Practical Tunnel Construction

Gary B. Hemphill

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Cover design: Anne Michele Abbott

This book is printed on acid-free paper.

Published by John Wiley & Sons, Inc., Hoboken, New Jersey
Published simultaneously in Canada

Library of Congress Cataloging-in-Publication Data:

Hemphill, Gary B.
Practical tunnel construction / Gary B. Hemphill.
pages cm
Includes bibliographical references and index.
ISBN 978-0-470-64197-2 (cloth); ISBN 978-1-118-33000-5 (ebk); ISBN 9780-1-118-33276-4 (ebk); ISBN 978-1-118-33444-7 (ebk); ISBN 978-1-118-35017-1 (ebk); ISBN 978-1-118-35021-8 (ebk); ISBN 978-1-118-35027-0 (ebk)
1. Tunneling. I. Title.
TA805.H45 2013
624.1′93—dc23

2012023642

Printed in the United States of America
10 9 8 7 6 5 4 3 2 1

To my father, Clayton Hemphill, 1913–2001, and all of the other tunnellers who have passed before and those to follow.

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PREFACE

This book is not written for my peers. I would not be so presumptuous to think I could teach them anything. Rather, the book is intended to give those with little or no knowledge a basic understanding of the process of tunnel construction. The book is intended to be useful to students, construction managers, tunnel designers, municipal engineers, or engineers who are employed by agencies that are exploring the feasibility of building a tunnel, planning and designing a tunnel, or building a tunnel.

The book is not all inclusive; one can only guess at the millions of words that would be required to completely cover all aspects of tunnel construction. In the space allowed, I have tried to address those areas that I think would be useful by providing the reader with a basic understanding of the tunneling process.

The amount of coverage given to each subject was based on three factors: space available, criticality, and availability of the information elsewhere. For example, there is a lot of information available about geotechnical issues as they relate to tunneling; however, because it is critical to understand the subject, a considerable amount of time is spent discussing it. The discussion on blasting spends some time on surface blasting, because it is the best way to understand the theory and it is easily adapted to underground blasting. Some rules of thumb are provided to facilitate understanding the magnitude of the activity. For example, the rule of thumb given to provide an estimate of the spacing of blastholes is probably not the actual distance. If the equation indicates the spacing between two holes is 600 mm (24 in.), what it really means is that they are located less 2 m (79 in.) apart. However, it can be useful for scheduling and cost estimating. This is true of all equations and rules of thumb in the book. Their accuracy is not dependable, because there is no way to know all the existing conditions and variables. Therefore, all of the estimates given should be taken with a grain of salt. They are included to help in understanding the subject, not as definitive answers.

The book begins with a brief history of, and the various uses for, tunnels. Hopefully, the historical perspective will prove interesting to the reader.

The technical part of the book begins with a discussion of the various geotechnical issues and how they relate to tunneling; both rock and soil are discussed. The geologic medium is used to determine the method, productivity, and ground support required, that is, the methods used to mine through the various types of geology. They range from hand excavation to the use of explosives to totally mechanized excavation.

Mechanical methods using tunnel-boring machines (TBMs) are given cursory coverage. The reason for this is the availability of information on this topic from other sources. Books by Maidl et al. (1996, 2008) provide information on both rock and soil TBMs. These books concentrate on TBMs and provide very good coverage; I can add nothing more.

The book then introduces the various types of ground support. The discussion starts by defining the types of ground support and then reviews the use of steel sets, wall plates, and crown bars for steel erection, lattice girders, and lagging.

In addition to discussing the construction of shafts and portals for tunnel access, mucking methods and tunnel haulage are discussed. The chapter on grout is longer because of the amount of material available and the importance of understanding grouting in tunneling. If the geologic material is weak, with a short standup time, or if the presence of water or both are present and they are not handled properly, it will eat your lunch. Among the books I used for information on grouting, I recommend Henn (2003, 2010) and Warner (2004) to anyone who wishes to increase their knowledge beyond what is found in this book. Other references are in the bibliography.

Since water can be such a problem, there is a chapter on water-handling methods. To complete the subject, it is necessary to include the water control elements of grouting.

In addition to trenchless excavation, sequential excavation is discussed, with a cursory look at methods. Shotcrete warranted its own chapter and is a factor in sequential excavation.

Ventilation is also discussed. The purpose of this is to give the reader an idea of how to read fan curves and how to determine the fan size. In reality, the vendor will provide a recommendation or the tunnel superintendent or manager will size it based on his or her experience.

Finally, I looked at tunnel linings. There is a great deal of information available on this subject, so I limited the discussion to key points about which readers should know. If more knowledge is desired, there are several good references available, for example, O'Carroll (2005) and Bickel et al. (1996).

To close, I hope that the reader will learn as much from reading this book as I learned in writing it. Tunneling has continued to evolve with the mechanization of excavation and lining. No one can guess what the next developments will be, but the technology will continue to improve the safety, schedule, and cost of tunnel construction.

ACKNOWLEDGMENTS

No undertaking as comprehensive as writing a technical book can be done without assistance from others, whether it is in the form of providing information or reviewing and providing criticism. My friends and colleagues Raymond Henn, Richard Flanagan, Lou Maestas, Charlie Daugherty, Eric Smith, and William Daly all reviewed sections of the book (and we're still friends), providing critiques that improved the book. Nick Barton kindly provided suggestions and encouragement. I have found my relationship with John Wiley & Sons to be informative and pleasurable, which is largely due to the patience, encouragement, and understanding given to me by my editors Robert Argentieri and Daniel Magers.

The following is a list of all those professionals who provided me with information and encouragement, for which I wish to express my gratitude. I sincerely apologize if, in the furor of getting the book published, I missed anyone.

Andrew Marshall	UK Pipe Jacking Association
Atlas Copco	Photo archives
Brian Matheson	METGROUP
Daniela A. Bedward	American Concrete Institute
David Baca	Aardvak Packers LLC
Dmitri Kopeliovich	www.subtech.com
Ellie Swinbank	National Mining Museum Scotland
Emily Lemiska	Massachusetts General Hospital Public Affairs Office
Eric Smith	Sub-technical, Inc.
Richard Flanagan	Parsons Brinckerhoff
Florian Kulke	Herrenknecht AG
George M. Morris	Macro Enterprises, Ltd
Graeme Paterson	The National Archives (UK)
Janelle	Mining Equipment, Ltd
Jim Perry	Tunnelcorp Pty. Ltd
John Capers	Austin Powder Company

Johnny Xu	Construction Planning Consultant
Kelly Arnold	Dyno Nobel North America
Kelly Balmes	Sandvik Mining and Construction
Kelly Foos	American Augers
Loures Caleres	Sandvik Mining and Construction
Luis Maestas	Shotcrete Consultant
Malcolm Mathews	Miner
Mary Hedge	MTA Bridges & Tunnels
Nick Barton	Nick Barton and Associates
Professor Peter Calder	Queen's University
Raymond Henn	Brierley Associates
Richard Flanagan	Parsons Brinkerhoff
Ricardo Martin Gosalvez	EUROHINCA, S.A.
Rob Galbavy	ChemGrout
Ryo Nakamura	Hitachi Zosen U.S.A., Ltd
Steve Kral	Society for Mining, Metallurgy, and Exploration
Susan Sekerka	Dyno Nobel North America
Susan Wamsley	Parsons Brinckerhoff
Terry Aylward	Griffin Dewatering Construction Services Group
Thomas J. Clemens	DSI Underground Systems, Inc. American Commercial Division
Tip Johnson	D'Angelo International
Wilhelm Chip Kogelmann	Alpine Equipment Corporation
William Daly	Parsons Brinckerhoff
Yoshiki Sukeyaso	Hitachi Zosen U.S.A., Ltd

I cannot forget my wife Patricia and my daughter Jennifer, who made me realize early on that if I did not agree to write the book, they would see to it that I would never have a moment's peace. I'm at peace.

In closing, I cannot overlook the contributions of people like Bieniawski, Heuer, Deere, Széchy, Monsees, Terzaghi, Barton, and Hoek, to name just a few of the professionals who have given so much to the tunneling industry.

1 Introduction

What sent humans underground? Was it for protection, to look for minerals, or for shelter? There is evidence of tunneling long before recorded history. Perhaps, after the surface outcropping of flint was depleted, humans went underground and became the first miners. In this book, the terms *mining* and *tunneling* will be used interchangeably; however one could argue that tunneling came first. Humans mined on the surface until they ran out of surface mineral and then had to tunnel to reach the minerals they sought. Simply put, mining is defined as digging underground to extract a substance of value, whether it be an ore such as gold or a material with an industrial use such as mica, whereas tunneling is digging under the surface of the earth for the sake of making the hole itself. The hole may be for transportation, water and wastewater, defense, storage, or any other need to go underground. Historically, the two principal reasons for tunneling were for water and warfare. Many engineering disciplines have existed for hundreds of years. Military engineering as a discipline resulted from the need to fortify military structures and for breaching fortifications. With military engineering, civil engineering evolved at about the same time. Later aeronautical engineering, nuclear engineering, and electrical engineering were developed. Compared to early engineering, these disciplines are relatively young. Documentation indicates that geotechnical engineering, the most important discipline used in tunneling, has its origins in the eighteenth century. Metallurgy can be traced back to when early humans first started mining and processing metals, for example, copper and iron. Tunnel/mining engineering is probably the oldest engineering discipline. When humans used an antler and dug a hole, they had to make sure that the hole would remain open and not collapse. Some soils may have stood without support, but some did not. How they solved this problem is unknown. Possibly after they used logs to build shelters, they also used logs and branches for support. What about before this? Perhaps they learned to dig their tunnels below trees so that the roots would hold the soil. Maybe archeologists have some answers.

Since more tunnels were driven before tunnel engineering existed, what does this discipline bring to the endeavor? Tunnel engineering can be divided into two areas: tunnel design and tunnel construction. Although there is some overlap between these two areas, both require the same information, but for different reasons. The tunnel designer is interested in the geotechnical properties of the ground. This information, which includes ground strength parameters, water, and so on, is used to help determine tunnel alignment, depth, type of lining necessary, and other details to safely provide the end use. The tunnel designer must have a sense of what is practical. Because tunnels are expensive, the designer must come up with the most cost-effective design to achieve the goals for the tunnel. The tunnel designer must have enough knowledge of tunneling techniques to make sure the design is “constructible.” Often the tunnel construction engineer is involved with the designer to provide construction insights. The tunnel construction engineer may then assist construction by providing design insights to the tunneler.

Historically, tunneling has been filled with hazards. Miners have been killed or maimed by poisonous gases, cave-ins, falls, and fire; some have been crushed by equipment; and some have been worked to death, for example, slaves. Although many improvements have been made in the last 100 years, some of these hazards still exist. In the middle of the last century, one could assume that there would be one death per mile of tunnel. Fortunately, that statistic no longer exists. The most important contribution that the tunnel construction engineer can make to tunneling is safety. There is no need to discuss the moral and economic reasons for this; they are well known.

Like the designer, the tunnel constructor is interested in the geotechnical attributes of the tunneling medium. Like the designer, he or she needs to determine the ground support required, the best mining method, in the case of rock the anticipated bit life, and so on. The constructor also needs to know about the water, not so much from the final lining perspective, but how to handle the water during mining.

Tunnel engineering is still very much an art. There are models that will provide a great deal of insight as to the in situ geotechnical parameters; however, the decision in the field could very well be made by someone who does not know the name of the rock being tunneled through but knows what will hold it up. Tunneling is referred to as a “gray beard” business. That is, experience is a critical factor in tunnel engineering. In a typical civil engineering design office, there will be a few older engineers, but the majority of engineers will be younger engineers who are there to apply what they learned in school under the guidance of older engineers. Because it takes years to learn the art of tunneling, a typical tunnel design office generally has a larger percentage of older engineers. This is illustrated if we look at steel and concrete versus rock and soil. Steel and concrete have known strength parameters and it is known how they will act in various circumstances. This is not the case with rock and soil. All that one can truly depend on learning from a boring is what is in the boring. The geologic material could change inches away. Therefore, tunnel engineers have had to have seen similar material and have an opinion of how it acts under various conditions.

The tunnel engineer does not get to choose the tunneling conditions. Unlike surface construction where the conditions are generally visible, the tunneler is going into areas with conditions he or she can only assume. Later in the text. the engineering properties of rock and some of the exploration techniques used to try to determine those properties are discussed. Despite all of the exploration and engineering analysis, the most important piece of information the engineer needs to know about rock is the boring. The rock type and characteristics can change next to the borehole. There may be an outcropping that indicates the strike and dip of discontinuities, but in a deep tunnel that may provide little information; geophysical methods of exploration may be used, but these methods are limited. When all exploration tools are available, the tunnel engineer can be quite good at predicting the conditions. Despite this gloomy description of the knowledge that tunnel engineers have before tunneling commences, based on the exploration and analysis, they are good at predicting conditions. However, one still has to exercise caution and be prepared for surprises.

Some believe that these problems are what make tunneling interesting. Attracted to the unknown, the miner is prepared to enter the bowels of the earth and handle any problem, whether it be a change in the ground conditions or unexpected inflow of water. They may have to modify the mining method or ground support or both or grout to seal or slow the inflow of water, but they keep driving forward. Miners do not like to not advance the face. Although the work is hard and dirty, when a tunnel breaks through in the right place, the tunneler remembers it was done blind. As when early explorers had maps but the maps were incomplete, the miner has geotechnical reports that cannot tell them everything that will be encountered. Tunnellers, just as they did a thousand years ago, enter a world of unknown conditions and risks and succeed.

The oldest and largest known qanat is in the Iranian city of Gonabad and after 2700 years still provides drinking and agricultural water to nearly 40,000 people. The main well is more than 360 m (1180 ft) at its deepest and has a length of 45 km (28 miles). It took a special individual to build this water supply.

The military application, like the qanats, can also be traced back thousands of years. Early combatants would tunnel under the walls of the city or under the enemy's lines to plant explosives to breach the walls or to smuggle in soldiers to take the enemy from the rear. In early January of 1917, British General Sir Herbert Plumer issued orders for 20 mines to be placed under German lines at Messines. For the next five months more than 8000 m (5 miles) of tunnels were dug under enemy positions and 600 tons of explosives placed in position by miners hired by the military. Simultaneous explosion of the planted explosives occurred on June 7. The blast, which was so loud that it was heard in London, killed an estimated 10,000.

What makes a tunneler willing to violate the earth and venture into the unknown subsurface? To tunnelers the sounds of water flowing or air leaking from a pipe when equipment is not running, the texture of blasted rock, the smell of stale water and oils and other lubricants, the darkened environment where they learn to see with less light, and the taste of salt in their own sweat are what make them feel at home. In one of the most dangerous work environments in modern industry, the tunneler feels safe.

Why do tunnelers do this? Perhaps they are the third generation in their family to make this journey. And a journey it is. Perhaps they are attracted to going into an unknown environment. If one can forgive the use of a contemporary phrase, they do it because they go where no man has gone before.

2 Geotechnical Considerations—Rock

The geologic material/conditions through which a tunnel is driven is the most important information for the planning, engineering, and construction of the tunnel. This chapter will provide an introduction to geology, exploratory methods used, and the engineering properties of rock. In addition, rock mass classifications will be introduced, giving the reader a general understanding of the methods.

For the sake of this writing, the geology relates to the type and structure of the rock through which the tunnel is to be driven, for example, the type of rock. Geotechnical considerations are the engineering properties of the rock. This can be the rock tensile strength, compressive strength, and discontinuities in the tunnel area. The porosity and permeability will provide an indication as to the ability of the rock to permit water inflow into the tunnel.

SITE INVESTIGATION

The purpose of a site investigation is to learn about the geologic medium through which one will be tunneling by determining the origin and condition of the rock or soil. Direct investigation will provide the geology and engineering (geotechnical) properties of the soil and/or rock as well as information about the existence of groundwater. The purposes of subsurface exploration are to determine the origin and condition of the material to be blasted or through which tunnel alignment is proposed, the physical and engineering properties of the material, the presence of water, the presence of gases, and the geologic structure of the material.

The investigation program must be planned to determine borehole locations and other investigative methods. It is important that the designer be involved in planning and implementing the exploration program from the beginning.

Outcroppings or other geologic features may be exposed on the surface that can give an indication as to the subsurface geology. A gorge, cliff, or highway rock cut with drilling borings along the alignment to expose the stratigraphy of the rock may also be used to determine the dip of the formation.

Funding should not dictate the investigation program. Often, because the investigation phase is not done completely, for every dollar “saved” during the investigation, a multiple of it is spent during the construction. That is, it is often cheaper to spend the money for the investigation rather than use mining techniques that are not the best for the condition or run into problems that were not planned for. Also, it is better to have no information than have information that is insufficient. Knowing just a little can lead to a false sense of knowledge and disaster.

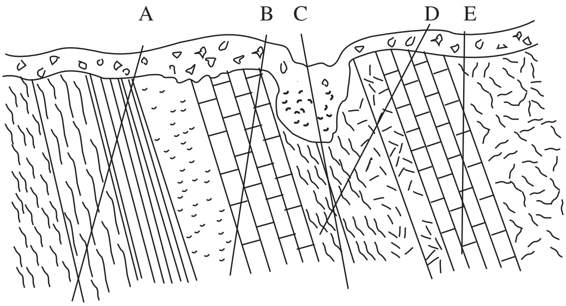
Borings

The cost of geotechnical investigation may equal approximately 2–7% of total project cost, and, in the case of a tunnel, the level of exploratory borings may average approximately 1.5 linear meters of borehole per route meter of tunnel alignment (U.S. National Committee on Tunneling Technology, 1984). That is, a 5000-m (16,400-ft) tunnel with 30 m (100 ft) of cover, requiring boreholes 35 m (115 ft) deep, should, according to the recommendation, have 215 boreholes, or a borehole at approximately every 25 m along the tunnel alignment. Unless the ground is very complex, it is unlikely that this many borings will be necessary. By many standards, this is more than is generally necessary. A borehole every 60 m (200 ft) along the tunnel alignment will be adequate for most tunnels (Proctor and White, 1977) and borings should not be more than 1 km (3280 ft) apart. However, if the material between boreholes varies, a more conservative standard may be required.

Perhaps a more reasonable approach for a long tunnel would be to start with borings 150 m (500 ft) apart along, or parallel to, the tunnel alignment. If the borings or surface features indicate differences in the material, then more borings should be placed between the borings. This should be done until the alignment between the borings can be characterized with a high degree of certainty. The borehole locations can be determined based on surface outcroppings. In addition to the alignment, portal locations and shaft locations should be thoroughly investigated.

The layout of borings should take into consideration the structure of the material. What is the strike and dip of the rock? Does the soil appear to be consistent or is there a chance of a change in the makeup of the soil? When tunneling in Houston one can experience a sudden change in material, for example, when the tunnel intersects sand pockets. When mining in Houston's Beaumont clay, a sudden change in ground from clay to sugar sand can cause the tunnel to fill with sand. In one case, a shaft had to be sunk from the surface to retrieve the machine. Of course, no boring program can intersect all sand pockets. However, if one or two do, one knows to be prepared for them.

For tunnels in mountainous terrain, borings from the surface may be prohibitive or it may be necessary to use very few borings or drive a pilot tunnel. When determining boring locations, knowledge of the geology of the area is necessary to determine fault locations. Also in steeply dipping rock, vertical borings from the surface may be of little use. It is better to have a small amount of good information than a lot of useless information. When the ground



Boring C is located to follow the presumed path to determine if there is gouge and, if so, how much.
Boring D is drilled at an angle to allow it to penetrate different strata and cross the path boring C.

Figure 2.1 Boring Locations in Dipping Beds

is steeply dipping, it is important to angle the holes to cross the strata. Also, valleys and low areas may represent faults in which a softer and potentially weak area may be examined. Figure 2.1 illustrates potential boring locations for a tunnel in steeply dipping beds.

Tunnels and underground blasting with very limited geologic information should have favorable contract terms to deal with changes or unanticipated conditions. If the contract lacks such flexibility, the tunnel support will have to be designed for the worst ground conditions. This will raise the bid prices and construction costs. However, if there is flexibility in the contract, the design can have less conservative ground support with a contract mechanism that will permit changes to the support, during construction, reducing bid prices. No exploration is truly accurate. One must remember that a 54.7-cm (2.16-in.) core provides information about the core only. A meter away the ground can change dramatically. Efforts to improve the accuracy of site investigation require additional methods such as geophysics.

Obtaining Samples and Testing of Rock

When determining ground support requirements and mining methods, the following basic properties of an in situ rock mass should be considered:

1. Rock deformation under load
2. Compressive strength
3. Tensile strength
4. Shear strength
5. Bearing capacity
6. Permeability

7. Internal stresses (because of overburden weight, pore water, tectonic stresses, etc.)
8. Discontinuities
9. Condition of discontinuities

Rock samples are generally obtained by core drilling the in situ rock, but samples may be cubic or prismatic. The core is generally designated "NX" and is 54.7 mm (2.16 in.) in diameter. Because of core breakage, the NX designation is the smallest practical diameter. The samples are logged as to the original location in the formation. The core is checked for the rock quality designation (RQD) while still in the field. The samples are then protected and shipped to the laboratory for testing. The samples are tested to render a practical identification of rock quality and to determine the probable resistance of the material to deformation of the discontinuities or weaknesses in the rock.

The testing program should provide sufficient data to make a probable determination of the ground's potential advance rates and stand-up time. To do this, the contractor, like the engineer, needs knowledge of the types of rock, rock stratigraphy, unit weight, unconfined compressive strength, Poisson's ratio, elastic modulus, angle of friction, cohesion, groundwater elevation, hardness, drillability, water chemistry, RQD, and permeability. The reasons the contractor needs to know these rock characteristics vary from those of the designer. The contractor is concerned with the most cost-effective blasting design or tunneling method. The type of material, the engineering properties, and the groundwater are paramount issues to the contractor. The contractor is concerned with bit wear, borehole wetness for determining the explosives needed, and, when underground, maintaining the opening. In addition, the contractor needs to have information regarding the strike and dip of the discontinuities as well as the joint spacing, conditions, fillings, and number of joint sets.

Geophysics

Geophysical investigations will identify general material types; locate inconsistent geologic conditions such as buried valleys, shear zones, and deeply weathered zones; determine the approximate elevation of the interface of the overburden and rock; and assist in determining effective locations for borings.

A nondestructive method for investigation for tunnels is geophysics. Although it is very subjective and depends on interpretation, with other exploration tools, this method is used to help determine the kind of ground through which the tunnel will be driven. Often done prior to the borings, geophysical methods help locate potential areas for placing the borings and may eliminate less favorable alternate borings sites.

Geophysical techniques determine the geologic sequence and structures of subsurface rocks by measuring certain physical properties. The properties most used are density, elasticity, electrical conductivity, gravitational attraction, and magnetic susceptibility. To determine these properties, seismic refraction and reflection surveys are conducted, electrical resistivity is measured, and gravity and magnetic surveys are performed.

Seismic methods, refraction and reflection, and resistivity are used to measure and record simulated fields of force applied to the area under investigation. Refraction method is based on the property of seismic waves to refract, or be bent, measuring the travel time of a seismic energy wave down to the rock or other areas that have distinctly different density. Seismic waves travel outward from an energy source and reach a detector. The detector first senses the waves that went directly to it along the ground surface, and then it senses waves that went downward, were bent (refracted) at a deep layer, and then left the deep layer and came back to the surface. The shock waves that return from the rock are refracted waves. Based on favorable density contrasts that generally exist between geological materials, the refraction method is utilized to provide detailed information on the distribution and thickness of subsurface layers with characteristic seismic velocities. Although reflection uses similar equipment as refraction, the field and processing time for a given lineal footage of seismic reflection survey are much greater than for seismic refraction. However, seismic reflection can be performed in the presence of low-velocity zones or velocity inversions, generally has lateral resolution vastly superior to seismic refraction, and can delineate very deep density contrasts with much less shock energy and shorter line lengths than would be required for a comparable refraction survey depth. They are applicable to the determination of horizontal or near-horizontal changes or contacts, whereas magnetic and gravimetric techniques are generally used to define lateral changes or vertical structures.

Seismic methods require a sudden release of energy. The energy may be the result of the detonation of explosives or mechanical pounding. The shock waves generated by the release of energy radiate out in a hemispherical pattern from the point of the energy source and are compressional, P; dilational shear, S; and surface waves. Since the elastic modulus increases with depth, the shock wave velocities generally increase the deeper the waves travel below the surface.

Seismic waves follow principles of propagation, reflection, and refraction similar to light waves. The equipment required for these techniques consists of an energy source, an instrument to detect the waves, a geophone, and a device that records the waves' travel time.

Reflection and refraction are based on the fact that they can detect the time that the shock wave is generated. Reflection is the most used technique for seismic investigation. Because it can gather information from many horizons several kilometers deep, it is commonly used in petroleum exploration. Refraction is the most common method and consists of sending seismic waves into the ground and measuring their arrival at various points. It measures the change in direction of a wave due to its change in velocity. The velocity of the elastic wave passing through a geologic material is a function of the structure of the material, composition, and in situ stress. Velocities increase with density, water content, and low porosity.

Compression waves travel faster and are generated and recorded more easily than shear waves. They are used almost solely in seismic exploration. Shock wave velocity depends on several variables, including rock fabric, mineralogy, and pore water. Generally velocities in crystalline rocks are high to very high. Velocities in sedimentary rocks increase according to amount of consolidation, decrease in pore water, and increase according to degree of cementation and diagenesis. Unconsolidated sedimentary deposits have maximum velocities that vary as a function of the volume of the void, either air filled or water filled, mineralogy, and grain size.

The reliability of using seismic wave velocities to identify features improves above the water table. Differences in wave propagation velocity between two adjacent materials increase. To identify sloping features, the seismic lines must be reversed and vertical features narrower than three times the geophone spacing will not be identified and isolated.

Also, features with lower velocities will not be detected if under a higher velocity feature. Water velocity is about 1500 m/sec (5000 ft/sec), which is the same as for some compact gravels, mudstones, and weathered rock. It is also difficult to obtain meaningful data when the ground water table is at or near the rocks.

Resistivity refers to the resistance of a material to electric current and is dependent on the chemical composition and degree of saturation of the material. The electric current is applied to the ground through two electrodes. Changes in potential across the known distance between the two electrodes are used to help evaluate the types of material. The preferred unit of measurement is ohms. Wet clays and silts, mineralized water, and some metal ores are all good conductors. Conversely, dry sands and gravels and crystalline rocks without metal ore are poor conductors. Moisture content has a significant impact on the results. Resistivity is most helpful when used with seismic techniques.

The resistivities of superficial deposits and bedrock differ from each other. Resistivity may be used in their detection and gives their approximate thicknesses and relative positions and depths. Tests should be conducted at the borehole to establish a correlation between resistivity and lithologic layers. If the correlation cannot be established by this method, an alternative method should be used.

TYPES OF ROCK

Igneous Rocks

Lavas are magma that reach the earth's surface and flow out over it. This magma cools to form extrusive rocks, which are either of a fine crystalline grain or of a glossy texture. Intrusive rocks have a coarse crystalline texture. The most common intrusive rock is granite. Large bodies of intrusive rock deep in the earth's crust are known as "plutonic rocks." Hypabyssal rocks are intrusive rocks that are fairly near the earth's surface and fill cracks or fissures, forming sheets between existing layers. The hypabyssal rocks cool more quickly than plutonic rocks and therefore have a smaller grain. Examples of hypabyssal rocks are felsite and dolerite.

Batholiths are the largest form of plutonic intrusions. They consist of a mass of igneous rock that has been pushed up through the overlying rock strata, causing the overlying rock to be displaced and broken. Because of the disruption to the overlying rock, batholiths are called "discordant intrusions." After the displaced rock has eroded away, the batholith will appear on the surface. To examine a rock in the field, the rock should be broken, and the color and grain size should be examined to determine the rock origin. Table 2.1 is a listing of common igneous rocks grouped as intrusive and extrusive. There are two types of minerals that form igneous rocks: the *felsic*s (light colored), which consist of quartz, feldspar, and feldspathoid, and the *mafic* (darker colored), which consist of olivine, pyroxene, amphiboles, and micas. There are subdivisions based on color, but it is somewhat inconclusive because weathering will change the surface coloring of the rock. Igneous rocks tend to be more abrasive and consequently cause greater bit and cutter wear. With igneous rocks, the key to fragmentation are the discontinuities. Most igneous rock is complexly jointed. As with mechanical tunneling and blasting, an understanding of the discontinuities will help achieve the desired fragmentation and provide insights into the required ground support. Sometimes igneous rocks can be deeply altered by weathering, and hydrothermal fluids and gases may help to precipitate fragmentation but can increase the production of fines in the muck. Also, in these highly weathered areas, when blasting, a propensity for gas to vent exists, because weak fractures affect breakage. Highly weathered areas may increase ground support requirements and act as a conduit for water flow into the tunnel.

