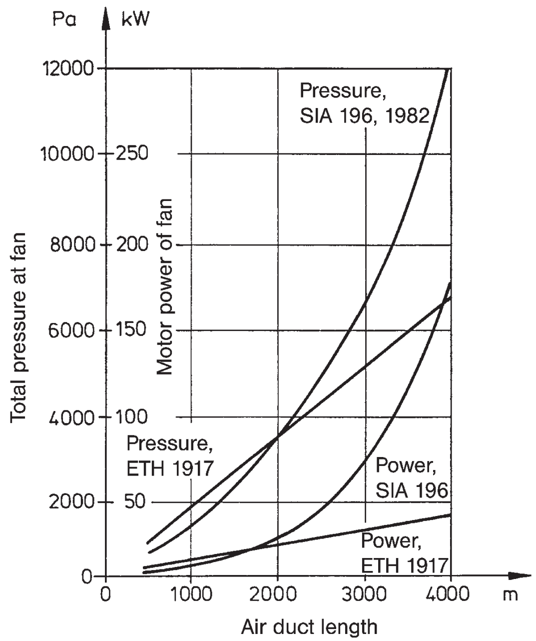


Figure 8-11 Comparative calculation of an air duct with pipe friction calculation and the leaking pipe theory [116], [326]

The basic equations for the change of the static pressure in the air duct and for the change of the airflow velocity as a result of quantity loss in the leaking pipe result as follows:

$$\frac{dp}{dx} = -\lambda \frac{1}{D} \frac{\rho}{2} u^2$$

$$\frac{du}{dx} = \frac{4f'}{D} \sqrt{\frac{p}{\frac{\rho}{2}(1+\zeta)}}$$

The differential equation pair is not very suitable for practical application in air duct calculations. In the recommendation SIA 196 “Construction ventilation of underground works” [326], the calculation is converted into easily used nomograms.

Fig. 8-12 shows the nomogram for an air duct of quality class A with a low friction coefficient of $\lambda = 0.018$ and a small total leakage area of $f^* \leq 10$ mm^2/m^2. An example of an air duct of this nature is provided by new fabric ducts with simple operation. For longer duct runs, such as are used with air duct cassettes, the f^* value is somewhat more favourable, as with an A class duct.

The example entered in the nomogram corresponds to a 2,200 m long air duct with a diameter of 1.0 m, a Q_0 of 10 m^3/s and an airtightness ρ of 1.2 kg/m^3.

The calculation gives: $L/D = 2200$, $p_o = 0$, $u_o = 12.7$ m/s;

$\omega = 1.4$, $\pi = 53$, both from the nomogram.

The static pressure is calculated as

$$p_1 = \Pi_1 \frac{\rho}{2} u_0^2 = 5150Pa$$

The air volume to be provided by the fan is: $Q_l = \omega\, Q_0 = 14$ m^3/s.

The required force demand is given as:

$$N = \frac{Q_1 \cdot p_{tot}}{\eta_{vent} \cdot \eta_{motor} \cdot 10^3} [kW] \qquad p_{tot} = (p_1 - p_{dyn} - p_x)$$

The recommendation SIA 196 also gives practical values for the various possible individual pressure losses [326].

In this way, the duct run can be calculated quite simply. It is assumed in this case that the air volume Q_0 corresponds to the air volume required to thin contaminants in the tunnel air. This applies correctly for the emission sources at the face. If diesel traction is used, however, the tunnel air is polluted along its entire length. It is therefore permissible to blow the fresh air to thin these diesel emissions from locations along the entire length corresponding to the air pollution along the length. This method of working is particularly cost-effective for long lengths of air duct. The leakage volume as the difference of air volume Q_l-Q_0 may be considered for the thinning of such diesel fumes. This air leakage is often not sufficient and air vents have to be provided along the duct, for example by installing Coanda effect ducts at periodic intervals.