Sedimentary Rocks

Sedimentary rocks are formed near the earth's surface by deposition and accumulation of clastic and biogenic sediments. Clastic sediments are rocks that are

Table 2.1 Intrusive and Extrusive Rocks

Intrusive Rocks	Extrusive Rocks
Quartz	Andesite
Diorite	Basalt
Peridotite	Diabase
Gabbo	Scoria
Pegmatite	Trachyte
Granite	Obsisian
Syenite	Rhyolite

eroded either mechanically or chemically or both, and the resultant particles are transported and deposited to form sediments. Water is the primary transporter of material that forms sediments; however, wind and ice also transport sediments.

Chemical erosion is caused by water that has substances or chemicals dissolved in it that attack the rock, causing portions to weaken and so erode mechanically with less effort.

Mechanical erosion is erosion caused by pressure or forces inflicted on the rock. Examples of mechanical erosion are the expansion of a tree root causing a rock to crack and, as discussed below, water freezing in cracks in the rock and fracturing the rock when it expands.

Weathering is the breakdown of rocks in situ and may be either mechanical or chemical; however, most climatic weathering is both. Chemical weathering is caused mainly by water, whereas mechanical weathering is caused by the creeping of rocks and slides or slips. Mechanical weathering can result in the physical shattering of the rock, as when a boulder rolls down a hill and shatters when it reaches the bottom and impacts an obstacle.

As mentioned above, frost is the main climatic weathering source. As water freezes, its volume increases approximately 9%. Therefore, when water is trapped in a crack or pores in rock and freezes, the physical expansion of the ice creates pressure on the rock that may reach 13.8 MPa (2000 lb/in.^{2}). Weathering due to frost is most common at high altitudes and in cold regions. The primary areas for this type of weathering are where the temperature often goes above and below the freezing temperature of water. The cycle of frequent freezing and thawing causes fatigue, resulting in rock to failure. Soft rocks tend to be more affected by rain than harder rocks. Hard rocks, such as quartz, resist weathering. Although sandstones and shales are resistant to chemical weathering, the cement that binds the particles together is not resistant and can erode.

Erosion by water is generally by rivers, oceans, and groundwater. Rivers erode the terrain or the conduit in which they flow, that is, riverbeds and banks. The eroded material, called “load,” is transported downstream. The distance the eroded particles are transported depends on the particle size in relation to the speed and turbulence of the water flow. The faster and more turbulent the water flow, the farther the larger particles will be carried. The largest of the particles that the water flow can move will be rolled or bounced along the river bottom. As the particles decrease in size, some of the larger particles will be moved in short, sporadic leaps or jumps; this type of movement is referred to as “saltation.” As the slope of the flow line of the river decreases and the velocity of the water flow decreases, the river drops the load. Table 2.2 lists the types of particles with their corresponding sizes.

Clay, the finest particle, drops to the bottom when the water becomes still; this is common in floodplain areas along rivers. Wind and ice move particles similarly, but water is the main cause and transporter of eroded particles.

Table 2.2 Soil Types and Particle Sizes

Soil Type	Particle Size (mm)
Boulder	>300.0
Cobble	150.0–300.0
Gravel	2.0–75.0
Sand	0.05–2.0
Silt	0.002–0.05
Clay	<0.002

The process by which loose, unconsolidated particles are changed into cohesive sedimentary rocks by low temperature and pressure near the surface is called “diagenesis.” This process requires no movement of the earth’s crust, and the most common method is by the compaction and cementation created by the weight of the additional sediments above. That is, as sediments are added, the sediments on the bottom are subjected to the weight of the sediments deposited on top. As the material is compacted, the water is forced, or squeezed, out. The soluble and unstable particles or minerals dissolve and enter the pores of the sediment, forming cement; or the cement may be added to the sediment by groundwater. The common cements of sedimentary rock are silica and iron hydrates. The final stage in the sedimentation process, where loose particles are cemented and form a rock mass, is called “lithification.”

There are two major types of sedimentary rocks: clastic and nonclastic. *Clastic* rocks are formed by the cementing of particles from other rocks and are classified by base or grain size. *Nonclastic* rocks are formed by chemical or organic processes. The most common types of sedimentary rocks are sandstone, shale, conglomerates, breccias, limestone, dolomite, and bituminous coal.

Sandstone is composed of sand and cement. It is porous and permeable to fluids. Sandstones are clastic with many of the particles being quartz. Although sandstone is fairly soft and easily penetrated with a drill or mechanical means, it is very abrasive, and bit cutter tool wear can be high. Siltstone is quite similar to sandstone, but the particle sizes are much smaller. Shale is formed from the compaction of mud and clay. It has a hardness of 2 on the Mohs scale of hardness and is easily penetrated with a drill. Limestone is almost entirely calcium carbonate, with some clay and sand. Limestone is formed on the sea bottom, and its proximity to the shore determines the quantity of clay found in it. That is, the closer to the shore, the higher the clay content. It is 3–4 on the hardness scale, but depending on the how the particles are cemented, penetration hardness can vary considerably.

Dolomite is similar to limestone except that it contains magnesium carbonate. A chemical analysis is required to distinguish it from limestone. Bituminous coal is nonclastic soft coal that is primarily carbon from decayed vegetable matter.

Sedimentary rocks can be very diverse in its stratification. That is, the bedding can vary from limestone to shale in the adjacent bed. These abrupt changes in the rock can create tunneling problems. Despite the potential problem of diverse stratification, when tunneling in sedimentary rock, thinly, tightly bedded rock with numerous tight joints can be very favorable for blasting and mechanical tunneling conditions.

Metamorphic Rocks

Metamorphic rocks are rocks that have undergone a change in mineral content, texture, or both from their igneous or sedimentary predecessors. Metamorphic rocks are formed by the high pressure deep within the earth at temperatures ranging from 200 to 980°C (400 to 1800°F). The pressure is from the weight of the overlying rock and from movements of the earth's crust. The pressure and heat of the metamorphic process cause a rearrangement of the molecular structure of the minerals, creating new minerals. For example, garnet is a product of the metamorphic process on granite. In addition, the reduction in volume creates denser materials. Table 2.3 is a listing of some metamorphic rocks and the rocks from which they are formed.

The most common or easily traceable process is that of anthracite. Anthracite is the metamorphic derivative of bituminous coal. Figure 2.2 illustrates the developmental stages of anthracite.

Table 2.3 Common Metamorphic Rocks

Name	Rock Texture	Commonly Formed by Metamorphism by
	Unfoliated	
Quartzite	Granular (breaks through grains	Sandstone
Marble	Granular	Limestone, dolomite
Hornfels	Dense	Fine grained rocks
	Foliated	
Slate	Fine grained	Shale, mudstone
Pyrite	Fine grained	Shale, mudstone
Schist	Fine grained	Shale, mudstone, andesite, basalt
Gneiss	Coarse grained	Granite

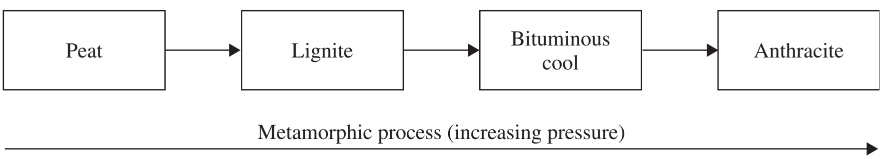


Figure 2.2 Formation of Anthracite

Metamorphic rocks are generally very complex in composition, fabric, and structure. They are generally very hard and abrasive to drill. Anyone who has drilled or mined in quartzite can appreciate how tough the quartzite can be on drilling tool bits.

As with other rock types that are fractured with tight discontinuities that do not vent the gases, one can achieve good fragmentation from blasting or good penetration rates when tunneling. However, the fractured rock may cause ground support problems.

Conglomerates are sedimentary rocks formed from gravel and are sometimes referred to as "pudding stone." Conglomerates have a visual likeness to concrete and are clastic. Brecca is similar to conglomerates, but the particles are angular rather than rounded. These conglomerates are analogous to concrete made with river-wash or manufactured aggregate.

Discontinuities

When determining a rock's engineering properties, the recognition of discontinuities is critical. Discontinuities are natural fractured surfaces of a rock mass and their type depends on their origin. The most common discontinuities are bedding planes in sedimentary rock. The metamorphic equivalent of a bedding plane is a foliation plane. The most common discontinuities in all rocks are joints. Joints were formed by the cooling of crystalline and extrusive rocks and in regional tectonic compressive and tensile stress fields associated with plate tectonic movement. Joints that have undergone limited shear displacement are called shear planes. However, those that have undergone more than 25 mm (1 in.) or so of displacement are called "faults." Displacement discontinuities found in bands of parallel and semiparallel fractures are called shear zones and fault zones.

When conducting rock mass characterization, the frequency, spacing, orientation, length, material, if any, in the joint, and surface characteristics of the discontinuity are important.

Rock can be either elastic or plastic. Elastic rock will deform up to the point of rupture and will fracture from shear, producing joints and faults that relieve some of the stresses in the rocks. Plastic rock is weak and will deform easily. It is abundant in clay-type material and usually contains few fractures. If fractures do develop, they are the result of tensile stresses. Very stiff plastic rock may fracture in either tension or shear.

Formations devoid of stratification are much more favorable for tunneling than formations composed of several layers of shales or granular masses of varying degrees of solidification. The direction of the strike and dip of the layers is extremely important.

Strike and dip are descriptions of the orientation of the discontinuities and bedding. Strike is the compass direction of a line considered to be drawn along the bedding plane that is horizontal. The dip is the angle between the horizontal plane and the plane of the bedding measured at right angles to the

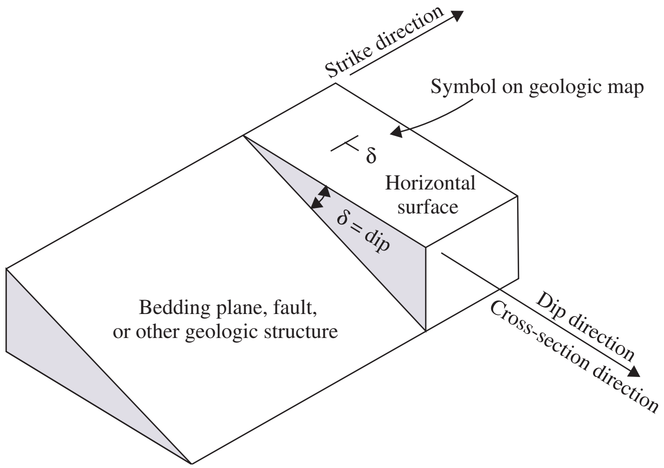


Figure 2.3 Strike and Dip (Goodman, 1989)

strike. Figure 2.3 illustrates how to determine the strike and dip of a rock mass.

Generally, steeply dipping strata facilitate weathering and infiltration of surface water. The orientation of the bedding can have great effects on tunneling. When the tunnel axis is perpendicular to steeply dipping bedding, the mining will be favorable, because the bedding spans the tunnel, generally requiring less ground support. Steeply dipping strata may facilitate water penetration and weathering action. When the tunnel axis is parallel to steeply dipping bedding, the bridging action may be limited to the shear strength due to friction and cohesion between adjacent layers.

Horizontal bedding with relatively thick layers is generally good for stand-up time. It can be hazardous if it is dipping at 5°–10°. This may result in roof spalling. Figure 2.4 illustrates locations of tunnels relative to strike and locations of tunnels relative to syncline and anticline.

Folds

When horizontal compressive forces are present (tectonic), rock will distort into a wavy pattern called folds. Folds are wrinkles, or warps, of the rock

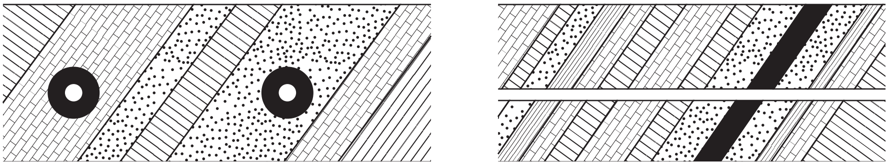


Figure 2.4 Locations of Tunnels Relative to Strike (Széchy, 1973)

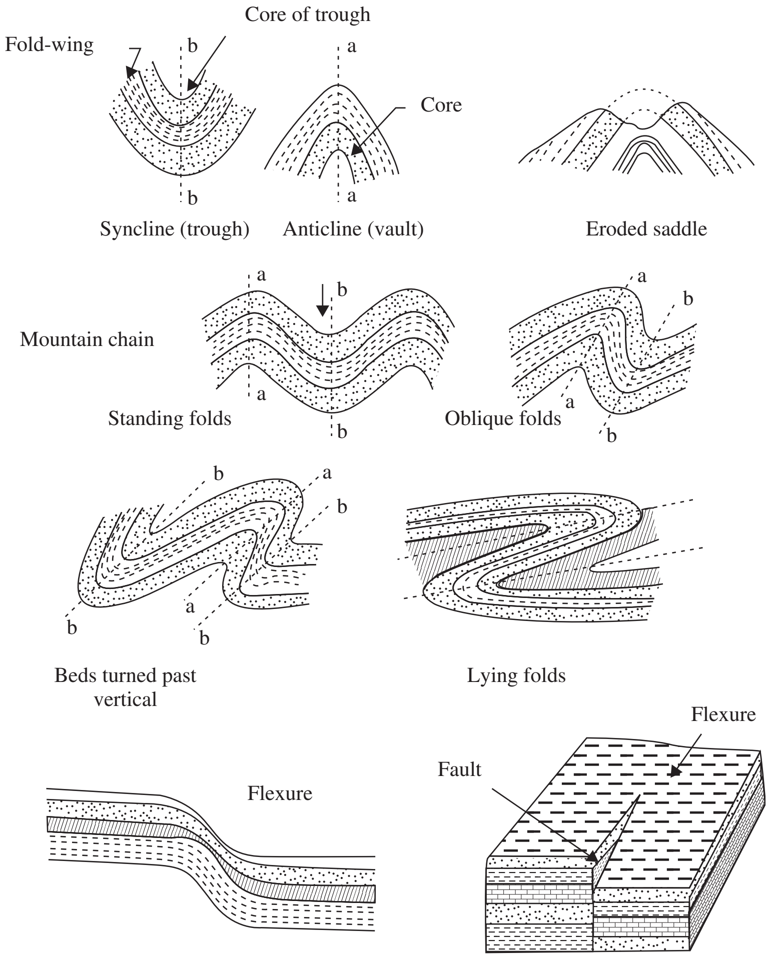


Figure 2.5 Main Fold Formations (Széchy, 1973)

bedding where regular patterns of crests and troughs can be observed in a rock formation. Folds are generally more often observed in layered rocks, where the differences in color or texture of the rock make it easier to see the folds. When a fold is concave, that is, forming a trough, it is called a “syncline,” and when the fold is convex, forming a crest, it is called an “anticline.” See Figure 2.5.

Folding strata give rise to the pressure on the core and tension in the crown. If the tunnel must be driven following the strike, the tunnel should be located in the anticline. Passing through the crest of the fold will be subject to less pressure. Water will tend to seep away from the tunnel.

A permeable layer in an anticline will bear little or no water, whereas a permeable layer in a syncline will bear considerable water. This can affect the degree of rock weathering. More water will percolate through longitudinally disturbed zones and some rocks will lose strength if small pieces become saturated, potentially manifesting in ground stability problems.

There are two main types of folds: flexural and slip folds. Flexural folds are created when adjacent layers slip past each other with the layers retaining their original thickness. Slip folds, also known as shear folds, are found primarily in sedimentary or metamorphic rocks under extreme confining pressure; the folding is caused by slipping along closely spaced slip planes.

Fractures

Fractures are cracks in the rock mass. Their orientation is very important because the shear strength along the fractures is less than that of the rock mass, thus forming potential failure surfaces. The three types of fractures are faults, joints, and shear zones.

Faults

Faults are the result of an accumulation of stresses in the rock to the point of rupture and a displacement along the area of rupture and are often associated with earthquakes.

They are similar to a shear zones except they have undergone greater shear displacements. Many geologists regard 75 mm (3 in.) of displacement as the threshold from the shear zone to the fault. If groundwater is near a fault, the temperature of the water is increased and chemical reactions are assisted, causing changes in the rock. This change or alteration of the rock about faults is called “argillization.” Argillization produces claylike minerals at the cost of feldspars, causing the materials within the fault area to have a high degree of plasticity. Faults are generally the shear planes along which the movement or concentration of forces takes place.

There are three types of faults: normal, reverse, and wrench faults. As illustrated in Figure 2.6, normal faults are faults in which the block lying above an inclined fault surface moves downward relative to the other block. Normal faults are the result of tensile stresses.

Reverse, or thrust, faults are faults in which the block above the incline moves upward relative to the other block. Mylonites (zones of soft, pulverized rock that are most common with granite) are formed in areas that have been subjected to reverse faults.

Wrench faults are the result of shear stresses along a vertical plane that cause a horizontal movement of the blocks relative to each other.

Joints

A joint is a fracture that has not undergone any shear movement. Joints can be the result of cooling of igneous rocks, tensile tectonic stresses, and tensile

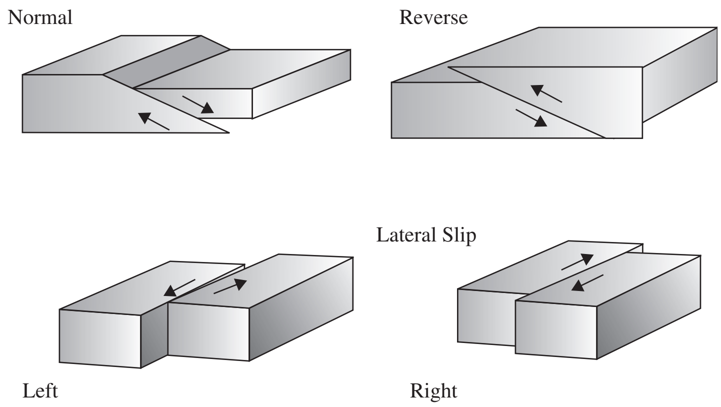


Figure 2.6 Faults

stresses from the lateral movement of adjacent rock. They usually occur at fairly regular spacings. The joint patterns are never random and a group of such joints is called a set. It is the absence of movement that distinguishes a joint from a fault.

Shear Zones

Small fractures that have experienced a small shear displacement, perhaps only a few centimeters, are called shear zones. They may be caused by various stresses in the ground. Unlike joints, they do not appear as sets and are often conduits for water.

The uniaxial compression (unconfined compressive strength) test is generally known to be the most rigorous strength test of a material, and it is also the most common test (Jumikis, 1983).

The engineering properties indicate the strength and durability of rock. For most of these properties the reasons are self-evident. However, before looking at any engineering property of a rock, one must keep in mind that rock is a nonhomogeneous material and the engineering parameters will vary according to its location and direction and characteristics of the discontinuities affect the ground support required and how efficiently the tunnel can be driven. In short, the rock's material has a nonuniform structure and, because of planes of weakness and other properties, the material will vary within the rock mass. However, these parameters are still of interest.

Strength is the ability of a material to resist external forces. The strength of a rock may be determined by laboratory testing, a loading test in situ, or geophysical methods. The strength in question refers to both compressive and tensile strength. Generally, because of fabric and microfractures in the rock, tensile strength is about 10% of compressive strength.

Basically, the stronger the rock, the more energy will be required to fragment or penetrate it. Therefore, the assumption might be made that stronger rock will require more explosives or a more powerful tunnel-boring machine (TBM). This is not necessarily the case. If the stronger rock is intact and free of discontinuities, a more powerful explosive will probably be needed but the reduced ground support required may improve advance rate. However, should the rock mass have many cracks and other discontinuities, it is likely that good fragmentation will be achieved regardless of the characteristics of the explosive, the TBM will advance faster, and more ground support will be required. The TBM can be designed to reduce ground support needs. This will be covered later in Chapter 7. The most important thing that one should know about the rock mass to be blasted or tunneled is the amount and type of discontinuities in the rock mass. Generally speaking, a rock mass with many discontinuities that are very dense or filled (so that gases do not escape) will fragment efficiently.

Hardness

Hardness is measured with the Mohs scratch test (Table 2.4). The test is a simple field-friendly way to determine the resistance of a rock to scratching or penetration, as with cutter tools or drills. The drillability of a rock provides a measurement or indication of the difficulty or ease with which a rock is penetrated with cutter tools or rock drills.

To test the hardness of a mineral, try to scratch the surface of the sample with another mineral or other object. If the unknown sample cannot be scratched by

Table 2.4 Mohs Scale of Hardness

Hardness	Defining Mineral	Tool
10	Diamond	
9	Corundum Tungsten carbide	
8	Topaz Hardened steel	
7	Quartz	
6.5		Steel file
6	Feldspar	
5.5		Glass, pocket knife
5	Apatite	
4	Fluorspar	
3.5		Penny
3	Calcite	
2.5		Finger nail
2	Gypsum	
1	Talc	

the known mineral or object, the hardness of the sample is greater. For example, if a sample was not scratched by fluorite, the hardness of your unknown sample is greater than 4. If a sample is scratched by penny the hardness is more than 3.5. If a sample was scratched by apatite then you know the hardness of the sample is less than 5. Therefore, the actual hardness of the sample would be between 4 and 5.

The most common parameters used to determine and therefore predict bit and cutter life are as follows (Nilsen and Ozdemir, 1999; Büchi et al., 1995):

Vickers hardness number (VHN), from the Vickers test

Cerchar abrasivity index (CAI), the result of the Cerchar test

The ABR abrasimetre abrasion coefficent

The LCPC abrasivity index, determined from the LCPC test, which was developed in France by Laboratoire Central des Points et Chaussée

The Norwegian University of Science and Technology (NTNU, formerly the NTH) abrasion test, which provides the abrasion value/abrasion value cutter steel (AV/AVS)

The Vickers test defines the microindentation hardness of a mineral, yielding the VHN. The basic principle, as with all common measures of hardness, is to observe a material's ability to resist plastic deformation from a standard source. This gives the load applied to the indenter, in grams or kilograms force, divided by the area of the indentation. In other words, the hardness number is determined by the load over the surface area of the indentation and not the area normal to the force and is therefore not a pressure.

The Vickers hardness test is conducted using a diamond indenter in the form of a right pyramid with a square base and an angle of 136° between opposite faces of the test material. The full load is normally applied for 10–15 sec. The two diagonals of the indentation left in the surface of the material after removal of the load are measured and their average calculated. The area of the sloping surface of the indentation is calculated. The Vickers hardness is the quotient obtained by dividing the kilogram force [poundal (pdl)] load by the area of indentation in square millimeters (square inches). Figure 2.7 gives the dimensions of the indenter.

Cerchar Abrasive Index The Cerchar scratch test is one of the most common laboratory procedures for determining abrasivity. It is performed by, with a sharp pin, scratching a freshly broken rock surface. The pin is made of heat-treated alloy steel with a defined geometry. The pin is dragged across the rock specimen for a distance of 10 mm (0.4 in.) under a static load of 70 N (506 pdl). Using the measured diameter of the resulting wear on the testing needle, the CAI is calculated. Conducting the test on an irregular surface is more representative of actual conditions. The test results are directly related to TBM cutter life. The values generally range from 0.2 for soft rocks like shale

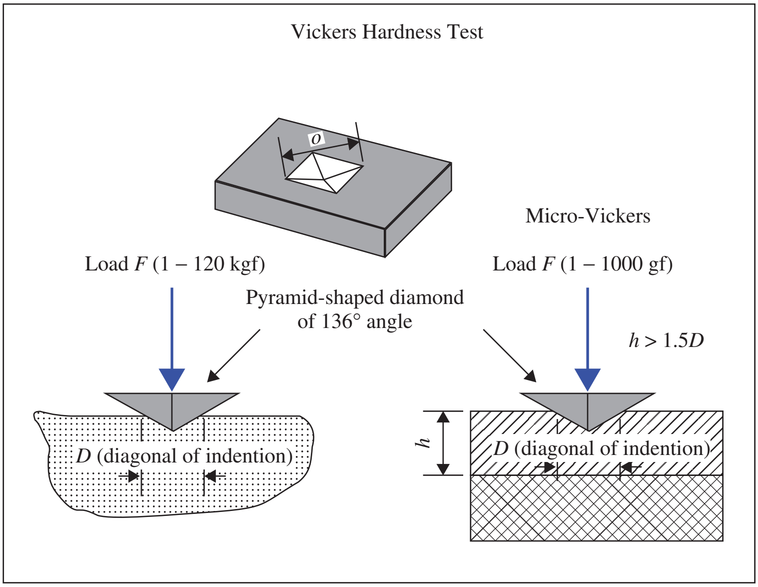


Figure 2.7 Diamond Indenter (Subtech.com)

to 0.5 and higher for very hard rocks, such as quartz. It is important that the properties of the steel be known as the variance in the steel can affect the test results greatly.

LCPC Abrasimeter Test Although principally used for rock tests, the LCPC abrasimeter test correlates well with the Cerchar and Unconfined Compressive Strength (UCS) of the rock. It is useful primarily in determining the abrasiveness of soils. This test is discussed in Chapter 4.

The NTNU Abrasion Test A drillability analysis may be done to determine the durability of the rock, that is, the effect of the rock being tested on the wear, bit, and cutter life. A method developed by the NTH/NTNU Department of Building and Construction consists of three indices: drilling rate index (DRI), bit wear index (BWI), and cutter life index (CLI). These indices are used to determine the rock brittleness, surface hardness, and wear capacity. The brittleness value S_{20} provides a measurement of the rock's brittleness and is determined by using an apparatus to impact the rock. The value is determined from the percentage of a presieved fraction that passes through it after 20 impacts.

The surface hardness, Siever's J-value (SJ), is the measurement of the penetration of a drill bit 1/10 mm in diameter after 200 revolutions that is powered by a miniature drilling apparatus.

The third index, the wear capacity, using the AV, the abrasion value and AVS, which uses the same equipment as the AV but instead of using tungsten carbine, as in the AV, the test uses steel samples of from the cutter. This yields a measurement of the rock's wearing capacity on tungsten carbide and cutter ring steel. The tungsten carbide is the principal metal in a drill bit and the cutter ring steel is the part of the TBM disc cutter that has contact with the rock.

The AV and AVS are defined as the weight loss of tungsten carbide and cutter ring steel test pieces in milligrams after 5 min for tungsten carbine and 1 min for cutter ring steel.

The brittleness (S_{20}) and surface hardness (SJ) are used to calculate the DRI. The DRI and the wear capacity on tungsten carbide (AV) are used to calculate the BWI. The SJ and the wear capacity on cutter ring steel (AVS) are used to calculate the CLI.

Barre, Vermont, Comparison Rock drillability is measured by using the drilling speeds of the granite in Barre, Vermont, as the datum. There the drilling rate is given as 1.0. Based on the Barre rate, rock may be placed into one of four classifications.

If the comparative drilling rate is 0.5 or less, it takes twice the time to drill the hole, the rock is classified as very hard, for example, iron ore and taconite. Hard rocks have comparative drilling speeds in the range of 0.6–1.0. Most granite, quartzite, gneiss, and some schists are this category. Quartzite is very abrasive and will wear down bits. Although some limestones are classified as hard rock, along with dolomite, marble, and some schists, limestones with a drilling rate of 1.0–1.5 are categorized as medium rocks. Soft rocks, rocks that have a drilling speed of 1.5 and higher, include shale, tuffs, Ohio sandstone, and Indiana limestone. Some sandstone can contain quartz, which, as mentioned earlier, is very abrasive and will cause bit wear.

Tunnel Orientation

The orientation of bedding can be critical relative to progress, stand-up time, and water make. The options are perpendicular to the strike, horizontal bedding, and parallel to the tunnel axis.

Tunnel Axis Perpendicular to Strike Generally, a tunnel that is parallel to the strike with steeply dipping bedding will be favorable, provided there is no water inflow. The competence of the rock, the stand-up time, will be influenced principally by the tightness of the bedding joints or cleavage. This is a potential problem but is more of an issue with larger tunnels. The steep bedding rock may act as a conduit of water or may have caused weather zones. Figure 2.8 illustrates a tunnel driven perpendicular to the strike.