Recirculation systems. Recirculation systems do not normally lose any volume, so the Bernoulli pipe equation can be used. The static pressure difference can be calculated as:

$$p_1 = \lambda \cdot \frac{L}{D} \cdot \frac{\rho}{2}\, u^2 \, [Pa]$$

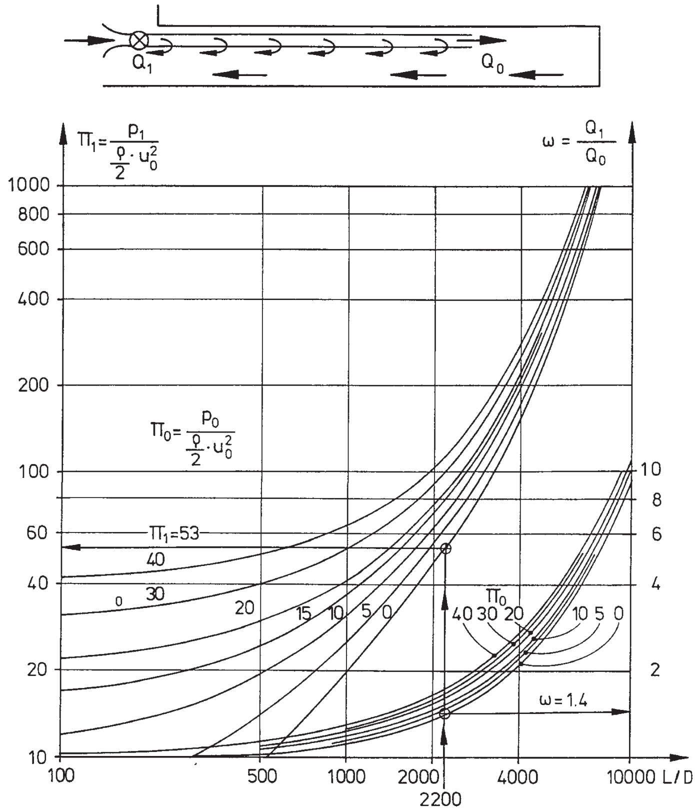


Figure 8-12 Nomogram for the calculation of air ducts, air duct of quality A

The following values of the pipe friction coefficient l can be used depending on the wall roughness:

– with formed concrete walls		λ = 0.02
– tunnel bored by a TBM and not lined		λ = 0.03
– blasted tunnel without installations		λ = 0.08
– tunnel with steel lining		λ = 0.10
– Profiles with shotcrete	small	λ = 0.04
	large	λ = 0.03

Since recirculation systems can with few exceptions be operated without losses, they can be considered particularly cost-effective.

Operation of ventilation systems. A ventilation system cannot be left to itself for the entire period of operation after it has been set up. The necessary maintenance requires periodic checking, which cannot be limited to mere visual inspection. Rather the pressure applied and the transported air volume should be monitored continuously. Continuous comparison of measured values to calculated values can quickly detect irregularities.

High operating pressures should be avoided as far as possible because leakage volumes increase progressively with pressure and more pressure results in still more leakage loss, a vicious circle.

High pressures should particularly be avoided if there is a danger of the occurrence of methane, since air ducts can then no longer be repaired in operation, and longer stoppages also lead to increased methane concentration and could lead to risk of explosion.

Operating costs of air duct systems. The operating costs of ventilation systems are directly dependent on the pressure applied by the fan. Figs. 8.13 to 8.15 make this statement very clear.

High pressures also result in high purchase costs for the fans. The cost of air ducting also increases with increasing diameter, but this is normally less than the alternative of more powerful fans. For small-diameter air duct systems, therefore, increased pressure applied by the fan leads to increased total operating costs and also increased investment costs. Fig. 8-14 shows the cumulative operating costs of a ventilation system for a tunnel with a cross-sectional area of 20 m² dependent on the duct diameter for an assumed average advance rate of 7 m/d.

The common intention to use air ducts with the smallest possible diameter since they obstruct the works less is therefore counterproductive regarding safety and also not cost-effective because of the very high pressures that have to be applied by the fans, even with special quality ducting. Fig. 8-15 shows this clearly.