Horizontal Bedding Relatively thick layers of horizontal bedding generally provide good stand-up time. Because of the potential for spalling, dipping at 5°–10° can be hazardous. Figures 2.9 and 2.10 are sketches of a tunnel

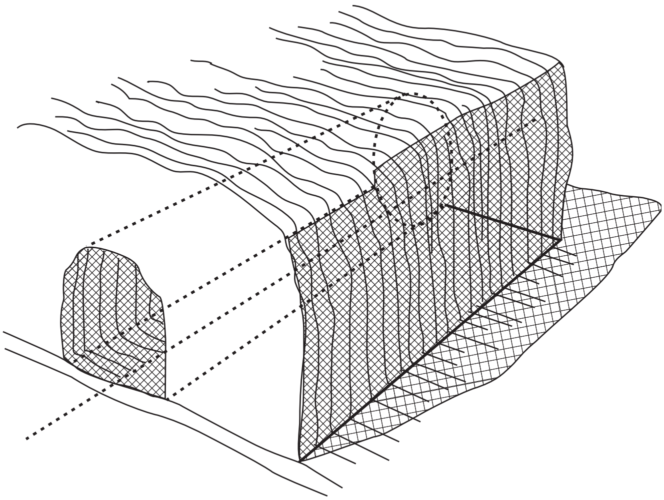


Figure 2.8 Tunnel Perpendicular to Strike (Széchy, 1973)

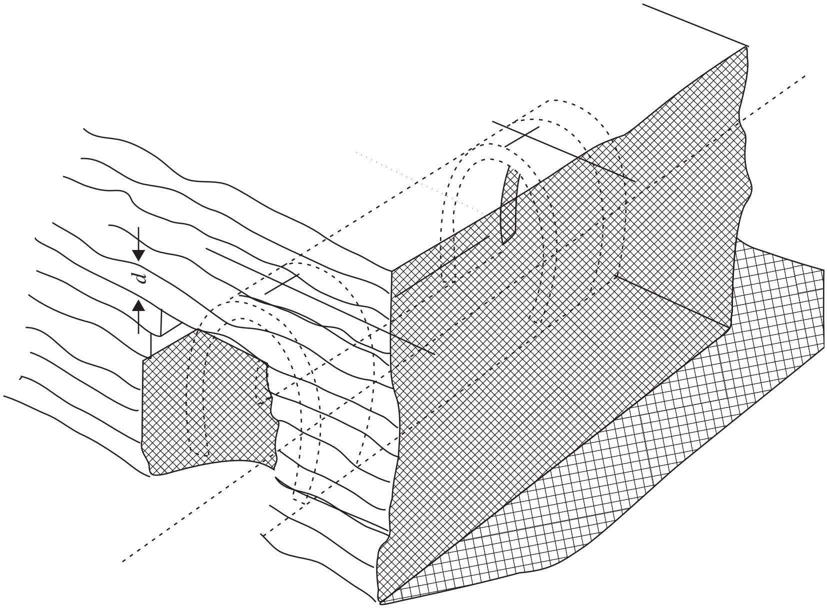


Figure 2.9 Tunnel through Horizontal Bedding (Széchy, 1973)

driven through horizontal bedding. Note, other than the type and amount of discontinuities, the width of the opening is the dominate determinant of stand-up time.

Driving Parallel to Bedding Tunneling is generally unfavorable when the tunnel axis is parallel to steeply dipping bedding; the bridging action may

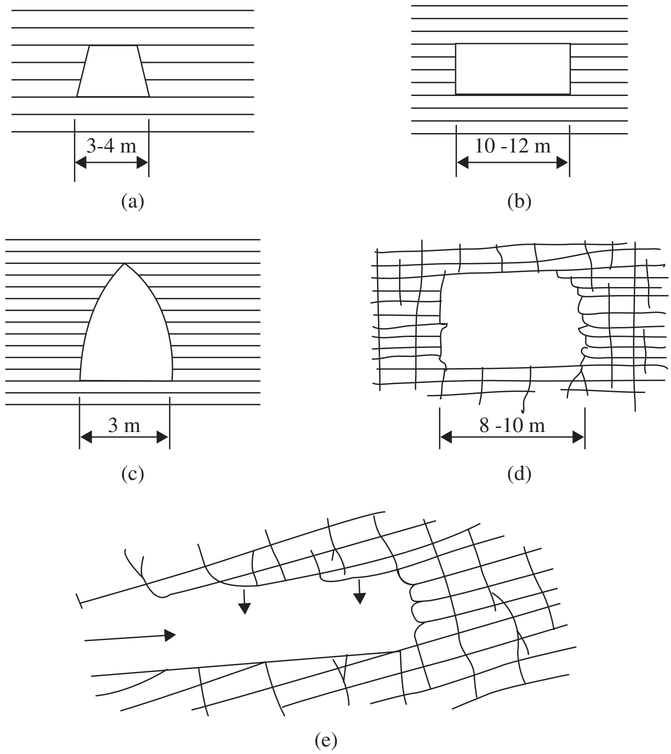


Figure 2.10 Cavity Sections Related to Driving through Horizontal Bedding (Széchy, 1973)

be limited to the shear strength (due to friction and cohesion between adjacent layers). Thus the fill material is critical for roof support. Steep bedding can also act as a means for water infiltration, creating the necessity of handling more water at the face, and washing out the fill material from the discontinuities creates ground control issues. As illustrated in Figure 2.11, in steeply dipping strata, the bridge action is limited to the friction and cohesion between adjacent layers.

When using drilling and blasting techniques to drive a tunnel through rock with parallel bedding, the resulting breakage is generally poor.

Tunneling Under the Water Table

Few would disagree that water in a tunnel has the greatest effect on tunneling. Not only does it have to be removed by pumping it or by creating an effective drainage ditch, but it also can have adverse effects on ground stability. It can wash material from the discontinuities, thus lessening the rock's shear strength, and it can cause soil to be running to flowing ground and thus have adverse effects on stand-up time.

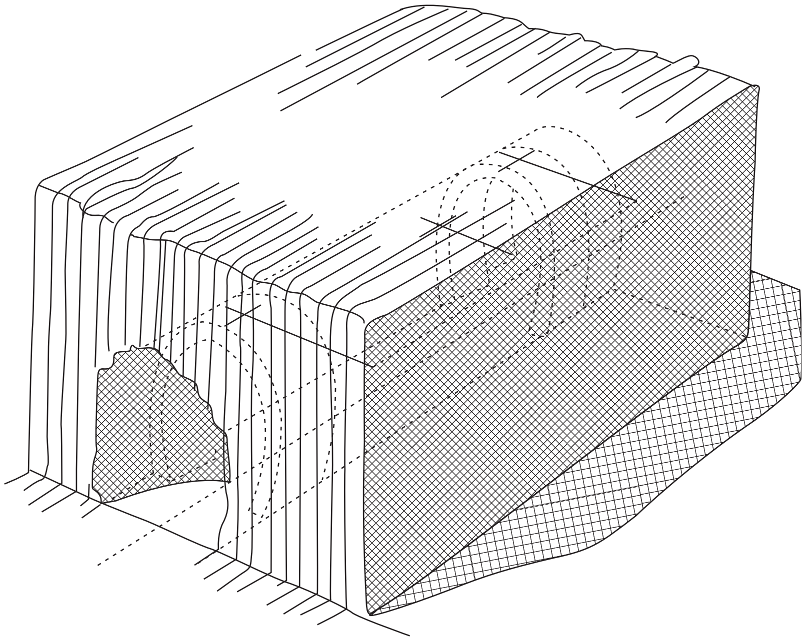


Figure 2.11 Tunnel Driven Parallel to Bedding (Széchy, 1973)

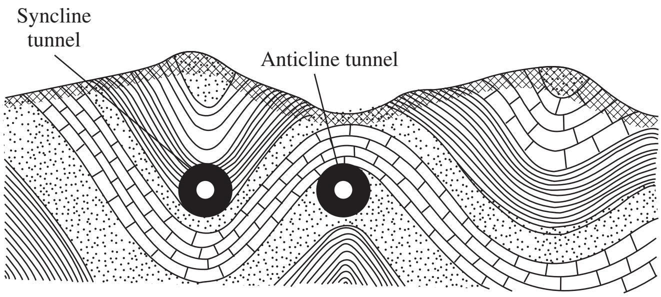


Figure 2.12 Location of Tunnels Relative to Syncline and Anticline (Széchy, 1973)

Synclines and Anticlines When planning a tunnel's location water should be a major consideration. The tunnel's location relative to structural features and the water table must be considered. The relationship of the synclines and anticlines is critical. When given the choice of locating a tunnel in a syncline or an anticline, choose the anticline. The folding strata give rise to pressure on the core and tension in the crown. If the tunnel alignment necessitates that the tunnel will be driven along the strike, it should always be located in the anticline. If the tunnel is driven through the crest of the fold, it will be exposed to less pressure. As is illustrated in Figure 2.12, a tunnel in the syncline fold will act as a basin for the groundwater above it. However, with the anticline, even if the tunnel is below the water table, the relationship of the bedding facilitates water flowing away from the tunnel.

Hydrologic Survey The effect of water on tunnels is illustrated in three ways: static dynamic head; load on the tunnel lining, which modifies the rock or soil physically by dissolving it or chemically; and decomposing and attacking the lining material by water containing corrosives such as sulfides. Generally seeping or moving water is a greater problem than stationary water.

The structure can result in little flow to large flow in water-bearing areas. As referred to above, a permeable layer in an anticline will result in little or no water, whereas a permeable layer in a syncline will cause considerable water inflow.

The degree of rock weathering can affect the water inflow at shallow depths. In particular, a steep dip of the bedding will be weathered to a certain depth. As discussed earlier, if possible, a boring should be located close to the weathered area to measure its depth. However, more water will percolate through longitudinally disturbed planes.

Mining under the water table should be avoided; however, if the tunnel must be made, certain effects such as a deterioration to the rock quality or causing soil to be either reduced in strength or fail, may be encountered. The depth below the water table will determine the pressure to which the lining will be exposed. The volume of the water inflow increases with pressure. Water control may be required. This can include panning, a closed face, grouting, and/or other water control methods to be discussed later.

Another consideration with higher water inflows is ground control. With soil, will running or flowing ground be encountered, requiring additional support? With rock, will additional ground support be required? Some rocks loose strength when small pieces become saturated. Also, the water movement may reduce the material in discontinuities, weakening the rock.

Rock Mass Classification

Rock Quality Designation The RQD was developed by D.U. Deere as a technical description of rock cores for engineering purposes. Since its development, it has come to be used as commonly as unconfined compressive strength when describing rock quality. There have been occasions where the RQD was the sole parameter for determining rock mass quality. However the RQD cannot be used as the only parameter because it does not provide joint orientation and joint fill material.

The RQD may indicate the fragmentation characteristics of the rock. A low RQD could indicate the rock will break into smaller fragments, thereby reducing the powder factor and associated drilling pattern and penetration rate of the mechanical miner and provide insight into the ground support requirements.

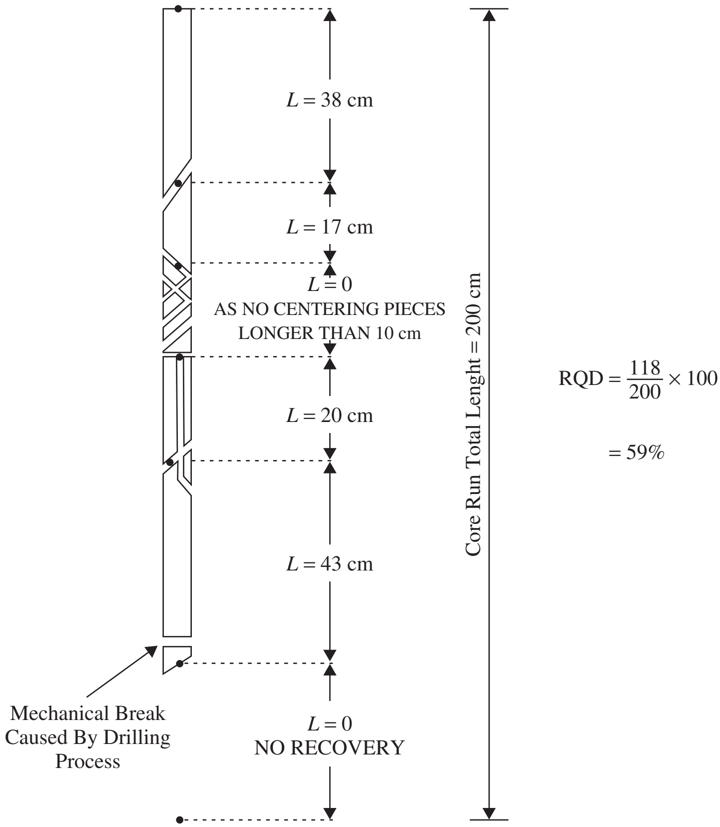


Figure 2.13 Procedure for Measurement and Calculation of RQD (Deere, 1989)

The RQD is defined as the percentage of recovered core pieces that are longer than 4 in. (10 mm) of the total core:

$$\text{RQD} = \frac{\sum \text{length of core pieces} \geq 10\ \text{cm}(4\ \text{in.})}{\text{length of core run}} \times 100\%$$

The higher the RQD is, which means there is a higher percentage of core pieces that are greater than 10 cm, the more competent the rock. Figure 2.13 illustrates the RQD measurement.

The relationship between the RQD and the quality of rock proposed by Deere is illustrated in Table 2.5.

Since its introduction there have been many attempts to relate the RQD to various qualitative classification systems. Table 2.6 is a representation of the relationship between Terzaghi's rock load factor and the RQD.

The RQD is used in other qualitative analyses of rock quality and recommended ground support, such as the Q-system and Geomechanics classification.

Table 2.5 RQD Compared to Rock Quality

Rock Quality Designation (%)	Description of Rock Quality
90–100	Excellent
75–90	Good
50–75	Fair
25–50	Poor
0–25	Very poor

Source: From Deere and Deere (1988).

Table 2.6 Terzaghi's Rock Load Concept

Rock Class	RQD, %	Rock Load, H_p	Remarks
I. Hard and intact	95–100	0	Same as Table 5.2
II. Hard stratified or schistose	90–99	0–0.5B	Same as Table 5.2
III. Massive moderately jointed	85–95	0–0.25B	Same as Table 5.2
IV. Moderately blocky and seamy	75–85	0.25B–0.35($B + H_t$)	Types IV, V, and VI reduced by about 50% from Terzaghi values because water table has little effect on rock load (Terzaghi, 1946; Brekke, 1968)
V. Very blocky and seamy	30–75	(0.2–0.6)($B + H_t$)	Same as above
VI. Completely crushed	3–30	(0.6–1.10)($B + H_t$)	Same as above
VIa. Sand and gravel	0–3	(1.1–1.4)($B + H_t$)	Same as above
VII. Squeezing rock at moderate depth	NA	(1.10–2.10)($B + H_t$)	Same as Table 5.2
VIII. Squeezing rock at great depth	NA	(2.10–4.50)($B + H_t$)	Same as Table 5.2
IX. Swelling rock	NA	Upto 80 m irrespective of value of $B + H_t$	Same as Table 5.2

Source: As Modified by Deere et al. (1970).
Notation: B = tunnel span; H_t = height of opening; H_p = height of loosened rock mass above tunnel crown developing load.

The most common laboratory tests are the uniaxial compression test, bending or flexural tests, and direct shear. These tests indicate the rock's strength in compression, bending, and shear, respectively, and are correlated to rock drillability, explosives requirements, drill pattern, stand-up time, boreability, and penetration.

Rock Mass Determinations As with any engineering problem the loading of the structure is necessary to design the load-carrying capacity; the same is true for rock. Unlike a foundation that is transferring the load to the ground, generally the load on a tunnel crown is the weight of a portion of the rock itself. Current designs are based on rock strength and other engineering properties. The newer methods do not consider rock loading (weight) because they are designed to assist the rock in being self-supporting. Although not used frequently today, below is a discussion of the method of determining rock mass quality and rock loading developed by Karl Terzaghi.

Terzaghi This method considered rock-loading based on the amount of rock that is being carried by the supports. Obviously the tunnel supports are not required to support the rock all the way to the ground surface; therefore the method developed by Terzaghi provides a means of determining the amount of rock load the steel ribs have to support based on the discontinuities.

Based on the discontinuities, Terzaghi categorized rock into nine classifications. The discontinuities and the effects on rock strength are defined in Table 2.7.

Class I is defined as hard and intact. It is the best quality of rock and generates no load requiring support but may need a light coat of shotcrete for spalling and surface protection. Class II rock is hard, stratified, or schistose and should require little support for spalling. Massive and moderately joined Class III rock requires little or no support, and because the joints are widely spaced, the large blocks will interlock and thus there is no lateral load and no support is required in the walls. The moderately blocky and seamy rock in Class IV has joints that are closely spaced and is about 1 m (3 ft) in size. Class V rock is very blocky and seamy and has closely spaced joints with block sizes of less than 1 m (3 ft). Class VI rock is completely crushed but chemically intact rock not unlike crusher run aggregate. There is no interlocking and the block sizes can vary from just a few centimeters to 30 cm. Rock that squeezes a moderate depth, Class VII rock squeezes into the tunnel opening without appreciable increase in volume. The depth may be from 150 to 1000 m (500 to 3300 ft). Class VIII is squeezing rock at great depth. The maximum recommended depth is 1000 m (3300 ft), possibly 2000 m (6600 ft) in very goodrocks. Class IX is swelling rock, and its volume changes due to chemical change in the rock, usually in the presence of moisture. Table 2.8 illustrates the loading on the tunnel by different types of rock.

Rose (1982) proposed that if the tunnel is above the water table, the requirements of Class IV and Class VI rock can be reduced by 50%.

The capacity whereby the overburden weight of the thickness of the rock above the roof of a tunnel is transferred onto the rock on both sides of the tunnel is referred to as "arch action." The rock that transfers the load to the tunnel walls is the ground arch. Figure 2.14 illustrates the ground arch.

The load stress relaxation caused by tunneling results in the arching action. There are conditions for arch action and elements which determine the load on the crown support in tunnels through crushed rock and cohesionless soil above

Table 2.7 Rock Classes in Terzaghi's Rock Load Theory

Rock Class		Definition
I.	Hard and intact	The rock is unweathered. It contains neither joints nor hair cracks. If fractured, it breaks across intact rock. After excavation the rock may have some popping and spalling failures from roof. At high stresses spontaneous and violent spalling of rock slabs may occur from sides or roof. The unconfined compressive strength is equal to or more than 100 MPa.
II.	Hard stratified and schistose	The rock is hard and layered. The layers are usually widely separated. The rock may or may not have planes of weakness. In such rock, spalling is quite common.
III.	Massive moderately jointed	A jointed rock. The joints are widely spaced. The joints may or may not be cemented. It may also contain hair cracks, but the huge blocks between the joints are intimately interlocked so that vertical walls do not require lateral support. Spalling may occur.
IV.	Moderately blocky and seamy	Joints are less spaced. Blocks are about 1 m in size. The rock may or may not be hard. The joints may or may not be healed, but the interlocking is so intimate that no side pressure is exerted or expected.
V.	Very blocky and seamy	Closely spaced joints. Block size is less than 1 m. It consists of almost chemically intact rock fragments which are entirely separated from each other and imperfectly interlocked. Some side pressure of low magnitude is expected. Vertical walls may require supports.
VI.	Completely crushed but chemically intact	Comprises chemically intact rock having the character of a crusher run aggregate. There is no interlocking. Considerable side pressure is expected on tunnel supports. The block size could be a few centimeters to 30 cm.
VII.	Squeezing rock, moderate depth	Squeezing is a mechanical process in which the rock advances into the tunnel opening without perceptible increase in volume. Moderate depth is a relative term and could be up to 150–1000 m.
VIII.	Squeezing rock, great depth	The depth may be more than 150 m. The maximum recommended tunnel depth is 1000 m (2000 m in very good rocks).
IX.	Swelling rock	Swelling is associated with volume change and is due to chemical change of the rock usually in the presence of moisture or water. Some shales absorb moisture from air and swell. Rocks containing swelling minerals such as montmorillonite, illite, and kaolinite can swell and exert heavy pressure on rock supports.

Source: From Terzaghi (1946) Sinha (1989).

Table 2.8 a Rock Loads in Tunnels within Various Rock Classes

Rock Class		Rock Load Factor, H_p	Remarks
I.	Hard and intact	0	Light lining required only if spalling or popping occurs.
II.	Hard stratified or schistose	$0-0.5B$	Light support mainly for protection against spalling. Load may change erratically from point to point.
III.	Massive moderately jointed	$0-0.25B$	No side pressure
IV.	Moderately blocky and seamy	$0.25B-0.35\ (B+H_t)$	No side pressure
V.	Very blocky and seamy	$(0.35-1.10)(B+H_t)$	Little or no side pressure
VI.	Completely crushed	$1.10\ (B+H_t)$	Considerable side pressure. Softening effects of seepage toward bottom of tunnel require either continuous support for lower ends of ribs or circular ribs.
VII.	Squeezing rock, moderate depth	$(1.10-2.10)(B+H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
VIII.	Squeezing rock, great depth	$(2.10-4.50)(B+H_t)$	-do-
IX.	Swelling rock	Up to 250 ft (80 m), irrespective of value of $B+H_t$	Circular ribs are required. In extreme, cases, use of yielding support is recommended.

Source: From Terzaghi (1946).
Notation: B = tunnel span in meters; H_t = height of opening in meters; H_p = height of loosened rock mass above tunnel crown developing load.

Table 2.8 b

Rock Condition	Rock Load H_pf	Remarks
1. Hard and intact	0	Light lining, required only if spalling or popping occurs.
2. Hard stratified or schistose[a]	$0-0.5B$	Light support.
3. Massive, moderately jointed	$0-0.25B$	Load may change erratically from point to point.
4. Moderately blocky and seamy	$0.25B-0.35(B+H_t)$	No side pressure.

(*continued overleaf*)

Table 2.8 b (*Continued*)

Rock Condition	Rock Load H_pf	Remarks
5. Very blocky and seamy	$(0.35–1.10)(B + H_t)$	Little or no side pressure.
6. Completely crushed but chemically intact	$1.10\ (B + H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.
7. Squeezing rock, moderate depth	$(1.10–2.10)(B + H_t)$	Heavy side pressure; invert struts required. Circular ribs are recommended.
8. Squeezing rock, great depth	$(2.10–4.50)(B + H_t)$	
9. Swelling rock	Up to 250 ft irrespective of value of $B + H_t$	Circular ribs required. In extreme cases use yielding support.

Note: Rock load H_p in feet of rock on roof of support in tunnel with width B (ft) and height H_t (ft) at depth of more than $1.5(B + H_t)$

The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4–6 can be reduced by 50%.

[a]Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term *shale* is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

the water table. Figure 2.15 illustrates the mechanics of arch action in sand. Sand is used because totally crushed rock would act similarly to sand.

The ground arch is represented by the shaded area bound by $a–c–d–b$. The width of the ground arch is represented by B_1. The crushed rock will move downward as the tunnel is excavated and during the time it takes to install the ground support. The downward movement is resisted by the boundaries represented by ac and bd. The loading results in the friction forces transferring the bulk of the load of the height overburden represented by H to both sides of the tunnel, resulting in the roof support being required to carry the balance of the load, which is equivalent to H_p. The thickness of the ground arch, represented in the illustration as D, is roughly equal to $1.5B$. Above the ground arch the ground is unaffected by the excavation.

The slight downward settlement of the crown will reduce the rock load on the support of the intrados of the arch, represented by $H_{p,\min}$, which is considerably less than the thickness of the ground arch, D. Should the ground subside more, for example, if the ground support in not installed soon enough,

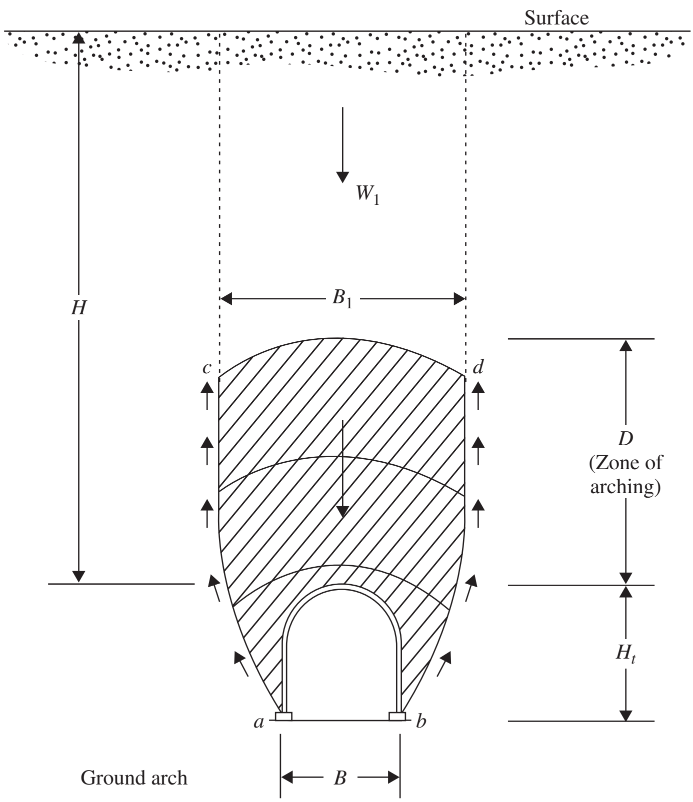


Figure 2.14 Ground Arch (Proctor and White, 1977)

the ground arch will settle more, increasing the load on the supports. This value is represented by $H_{p,\text{max}}$; this dimension is still less than D.

Once the support is installed and sufficiently blocked, the increase in rock load will increase at a decreasing rate by approximately 15% (Proctor and White, 1977) from H_p to $H_{p,\text{ult}}$. Thus,

$$H_{p,\text{ult}} = 1.15H_p$$

As indicated by c in Figure 2.15, the roof load increases as the depth of the overburden on a tunnel with a given cross section increases from zero. As the depth increases, the depth of the rock load will reach the depth H_p, which is independent of depth. All things being equal, the value of H_p (Figure 2.15) increases approximately in direct proportion to the width B_1 of the ground arch.

The value of the constant C is related to the degree of compactness of the crushed rock or sand and the distance d through which the crown of the ground arch subsided while the tunnel was being driven and ground support was being installed. The values of C are given in Figure 2.16.

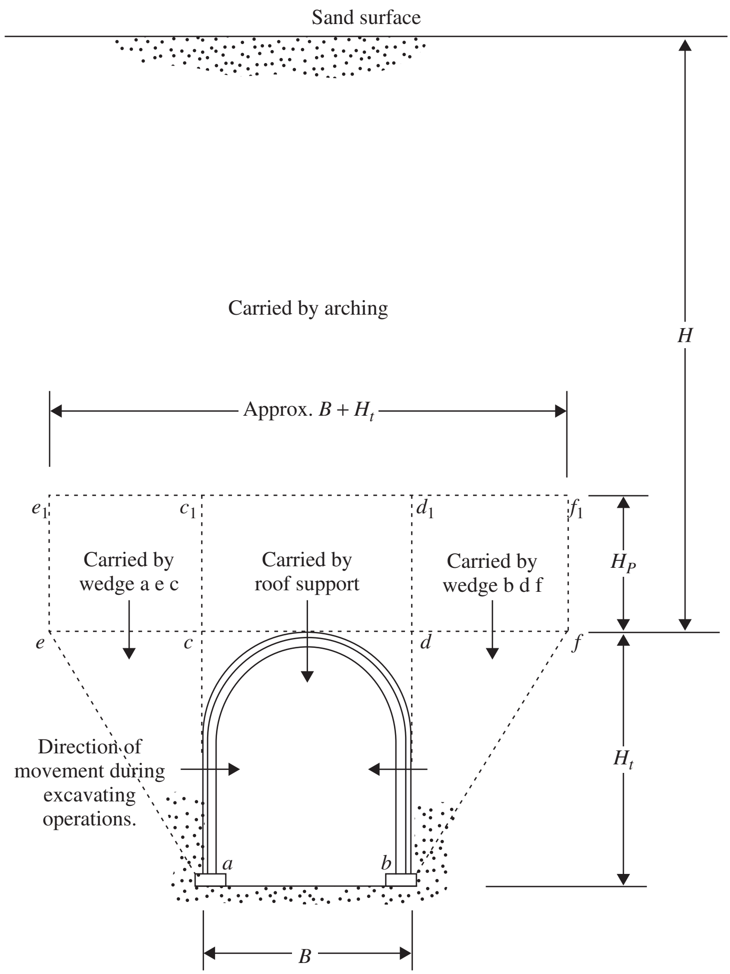


Figure 2.15 Loading of Tunnel Support in Sand (Proctor and White, 1977)

The cross section shown in Figure 2.15 is through a tunnel of rock crushed to sand at a great depth from the surface. At the lower level just above the tunnel, the boundary from Figure 2.15 (a–b–d–c) tends to move into the tunnel rise from the outer edges of the tunnel at a slope of approximately 2:1. Thus the ground arch B_1 approximately equals

$$B_1 = B + H_t$$

where B = width of tunnel
H_t = height of tunnel

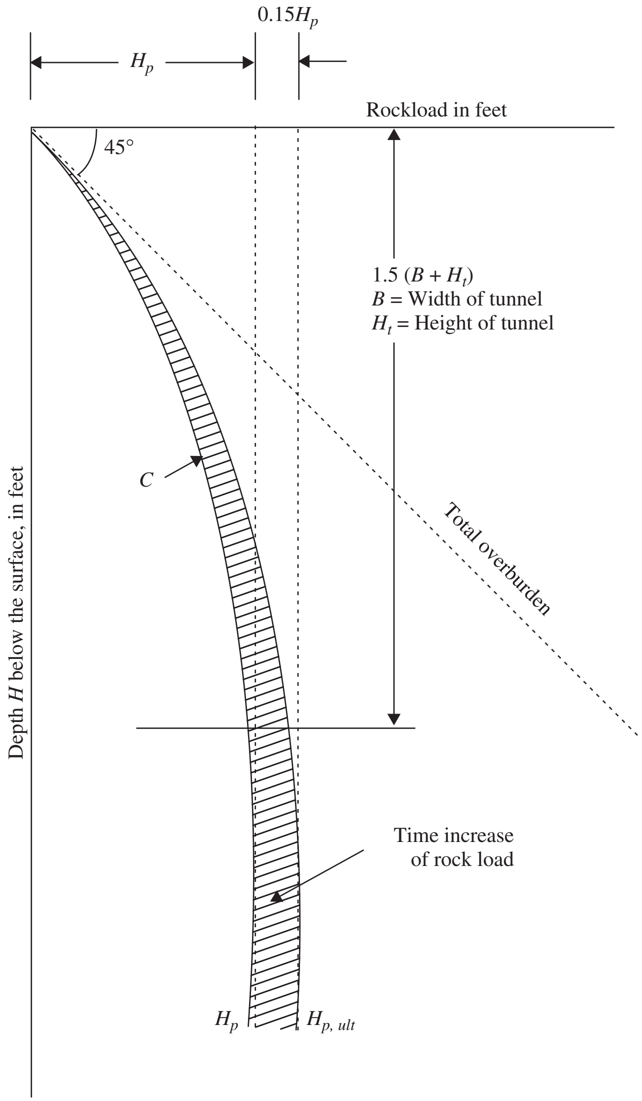


Figure 2.16 Load–Depth Relationship for Tunnel in Sand (Proctor and White, 1977)

The rock load H_p is represented in the figure by the rectangle e–f–f_1–e_1. The remainder of the overburden weight is supported by the ground arch. The weight of the center section c–d–d_1–e_1 is transferred to the floor of the tunnel through the ribs. The tops of the wedge-shaped bodies tend to slide into the tunnel, increasing the horizontal loading pressure caused by the surcharge from the weight of the outer part. The rock load H_p is determined by the equation $H_p = CB_1$.

Figure 2.16 illustrates that when the depth of the overburden on a tunnel with a given cross section increases from zero, as indicated by curve C, the roof load increases. With increasing depth H, the rock load approaches a value H_p which is independent of depth. That is, as the depth increases, the load decreases, until there is no longer any effect of the overburden on loading.

Solid, very competent rock will probably not need steel ribs; it may require only a few rock bolts. In blocky and seamy rock blocks are located between joints that are neither interconnected nor intimately interlocked. Both closely jointed and badly broken rock can be encountered. Joints may be empty or filled with the products of rock weathering and may be narrow or wide. This type of rock shares the character of dense sand with little or no cohesion with very large grains. Rock with randomly oriented joints is likely to have a roof load that is associated with horizontal pressure on the sides of the tunnel support.

Because there is no intimate interlocking between adjoining blocks or cross connections, the loading on the roof of the tunnel is determined by laws similar to that of sand in that arching is realized. At considerable depth, the load H_p on the roof support in tunnels is independent of depth and increases in direct proportion to the sum of the tunnel width B and the height H_t. Therefore, for a tunnel with a width and height of 3 m (10 ft) and a corresponding roof load of H_{p10} with any width B and any height H_t through the same rock in feet, H_p is given as

$$H_p = H_{p10}B + H_t/20$$

where H_{p10} represents empirical values which are based on observations. Values for rock loads on tunnel supports in moderately and very blocky rock are established as shown in Table 2.9.

In wet tunnels with moderately blocky rock, H_p may have an initial value of zero and may increase up to 7 ft of rock.

Wet tunnels in moderately blocky and shattered rock, the initial load after a day or two may be as high as 12 ft and may increase to a final value of as much as 21 ft. Table 2.10 is the result of using the aforementioned values for H_{p10}.

The H_p values can be much lower in dry tunnels than in wet tunnels. During long, wet spells and spring thaws tunnels that do not have something above to

Table 2.9 Rock Loads (ft) in Blocky and Seamy Ground Using Various Values of H_{p10}

Rock Condition	Initial Value	Ultimate Value
Moderately blocky rock	$H_p = 0$	$H_{p,\text{ult}} = 0.25B$ to $H_{p,\text{ult}} = 0.35(B + H_t)$
Very blocky and shattered rock	$H_p = 0$ to $H_p = 0.60(B + H_t)$	$H_{p,\text{ult}} = 0.35(B + H_t)$ to $H_{p,\text{ult}} = 1.10(B + H_t)$

Source: From Proctor and White (1977).

Table 2.10 Comparison between Rock Load (ft) in Sand and in Blocky and Seamy Rock

	Above Water Table		Below Water Table[a]	
Material	$H_{p,\min}$	$H_{p,\max}$	$H_{p,\min}$	$H_{p,\max}$
Dense sand[b]				
Initial	$0.27(B + H_t)$	$0.60(B + H_t)$	$0.54(B + H_t)$	$1.20(B + H_t)$
Ultimate	$0.31(B + H_t)$	$0.69(B + H_t)$	$0.62(B + H_t)$	$1.38(B + H_t)$
Loose sand[b]				
Initial	$0.47(B + H_t)$	$0.60(B + H_t)$	$0.94(B + H_t)$	$1.20(B + H_t)$
Ultimate	$0.54(B + H_t)$	$0.69(B + H_t)$	$1.08(B + H_t)$	$1.38(B + H_t)$
Moderately blocky[c]	$H_{p,\text{in}} = 0$	increasing up to $H_{p,\text{ult}} = 0.35(B + H_t)$		
Very blocky and shattered	$H_{p,\text{in}} = 0.60$ $(B + H_t)$	increasing up to $H_{p,\text{ult}} = 1.10(B + H_t)$		

Source: From Proctor and White (1977).
[a]Values are roughly equal to twice those for dry sand.
[b]Values computed on basis of laboratory tests.
[c]Values computed on the basis of the results of observations in railroad tunnels.

prevent water from percolating to them, such as being under a city with paved streets, are likely to be wet and should not be treated as dry tunnels.

Roof load estimates for blocky and seamy rock may be considered for a crushed rock with very large grain and low porosity. Values for rock tunnels in Table 2.10 are the result of observations in tunnels below the water table. And, because the values are from below the water table, they should be compared with those for cohesionless sand below the water table.

Table 2.10 illustrates that the ultimate rock load in moderately blocky and seamy rock is quite a bit lower than the ultimate minimum value for dense sand, whereas the value for very blocky and shattered rock is approximately equal to the ultimate minimum value for loose sand. This indicates that the inevitable yield of the rock toward the tunnel prior to wedging and backpacking is enough to develop the arch action to the fullest extent. Any yield in excess of this that may be due to careless mining or inadequate blocking and backpacking will probably increase the ultimate rock load. The table indicates that the initial rock load on the roof support in blocky and seamy ground is very much smaller than in sand but, everything else being equal, the ultimate rock load is similar.

Table 2.11 provides types of ground control that can be required based on the type of ground encountered. The approximate rock loads can be anticipated based on the principal rock conditions. Since boundaries for different rock conditions are not well defined, the rock load corresponding to each rock condition in the table is represented by a range of values.

An estimate of load and roof support based on the rock quality will be made for a tunnel where width B and H_t both equal 4.5 m (15 ft), the average depth is 91 m (300 ft) in granite, and the unit weight is 75 kg (165 lb). In areas

Table 2.11 Rock Load (ft) of Rock on Roof Support in Tunnel

Rock Condition	Rock Load H_p, ft 0	Remarks
1. Hard and intact	0	Light lining, required only if spalling or popping occurs.
2. Hard stratified or schistose[a]	$0-0.5B$	Light support.
3. Massive, moderately jointed	$0-0.25B$	Load may change erratically from point to point.
4. Moderately blocky and seamy	$0.25B-0.35(B+H_t)$	No side pressure.
5. Very blocky and seamy	$(0.35-1.10)(B+H_t)$	Little or no side pressure.
6. Completely crushed but chemically intact	$1.10(B+H_t)$	Considerable side pressure. Softening effect of seepage toward bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.
7. Squeezing rock, moderate depth	$(1.10-2.10)(B+H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
8. Squeezing rock, great depth	$(2.10-4.50)(B+H_t)$	
9. Swelling rock	Up to 250 ft irrespective of value of $B+H_t$)	Circular ribs required. In extreme cases use yielding support.

Source: From Proctor and White (1977).
Note: Rock load H_p in feet of rock on roof of support in tunnel with width B (ft) and height H_t (ft) at depth of more than $1.5(B+H_t)$.
The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4–6 can be reduced by 50%.
[a]Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

of a complex geologic history there can be considerable changes in the rock condition over short distances. The roof load for various conditions that may be encountered are obtained by introducing the values B, H_t, and w into the previously discussed equations. See Table 2.12.

Table 2.13 provides guidelines for steel supports. The rock mass is an integral part of the support system. The table is only applicable if the rock mass is not allowed to loosen and disintegrate extensively. Generally mechanically excavated tunnels will reduce loads 20–25% from that of blasted tunnels.

Table 2.14 is the guidelines from (Bieniawski, 1989) on the support pressure for rock tunnels and caverns based on RMR values. It use to be that support pressure was determined with estimates from Terzaghi's (1946) road load theory and found support pressure in rock tunnels and caverns doesn't increase directly with excavation size as assumed by Terzaghi approach. Others subscribe to

Table 2.12 Numerical Example of Rock Load Computation

Spalling state: 0–400 lb/ft^2

Moderately jointed

$H_{p,\min} = 0$

$H_{p,\max} = 0.25B = 0.25 \times 15\,\text{ft} = 3.75\,\text{ft} = 620\ \text{lb/ft}^2$

Moderately blocky and seamy

$H_{p,\min} = H_{p,\max}$ for moderately jointed $= 620\ \text{lb/ft}^2$

$H_{p,\max} = 0.35(B + H_t) = 0.35(15 + 15) = 10.5\,\text{ft} = 1730\ \text{lb/ft}^2$

Intensely shattered but chemically unaltered

$H_{p,\min} = H_{p,\max}$ for moderately blocky $= 1730\ \text{lb/ft}^2$

$H_{p,\max} = 1.10(B + H_t) = 1.10(15 + 15) = 33.0\,\text{ft} = 5440\ \text{lb/ft}^2$

If the granite has been chemically altered, squeezing and even swelling conditions may prevail. Loads may then, at this moderate depth, attain the following values:

Squeezing $H_p = 2.10(B + H_t) = 2.10 \times (15 + 15) = 63\,\text{ft} = 10400\ \text{lb/ft}^2$ *maximum*

Swelling = 20,000–40,000 lb/ft^2 maximum

Source: From Proctor and White (1977).

In the squeezing and swelling stretches full circle ribs should be used. Hence it is advisable to include an adequate supply of such ribs in the first procurement.

that loads are determined due mainly to dilatant behavior of rock masses, joint roughness, and prevention of loosening of rock mass by improved tunneling methods. However, the support pressure will likely increase directly with the excavation width of the tunnels that pass through slickened sided shear zones, thick clay filled fault gouges, weak clay shales, and running or flowing ground conditions where interlocking blocks are likely to be missing or where join strength is lost and rock wedges are allowed to fall due to excessive roof convergence on account of delay supports beyond stand-up time. Even though support pressure doesn't increase with the tunnel size, tunnels that are wider will require reduced spacing of bolts or steel arches and thicker linings since road load increases directly with tunnel width.

Terzaghi's approach was used with success prior to the advent of modern mechanical means of tunneling. The conventional drill blast methods of mining using steel sets for support were used in tunnels of comparable size.

Rock Mass Rating Developed by Bieniawski, the RMR rates rock quality on a scale from 0 to 100. Its principal use is to predict the ground support required. It is based on six universal parameters:

Strength of rock (uniaxial compressive strength)
Drill core quality (RQD)
Groundwater conditions
Joint and fracture spacings
Joint characteristics
Joint orientation

Table 2.13 Guidelines for Selection of Ground Support for 6- and 12-m-Diameter Tunnels in Rock

		Steel Sets		Rock Bolt		Total Shotcrete Thickness (cm)		
Rock Quality	Construction Method	Weight of Steel Sets	Spacing	Spacing of Pattern Bolt	Additional Requirements	Crown	Sides	Additional Supports
Excellent RQD >90	Boring machine	Light	None to occasional	None to occasional	Rare	None to occasional	None	None
	Drilling and blasting	Light	None to occasional	None to occasional	Rare	None to occasional	None	None
Good RQD 75–90	Boring machine	Light	Occasional to 1.5–1.8 m	Occasional to 1.5–1.8 m	Occasional mesh and straps	Local application 5–7.5 cm	None	None
	Drilling and blasting	Light	1.5–1.8 m	1.5–1.8 m	Occasional mesh or straps	Local application 5 – 7.5 cm	None	None
Fair RQD 50–75	Boring machine	Light to medium	1.5–1.8 m	1.2–1.8 m	Mesh and straps as required	5–10 cm	None	Rock bolts
	Drilling and blasting	Light to medium	1.2–1.5 m	0.9–1.5 m	Mesh and straps as required	10 cm or more	10 cm or more	Rock bolts

(*continued overleaf*)

Table 2.13 (*Continued*)

		Steel Sets		Rock Bolt		Total Shotcrete Thickness (cm)		
Rock Quality	Construction Method	Weight of Steel Sets	Spacing	Spacing of Pattern Bolt	Additional Requirements	Crown	Sides	Additional Supports
Poor RQD 25–50	Boring machine	Medium circular	0.6–1.2 m	0.9–1.5 m	Anchorage may be hard to obtain; considerable mesh and straps required	10–15 cm	10 to 15 cm	Rockbolts as required (1.2–1.8 m center to center)
	Drilling and blasting	Medium to heavy circular	0.2–1.2 m	0.6–1.2 m	As above	15 cm or more	15 cm or more	As above
Very poor RQD <25	Boring machine	Medium to heavy circular	0.6 m	0.6–1.2 m	Anchorage may be impossible; 100% mesh and straps required	15 cm or more on whole section		Medium sets as required
	Drilling and blasting	Heavy circular	0.6 m	0.9 m	As above	15 cm or more on whole section		Medium to heavy sets as required
Very poor squeezing and swelling ground	Both methods	Very heavy circular	0.6 m	0.6–0.9 m	Anchorage may be impossible; 100% mesh and straps required	15 cm or more on whole section		Heavy sets as required

Source: From Deere et al. (1970).

Table 2.14 Guidelines for Excavation and Support in Accordance with RMR (Bieniawski, 1989)

		Support		
Rock Mass Class	Excavation	Rock Bolts (20 mm Diameter Fully Grouted)	Shotcrete	Steel Sets
I. very good rock, RMR 81–100	Full face, 3 m advance	Generally, no support required except for occasional spot bolting		
II. Good rock, RMR;61–80	Full face, 1.0–1.5 m advance, complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh	50 mm in crown where required	None
III. Fair rock, RMR: 41–60	Top heading and bench, 1.5–3 m advance in top heading, commence support after each blast, complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown and 30 mm in sides	None
IV. Poor rock, RMR: 21–40	Top heading and bench, 1.0–1.5 m advance in top heading, install support concurrently with excavation 10 m from face	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
V. Very poor rock, RMR: <20	Multiple drifts, 0.5–1.5 m advance in top heading, install support concurrently with excavation, shotcrete as soon as possible after blasting	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh, bolt invert	150–200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required, close invert

Source: From Bieniawski (1989).
Note: Shape: horoscope: width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

The rock strength is evaluated using laboratory compression tests on prepared cores. For the RMR, it is sufficient to use an approximation of strength based on point load tests on intact pieces of core. Table 2.15 provides qualitative description of rock based on compressive strength and point load strength.

Table 2.15 Strength of Intact Rock Material

Qualitative Description	Compressive Strength (MPa)	Point Load Strength (MPa)	Rating
Exceptionally strong	>250	8	15
Very strong	100–250	4–8	12
Strong	50–100	2–4	7
Average	25–50	1–2	4
Weak	10–25	Use of uniaxial compressive strength is preferred	2
Very weak	2–10	-ditto-	1

Source: From Bieniawski (1989).

Table 2.16 Rock Quality Designation

Description	RQD	Rating
Excellent	90–100	20
Good	75–90	17
Fair	50–75	13
Poor	25–50	8
Very poor	<25	3

Drill Core Quality Rock is rated according to an evaluation based on the percentage of core recovery greater than twice its diameter. Table 2.16 describes rock quality relative to the RQD.

Joint Spacing Joint spacing is the distance between discontinuities. They can be joints, shear zones, bedding or foliation, or minor faults, basically any crack in the rock mass that will reduce tension or shear strength.

When not visible on the surface, joints spacing are evaluated from the drill core. Generally it is assumed that the rock mass contains three sets of joints. The measurement is conducted on what is assumed to be the most influential joint. Note that in Table 2.17, the greater the distance between the joints, the higher is the rating.

Condition of Joints The joints are also examined with respect to the joint sets that are most likely to influence the work. This is a subjective parameter, examined according to specific guidelines. Generally, the descriptions of discontinuity surface roughness, infilling, continuity or length, separation of the joint walls, and weathering and coating material are weighted toward the smoothest and weakest joints and gouge. Table 2.18 illustrates the rating based on the conditions of the joints. That is, we need to view descriptions with conservatism. When characterizing the surface of discontinuities, roughness is

Table 2.17 RMR Value for Spacing of Joints

Joint Spacing (m)	Rating
>2.0	20
0.6–2.0	15
0.2–0.6	10
0.06–0.2	8
<0.06	5

Source: Adapted from Bieniawski (1979).

Table 2.18 Condition of Joints

Description	Rating
1. Very rough and unweathered, wall rock tight and discontinuous, no separation	30
2. Rough and slightly weathered, wall rock surface separation <1 mm (0.04 in.)	25
3. Slightly rough and moderately to highly weathered, wall rock surface separation <1 mm (0.04 in.)	20
4. Slick sided wall rock surface, 1–5 mm (0.04–0.2 in.)	10
5. Soft gouge, 5 mm (0.2 in.) thick, 5 mm wide, continuous discontinuity	0

Source: Adapted from Bieniawski (1979).

an important factor. The roughness that occurs where the joint surfaces meet if the surfaces are close and clean will inhibit movement along the surface of the joint. As well as controlling water inflow, the separation, that is, the distance between surfaces of joint walls, controls the interlocking. The smaller the separation is, the more likely the joint roughness will interlock. If there is no interlocking, the shear strength will be controlled by the gouge. Thus, both the interlocking and gouge, if there is any, and the degree of roughness, are controlling factors of shear strength. The continuity of the discontinuities affects the behavior of the rock mass. When conducting a continuity assessment, the length of the discontinuity should be determined.

Groundwater Groundwater can strongly influence rock mass behavior. It weakens the shear strength of the gouge and/or in the case of water under pressure will actually remove gouge. There are four categories of water: completely dry, moist, water under moderate pressure, or severe water problems. High water inflow reduces the RMR. As seen in Table 2.19 dry joints are rated 15, whereas water under pressure is zero. Since the RMR is from zero to 100, the lower the rating of any parameter, the lower will the total rating be, and thus the more ground support and the less stand-up time.

Joint Orientation The orientation of the joints relative to the work can influence rock behavior. It can affect the difficulty or ease of mining and have a

Table 2.19 Groundwater Condition

Inflow per 10 m (liter/minute) (33 ft) of tunnel	None	<10	0–25	5–125	>125
Joint water pressure/major principal stress	0	0–0.1	0.1–0.2	0.2–0.5	>0.5
General description	Completely dry	Damp	Wet	Dripping	Flowing
Rating	15	10	7	4	0

Source: Adapted from Bieniawski (1997).

Table 2.20 Joint Orientation Effect on Tunnel

Strike Perpendicular to Tunnel Axis				Strike Parallel to Tunnel Axis		Irrespective of Strike
Drive with Dip		Drive against Dip				
Dip 45°–90° Very favorable	Dip 20°–45° Favorable	Dip 45°–90° Fair	Dip 20°–45° Unfavorable	Dip 20°–45° Fair	Dip 45°–90° Very unfavorable	Dip 0°–20° Fair

Source: From Bieniawski (1989).

Table 2.21 Influence of Orientation on Assessment

Joint Orientation Assessment for Tunnels	Value
Very favorable	0
Favorable	−2
Fair	−5
Unfavorable	−10
Very unfavorable	−12

Source: Adapted from Bieniawski (1989).

definite effect on ground control. Table 2.20 illustrates the effect of the joint orientation on tunneling. Note that the best tunneling condition is when the dip is between 45° and 90° and the tunnel is being driven with the dip, that is, perpendicular to the strike.

Table 2.20 is consistent with Figures 2.9–2.11 where driving perpendicular to the strike is preferred and driving parallel with the strike is less than favorable.

The sum for the first five ratings is adjusted to reflect joint orientation. As shown in Table 2.21, no points are added for favorable joint orientation and up to 12 points may be deducted for unfavorable joint orientation.

The determination of the rock mass is the algebraic sum of the ratings of all of the parameters. To illustrate an analysis follows of the quality of a rock mass based on our determination of the RMR.

Table 2.22 Total of RMR Parameters

Strength	Average	4
RQD	Good	17
Joint spacing	<2 m	20
Joint condition	Slightly rough	20
Groundwater	Wet	7
Orientation	Favorable	−2
Total		**66**

Source: From Bieniawski (1989).

Table 2.23 Geomechanics Classification of Rock Masses

	RMR	
Class	Description of Rock Mass	Sum of Ratings Increments
I	Very good rock	81–100
II	Good rock	61–80
III	Fair rock	41–60
IV	Poor rock	21–40
V	Very poor rock	0–21

Source: From Bieniawski (1989).

A rock mass that has an unconfined compressive strength of 40 MPa (5800 psi) is considered average rock and yields an average rating of 4. The RQD measurement of 80 receives a rock quality fair rating of 17.

When the distance between discontinuities is <2 m (<6.5 ft), the rating is 20. If the joints are slightly rough, the rating is 20. If the water inflow is wet, the rating is 7. If the orientation is favorable, the rating is −2.

Table 2.22 gives the total of the parameters.

The parameters used yield a RMR rating of 66. When the rating is compared with the above chart, the rock is determined to have a rating of Class II, Good rock. Applying the rating of 66 to the guideline for excavation and support one gets a recommended ground support.

Excavation can be full face with a 3 m (10 ft) advance and ground support should be completed within 20 m (66 ft) of the face. The ground support should consist of local bolts in the crown that are 3 m (10 ft) in length and spaced 2.5 m (8ft) with occasional wire mesh. Fiber-reinforced shotcrete can be used in lieu of wire mesh.

The above example uses Table 2.24 to determine the recommended excavation techniques and ground support for the RMR. The table has been modified to include fiber-reinforced shotcrete (FRS). Where the option of wire mesh or shotcrete is provided, the shotcrete prescribed is with fibers for reinforcement.

Table 2.24 Guidelines for Excavation and Support of 10-m Span of Rock

Rock Mass Class	Excavation	Rock Bolts (20 mm diameter, fully grouted)	Shotcrete	Steel Sets
I. Very good rock RMR: 81–100	Full face, 3 m advance	Generally no support required except spot bolting		
II. Good rock, RMR: 61–80	Full face, 1–1.5 m advance, complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh or FRS	100 mm of FRS crown where required	None
III. Fair rock, RMR: 41–60	Top heading and bench, 1.5–3 m advance in top heading, commence support after each blast, complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown or fiber-reinforced shotcrete	50–100 mm in crown and 50 mm in sides of fiber-reinforced shotcrete	None
IV. Poor rock, RMR: 21–40	Top heading and bench, 1.0–1.5 m advance in top heading, install support concurrently with excavation, 10 m from face	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and walls with wire mesh or fiber-reinforced shotcrete	100–150 mm in crown and 100 mm in sides of fiber-reinforced shotcrete	Light to medium ribs spaced 1.5 m where required
V. Very poor rock RMR: <20	Multiple drifts 0.5–1.5 m advance in top heading, install support concurrently with excavation, shotcrete as soon as possible after blasting	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh or FRS bolt invert	150–200 mm in crown, 150 mm in sides, and 50 mm on face w/wo FRS	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required, close invert

Source: After Bieniawski (1979).

The recommendations are based on a 10-m (33-ft) span. Support may vary for spans that are significantly smaller or larger.

Figure 2.17 illustrates the relationship between the RMR and the stand-up time of the rock. As one would expect, the stand-up time is related to the rock quality as determined by the RMR.

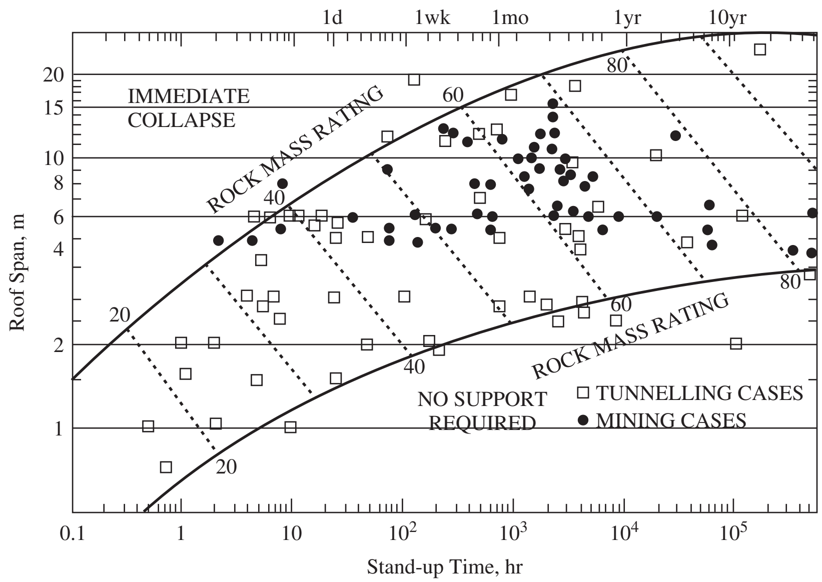


Figure 2.17 RMR and Stand-Up Time Versus Span (Bieniawski, 1989)

A summary of the RMS system is given in Table 2.25.

Q-*System* Barton, Lien, and Lunde of the Norwegian Geotechnical Institute developed the Q-system of rock mass classification in 1974. The Q-system uses numerical values to determine rock mass competence and the ground support required. The rock quality is based on a logarithmic rock mass quality scale from $Q = 0.001$ to $Q = 1000$ with the quality increasing with an increase in Q.