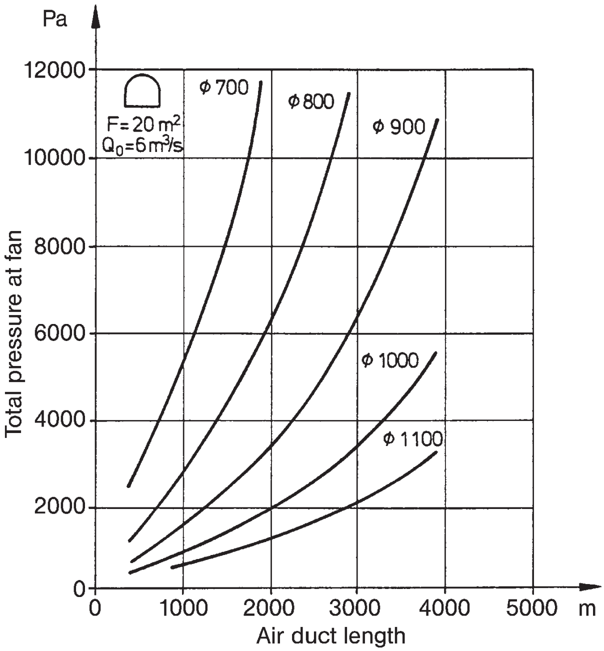


Figure 8-13 Total pressure applied by the fan for air duct diameters 700 to 1100 mm for a tunnel of 20 m² cross-section and a flow volume Q_0 of 6 m³/s

When diesel traction is used in longer tunnels, filter systems can very quickly become a cost saving when considered overall because as explained above, the necessary air swirling can be greatly reduced.

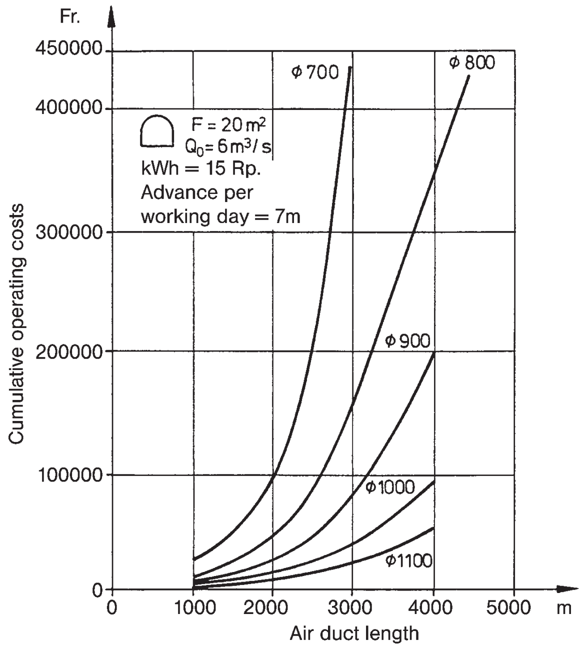


Figure 8-14 Cumulative operating costs (in Swiss Franks) of a ventilation system for a tunnel of 20 m^2 cross-section