There are six parameters for making the determination:

RQD

Number of joint sets

Roughness of the most unfavorable discontinuity

Degree of alteration or filling along the weakest joint

Water inflow

Stress condition

These six parameters are grouped into three quotients to obtain the overall rock mass quality Q:

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

where RQD = rock quality designation

J_n = joint set number

Table 2.25 Rock Mass Rating System (Geometric Classification of Rock Masses)[a]

Parameter			Range of Values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4–10 MPa	2–4 MPa	1–2 MPa	For this low range uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100–250 MPa	50–100 MPa	25–50 MPa	5–25 MPa	1–5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
	Don Drill core Quality RQD		90%–100%	75%–90%	50%–75%	25%–50%	<25%		
2	Change Rating		20	17	13	8	3		
	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm		
3	Rating		20	15	10	8	5		
4	Condition of discontinuities (see E)		Very rough surfaces, not continuous, no separation, unweathered wall, rock	Slightly rough surfaces, separation <1 mm, slightly weathered walls	Slightly rough surfaces, separation <1 mm, highly weathered walls	Slicken sided surfaces, or gouge <5 mm thick or separation 1–5 mm, continuous	Soft gouge >5 mm thick or separation >5 mm, continuous		
	Rating		30	25	20	10	0		
	Inflow per 10 m tunnel length (l/m)		None	<10	10–25	25–125	>125		
5	Ground water	(Joint water press)/(major principal σ)	0	<0.1	0.1–0.2	0.2–0.5	>0.5		
	General conditions		Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. Rating Adjustment for Discontinuity Orientations (See F)

Strike and dip orientations		Very favorable	Favorable	Fair	Unfavorable	Very Unfavorable
	Tunnels and mines	0	−2	−5	−10	−12
Ratings	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	

C. Rock Mass Classes Determined from Total Ratings

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	<21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. Meaning of Rock Classes

Class number	I	II	III	IV	V
Average standup time	20 yr for 15-m span	1 yr for 10-m span	1 week for 5-m span	10 hr for 2.5-m span	30 min for 1-m span
Cohesion of rock mass (kPa)	>400	300–400	200–300	100–200	<100
Friction angle of rock mass (deg)	>45	35–45	25–35	15–25	<15

E. Guidelines for Classification of Discontinuity Conditions

Discontinuity length (persistence)	<1 m	1–3 m	3–10 m	10–20 m	>20 m
Rating	6	4	2	1	0
Separation (aperture)	None	<0.1 mm	0.1–1.0 mm	1–5 mm	>5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slicker sided
Rating	6	5	3	1	0

(*continued overleaf*)

Table 2.25 (*Continued*)

Parameter	Range of Values				
Infilling (gouge)	None	Hard filling <5 mm	Hard filling >5 mm	Soft filling <5 mm	Soft filling >5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0

F. Effect of Discontinuity strike and Dip Orientation in Tunnelling

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip, dip 45°–90°	Drive with dip, dip 20°–45°	Dip 45°–90°	Dip 20°–45°
Very favourable	Favourable	Very unfavorable	Fair
Drive against dip, dip 45°–90°	Drive against dip, dip 20°–45°	Dip 0°–20° irrespective of strike	
Fair	Unfavorable	Fair	

Source: After Bieniawski (1989).

[a]Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

[b]Modified after Wickham et al. (1972).

J_r = joint roughness number
J_a = joint alteration number
J_w = joint water reduction factor
SRF = stress reduction factor

Basically, the equation breaks the variables down into three sets of parameters, that is, RQD/J_n, by identifying discontinuities. The number of discontinuities indicates the overall block size, J_r/J_a represents the block shear strength by evaluating the condition of the discontinuities, and J_w/SRF is the ratio of the water pressure and the stress reduction factor that is considered the total stress and indicates the active stress.

For Q values, the RQD is determined by the RQD description. Table 2.26 provides the parameter values.

The equivalent dimension (D_e) is a function of both the size of the opening and the purpose of the excavation. It is determined by dividing the span, diameter, or wall height of the excavation by the excavation support ratio (ESR) (given in Table 2.27):

$$D_e = \frac{\text{excavation span, diameter, or height (m)}}{\text{ESR}}$$

The ESR equates Q to the use of the underground structure. That is, it provides a safety factor based on the use of the structure. Underground railway stations require a greater safety factor than a temporary mine opening. For this example, assume the structure is a major road with an excavation support ratio of 1.0.

The appropriate support measures are determined by the relationship between the equivalent dimension of excavation and Q.

As an example, determine the support requirements for the following conditions:

1. RQD = rock quality designation = 50
2. J_n = joint set number = 9
3. J_r = joint roughness = 0.5
4. J_a = joint alteration = 3
5. J_w = medium flow = 0.66
6. Stress reduction factor = multiple shear zones = 7.5

Assume a tunnel with a diameter of 6 m and the parameters given in Table 2.28.

Entering these variables into the equation gives

$$Q = \frac{50}{9}\frac{0.5}{0.75}\frac{0.66}{5} = 0.077$$

Table 2.26 *Q* Parameter Values

Description	Value	Notes
1. Rock Quality Designation	**RQD**	
A. Very poor	0–25	1. Where RQD is reported 1. Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25–50	
C. Fair	50–75	
D. Good	75–90	2. RQD intervals of 5. i.e., 100, 95, 90, . . . , are sufficiently accurate.
E. Excellent	90–100	
2. Joint Set Number	J_n	
A. Massive. no or few joints	0.5–1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $3.0J_n$.
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, “sugar cube,” etc.	15	2. For portals use $2.0J_n$.
J. Crushed rock, earthlike	20	
3. Joint Roughness Number	J_r	
a. Rock Wall Contact		
b. Rock Wall Contact before 10 cm Shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slicken sided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	
G. Slicken sided, planar	0.5	2. $J_r = 0.5$ can be used for planar, slicken sided joints having lineations, provided that the lineations are oriented for minimum strength.
c. No Rock Wall Contact When Sheared		
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
J. Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

Table 2.26 (*Continued*)

Description	Value	Notes
4. Joint Alteration Number	J_a	o_r degrees (approx.)
a. Rock Wall Contact		
A. Tightly healed, hard, nonsoftening, impermeable filling	0.75	1. Values of, the residual friction angle, are intended as an approximate guide to the mineralogic properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25-35
C. Slightly altered joint walls, nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25-30
D. Silty or sandy clay coatings, small clay fraction (nonsoftening)	3.0	20-25
E. Softening or low-friction clay mineral coatings. i.e., kaolinite, mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays (discontinuous coatings, 1–2 mm or less)	4.0	8–16
b. Rock Wall Contact Before 10-cm Shear		
F. Sandy particles, clay-free, disintegrating rock, etc.	4.0	25–30
G. Strongly overconsolidated, nonsoftening clay mineral fillings (continuous <5 mm thick)	6.0	16–24
H. Medium or low overconsolidation, softening clay mineral fillings (continuous <5 mm thick)	8.0	12–16
J. Swelling clay fillings, i.e., montmorillonite (continuous <5 mm thick); values of J_a depend on percentage of swelling clay-size particles and access to water.	8.0–12.0	6–12
c. No Rock Wall Contact When Sheared		
K. Zones or bands of disintegrated or crushed	6.0	
L. rock and clay (see G, H, and J for clay	8.0	
M. conditions)	8.0–12.0	6–24

(*continued overleaf*)

Table 2.26 (*Continued*)

Description	Value	Notes	
N. Zones or bands of silty or sandy clay, small clay fraction, nonsoftening	5.0		
O. Thick continuous zones or bands of clay	10.0–13.0		
P, R. See G, H, and J for clay conditions	6.0–24.0	—	
5. Joint Water Reduction	J_w	Approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow, i.e., <5 l/m locally	1.0	<1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0–2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5–10.0	1. Factors C–F are crude estimates: increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5–10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2–0.1	>10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1–0.05	>10	

Description	Stress Reduction Factor	SRF
(a) *Weakness Zones Intersecting Excavation, Which May Cause Loosening of Rock Mass When Tunnel is Excavated*		
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)	5
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)	2.5
D	Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E	Single shear zones in competent rock (clay free) (depth of excavation ≤50 m)	5.0
F	Single shear zones in competent rock (clay free) (depth of excavation >50 m)	2.5
G	Loose, open joints, heavily jointed or “sugar cube,” etc. (any depth)	5.0

Table 2.26 (*Continued*)

Description	Stress Reduction Factor	SRF		
		σ_0/σ_1	σ_θ/σ_c	SRF
(b) *Competent Rock, Rock Stress Problems*				
H	Low stress, near surface, open joints	>200	<0.01	2.5
J	Medium stress, favorable stress condition	10–200	0.01–0.3	1
K	High stress, very tight structure; usually favorable to stability, may be unfavorable for wall stability	5–10	0.3–0.4	0.5–2
L	Moderate slabbing after >1 h in massive rock	3–5	0.5–0.65	5–50
M	Slabbing and rock burst after a few minutes in massive rock	2–3	0.65–1	50–200
N	Heavy rock burst (strain burst) and immediate dynamic deformations in massive rock	<2	>1	200–400
		σ_θ/σ_c	SRF	
(c) *Squeezing Rock: Plastic Flow of Incompetent Rock Under Influence of High Rock Pressure*				
O	Mild squeezing rock pressure	1–5	5–10	
P	Heavy squeezing rock pressure	>5	10–20	
		SRF		
(d) *Swelling Rock: Chemical Swelling Activity Depending on Presence of Water*				
R	Mild swelling rock pressure	5–10		
S	Heavy swelling rock pressure	10–15		

Source: After Barton et al. (1974) and Barton (2002).
Notes: (i) Reduce these values of SRF by 25–50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterization. (ii) For strongly anisotropic virgin stress field (if measured): When $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.75\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\sigma_c$, where σ_c is the unconfined compression strength, σ_1 and σ_3 the major and minor principal stresses, and σ_θ the maximum tangential stress (estimated from elastic theory). (iii) Few case records available where depth of crown below surface is less than span width, suggest an SRF increase from 2.5 to 5 for such cases (see H). (iv) Cases L, M, and N are usually most relevant for support design of deep tunnel excavations in hard massive rock masses, with RQD/J_n ratios of about 50–200. (v) For general characterization of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0, and 0.5 is recommended as depth increases from say 0–5, 5–25, 25–250 to > 250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterization values of J_w. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed. (vi) Cases of squeezing rock may occur for depth $H > 350Q^{1/3}$ according to Singh (2000). Rock mass compression strength can be estimated from $\text{SIGMA}_{\text{cm}} \approx 5\gamma Q_c^{1/3}$ (MPa) where γ is the rock density in t/m^3 and $Q_c = Q \times \sigma_c/100$, (Barton 2000).

The equivalent dimension of the tunnel is given as

$$\text{Equivalent dimension} = \text{tunnel diameter}/\text{ESR} = 6/1 = 6$$

Figure 2.18 provides a graphical representation correlating Q, equivalent dimension, and ESR of 1, the ESR commonly used for tunnels. As an example,

Table 2.27 Excavation Support Ratio

Excavation Category	ESR
A. Temporary mine openings	3–5
B. Permanent mine openings, water tunnels for hydropower (excluding high-pressure penstocks), pilot tunnels, drifts, headings for large excavations	1.6
C. Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
D. Power stations, major road and railway tunnels, civil defence chambers, portal intersections	1.0
E. Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

Source: After Barton et al. (1974).

Table 2.28 Parameters for Tunnel 6 m in Diameter

Parameter	Description	Value
1. Rock quality	Fair	RQD = 50%
2. Joint sets	Three sets	$J_n = 9$
3. Joint roughness	Slickensided	$J_r = 0.5$
4. Joint alteration	Silty	$J_a = 3$
5. Joint water	Medium flow	$J_w = 0.66$
6. Stress reduction factor	Loose, open joints	SRF = 5

Source: Adapted from Barton (1974).

from the left y axis 6 is the equivalent dimension and $Q = 0.077$. Determining the point where the two lines intersect gives us the rock bolt spacing of 1.3 m (4 ft) and a reinforcement category of 6, with fiber-reinforced shotcrete approximately 150 mm (6 in.) thick. Following the line for the equivalent dimension of 6 to the right y axis indicates the rock bolt length should be approximately 2.5 m (8 ft).

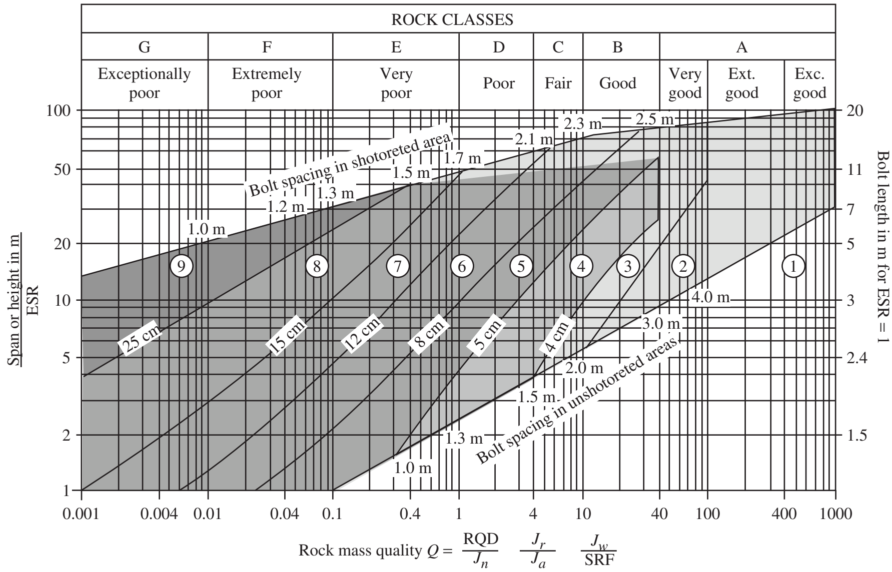
Table 2.29 provides the rock quality based on the Q rating. Above a Q rating of 400 the rock is considered exceptionally good, whereas, for a Q rating below a value of 0.001 the rock is considered exceptionally poor. Table 2.30 is support recommendations based on the Q.

The length of rock bolts can be estimated by using the excavation width B and the ESR:

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

For example, for an excavation with a minor road tunnel of width 5 m (16 ft) and ESR of 1.3, the bolt length is given as

$$L = 2 + 0.15(5) = 2.1\ \text{m}/1.3$$



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
4) Fiber-reinforced shotcrete and bolting, 5 - 9 cm
6) Fiber-reinforced shotcrete and bolting, 9 - 12 cm
7) Fiber-reinforced shotcrete and bolting, 12 - 15 cm
8) Fiber-reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolling
9) Cast concrete liring

Figure 2.18 Estimated Support Categories Based on Tunneling Quality Index *Q* (After Grimstad and Barton, 1993)

Table 2.29 Rock Quality Based on *Q* Rating

Group	Classification	Q
1	Good	010.000-0040.00
	Very good	040.000-0100.00
	Extremely good	100.000-0400.00
	Exceptionally good	400.000-1000.00
2	Very poor	000.100-0001.00
	Poor	001.000-0004.00
	Fair	004.000-0010.00
3	Exceptionally poor	000.001-0000.01
	Extremely poor	000.010-0000.10

Source: From Barton (1974).

Table 2.30 Q Support Recommendations

Support Category	Q	Conditional Factors RJD/J_a	J_r/J_a	Span/ESR (m)	P^a (kg/cm^2)	Span/ESR (m)	Type of Support	Supplementary Notes	
1^b	1000–400				<0.01	20–40	sb (utg)		
2^b	1000–400				<0.01	30–60	sb (utg)		
3^b	1000–400				<0.01	46–80	sb (utg)		
4^b	1000–400				<0.01	65–100	sb (utg)		
5^b	400–100				0.05	12–30	sb (utg)		
6^b	400–100				0.05	19–45	sb (utg)		
7^b	400–100				0.05	30–65	sb (utg)		
8^b	400–100				0.05	48–88	sb (utg)		
9	100–400	≥20			0.25	8.5–19	sb (utg)		
		<20					B (utg)	2.5–3 m	
10	100–400	≥30			0.25	14–30	B (utg)	2.3 m	
		<30					B (utg)	1.5–2 m + clm	
11^b	100–400	≥30			0.25	23–48	B (tg)	2–3 m	
		<30					B (tg)	1.5–2 m + clm	
12^b	100–400	≥30			0.25	40–72	B (tg)	2–3 m	
		<30					B (tg)	1.5–2 m + clm	
13	40–10	≥10	≥1.5		0.5	5–14	sb (utg)		I
		≥10	<1.5				B (utg)	1.5–2 m	I
		<10	≥1.5				B (utg)	1.5–2 m	I
		<10	<1.5				B (utg)	1.5–2 m + S 2–3 cm	I
14	40–10	≥10			0.5	9–23	B (tg)	1.5–2 m + clm	I, II
		<10					B (tg)	1.5–2 m + S (mr) 5–10 cm	I, II
							B (utg)	1.5–2 m + clm	I, III

15	40–10	>10			0.5	15–40	B (tg)	1.5–2 m + clm	I, II, IV
		≤10					B (tg)	1.5–2 m + S (mr) 5–10 cm	I, II, IV
16[b,c]	40–10	>15			0.5	30–65	B (tg)	1.5–2 m + clm	I, V, VI
		≤15					B (tg)	1.5–2 m + S (mr) 10–15 cm	I, V, VI
17	10–4	>30			1.0	3.5–9	sb (utg)		I
	≥10, ≤30						B (utg)	1–1.5 m	I
	<10	≥6					B (utg)	1–1.5–2 m + S 2–3 cm	I
	<10	<6					S 2-36 cm		I
18	10–4	>5		≥10	1.0	7–15	B (tg)	1–1.5 m + clm	I, III
		>5		<10			B (utg)	1–1.5 m + clm	I
		≤5		≥10			B (tg)	1–1.5 m + S 2–3 cm	I, II, III
		≤5		<10			B (utg)	1–1.5 m + S 2–3 cm	I
19	10–4			≥20	1.0	12–29	B (tg)	1–2 m + S (mr) 10–15 cm	I, II, IV
				<20			B (tg)	1–1.5 m + S (mr) 5–10 cm	I, II
20[b]	10–4			≥35	1.0	24–52	B (tg)	1–2 m + S (mr) 20–25 cm	I, V, VI
				<35			B (tg)	1–2 m + S (mr) 10–20 cm	I, II, IV
21	4–1	≥12.5	≤0.75		1.5	2.1–6.5	B (tg)	1 m + S 2–3 cm	I
		<12.5	<0.75					S 2.5–5 cm	I
			>0.75				B (utg)	1 m	I
22	4–1	>10, <30	>1.0		1.5	4.5–11.5	B (utg)	1 m + clm	I
		≤10	>1.0					S 2.5–7.5 cm	I
		RJD/J_n	J_r/J_a						
		≥30					B (utg)	1 m	I
23	4–1			≥15	1.5	8–24	B (tg)	1–1.5 m + S (mr) 10–15 cm	I, II, IV, VII
				<15			B (utg)	1–1.5 m + S (mr)	I

(*continued overleaf*)

Table 2.30 (*Continued*)

Support Category	Q	Conditional Factors RJD/J_a	J_r/J_a	Span/ESR (m)	P^a (kg/cm^2)	Span/ESR (m)	Type of Support	Supplementary Notes
24[c,d]	4–1	≥30	1.5	18–46	B (tg) 1–1.5 m + S (mr) 15–30 cm			I, V, VI
		<30			B (tg) 1–1.5 m + S (mr) 10–15 cm			I, II, IV

Source: From Barton et al. (1974).

[a] Approximately.

[b] Original authors' estimates of support. Insufficient case records available for reliable estimation of support requirements. The type of support to be used in categories 1–8 will depend on the blasting technique. Smooth-wall blasting and thorough barring down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is >25 m. Future case records should differentiate categories 1–8. *Key*: sb = spot bolting; *B* = systematic bolting; utg = untensioned, grouted; tg = tensioned (expanding shell type for competent rock masses, grouted posttensioned in very poor quality rock masses; S = shotcrete; mr = mesh reinforced; clm = chain-link mesh; CCA = cast concrete arch; sr = steel reinforced. Bolt spacings are given in meters. Shotcrete or cast concrete arch thickness is given in centimeters.

[c] See supplementary note XII.

Q-System: Support Measures—Supplementary Notes

I. For cases of heavy rock bursting or "popping," tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when popping activity ceases.

II. Several bolt lengths often used in same excavation, i.e., 3, 5, and 7 m.

III. Several bolt lengths often used in same excavation, i.e., 2, 3, and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2–4 m.

V. Several bolt lengths often used in same excavation, i.e., 6, 8, and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4–6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain-link mesh and a free-span concrete arch roof (25–40 cm) as permanent support.

VIII. Cases involving swelling, e.g., montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.

XI. According to the authors (Barton et al.) experience, in case of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell expansion type) if the value of RQD/J_n is sufficiently high (i.e., >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e., RQD/J_n < 1.5, for example, a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when RQD/J_n < 1.5 or when a lot of clay is present unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick-setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

[d] See note XII in Table 5.6.

[e] See note XIII in Table 5.6.

For maximum unsupported span:

$$\text{Maximum unsupported span} = 2\text{ESRQ}^{0.4}$$

If the rock quality is 10, the maximum unsupported length can be

$$2(1.3)(10^{0.4}) = 6.5 \text{ m (21 ft)}$$

Utilization of the rock mass quality helps both engineers and contractors determine the best type of support for the rock type in which they are mining. However, each classification system has its own strengths and the results should always be compared. It is recommended that at least two classification systems be used to allow a comparison of the results.

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3 Soft Ground (Soil)

Soil is decayed organic matter (solid particles) and the uncemented aggregate of mineral grains with liquid and gas in the void spaces between the solid particles. Soil is a natural accumulation of mineral grains that can, by temperate mechanical means such as agitation in water, be separated.

Soil is two types: residual and transported. Soil and unconsolidated sediments have different origins. They are chiefly the result of physical (mechanical) and chemical weathering and of organic origin. Physical weathering generally results in coarsely grained deposits. Chemical weathering results in varied deposits. Weathered sandstones form sandy soils whereas shales form silty and clayey soils. They can be transported by water, wind, or glaciers and some remain at their source of origin. The method of transportation will affect the nature of tunneling. Transported soils may be classified according to the method of transportation and deposition being transported by rivers, glacial, lake, marine, or wind.

Alluvium is material deposited by rivers. It usually extensively forms in the lower part of the river flow, where floodplains and deltas are formed. However, soil may be deposited anywhere the river overflows its banks or where the velocity of a river is slowed. The size of the particles is directly related to the velocity of the river. High velocity will enable the river to carry larger rocks; in some cases the rock will be moved along the river bottom and be bounced around until the velocity of the river flow slows and no longer can move the larger rock. As the river velocity decreases, the larger particles drop out. Once the river reaches a floodplain, the slow water will drop the fine clay particles. This change in particles adds to the difficulty of mining through soils. Changes in soil types, and thus changes in strength, the effects of water, and stand-up time, have considerable effects on tunneling.

Glacial deposits are, as the name implies, deposits transported by glaciers. As the glacier retreats, it leaves rocks and soils that had been picked up and moved by the glacier; this material is called glacial till or moraine. If the retreat of the glacier stops and remains stationary for a long time, the distribution of the material is unevenly located at the rim of the glacier. When the glacier retreats, the uneven deposit left is referred as terminal moraine. These deposits can cause considerable problems to tunneling. The soil will contain boulders of various sizes that will hamper production; thus tunnel boring machines are required to have the capability of breaking the boulders to a size that can be handled by the machines' mucking system. For open shields or liner plates the tunnels may have to use pavement breakers or hoe rams, if the tunnel is big enough, to break up the boulder. In an extreme case, such as the boulder being very large, blasting may be the only effective method. The last glacial period covered most of Canada and most of the northern United States. Where the glacier left the bedrock fairly level, tunneling in the bedrock can be done without concerns about leaving rock and mining in the soil. If the bedrock is not level but has peaks and valleys, the tunnel would be mined in both soil and rock, an undesirable situation. If the tunnel has areas of soil one not only is dealing with a mixed-face condition but could encounter boulders.

When the wind erodes or picks up sediments and deposits them at another location, these are known as *aeolian deposits*. The best examples of aeolian deposits are sand dunes. Their shape and content are constantly changed by the wind. The smaller particles that are carried greater distances form deposits. These dusty, powdery soils are known as loess.

Deposits formed at the bottom of freshwater pools and lakes are referred to as *lacustrine deposits* and can sometimes be deposited by glaciers. Lakes differ from marine environments in that the depth of the sedimentation in lakes is greater than in marine environments. Lakes are also smaller and nearly closed systems in which water movement such as wave action is less, resulting in lower energy levels depositing coarser sediment (sand and gravel) in the shallow water areas of lakes. Finer grained sediment (silt and clay) is deposited in deeper water areas of lakes. Alternating thin layers of light-colored, coarser grained sediment and dark-colored, finer grained sediments are a type of lacustrine deposit found in both glacial and nonglacial lakes.

Marine deposits are the result of river flows depositing sediments in the marine waters. This physical processes mainly reworks and distributes carbonate materials on marine shelf but can also help in the production of carbonates. Moderate water circulation on marine shelf brings nutrients from deeper water to a shallow shelf region which aids in organic growth of voids, fecal pellets that eventually become cemented together. Waves constantly move fine carbonate mud and coarser sediment to form sand or gravel-covered tidal flats, beaches, dunes, marshes, lagoons, and swamps or transports these sediments seaward to form spits, tidal deltas and bars, and barrier islands. Waves pounding against coastal rocks also contribute rock particles and sediment to the coastal shelf and seaward. The Outer Banks of North Carolina is a chain of barrier islands that contain beaches, lagoons, and spits. Reefs can be characterized as either thick masses of living carbonate "rock" or structures produced by sediment-binding, live organisms. The Great Barrier Reef in Australia is the largest coral reef in the world.

Bioherms are mounds of dead organic material that collects in rocks of different composition. Organisms are capable of extracting calcium carbonate ($CaCO_3$) from seawater to build protective shells or skeletons, although the availability of $CaCO_3$ in seawater is controlled by pH, temperature, and carbon dioxide content. When these organisms die, their remains collect and form carbonate deposits known as bioherms. Carbonate form in the earth's current oceans are not the same as those that formed carbonates in ancient oceans.

Other marine deposit environments include deltas, beaches, barrier bars, estuaries, lagoons, and tidal flats. Beach and barrier bar deposits mostly contain fine- to medium-grained well-sorted sand as well as placer gold, platinum, and other minerals.

Estuarine deposits, such as in the Chesapeake Bay and San Francisco Bay, consist of cross-bedded sands and mud or a mixture of both sand and mud. Lagoon deposits include evaporites, fine-grained sediments, and black shales. Delta deposits and tidal flat deposits, such as in the Mississippi River Delta, primarily contain muds in the upper zone, mud and sand in the middle zone, and sand in the lower zone.

Autochthonous deposit refers to residual soils which cover rock outcroppings of large areas. It covers the rock with various deposits of organic origin, such as lake marl, peat, or diatomaceous earth, which remain at the location of their origin. Some sediment is the result of rivers flowing into lakes. This may add mechanically weathered particles to the lake bottom. The nature of the deposit affects the ability to tunnel through it. When planning the project, the following question should be asked: Does the soil appear to be consistent or is there a chance of a change in the makeup of the soil? Although river locations are the most common, mixed sediments can be found in other deposit areas. For example, when tunneling in areas of Houston one can experience a sudden change in material when the tunnel intersects sand pockets. A contractor tunneling in Houston's Beaumont clay found a sudden change in ground from clay to sugar sand that caused the tunnel to fill with the sand. A shaft had to be sunk from the surface to retrieve the machine. Of course, no boring program can intersect all sand pockets. However, if one or two borings indicate sand pockets, one knows to be prepared for them.

ENGINEERING PROPERTIES

Engineering properties are those characteristics which allow engineers to determine the potential problems encountered while tunneling. The engineering qualities that should be considered are grain size, density, Atterberg limits, porosity, and swelling.

The Unified Soil Classification System, or USCS, is a soil classification system used in engineering and geology disciplines to describe the texture and grain size of a soil. The soil is divided into two categories, coarse and fine grained as separated by a No. 200 sieve.

The USCS is used to describe the texture and grain size of a soil. The classification system is represented by a two-letter symbol and can be applied to most unconsolidated materials. With the exception of Pt, the letters are described below. Table 3.1 gives the definitions.

Table 3.2 is a chart of the USCS symbols. If the weight of fines passing a No. 200 sieve of the soil has 5–12% (5% $<$ #200 $<$ 12%), both grain size distribution and plasticity have a significant effect on the engineering properties

Table 3.1 Symbol Definitions

First and/or Second Letters		Second Letter	
Symbol	Definition	Letter	Definition
G	Gravel	P	Poorly graded (uniform particle sizes)
S	Sand	W	Well graded (diversified particle sizes)
M	Silt	H	High plasticity
C	Clay	L	Low plasticity
O	Organic		

Table 3.2 Symbols of the USCS

Major Divisions			Group Symbol	Group Name
Coarse-grained soils more than 50% retained on No. 200 (0.075-mm) sieve	Gravel >50% of coarse fraction retained on No. 4 (4.75-mm) sieve	Clean gravel < 5% smaller than No. 200 sieve	GW	Well-graded gravel, fine to coarse
			GP	Poorly graded gravel
		Gravel with > 12% fines	GM	Silty gravel
			GC	Clayey gravel
	Sand ≥ 50% of coarse fraction passes No. 4 sieve	Clean sand	SW	Well-graded sand, fine to coarse sand
			SP	Poorly graded sand
		Sands with > 12% fines	SM	Silty gravel
			SC	Clayey sand
Fine-graded soils more than 50% passes No. 200 sieve	Silt and clay liquid limit < 50	Inorganic	ML	Silt
			CL	Clay
		Organic	OL	Organic silt, organic clay
	Silt and clay liquid limit ≥ 50	Inorganic	MH	Silt of high plasticity, elastic silt
			CH	Clay of high plasticity, fat clay
		Organic	OH	Organic clay, organic silt
Highly organic soils			Pt	Peat

of the soil and thus a dual notation is used. For example, GW-GM represents “well-graded gravel with silt.” If more than 15% by weight is retained on a No. 4 sieve (R#4 > 15%), the soil has a significant amount of gravel, and the suffix “with gravel” may be added to the group name but the group symbol does not change. Thus, SP-SM may refer to “poorly graded SAND with silt and gravel.”

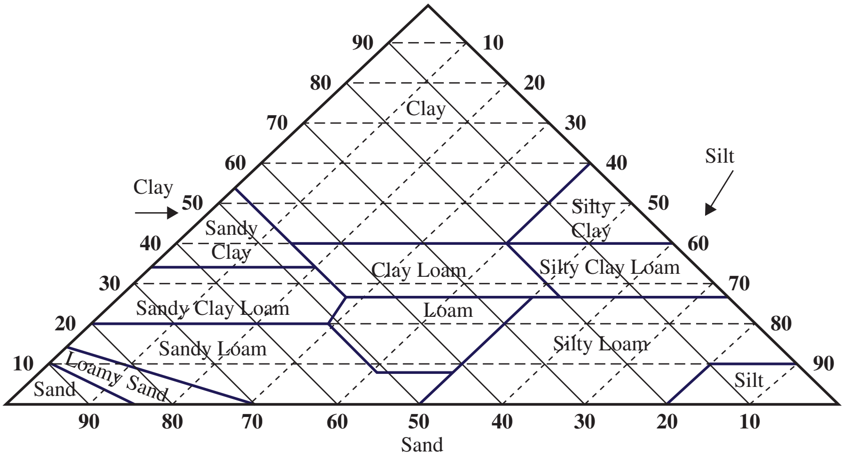

Figure 3.1 USDA Soil Classification

The U.S. Department of Agriculture (USDA) classifies soils into 12 types, which are referred to as “textures.” The classification is based to their relative percentages of sand, silt, and clay. Since these three percentages total 100, any soil sample can be plotted on a ternary graph, which is an equilateral triangle whose apexes represent pure clay, pure silt, and pure sand. Figure 3.1 shows the typical 12 soil textures as polygons superimposed on a triangle.

Cohesive soils are clay, silty clay, and sand clay mixtures where cohesionless soil consists of sand, gravel, and sand and gravel mixtures. The grain size for sand is between 0.05 and 2.0 mm (0.08 in.). To appreciate how small a grain size can be, 0.05 mm is about one-tenth the thickness of a standard mechanical pencil lead (0.5–0.7 mm). Table 3.3 provides the diameters of soils by their common names.

The cohesive strength of a soil is the most important indicator of strength. It is generally measured by its unconfined compressive strength or undrained shear strength; these are the best indicators of stand-up time, defined as the time

Table 3.3 Comparison of Particle Sizes

Soil Name	Diameter, mm (USDA Classification)
Clay	<0.002
Silt	0.002–0.05
Very fine sand	0.05–0.10
Fine sand	0.10–0.25
Medium sand	0.25–0.50
Coarse sand	0.50–1.00
Very coarse	1.00–2.00

that elapses before the unsupported sections of the removed materials begin to fall. The stability of cohesive soil is dependent on plastic behavior. The shear strength of soft clays will generally increase with depth.

Sand and gravel are cohesionless, which means the stand-up time is low. Angular sand has a longer stand-up time than rounded particles and the stability of noncohesive mixed soil is affected by the presence or lack of water. A little bit of moisture, which results in negative pore pressure, will help the sand stand longer. As it desiccates, it loses stand-up time.

The principal indicator of the gravel and sand competence is density. Density is the property that determines the tightness of the material; the higher the density, the stronger the material. If it is low, the material is weak and there is a greater chance of water entering or passing through the material into the tunnel.

Field tests are part of the site investigation. One of the most important tools of the investigation is the split-spoon sampler. Retrieving a sample disturbs the sample, and therefore the split spoon is only used to identify soil type and in laboratory tests. In addition to taking samples, the split spoon can be used to determine density. This test is known as the standard penetration test (SPT) and is performed by dropping a 64-kg (140-lb) weight 762 mm (30 in.), where N is defined as the number of blows it takes to drive the split spoon 25.4 mm (1 ft). The more blows required, the denser the material. Table 3.4 gives the density of fine-grained soil according to the number of blows N.

One key property of clay and silt is the plasticity index (PI). Soil samples that have the same PI will have other properties that are the same. As part of the "Atterberg Limits" (title) the PI is the range of plasticity, that is, the range of moisture content in the soil that still remains plastic. It is the difference in the moisture content of the liquid limit (LL) and the plastic limit (PL). The LL is the moisture content at which the soil passes from the liquid state to the plastic state. The PL is the content of moisture where the soil goes from the plastic state to a semisolid as moisture is removed. The PI is equal to LL−PL. The PI is the property used to determine the characteristic of the material and provides a basis for cohesive soil classification. The typical grain sizes of silt are between 0.05 and 0.002. There are few samples that do not fall within this range.

Table 3.4 NAVFAC Guide for Consistency of Fine-Grained Soil

SPT Penetration (blows/ft)	Estimated Consistency	S_{uc} (tons/ft^2)
2	Very soft	0.25
2–4	Soft	0.25–0.50
4–8	Medium	0.50–1.0
8–15	Stiff	0–2.0
5–30	Very stiff	0–4.0
>30	Hard	4

The physical and engineering properties determine the stand-up time of a soil. Soils are classified as firm, raveling, running, flowing, squeezing, and swelling.

The stand-up time of firm ground can be long enough to allow the arch concreting. Therefore, a roof section may be left unsupported in firm ground for several days without any noticeable movement in the crown or walls. Firm ground has soils that range between coarse grained, such as sand or sandy gravel including a clay binder, and extremely fine-grained soil, such as stiff, intact clay. However, raveling ground gradually breaks up into chunks, flakes, or angular fragments. The stand-up time increases as the width of the tunnel decreases. The most common type of ground behavior is raveling. Moderate cohesion that dries over time causes fragmentation and chunks to fall. The stand-up time of soils is greatly dependent on cohesion. That is, soils that have very little cohesion, such as sands and gravels, will flow, whereas soils with high cohesion will remain firm.

Fine and moist sand, in addition to sand-and-gravel mixtures containing some silt or clay to act as a binder, and stiff, fissured clays are the most common types of raveling. The surface tension of the films of water which surround the points of contact cause the sand's cohesion. The cohesion is gone once the particles are separated. Once the cohesion is lost, raveling begins. Raveling is followed by a cohesionless run.

In the slaking process, once the soil is exposed to the air, capillary action is reduced, causing an increase in the rate of raveling. Capillary pressure is the pressure exerted on the skin's particles. When water evaporates, the film shrinks, causing the particles to be pulled closer together and the chunk to get harder. If the chunk of soil shrivels and is then submersed in water, the chunk will swell and finally disintegrate. The increase in water content will reduce and finally eliminate the capillary pressure of the particles of which the chunk is composed.

Slaking also occurs when the chunk is surrounded by moist air with a higher temperature than the chunk. The lower temperature in the chunk causes the water vapor on the surface of the chunk to condense and thus the capillary pressure to decrease, thereby allowing the chunk to swell and finally fall apart.

Raveling is affected by the tunnel's position relative to the water table. Above the water table the exposed soil in the tunnel is probably dry (desiccated). The drying out of the intact raveling soil, such as dense silty sand, increases its cohesion, thereby delaying raveling. Conversely, in stiff, fissured clay the desiccation causes the fissures to expand and the raveling to accelerate. Below the water table the water migrates toward the exposed surfaces from the adjacent ground. The water enters the voids of the soil exposed to the air in the tunnel. Before the tunnel was excavated and the soil disturbed, the soil through which the tunnel would pass was very compressed by the weight of the soil above it. The excavations of the tunnel caused the pressure in the exposed soil to reduce. With the loss of load from the overburden, the soil tends to expand the drawing water from the soil further from the tunnel walls into the expanding voids. The effects on ground stability are aggravated by the water drawn from the adjoining soil. As the load increases, the water creeping toward the tunnel causes the raveling process to accelerate.

When the water and soil move into the tunnel from all sides, the tunnel is in flowing ground. The water must flow or seep into the tunnel for the ground around the tunnel to be considered flowing. The soil has to be cohesionless to be considered flowing. Flowing ground depends on the permeability, cohesion, and hydrostatic head of the groundwater (water pressure).

Squeezing Ground

With squeezing ground, all exposed surfaces slowly but continually move toward the tunnel. These conditions are present only in soft and medium clay. When a tunnel is driven through by something other than a shield, the mining of the face removes the lateral support of the block of soil ahead of the face. This may be avoided or mitigated by properly installed tight face boards or compressed air. Prior to the removal of the lateral support, the block carries the entire weight of the soil above it. As the lateral support is removed, except in the case of soft clay, part of the overburden load is temporarily transferred by dome action (see Figure 3.2) on the surrounding soil. The remainder is carried by the block. If the block is not strong enough, this load and the weight of the soil contained in the block tend to crush the block, causing it to bulge into the tunnel. The weight of the prism that squeezes clay into the tunnel is equal to the gross driving force. The loading and the vertical compression of the block cause a downward movement that is resisted by shearing stresses that transfer the load to the adjacent ground. To mobilize the shearing forces essential to create the half-dome action, a downward movement of the clay is needed. For this reason, to have a noticeable amount of surface settlement due to loss of ground is a threat when tunneling in medium or soft clay by hand in a shallow tunnel. In a deep tunnel the void of the tunnel is dissipated before it can affect the surface. This results in face movement, a semi-elastic compression of the block of clay adjacent to the working face. Squeeze can be caused by the load on the block being high, resulting in a further increase of creep of the clay. Loss of ground and settlement of the ground surface are practical limitations of squeeze.

Swelling Ground

Soils that gradually expand into the tunnel because of volume increase are said to be swelling ground. In tunnels less than 100 ft deep, swelling will occur only in clays that had been previously heavily surcharged by the weight of temporary overburden. Stiff, intact clay or stiff clay with a network of joints is needed for swelling ground. During site investigation, if it is learned that the clay strata through which the tunnel will be driven have been surcharged in the past, the potential for squeezing ground should be considered.

Table 3.5 serves as an abbreviated look at the classifications behavior, and types of soil, allowing us to predict the behavior of the soil through which we

Table 3.5 Tunnelman's Ground Classification

Hard	Excellent standup time. Tunnel may be advanced without crown support.	Very hard calcareous clay: cemented sand and gravel.
Firm	Good standup time. Heading can be advanced without initial support, and final lining can be constructed before the ground starts to move.	Loess above water table, hard clay, marl, cemented sand, and gravel when not highly overstressed.
Slow raveling	Sometimes after the ground has been exposed, chunks or flakes of material begin to drop out of the crown or walls due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground).	Depending upon degree of overstress. Slow raveling occurs in stiff fissured clays above the water table.
Fast raveling	Very short standup time. The process starts within a few minutes. In fast-raveling ground; otherwise the ground is slow raveling.	Below the water table residual soils or sand with small amounts of binder may be fast raveling. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
Squeezing	Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Occurs at shallow to medium depth in clay of very soft or medium consistency. Rate of squeeze depends on degree of overstress. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depth behind face.
Cohesive running	If the running soil is preceded by a brief period of raveling, the ground is considered to be cohesive running.	Occurs in clean, fine, and moist sand.
Running	Granular materials without cohesion are unstable at slopes greater than their angle of repose ($\pm$ 30°–35°). When lateral support is removed, exposing steeper angle slopes, they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive running.
Flowing	A mixture of soil and water flows into the tunnel like viscous fluid. The material can enter the tunnel from the invert as well as from the, face, crown, and wall and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. Any effective grain size in excess of approximately 0.005 mm (0.20 in.) May also occur in highly sensitive clay when such material is disturbed.

(*continued overleaf*)

Table 3.5 (*Continued*)

Swelling	Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.
Squeezing	The soil advances rapidly into the tunnel as a plastic flow.	Clays and silts with a high plastic index.
Boulders	Difficulty advancing a shield or forepoling. May require hand mining or blasting ahead of the machine.	Boulder, glacial till; some residual soils; some landside deposits; rip rap fill. The material around boulders may be sand, gravel, silt, clay, or combinations.

Source: From Terzaghi (1950); modified by Heuer (1974).

are tunneling. Above the water table cohesive (clayey) soils act like ductile plastic material that theoretically moves into the tunnel in a uniform manner.

Table 3.6 illustrates tunnel behavior in clean gravel and sand.

Dry or partially saturated sand and gravel above the groundwater table may possess some temporary apparent cohesion from negative pore pressure. Below the water table, the material lacks sufficient cohesion or cementation and can easily run or flow into the excavation.

Terzaghi (1977) summarized the behavior of sands and gravels in tunneling as shown in Table 3.6. Cleaner sand is more likely to run or flow when the face is unsupported during tunnel construction.

Figures 3.2 and 3.3 are representations of the loading of the tunnel by granular material and clay, respectively. Note how wide the arch is in granular material relative to that in cohesive soil. This indicates not only a reduced stand-up time but greater loading of the sand. This can necessitate more robust support for the tunnel.

Cohesionless ground generally has low stand-up time and thus needs immediate support. The term *running ground* relates to the clean gravel and clean coarse sand located above the water table that is considered to have a stand-up time of zero. With no cohesion the running ground will run with a slope angle of more than 34° when the lateral support is removed (Proctor and White, 1977). Because of the inability to bridge the gap in the crown support if it is greater than a few times the diameter of the largest grains, tight spiling or forepoling is necessary. Even though running ground has great mobility, the roof load of the tunnel support, except in a very shallow tunnel, is no more than a small fraction of the weight of the ground above the tunnel. The load is nearly independent of the depth if the depth of the tunnel is more than 1.5 times the width and height of the tunnel combined. In circular tunnels the height of the load on the tunnel is considered to be zero. The action of the tunnel support carrying only a small fraction of the soil above the tunnel is called arching.

Table 3.6 Tunnel Behavior in Soft Ground

	Significant Properties of Clean Gravel and Sand			
	>= greater than		<= smaller than Probable Behavior in Tunnel	
			Below Water Table	
Designation	Grain Size in mm.	Above Water Table	Free Air	Compressed Air
Gravel	>2.0	Running ground. Uniform ($U^* < 3$) and loose. ($N^{**} < 10$) materials with round grains run much more freely than well-graded ($U > 6$) and dense ($N > 10$) ones with angular grains.	Flowing conditions combined with extremely heavy discharge of water.	Running ground, excessive loss of air.
Sandy gravel	>0.2		Flowing conditions combined with heavy discharge of water.	Running or cohesive-running ground, heavy loss of air.
Coarse-to-medium sand	2.0–0.2			
Very fine sand	0.1–0.05	Loose ($N < 10$): cohesive-running Dense ($N > 30$): fast-raveling	Flowing ground.	Fast-raveling ground, moderate loss of air.
(Quicksand)				

Source: Terzhagi. (1977).

$^*U = \frac{D_{60}}{D_{10}}$ Uniformity coefficient. $^{**}N$ = Number of blows per foot of penetration in standard penetration test.

Field Identification: Disintegrate completely upon drying. In *uniform sand* or gravel all particles are of approximately the same size. Grains of *very fine sand* can hardly be discerned with the naked eye. Coarser grains can be clearly seen. *Well-graded sand* or sandy gravel contains all sizes between large and small. *Poorly-graded gravel* is a gravel mixed with fine or medium sand, without any coarser sand.

Laboratory Investigations: Mechanical analysis, examination with hand lens, record of prevalent shape of grains and of mineral composition.

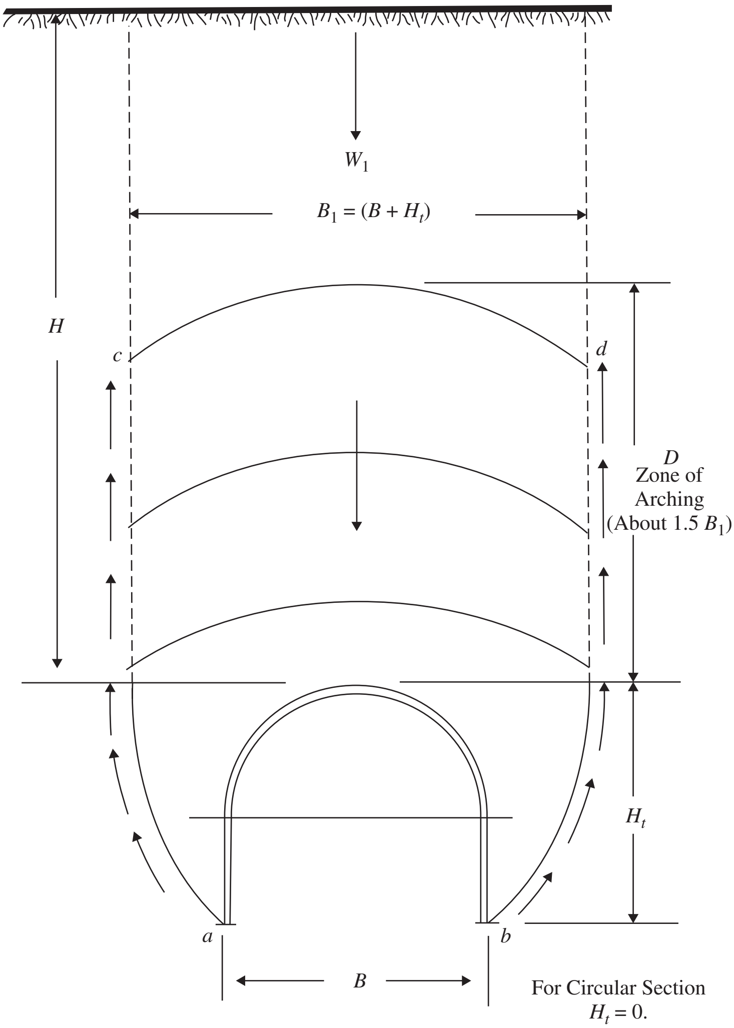

Figure 3.2 Granular Material Arching Concept (Proctor and White, 1977)

The ground arch is the soil that transfers the load to the ground located on both sides of the tunnel. This action indicates that the majority of the load of the overburden is carried by the ground located beside the tunnel.

Research has generated conclusions regarding the criteria for the arch action as well as determining the roof support load in running ground and those factors which determine load on the roof support in flat-roof tunnels through cohesionless sand situated above the water table (Proctor and White, 1977).

Local stress relaxation, caused by tunneling, inevitably results in the arching action to occur. In Figure 3.2 the ground arch width B_1 is represented by the

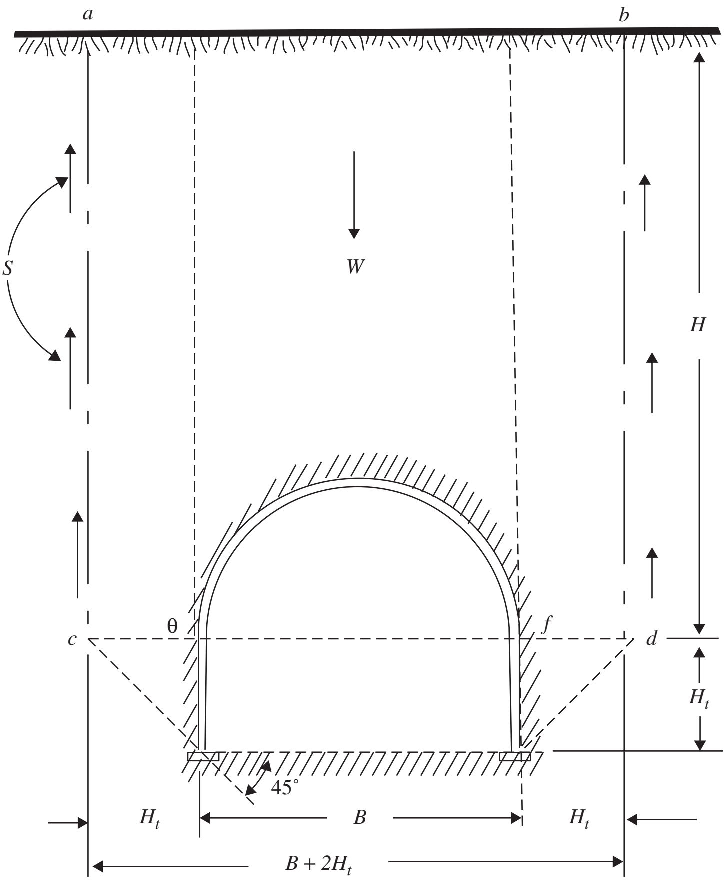

Figure 3.3 Load Action on Tunnels through Homogeneous Clay

area within *a–c–d–b*. The mass of sand that comprises the ground arch tends to move downward into the tunnel while the tunneling and erection of supports occur. Friction along the vertical boundaries *ac* and *bd* of the mass resists this downward movement. Most of the weight of the overburden is transferred by the friction forces, with height *H*, onto the ground on both sides of the excavation. The balance of the load, equivalent to a height H_p, is what the roof is required to support, where H_p is the equivalent height of the material that the tunnel supports will be required to carry. It is the product of the constant *C*, whose value depends on the compactness of the sand, and the distance *d* that the crown of the ground arch subsided during tunneling and ground support installation.

The thickness *D* of the ground arch is approximately equal to $1.5B_1$. The loading of the overburden above the ground arch on the tunneling operations is mostly unaffected.

The load on the support of the intrados of the arch value can be reduced to a value $H_{p,\min}$ by a slight downward movement of the crown of the ground arch. This is much smaller than the thickness D of the ground arch. The load on the roof support increases again if the crown of the ground arch is allowed to subside some more. The resulting increase in load on the roof support, approaching the value $H_{p,\max}$ is still smaller than D.

The load on the roof of the tunnel will increase about 15% immediately after the arching occurs. Because of the relatively short service period, this loading is inconsequential and can be ignored (Proctor and White, 1977).

The loading on the tunnel support increases as the depth of the overburden increases until H approximates the width $1.5B_1$ of the ground arch. Once this is reached, the load H_p becomes relatively independent of the depth of the overburden.

All other variables remaining constant, as the width B_1 of the ground arch increases, H_p increases in almost direct proportion. This can be expressed as

$$H_p = C \times B_1$$

An example of this equation with different sand density and constant C follows:

For dense sand:

$$H_{p,\min} = 0.27(B + H_t)$$

where $C = 0.27$;

$$H_{p,\max} = 0.60(B + H_t)$$

where $C = 0.60$.

For loose sand:

$$H_{p,\min.} = 0.47(B + H_t)$$

where $C = 0.47$;

$$H_{p,\max} = 0.60(B + H_t)$$

where $C = 0.60$.

For the roof loading, the weight can be determined per unit of length by the equation

$$Q_h = wH_pB$$

where w is the unit weight of the sand.

The type of ground will determine the amount of stand-up time. Running ground will cause the tunnel to require breasting of the block of ground just ahead of the working face. The amount of breasting is greatly mitigated by the lateral support. As a result, while breasting boards are being installed, some material will run into the tunnel. This loss of material reduces the compactness of the soil, causing the sand above the tunnel to settle a distance of a few percent of the height of the working face. However, the subsidence of the sand that is located above the tunnel will likely be a little greater, rather than smaller, than the yield required to reduce the roof load to $H_{p,\min}$. Even though the related load is a little greater than $H_{p,\min}$, the difference is insignificant.

Because of the circular walls, the width of the zone of arching is greatly reduced in circular tunnels. The roof load is further reduced because of the arched roof. Thus the estimate of the load on the tunnel roof can be safely determined by using $H_t = 0$ in the equations above.

Using the loose sand for the maximum condition gives

$$H_p = 0.60(B)$$

where $\quad B = 5\text{ m} \quad H_p = 0.60(5) = 3\text{ m}$

To be classified as raveling ground, the stand-up time must be long enough to allow the installation of at least one support unit, such as liner plate.

In stiff clay, if the unconfined compressive strength of the clay on both sides of the tunnel is considerably greater than the full overburden load, the block of clay above the roof will have a tendency to slide down along vertical sections through e and f, not along the sections ac and bd. This reduces the width B. Therefore the load on the roof support under this condition can be determined as

$$Q_{jc} = wBH - 2sH$$

where the average shearing resistance of the ground located above the roof is s. Therefore, the unit load is

$$p_{jc} = Q_{jc}/B$$

The working face must be kept more than B distance from the last support if the stiffness of the clay located ahead of the working face allows the clay above the roof level to mobilize its shearing resistance. This is required to mobilize its shearing resistance. However, although the load on the roof support will be greater than the load calculated by the previous equation, it will be less than $wH - p_c$.

The width of the block of clay will be greater than $B + 2H_t$ if the clay on both sides of the tunnel is appreciably softer than the clay above the roof. This softer clay will not have the strength to support to block, and thus the clay adjoining the walls will yield under the load of the arch, causing the arch to broaden and the ground arch to deepen.

Although all swelling clays are hard or stiff, many varieties of hard or stiff clays exist that do not visibly swell. Besides the consistency of the clay, many factors affect the degree of swelling. Tunneling through intact swelling clays is relatively easy. The clay does not start to swell for about a week. During this period, temporary supports are needed for generally smaller loads than similar supports in nonswelling clays of the same consistency. However, expanding clay can damage or destroy the supports if the final lining is delayed.

Jointed swelling clay will probably swell soon after the clay is exposed. In this type of ground it is important that support and backpacking be installed as soon as possible. Even with the quick installation of the support, the clay may expand and open, causing the load on the tunnel supports to increase to the full overburden load.

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4 Soft Ground Tunneling

Historically, one of the most common tools for mining in soil without a shield is the liner plate, a small rectangular steel plate that can be flanged on two or four sides. It is curved to the radius of the tunnel in the longitudinal direction. It is strengthened by deforming the plate with corrugations, providing a deeper section and thus increasing its section modulus. Some will have holes for grouting. They provide relatively lightweight support that supplies resistance to ground loading and, because of their relative weight and ease of handling, offer savings of time, less overexcavation, and less manpower. They have predetermined strength, are fire resistant, and can be obtained galvanized or with a corrosion-preventing coating. They provide continuous support and a maximum of support per given length with a minimum weight of steel. Figure 4.1 is a photograph of a corrugated and smooth liner plate.

The plate dimensions can vary based on utility, that is, primarily by ease of handling, weight and size, and size of the cavity dug to install the plate. Liner plates are generally the same length, 957 mm (37.7 in.), and although the width may vary, 406 mm (16 in.) is the norm. Note that the length is 304 mm (12 in.) $\times$ π (3.14159). Thus, in English units a tunnel 8 ft in diameter uses eight plates.

Because of its deflection capacity, it can use the surrounding ground to assist with support. That is, as the sides bulge out due to the load at the crown, the liner plate ring pushes against the surrounding ground, improving the vertical load capacity. Because of the slight deflection, the load is distributed around the liner plate, keeping the ring in compression and increasing liner plate strength. Since the ability to carry the load is affected by transferring the load to the surrounding ground, it is important that there is little over excavation and that it be backfilled with pea gravel or soil.

Liner plates are easy to transport and store. Because of their relatively light weight, each liner plate can easily be handled by a miner.