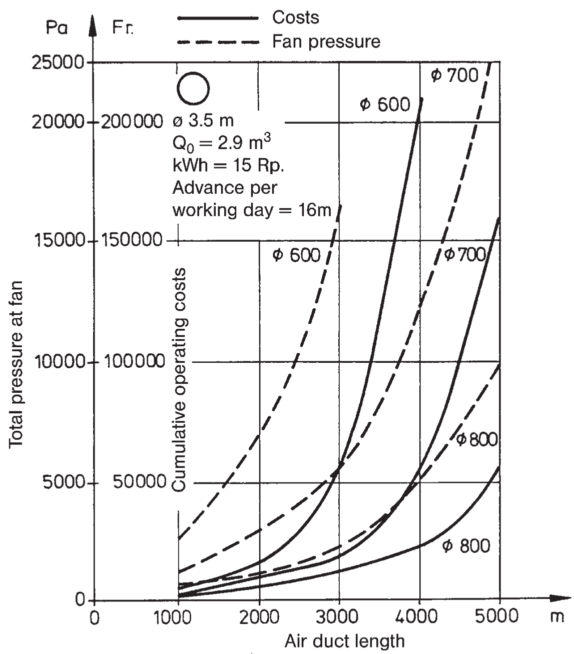


Figure 8-15 Cumulative operating costs (in Swiss Franks) of a ventilation system for a tunnel of 3.5 m diameter

8.5 Special ventilation systems

8.5.1 Ventilation for TBM drives

Basically, a tunnel being bored by a TBM can be divided into the following two zones:

- Forward machine area.
- Rearward area.

For the ventilation of the rearward area, the relevant national regulations apply such as SUVA for Switzerland, TBG for Germany and AUVA for Austria. The necessary ventilation systems and concepts do not vary greatly from those already described here.

For the application of a TBM, special ventilation measures have to be undertaken at the face.

The main problem with the ventilation of a TBM is the dust created by the excavation process. Full-face machines should therefore be fitted with a dust shield with a connection to the extract dust in the upper third. Equipment must also be provided that can work with an appropriately designed ventilation plant (details, see Section 8.3.3). Integration of a deduster into the tunnel boring machine is desirable (Fig. 8-16). The dedusting equipment must be powerful enough to keep the dust concentration below the permissible level, and must also be appropriately maintained.

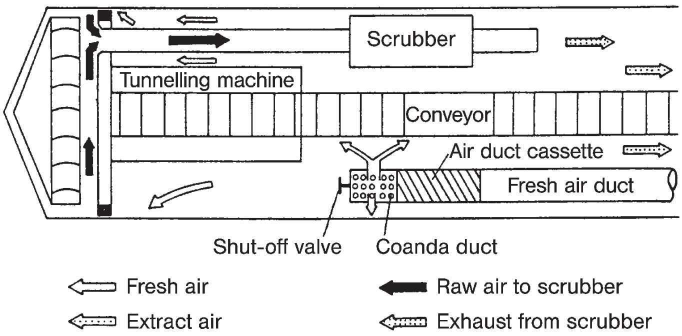


Figure 8-16 Ventilation and dedusting scheme of a full-face TBM

If only the dust produced in the excavation chamber by the disc cutters was considered, then the dust bulkhead effect integrated into all machines through the separation of excavation chamber from machine space would be sufficient, provided the dust extraction was adequate. If, however, there is a danger of methane occurence, whether due to the opening of the rock pores by excavation or from joints in the rock mass, which in an extreme case can lead to flooding of the tunnel, the ventilation of the excavation chamber by the dust extraction equipment integrated into larger machines is no longer sufficient. The excavation chamber then has to be through-ventilated. Fans installed in the bulkhead wall together with the dust extraction equipment ensure the necessary thinning of the gases in the excavation chamber.

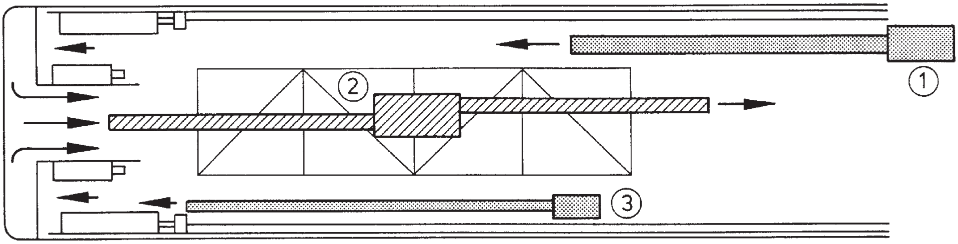


Figure 8-17 Ventilation scheme of a TBM drive [315]

In Fig. 8-17, the main air duct 1, operating with positive pressure, supplies air to the machine area. The flushing fan of the excavation chamber extracts the dust-laden air from the stator area of the TBM and feeds it to deduster 2. The outlet from the deduster extends backwards further than the supply outlet from the main air duct. Flushing fan 3, working with positive pressure, ensures sufficient fresh air supply to the backup area behind the TBM. The flushing fan extracts from the excavation chamber and blows to the deduster. Flushing fans 2 and 3 are mounted permanently in the backups. The main ventilation system normally consists of a duct, or for larger cross-sections or linger tunnels a pair of parallel ducts.