UNITARY EXCAVATION

Unitary excavation using liner plates begins at the crown in the center (12 o’clock) position. A horizontal opening, the cavity, is dug at the crown just large enough to install and bolt the plate in place. The letters a, b, c, d represent the upper surface of the cavity, whereas the ends are represented by

Figure 4.1 Liner Plate

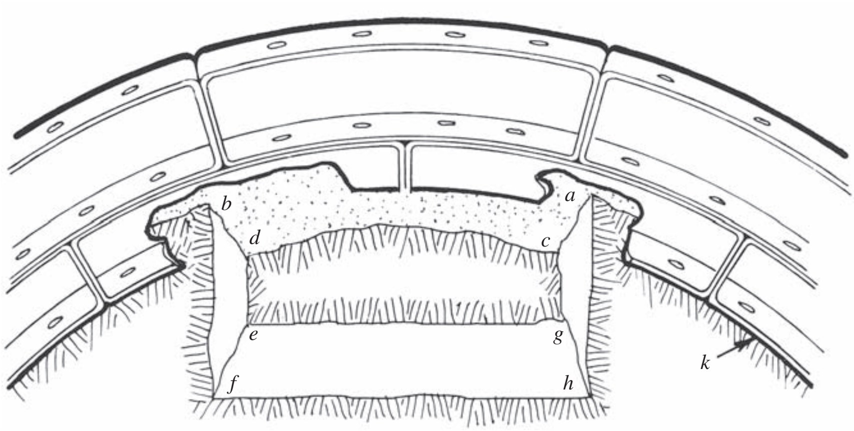

Figure 4.2 Unitary Excavation Initial Pocket at Crown (Proctor and White, 1974)

b, d, e, f and a, c, g, h. The plate patterns are offset so that the longitudinal seam is broken up (see Figure 4.2).

A support, usually a wood block, is placed in the cavity with one end wedged at the bottom of the cavity and the top against the plate to support it. See Figure 4.3. Hay is stuffed between the plate and excavated ground to prevent the ground from raveling, reducing stand-up time.

The process is continued on both the left and right of the first plate with the second and third plates bolted to the other plates; the plates are supported with blocking and hay is stuffed behind them.

Once three liner plates are installed, the face will be brought down. The center block can be removed and the face is cut to the spring line. The block is replaced by a trench jack. The process is repeated twice more with the other blocks being removed, the face cut, and a trench jack replacing the block. Again, the space between the liner plate and tunnel surface must be filled with hay. Then additional mining is done, placing plates at the spring line. At the spring line, it is important that the size of the excavation be accurate, because

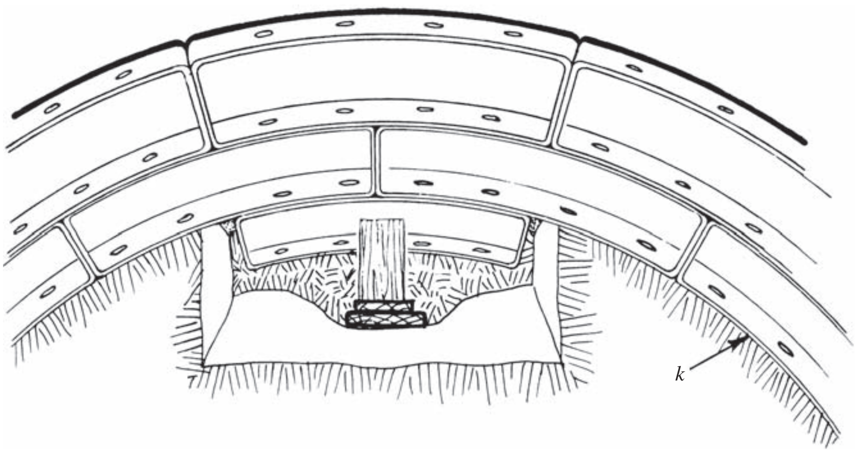

Figure 4.3 Unitary Excavation Liner Plate installed to Support Roof of Pocket (Proctor and White, 1974)

of the shape of the liner plate. It is important for the liner plate to be in contact with the ground to maintain its shape. If not, the plate will budge. Any over excavation will have to be filled with soil and packed. To prevent the plate from budging, it is recommended that the mining at the spring line be a little tight, to avoid having to pack soil in the annulus.

This will allow the plate to be jacked into the wall to prevent any bulging. The jacks used to support the second and third plates are now used to support the lowest liner plates (see Figure 4.4) and the plates are blocked.

Ten minutes of stand-up time is desirable, but it can be employed in as little as 5 min. If areas under buildings or any other structures are prone to settlement, it is imperative that the annular area between the liner plate and ground be grouted. The frequency of grouting is a function of the length mined. However, grouting should occur at least daily.

FULL-ARCH MINING

In full-arch mining, the area above the spring line is mined a distance of one or more courses of ground support prior to erecting any support. This method can be used when there is sufficient stand-up time. For a tunnel with a liner plate that has local ground arching for a span of 400 mm (16 in.), there must be a least 1 hr of stand-up time. If there is not enough stand-up time for the full-arch method with liner plate, the unitary method can be used.

When stand-up time is long enough, ribs and lagging can be used. Ribs are steel beams installed with lagging covering the ground between them. Lagging can be steel plates, which are generally made from light steel of varying

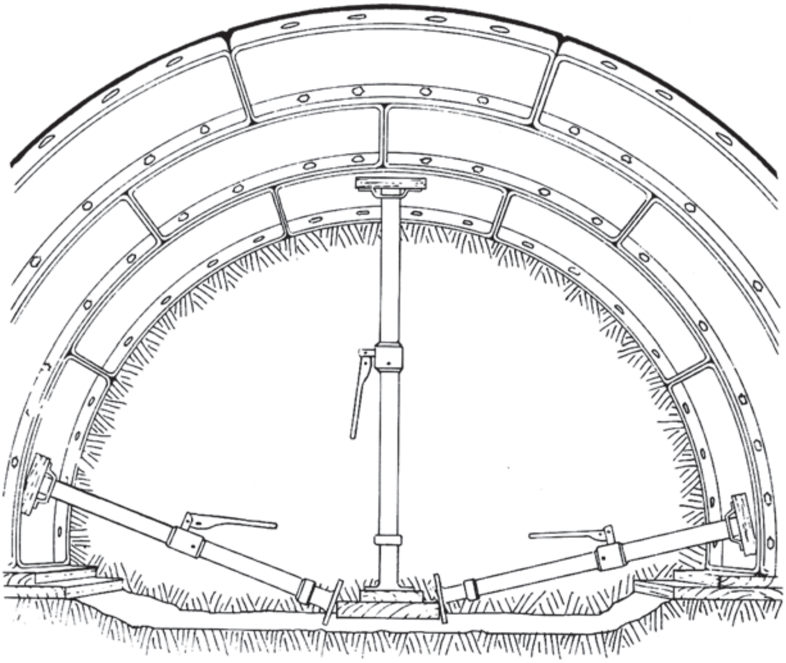

Figure 4.4 Unitary Excavation Mining to Spring Line (Proctor and White, 1974)

shapes of corrugations, wood lagging (Table 4.1 provides recommended lagging thicknesses), liner plates, and channel lagging (see Figure 4.5).

The type and thickness of the lagging are generally determined by the depth of the flange and the rib spacing and load. The tunnel is advanced a predetermined length based on the stand-up time of the soil. The length of advance is the spacing between ribs.

When mining, after the face is advanced a distance equal to the support course length, the steel rib is erected and held stable with tie rods or spacers. The lagging is then installed starting at the spring line and up to the crown equally from both sides while the annular space between the lagging and the ground is filled by ramming gravel or tunnel spoil.

At times, such as running ground, the face of the drive will lack sufficient cohesion to remain in place while the ribs and lagging are being installed. This will require breasting. Breasting is the placement of boards against the face to stabilize the face and has an ancillary effect of helping to maintain face moisture content. Figure 4.6 illustrates the advancement of breast boards in running ground.

Extending breast boards across the tunnel can usually be used in tunnels about 3 m (10 ft) wide. A diagonal brace at each end is used to individually brace each board to the completed lining.

Table 4.1 Recommended Wood Lagging Thickness

Soil Description	Unified Classification	Depth	Recommended Thickness, mm (in.), of Lagging (Rough Cut) for Spans, m (feet) of:					
Competent Soils			1.5 (5)	1.8 (6)	2.1 (7)	2.4 (8)	2.7 (9)	3.0 (10)
Silts or fine sands or Silt above the water table	ML SM-ML							
Sands and gravels (medium dense to dense	GW, GP, GM, GC, SW, SP, SM	0–7.6 (0–25)	50 (2)	75 (3)	75 (3)	75 (3)	100 (4)	100 (4)
Clays (stiff to very stiff)	CL, CH	7.6–18 (25–60)	75 (3)	75 (3)	75 (3)	100 (4)	100 (4)	125 (5)
Clays, medium consistency and γ(H/S) $\mu < 5$	CL, CH							
Difficult Soils								
Sands and silty sands (loose)	SW, SP, SM							
Clayey sands (medium dense to dense) below water table	SC	0–7.6 (0–25)	75 (3)	75 (3)	75 (3)	100 (4)	100 (4)	125 (5)
Clays heavily over-consolidated fissured	CL, CH	7.6–18 (25–60)	75 (3)	75 (3)	100 (4)	100 (4)	125 (5)	125 (5)
Cohesionless silt or fine sand and silt below the water table	ML; SM-ML							
Potentially Dangerous Soils								
Soft clays γ(H/S) $\mu > 5$	Cl, CH	0–4.6 (0–15)	75 (3)	75 (3)	100 (4)	125 (5)	—	—
Slightly plastic silts below the water table	ML	4.6 (7.6)	75 (3)	100 (4)	125 (5)	150 (6)	—	—
Clayey sands (loose), below the water table	SC	7.6–10.6 (25–35)	100 (4)	125 (5)	150 (6)	—	—	—

Source: From "Lateral support systems and underpinning," FHWA-RD-Report 75-128.
Note: The use of lagging in potentially dangerous soils is questionable.

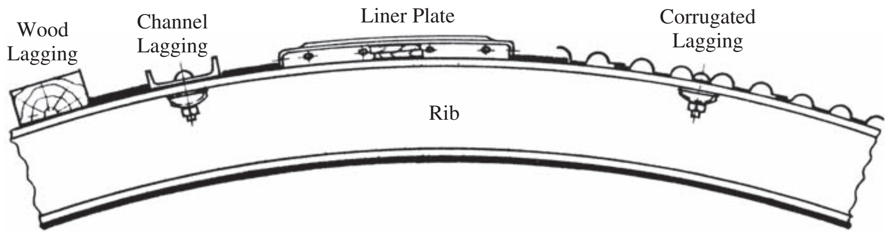

Figure 4.5 Types of Lagging (Proctor and White, 1974)

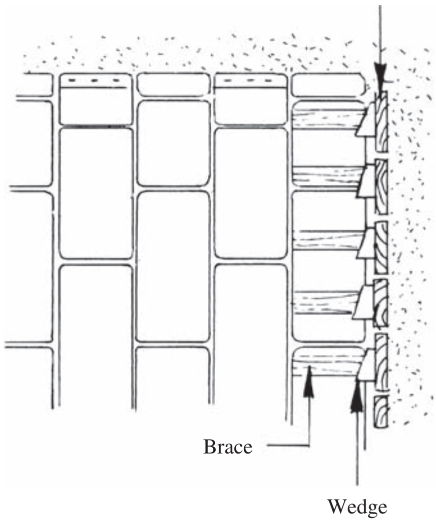

Figure 4.6 Bracing of Breast Boards in Small Tunnels (Proctor and White, 1974)

In running ground the openings in the breast board are protected by a board above, preventing the inflow of the ground. The breast boards are advanced from the top down.

When driving with a liner plate where soil has low stand-up time, spiles are used. Spiles are boards about 152–200 mm (6–8 in.) wide, 20 mm (3/4 in.) thick, and 600–710 mm (24–28 in.) long. The forward ends are sharpened to a chisel-like point. The spiles allow mining in ground with very poor stand-up time. The gap between the spiles is small, preventing the ground from flowing into the tunnel. However, if the ground still flows, the space between the spiles may be reduced.

The spiling cycle consists of five steps. In step 1 the top breast board is removed. A 13-cm (6-in.) miner's spade is driven into the face at an upward angle. The miner pulls down the spade, forcing out the sand. The spade is again driven, deepening the hole. With the spade in place, another miner inserts a spile above the spade while the spade is wiggled up and down, pushing it forward to

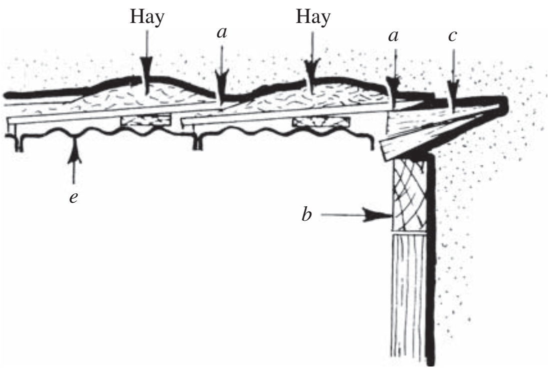

Figure 4.7 Spiling Methodwith the heading from the next advance (Proctor and White, 1977)

loosen more sand. Hay is then stuffed above the spile, preventing the ground from running. The spile is driven forward until it passes the liner plate's front flange. Figure 4.7 illustrate the spiling method. The letter *a* denotes the spiles, which in this case are about 150–200 mm (6–8 in.) wide and 710 mm (28 in.) long, with the forward end sharpened like a chisel, or they can be thin metal plates of approximately the same dimensions. The forward, or leading, edges are supported by wedges, *c*, driven into the ground at the crown level. The breasting boards, *b*, are used to hold, or confine, the face. The liner plate is used to support the spiles with hay stuffed behind the support.

The traditional method for advancing in flowing ground is forepoling. Poles (boards) are driven ahead of the advance on an upward slope from the tunnel axis. These poles, although more commonly driven in the upper tunnel such as the crown, can be driven from the entire perimeter depending on conditions. The length of forepoling boards is more than twice the spacing of the ribs. The forepoling board is driven above the rib closest to the face with the tail end of the board being under the rib second closest to the face. This allows the forepoling boards to act as a cantilever. The cantilever beam formed by the forepoling board is stronger than the beam formed by spiles and thus better suited in running ground. The face is advanced moving the breast boards until the next course, or set, of support is installed. Once the face is advanced enough for the next rib, the ribs and lagging are erected and the next length of poles is installed. As illustrated in Figure 4.8, the process continues until the tunnel is advanced to better, nonflowing, ground or the drive is completed. The length of the cantilever poles prevents the rear of the pole from rising and therefore the front of the board will not drop provided the board does not break.

Note in Figure 4.8 that the forepoling board extends from the underside of the second rib to the topside of the first rib and into the ground above the face. The design line indicated the boundary of the final lining. As illustrated, the boards can be cut prior to installing the final lining.

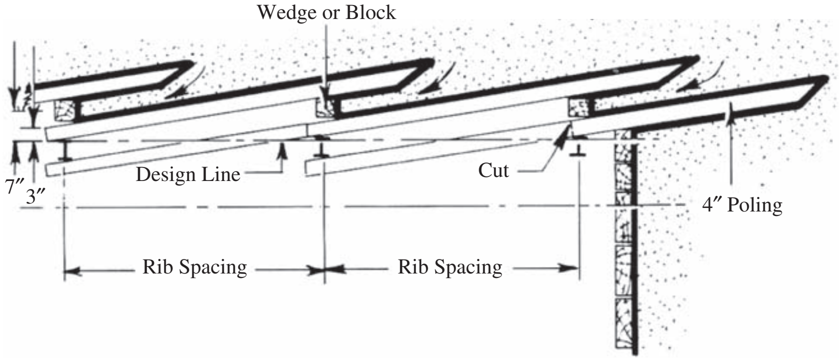

Figure 4.8 Forepoling Method (Proctor and White, 1974)

SHIELDS

When a tunnel is expected to be long enough to justify the capital costs or time needed for mobilization, a shield is the immediate choice for driving it in the soil. Marc Brunel invented the free air shield in the mid-1800s. In 1869, James Henry Greathead, an English engineer, built a tunnel under the Thames River using a circular shield he developed. Most shields derive their basic design from his shield. To support the tunnel behind the shield, cast iron segments were used. Later, in 1874, Greathead developed a shield with a fluid-supported face. See Figure 4.9.

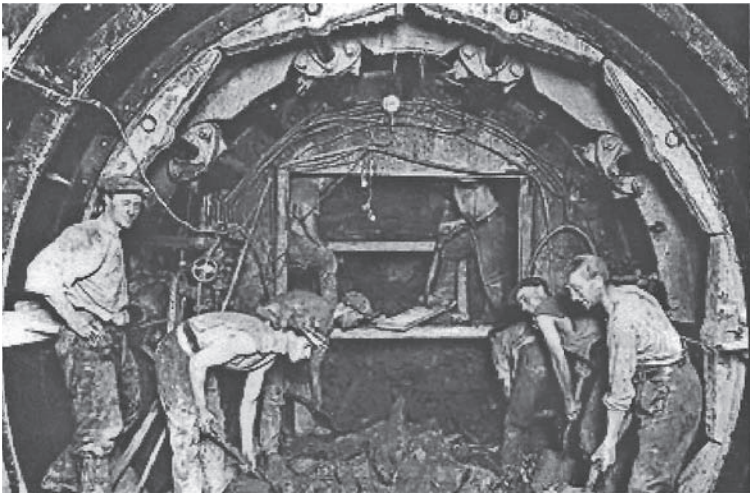

Figure 4.9 Workers Using the Greathead Shield to Construct a segment of the London Underground, London, England (Courtesy of The London Transport Museum)

Shields are suited for weak, noncohesive soils such as loose gravel, sand, silt, running ground, and weak plastic soils, that is, soft plastic clays in which the water content is critical and when worked will liquefy easily.

Shields are used for soils that are below bodies of water and subjected to water pressure. The same is true if under the water table unless the soil is dewatered.

There are three sections to a shield: the cutting edge located at the front of the machine, the middle of the shove ram section, and the tail shield.

The shield is made of rolled steel plate with the front end being the thickest at 80–120 mm (3–5 in.). When hand mining platforms can provide support; there is no shelving with the cutting wheel. The middle of the shield is supported by rings and the ring support frames for the erector are toward the tail shield.

When tunneling in soft ground, the material can be removed with hand tools or by mechanical means. Tools can vary from a shovel to a small backhoe. The material may be a combination of soft clays, sand, granular soils, glacial till, alluvial soils, cobbles, and boulders.

Mining with the open shield can be done with the face open and divided by bracing, which also acts to support work decks or partitions To mine with an open shield, the soil must be firm enough to stand or be held using breast boards. The face is divided into pockets much like one would see for sorting mail. In larger shields, it can function as a work deck for the miners in each partition. The principal function of the divided face is to support the face of the tunnel. The depth of the pockets should be such that the angle of repose of the soil is greater than the face opening; thus, the horizontal section of the pockets will prevent the soil from running into the tunnel.

The closed shield has the face closed by a steel bulkhead and is used in fine running sand, silt, or soft clay. There are ports in the shield face to permit excavating through the shield. Because of the additional load on the closed shield by the face, the closed shield requires more bracing. Breasting jacks are mounted on the upper face and control the breast boards, holding them against the face. While still maintaining pressure on the face, they are retracted during the shoving.

Externally, the shield must withstand loading from the soil and water above it. With the operating load, the shield is stressed by the soil excavation, the thrust caused by the push jacks, the weight of the components, and the erection of the lining segments. The body of the shield is a steel cylindrical plate about 2.5–3 m (8–10 ft) long. The open shield can be used if the material is competent enough for the face to stand without support. The shield is supported by a minimum of two circular ribs and longitudinal and vertical beams.

At the front of the shield, the size of the cutting edge is dependent on the material to be excavated and the tools used. If the pushed shield and cutting edge cannot remove the material, a digging arm may be placed in the shield to assist with the excavation. Larger shields, say 6–8 m (20–26 ft), can require plate thicknesses of 60–80 mm (2.4–3.1 in.) and the tail thickness can be 40–50 mm (1.5–2 in.).

Ahead of the cutting edge is the hood. It is the forward extension providing additional protection in loose or running ground. The hood must be strong enough to resist damage from striking boulders.

The operating principle is to propel the shield forward using shoving jacks that push off previously installed liner or ground support. The liner may be iron segments, steel segments, concrete segments, or ribs and lagging. The shield is subjected to loads both externally and internally. The greatest load to which the shield will be subject is from the jacks. The number and location of the jacks are determined by the size of the shield. The jacks generally have a capacity of 63,000 kg/m^2 (6.5 tons/ft^2) of face. The jacks push against the installed primary liner to advance the shield. Jacks are equally spaced around the periphery; however, sometimes there are more put in the bottom to counteract the tendency of the shield to dip. Jack strokes can range from 45 to 154 cm (18 to 60 in.). Generally it is a good idea to have more stroke available than needed for the shove, that is, 15 cm (6 in.).

The tail of the shield extending behind the body should be at least one and a half times the length of the lining segment. This allows the advance to be a full stroke and remain under the protection of the tail shield while installing segments or to replace a jack if required. Some prefer that the tail shield be the length of two lining segments so that if it needs to be removed or replaced it can be done under cover of the shield. Often, the weakness of a longer tail shield negates the practicality of the longer shield. The rule of thumb for the tail shield thickness is 25 mm (1 in.) for every 3 m (10 ft) of diameter. The tail houses the jacks and is where the liner is erected.

To avoid bending or binding in the tail shield area when negotiating curves, the shield should have a conical design, decreasing the diameter from the front to the tail shield. An articulating joint between the middle section and the tail shield, much like articulating steering on an earthmover, permits the shield to negotiate curves easier by reducing the turning radius of the shield, and the tunnel should be mined larger than the shield. In addition to assisting with negotiating curves, the gap created by the overcut reduces skin friction.

There are two types of loads on a shield: external and internal. The external loads, that is, the earth and water pressures acting vertically on the shield surface, include the water head and plastic ground or fluid supporting the gap, or annulus, between the shield skin and the surrounding ground. Also, an asymmetrical position of the shield in the excavation profile can load the shield.

The internal loads can be referred to as *operational loads.* Operational loads are both horizontal and vertical. The activities classified as operational loads include operation of the push cylinders, ring building, which includes operation of the erector, the weight of the various components, and the resistance of the face as the machine is advanced. The maximum load on the shield is the thrust force distribution generated by the cylinders pushing against the liner. Included in this load is, the friction caused by the shield skin, the cutting edge, and excavation tools.

When being pushed through the ground, shields tend to deviate from the planned course. In addition, the possible misalignment at the tail shield may cause difficulty in erecting the ring. The shield maintains the course by manipulating the jacking pressures and utilizing the annulus gap; in the case of an articulated shield, the direction of the front shield will control the direction. The most common deviation from the course is the tendency for the shield to dip. This is mitigated by applying thrust from the bottom jacks. The shield's tendency to roll is controlled by the use of retractable fins that are pushed into the wall to prevent it from rolling.

As the shield advances, while cutting the annulus, it is necessary to fill the void between the shield and the ground. This helps mitigate subsidence, prevents the shield from being put out of round, the shield becomes oval shaped or being warped due to uneven loading of the skin, and can cause the shield to be stuck due to the ground relaxing or squeezing the shield.

There are two common ways to fill the void: blowing in pea gravel or pumping in grout, with grout being the most common. The grout is a cement mortar that is pumped into the annulus. Being a fluid it is can flow into void areas. However, grouting requires skilled operators and the mix design must be such that the grout will not set for several hours. If it sets too quickly or the shield does not move, the hardened grout will trap the machine. Also, the grout can be very messy and run into the shield through the liner and the tail shield void.

Pea gravel is blown in behind the shield using compressed air. Pea gravel is often preferred because it is compact and uncompressible with voids allowing the air to escape. Pea gravel is also less messy as it does not move between the shield and liner. Also, with pea gravel standing time is not an issue.

When the ground is such that it cannot be maintained and flows into the tunnel, excavating without sealing the shield is not possible. The sealed shield needs a mechanical method to excavate the tunnel without having to flow into the shield. This called for certain innovations in underground mining methods. These innovations resulted in the slurry pressure balance machine (SPBM) and the earth pressure balance machine (EPBM). In addition, shielded machines were adapted for hard rock tunneling. Table 4.2 illustrates the various types of shielded machines and the type of geologic condition for which they are best suited.

Of course, there can be crossover of the tunneling machines. For example, jacking below the water table will have problems that may be better handled with a microtunneling machine. Also, many soils are a mixture of fine and coarse grained. In this case, the cost saving of using an EPB machine compared to a slurry machine may make it the preferred one to use.

Slurry Machines

To deal with the problems associated with mining in soil under the water table and stringent surface settlement requirements, new methods had to be developed. Tunnels can be efficiently excavated utilizing soft ground techniques, such as slurry pressure balance machines and earth pressure balance machines, SPBMs and EPBMs. However, shorter tunnels that cannot reasonably amortize

Table 4.2 Optimum TBM for Ground Conditions Encountered

TBM Geologic Selection		
Type of Machine	Typically Approximate Diameters	Optimum Ground
Pipe jacking machines	Up to 3–4 m (10–13 ft)	Any ground
Small-bore unit, microtunnel	Up to 2 m (6.5 ft)	Any ground
Shielded TBM	2–14 m (6.5–44 ft)	Soft ground above the water table
Mixed-face TBM, earth pressure balance (EPB)	2–14 m (6.5–44 ft)	Mixed ground above the water table
Slurry TBMs	2–14 m (6.5–44 ft)	Coarse-grained soft ground below the water table
EPB TBMs	2–14 m (6.5–44 ft)	Fine-grained soft ground below the water table
Hard-rock TBMs	2–14 m (6.5–44 ft)	Hard rock
Reamer TBMs	Various	Hard rock
Multihead TBMs	Various	Various

Source: Adapted from Kessler and Moore.

the capital cost of the equipment or the time required to mobilize the machine are still being excavated using basic tunneling methods.

The basic methods for stabilizing the tunnel face include natural and mechanical face support, compressed air support, slurry face support, and earth pressure support.

Compressed air machine shields are used below the water table or free bodies of water. It is good in porous ground for water control but should not be used to control ground. Compressed air can be used with hand mining in shields mechanical, or part or full-faced machines. The air pressure is used to keep the water from entering the tunnel. With the use of bentonite the risk of blowouts is reduced because the bentonite infiltrates the face forming a cake on it. Compressed air machines are assisted by natural or mechanical support methods.

Slurry machines are generally used in noncohesive soils, such as sand, gravel, or cobbles. In a mixed-face condition with clay, the clay can become sticky and create blockages in the line or separator. Figure 4.10 shows the slurry range of applications for different particle size distributions.

The slurry shield system performs two basic functions: it supports the face by applying pressure to oppose the hydrostatic and soil forces and it provides the transportation system for removing the excavated material from the heading. To do this, an elaborate control system is required to maintain the flow and pressure within the operating parameters for success.

Similar to slurry walls, the slurry is usually a mixture of bentonite clay and water and, in some cases, a polymer. In sandy conditions, as it does with

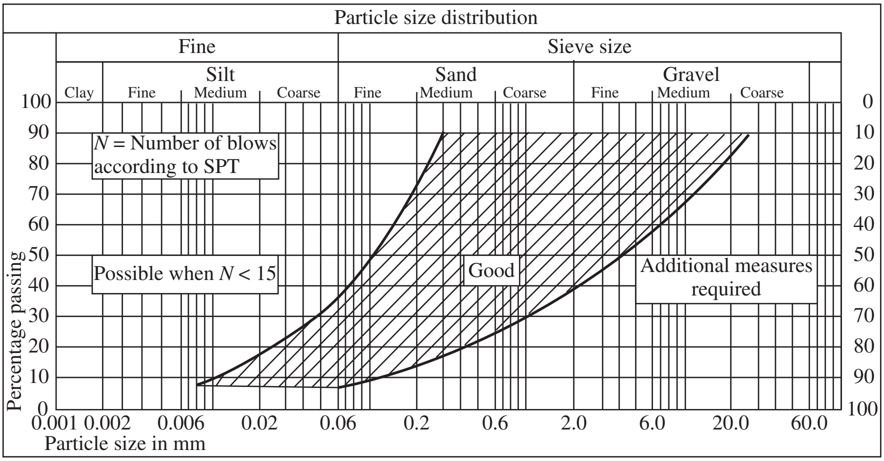

Figure 4.10 Slurry Shield Range of Application with Regard to the Type of Ground (Maidl et al., 1996)

compressed air, the bentonite penetrates the face and produces a membrane which creates a film effect creating structural resistance to assist in preventing collapse of the face, much as it does with slurry walls.