The deduster capacity can be roughly estimated from the TBM diameter. If the separation of the excavation chamber is adequate, with a minimal opening for the mucking conveyor – this opening can also be restricted with tarpaulins or plastic discs – then the deduster capacities determined from Fig. 8-18, which are based on various values from experience with various rock types, should be sufficient.

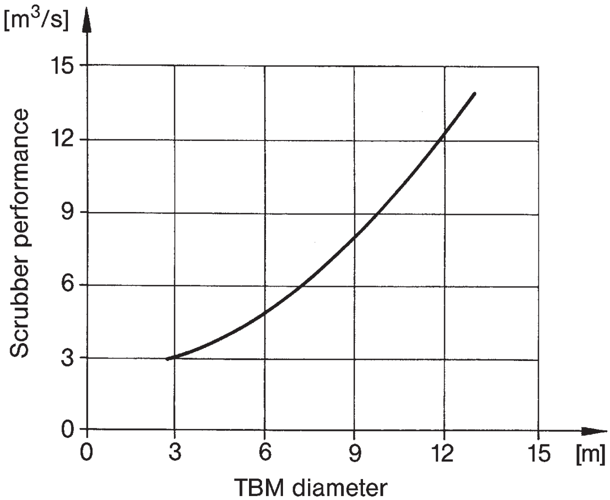


Figure 8-18 Deduster capacity dependent on TBM diameter [242]

8.5.2 Ventilation of roadheader drives

It is not practical to install a dust shield on a roadheader, which means that dedusting is problematic and often a feasibility criterion. A deduster is definitely needed in this case (Fig. 8-19). The extract inlets should be placed in the lower third of the cross-section. The extraction of dust through an extract inlet in the cutting head, which is often used, is not normally sufficient on its own.

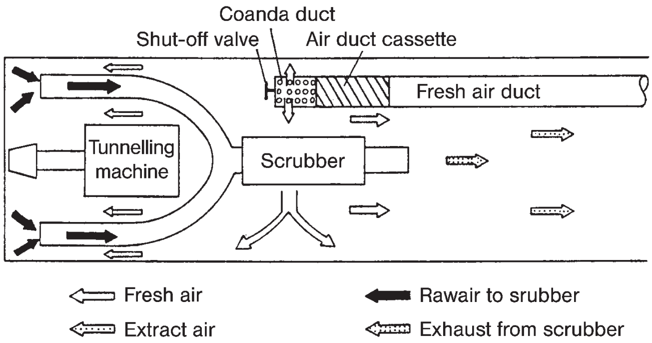


Figure 8-19 Ventilation and dedusting scheme for a roadheader machine

8.5.3 Automatic ventilation

A technical innovation was achieved in 2000 in the Landeck Tunnel, Austria, with the demonstration of an automatic tunnel ventilation system [33]. This system uses sensors to continuously monitor the dust concentration in the air and then evaluates the measured values against the permissible concentrations of fine dust. This automatically controls the output of the main air supply fan as required to provide the air necessary for the required thinning. This is done by processing information from many locations in the tunnel. This makes it possible to supply high quantities of air in good time, and also to reduce the fan output when the concentration of contaminants in the air is low. The ventilation system can thus be operated optimally and economically, which saves costs. Since the sensors currently only measure the general particle concentration but cannot evaluate the concentration of specific substances, the system first has to be calibrated on site.

This new ventilation technology has according to [33] already been used on several sites and has proved successful under various conditions.

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