When mining with slurry, the face is continually renewed. As material is removed, the slurry acts fast enough to continue supporting the face. The specific gravity of fresh slurry is between 1.08 and 1.0. At times, because the face is not permeable, the use of bentonite may not be necessary in soils that are predominately clay. The clay will naturally disperse within the water.

A slurry machine is a shield with a sealed head that is fed slurry which holds the face while it is cut by cutterhead blades or picks. The excavated material mixes with the slurry and is pumped from the head to a separator on the surface or on the trailing gear. The separator removes the excavated material and after treatment the slurry is pumped back to the face. The cutterhead contains a compressed air reservoir which monitors the pressure in the head, thus providing the information for determining slurry pressures. There is a balance between the flow of slurry into the head and that being pumped out. The inflow is controlled by pressure sensors. The outflow must have sufficient velocity to keep the particles of excavated material in suspension until it reaches the separator.

Figure 4.11 is a schematic of a slurry machine. Note the bentonite pipes feeding the head. As with other types of TBMs, the machine has the capability of installing segments.

The determination of slurry pressures and density depends on lateral soil pressure and the height of the water table above the tunnel. Soil pressure is difficult to calculate because it is dependent on the types of strata, the overburden height, and the precise soil properties which are dependent on the water content.

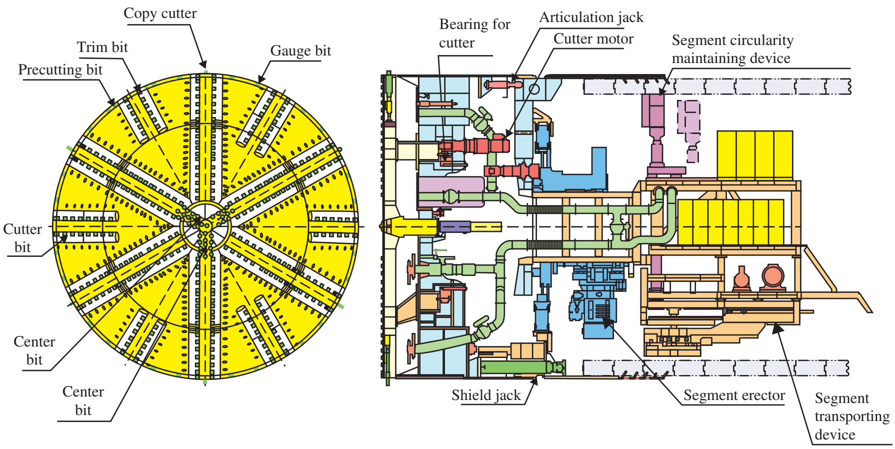

Figure 4.11 Schematic Representation of a Slurry-Type Shield Machine (Courtesy of Hitachi Zosen)

If the slurry pressure is too low, collapse can occur, whereas pressure that is too high can lead to excessive slurry loss into the ground and heave. Calculation of the slurry pressure the machine will encounter will result in as many different answers as there are people doing the calculations. If the pressure determined is incorrect, the pressure can be determined by trial and error. It is important to meter the output to avoid a loss of ground.

The cutterhead designs for slurry machines vary. Slotted discs can be used that consist of narrow slots of about 200 mm (8 in.). The disc provides additional mechanical face support yet is smooth, reducing the required torque. The narrow slots prevent the entry of large boulders, but the large stones that can enter are broken up by a crusher located at the entrance to the muck discharge system. A spoke-type crusher capable of crushing boulders in excess of 1 m^3 (1.3 yd^3) in located in the cutterhead chamber.

Generally drag-type cutter bits are used. However, if rock is anticipated, disc cutters are added. Usually there is an air lock system incorporated behind the shield. Therefore, when disc cutters need to be replaced, the fluid inside the cutterhead is displaced by compressed air, maintaining the face support and preventing the ingress while permitting the changing out of the disc.

The muck removal is a fluid system. The mixture of bentonite suspension and ground has to be economically removed from the excavated material, and what is not recycled has to be disposed of. Separating the solid and liquid components outside the tunnel can permit much of the conditioned bentonite suspension to be returned to the slurry system. Muck exits the cutterhead chamber and enters the discharge pipe where the flow is boosted by a large centrifugal pump located near the cutterhead support. The flow volume is determined by the rate of excavation and pipe size. The discharge density is generally about 1.25 and the ratio of slurry flow to muck is about 10 to 1. The discharge pipe is usually about 150 mm (6 in), approximately 3–4 times the maximum particle size.

The density should be as high as possible to reduce the volume of slurry required. However, the resulting viscosity does have a practical limit. In practice, a density of 1.4 has been found to be optimum, whereby the slurry is at a minimum volume but is still pumpable. The proportion by weight to obtain this yield is 16:40:75 for bentonite, ordinary clay, and water, respectively. Since bentonite is the most expensive ingredient, its proportion is minimized as much as possible yet still maintaining the mixture's expansive and lubricating properties.

The flowchart in Figure 4.12 illustrates the steps in the processing of the slurry mix. The muck slurry is transported from the machine to the separator plant. In short tunnels the muck is transported to the separator plant on the surface. As the tunnel advances, the discharge line is telescopically extended. In very long tunnels and larger tunnels it may be more practical to locate the separator plant on the trailing gear. If there are a great deal of fines to be removed, a plant with very large filter presses has to be located at the portal. After primary separation the slurry requires only a small discharge pipe.

Separator plants vary in design. Most use vibratory screens, hydrocyclones, and filter presses to process the slurry. Separator plants require a lot of space, which may result in the alternate selection of an EPB machine for tunnel excavation. The vibratory screen can remove material down to approximately 5 mm. This is usually done using a multistage process. Hydrocyclones can separate material of about 50–100 μm. If this is still too many fines, the slurry must then be processed through filter presses. Processing a large amount of fine requires presses with a large capacity.

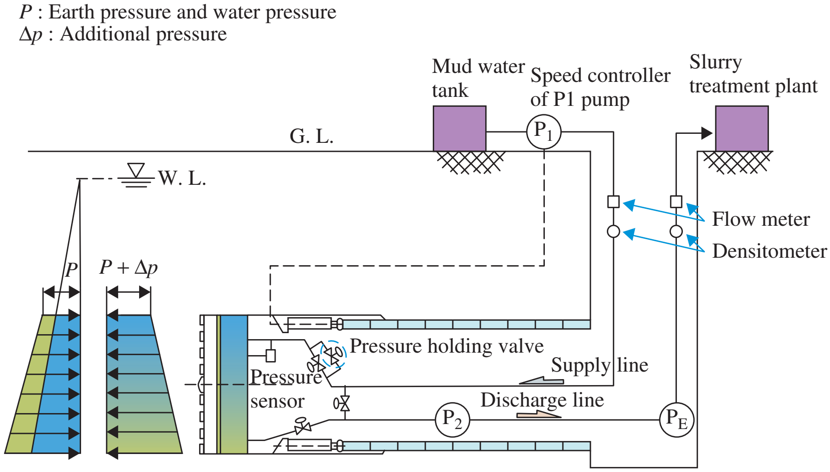

Figure 4.12 Flowchart of Slurry System (Courtesy of Hitachi Zosen)

The solids that are extracted are carried by conveyors to a hopper for loading trucks or other haulage equipment. The remaining slurry is recirculated through the system after the density has been adjusted.

Shielded machines are generally used with precast concrete segments. To prevent the slurry, water, or grout from entering through the gap between the tail shield and liner, a mechanism is used to seal the space. The most common seals consist of rows of wire brushes attached to the shield (see Figure 4.13).

Between the rows of wire brushes there are gaps, referred to as intermediate chambers, filled with grease under pressure to act as a sealant. The pressure of the grease is generally maintained at a higher pressure than the grouting pressure.

There may be up to five rows of brushes each of which has a capacity of about 2.5–3 bars (35–45 psi) per row. Thus three rows would have a capacity of 7.5–9 bars (100–130 psi). The brushes are about 25 cm (10 in.) deep and are spaced about 30–35 cm (12–14 in.) apart longitudinally. Figure 4.14 is a schematic of a wire brush seal. Note in the figure that there are three lines.

Figure 4.13 Wire Brushes for Tail Seal

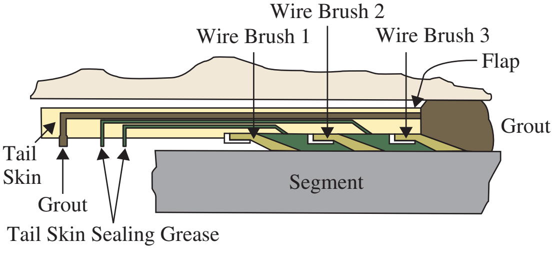

Figure 4.14 Wire Brush Seal Illustration (Loganathan, 2011)

Two have the grease and maintain the pressure by forcing grease into the intermediate chambers. The other line is the grout tube that passes through the seal.

In many, but not all, slurry machines the face support pressure in the excavation chamber is controlled by an air cushion. This system uses the change in air pressure to regulate the slurry suspension pressure in the feed line. The excavation chamber behind the cutting wheel is separated by a wall in the front of the bulkhead. Therefore, while the face side is totally filled with the slurry, the level of slurry behind the submerged wall is only a little above the machine axis.

A grid similar to a grizzly is located in front of the intake to keep the suction line from becoming congested or plugged by large rocks or hard clumps of soil, thus preventing problems with the slurry pump operation.

Because the compressed air regulating system accurately controls the compressed air cushion at the exact set point pressure, fluctuations in the slurry system can be more efficiently compensated for by changing or maintaining air pressure.

The compressed air cushion is controlled by the compressed air regulating system, maintaining it at the exact required pressure. This allows for the efficient compensation of fluctuations in the bentonite slurry system.

Earth Pressure Balance Machine

The EPBM, developed by Japan in the 1960s, has evolved into the primary method for mining in soil usually below the water table. The technique enables the construction of near-surface tunnels in poor ground conditions, such as running or flowing ground, with minimal surface effects and permits tunneling in urban areas under which tunnels could not be previously mined. The main types of tunnel material for which EPBMs are best suited are soft sensitive clays, silts and fine sands silty sands, and clayey sands. In these materials remolding in the machine head will produce a mass of soil of soft to pulpy consistency of low permeability and at most will require the addition of a small quantity of water. When mining in stiff clays the pressure balance can help to reduce ground movements. EPBMs are not well suited for highly plastic clays where large quantities of water are needed to change the water content sufficiently and the clay is very impermeable. It is difficult to achieve a uniform consistency at the right shear strength.

EPBMs work on the principle of utilizing the excavated material to balance the external ground pressure. The face is supported by remolded soil in the machine head. As the cutterhead rotates and the shield advances, the excavated material (muck) may be mixed with a foam material that contains water and compressed air, which alters its viscosity and changes it to a flowable material. The altered muck is then maintained under a controlled pressure in the cutterhead to balance the pressure exerted by the ground in the face. The muck is removed from the cutterhead via a screw conveyor. The speed of the conveyor and the guillotine gate can be adjusted to regulate the muck discharge rate. This maintains the stability as the machine advances.

Because of the weight of the muck in the cutterhead, the friction developed, and the thrust against the face, EPBMs require about to 2.5 times the amount of torque of a slurry machine. EPBMs generally operate in noncohesive, water-bearing ground where the loss of face stability exists. The use of soil to hold the face eliminates the need for secondary support such as compressed air suspension or breast plates. They are good for loose sand and/or gravel which makes them superior to compressed air machines. Figure 4.15 illustrates the range of grain size soil distribution relative to machine selection.

The thrust force is transferred to the pressure bulkhead against the face to prevent uncontrolled ground from entering the excavation chamber. Balance is reached when the earth slurry in the excavation chamber is not further consolidated by the earth and water pressure. If the support pressure of the earth slurry is increased above equilibrium, the earth slurry in the excavation chamber as well as the face will be further consolidated and may cause ground heave in front of the shield. With a reduction of earth pressure the ground can penetrate into the earth slurry and cause surface settlement. Removal from the muck/slurry excavation chamber must be done within a controlled transport system to avoid a reduction of earth pressure. This is done with the use of a screw conveyor. Figure 4.16 is a schematic of an EBPM.

To serve as a support medium the excavated material should have the following characteristics: There should be good plastic deformation and should have pulpy to soft consistency, low inner friction, and low permeability. Permeability is dependent on the fines content of the soil (less than 200 mesh) or the silt and clay content.

Since it is unlikely that any soil will possess these properties, the material must be altered with additives. To do this, the material is excavated with the cutterhead, but it does not fall into the excavation chamber. It is pressed through openings in the cutterhead wheel into the excavation chamber where it is mixed with foam and is kneaded in the excavation chamber, which alters its viscosity and turns it into a flowable plastic material.

Foam consists of water enriched with a surfactant, which lowers the surface tension of a liquid (tenside) as well as a stabilizing polymer. In noncohesive permeable soil, the muck can be conditioned by foam. The foam added to the excavated ground becomes part of the grain structure. The air bubbles on the grain structure foam with compressed air lower the density of the earth slurry and reduce grain friction. The compressed air content of foam is about 90%.

Through good plastic deformation resulting in a pulpy to soft consistency the support pressure can be applied regularly on to the tunnel face and a frequent flow of materials to the screw conveyor can be guaranteed.

Blockage of the cutting wheel and clogging of the excavation chamber in areas with a low pressure head can be avoided, allowing a reduction in torque in the screw conveyor and cutterhead. Too soft a consistency makes it difficult to facilitate transport by conveyor belts and muck skips unless using a solids

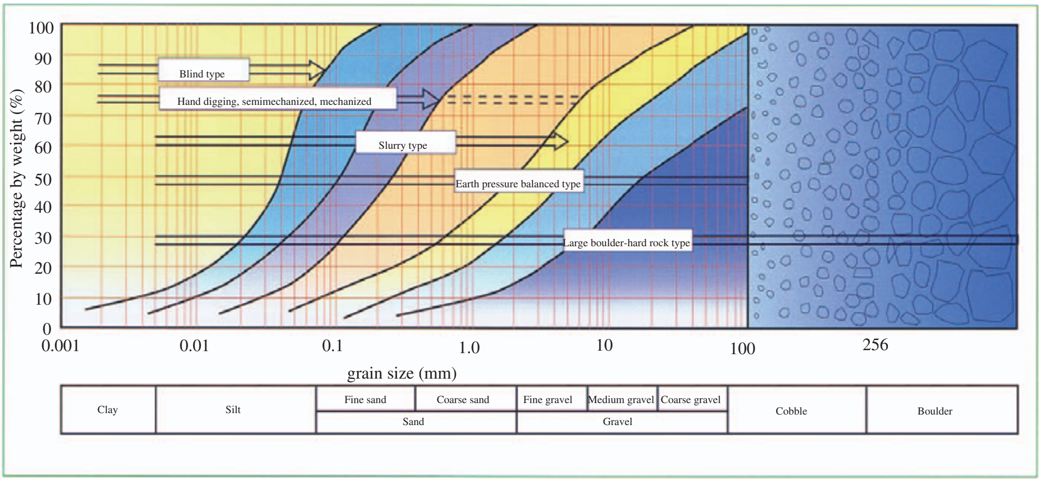

Figure 4.15 EPBM Shield Range of Applications with Regard to Ground Particle Size Distribution (Courtesy of Hitachi Zosen)

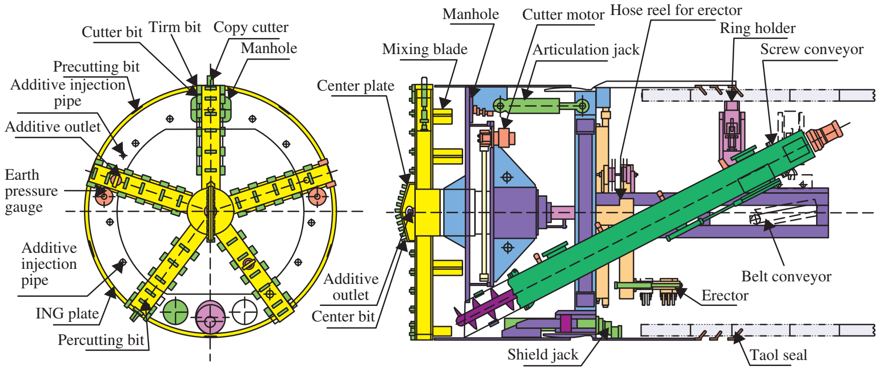

Figure 4.16 Schematic of EPBM (Courtesy of Hitachi Zosen)

handling pump. In soils that are mainly clean gravels or clean sands, the fine content will necessitate adding bentonite to reduce the permeability of the face and lubricate the material. Experience suggests that less than 15% of fine material necessitates the injection of bentonite or similar additive into the face in the form of a water-based slurry. Good flowability of muck is necessary to provide accurate control of screw conveyor discharge. The percentage of actual fines in the in situ soil that are less than 200 mesh is used to determine the quantity of additive to be injected per unit volume. Figure 4.17 illustrates the relationship for bentonite/clay additive relative to the density.

A specially developed polymer can be used as an alternative to clay slurry. It is a gel product and the quantity of gel per unit volume of slurry required is considerably less. Approximately 1% of the clay is required Therefore the logistics of mixing large volumes of clay are eliminated.

The injection ratios for clay and polymer slurries are compatible. The polymer can be mixed on the machine trailing gear or on the surface and delivered by car to the backup pumping system on the trailing gear. The process generally costs 20–30% more than for clay-based slurry.

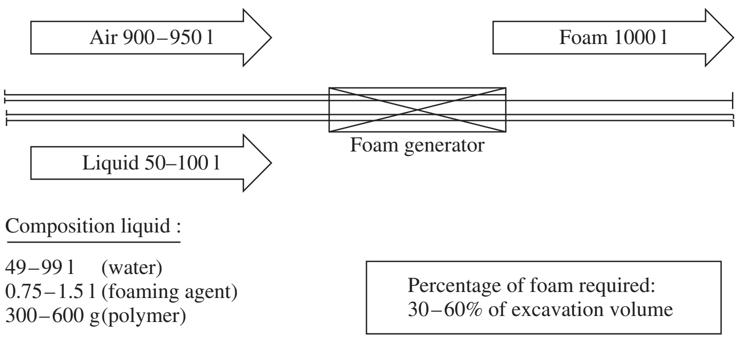

Figure 4.17 Foam Generation (Maidl et al., 1996)

The slurry in the tunnel is used to secure the face. It counteracts the earth pressure as well as the pressure of the water, and water can be used alone only in impermeable, cohesive material.

Clayey silty and silty sandy ground with pulpy to soft consistency is well suited for use with an EPM shield. Depending on the consistency of the ground, no water or only small quantities of water are needed. By arrangement of agitators and paddles/kneading tools in the excavation chamber, even very cohesive ground can be changed into a plastic slurry.

As the percentage of sand increases, addition of water is no longer sufficient because it is not possible to reduce the angle of internal friction. There is a danger of segregation of the earth slurry. The increase in water permeability makes sealing the screw problematic. The water in the pores can be bound through the swelling suspension and the muck be changed into a plastic earth slurry with very good flow properties and reduced permeability. Injecting a water-absorbing polymer suspension can achieve good results. EPB shields with injection holes in the cutting wheel and excavation chamber are used.

In Japan, because of the very permeable sandy to gravelly ground, an EPB shield with suspension back pressure was developed. Internal friction is reduced by reducing the effective tensions (growing pore water pressure) in the grain skeleton when pressurizing the excavation chamber with slurry pressure.

When water constitutes the transport and support medium, there will be no filter cake on the tunnel face. There is an increased danger of thrusts when entering the pressurized excavation chamber and reliable control of the groundwater is more difficult. Because of the uneven distribution of the support pressure on the tunnel face, these shields are more difficult to steer than the slurry shield.

Soil conditioning depends on the ground type and the ground parameters. These include grain size, water content (w), liquid limit LL (%), plasticity index (PI), and consistency index (CI). These parameters can be influenced by the addition of bentonite, clay, or polymer suspensions.

Conditioning of the ground should be done at the tunnel face in front of the cutterhead during excavation. This way clogging of closed cutterheads with small openings can be avoided.

Control of the support pressure at the face is possible when the modulus of volume change of the ground mixture is lowered, thus giving the ground ideal plastic properties over wide deformation areas. The gas phase within the grain structure will expand and the ground will deform with the pressure in the excavation chamber lowered. The size of the pores in the ground due to the rising pressure in the excavation chamber results in avoiding a rapid increase in the driving torque of the cutterhead.

In noncohesive water-bearing ground with little cover and strict surface settlement specifications, controlling the support pressure is of paramount importance. Unlike with slurry machines, the sudden collapse of the tunnel face into an excavation chamber filled with pulpy and compacted soil will not happen. The measuring and control of support pressure are adversely affected by material with a pulpy consistency theoretically based on the driving speed and excavation diameter. A key factor in controlling the face pressure is the control and monitoring of excavation quantities that are discharged. This method is referred to as the volume control method.

An EPBM is good in clayey ground. It uses the ground itself to serve as a support medium. Muck is removed from the excavation chamber via a screw conveyor or by being squeezed into the inside of the shield through extrusion openings. The appropriate screw speed can be determined by two monitoring systems: by earth pressure sensing in the chamber and by measurement of mud injection into the head. An earth pressure sensor mounted on the top of the cutterhead support senses the soil and water pressure. The pressure should be maintained at a value corresponding to the natural soil pressure at depth. If pressure is lost, one can reduce the speed of the machine. If reducing the speed does not increase the pressure, then earth slurry injection is increased to restore the pressure.

The amount of muck discharged can be compared with the machine advance to determine from the theoretical volume if overexcavation is occurring or in very soft ground whether underexcavation is occurring. The problem with this approach is that it does not provide the actual filling and degree of compaction of the material within the excavation chamber. In addition to the lack of precision, it obstructs the removal of muck. It is a qualitative determination that is not real time and loosening and swelling of the heading are not known. It does not provide the current condition of the face. If conditions are added at the same time, it is not possible to accurately determine the volume.

Other methods for monitoring the volume of muck have been developed utilizing more technical approaches. The belt weighing method weighs the muck for a fixed distance on the conveyor belt. The ultrasonic method measures the distance and calculation of the muck at a cross section on the conveyor belt using an ultrasonic transmitter, a receiver, and a processor. In addition, measuring the volume stream of the muck can be used to calculate and compare it to the theoretical volume of muck. In the laser method, the volume of the muck is calculated using a processor-coupled camera. A fanned-out laser beam is projected onto the measured cross section by a mirror to provide visualization of the surface. The ultrasonic and laser methods require relatively smooth surfaces. A series of earth pressure measuring cells provide more informative results for stability at the tunnel face.

Another approach to face pressure control is to constrict the flow through a gate device. Lovat uses a belt conveyor with doors which can control the entry of muck into the hopper. Hydraulic cylinders keep the doors closed until the soil pressure on the outside forces them open. The doors are located at the top of a muck ring inside the head. Muck is elevated by paddles inside the head. This system is different from other EPBMs in that the muck paddles effectively partition the head into a series of chambers instead of the single mass of soil inside an open head subject to uniform pressure.

The success of an EPB operation depends on the control of the flow of material from the cutterhead chamber to balance the advance rate of the machine and restrict water inflow. Water flow can result in the loss of material through the screw and cutterhead, leading to the creation of voids and potential ground loss.

The screw conveyors remove the material from the pressurized excavation chamber to the unpressurized tunnel and in permeable ground will seal against water under pressure. The screw conveyor is part of the support pressure control through controlling the removal of muck from the excavation chamber.

Part of the function of the EPBM is to keep water from getting into the tunnel. If the water pressure is atmospheric, say 1.5 bars of more, some type of pressure lock has to be on the discharge gate of the screw to prevent blowout.

Methods include a piston discharge unit (a positive-displacement pump) or a rotary feeder. The inside of the shield is totally sealed off from the ground and groundwater. A wire brush seal is used in the tail between the liner and the tail shield. Articulated shields have an elastomeric seal positioned at the joint.

Not having to provide a separation plant that requires a large surface operating space and is very uneconomic, particularly if there is a high percentage of silt/clay, is an advantage of EPBMs over slurry machines. Also, when mining with little cover, slurry machines risk the danger of blowouts through pulpy support slurry.

Comparisons of the machines as well as the types of projects for which they are best suited should be made. Although each machine has its ideal uses, in some situations where the parameters are that the same or almost so, the EPBM will be selected because of its lower cost. Instead of on the basis of the mining conditions, the selection may be made based on the contractor's experience with one type over the other, capital cost, or surface area, to name a few.

The EPBM has certain advantages over SPBMs. These advantages will assist the contractor in making the determination as to what tunneling system to select:

- EPBMS have the potential for greater advance rates than SPBMs as well as lower capital costs.
- EPBMS require less surface area and shaft cross section.
- Because there is no slurry circuit, less additives are used with EPBMS.
- EPBMs are simpler to operate and maintain.
- EPBMs can benefit from self-supporting ground.
- The amount of ground loss should the face collapse is limited with EPBMS.

However, there are attributes possessed by SPBMs that make them better suited to particular projects when compared to EPBMs:

- Because the SPBM does not have to rotate a head full of heavy muck when advancing, they require less torque and cutterhead power.
- The required pressure in SPBMs is determined and controlled by a system with precise controls.

- In SPBMs the tunnel is cleaner because the slurry and slurry/muck are maintained in pipes.
- There is less friction and hence less wear in SPBMs.

Often the tunnel alignment requires the machine to travel through varying material. It may be a mixture of rock and soil or soil and boulders. Provided the tunnel is above the water table, an open shield may be used. This type of ground can be dealt with at the face with equipment suitable for the material encountered. To mine this type of ground, slurry machines have a crusher at the bottom of the cutterhead that will break cobbles into fragments small enough to pass through the discharge pipe. The cutters will be a mixture of soft ground and hard rock cutters. Mixed ground tunneling can be abusive to hard rock cutters (cutters are discussed in Chapter 7). The stress on the hard rock disc cutters is greater, because when the cutterhead is turned in the soil and encounters a boulder, there is considerable impact stress. To mitigate this, a tougher disc cutter should be used.

There are few tunneling conditions for which an appropriate machine cannot be built. It was not that long ago that mining through water-bearing ground could only be done with compressed air. This is not the case anymore. Machines will sometimes use compressed air in the face to provide the support needed to permit a worker to enter the cutterhead to change cutters.

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5 Tunneling In Rock

Although rock is much stronger and more abrasive than soil, often greater progress can be achieved tunneling through rock than through soil. The penetration is more difficult, but generally it is easier to maintain the stability of the opening, reducing the time taken from tunneling to install ground support, thus improving overall advance. As in soil, the advance, or production, can vary from 0 to more than 100 m/day depending on many factors, including rock type and strength, rock stand-up time, and penetration method. In rock, often the strength of the rock has little bearing on the advance compared to the competence of the rock. Time spent on controlling incompetent rock with very low stand-up time is time not advancing. The zero production mentioned above is not the norm, and if it does occur, it is probably the result of the installation of ground support or equipment breaking down, as in the case of a TBM, or planned maintenance of the equipment of production.

When early humans first encountered flint and other minerals they thought precious, they would generally follow a surface outcropping of the flint underground often using flint or animal bones and antlers as tools. However, as humans evolved, they found other reasons to go underground, for example, for water, shelter, transportation, or military reasons. One of the earliest methods of mining in rock was to build a fire at the face, thus heating the rock. When the rock became hot, it was doused with water. The abrupt introduction of the cold water caused the rock to crack, giving the miners a start at advancing the face. As one can imagine, tending to the fire or dousing the rock with water were not healthy activities, but slaves were plentiful. Although available, copper and its alloy bronze did not have the strength to be effective at penetrating rock. The discovery of iron changed things. Humans now had an implement strong enough to penetrate rock—the iron chisel. Centuries later the advent of gunpowder improved the mining method and tunneling came in to the modern age. A later improvement was the invention of NG dynamite by Alfred Nobel. Nitroglycerin NG dynamite increased the available energy from black powder to a high explosive. From this modern explosives have developed to the level that many explosive products do not contain NG. With the evolution of non-NG explosives, a method of mechanical mining was being developed. This method, using the concept of the soft ground shields and principles introduced in the nineteenth century, allowed using mechanical means to tunnel through rock. Largely through the insight and efforts of James S. Robbins, these tunnel-boring machines, sometimes called "moles," have revolutionized tunneling.

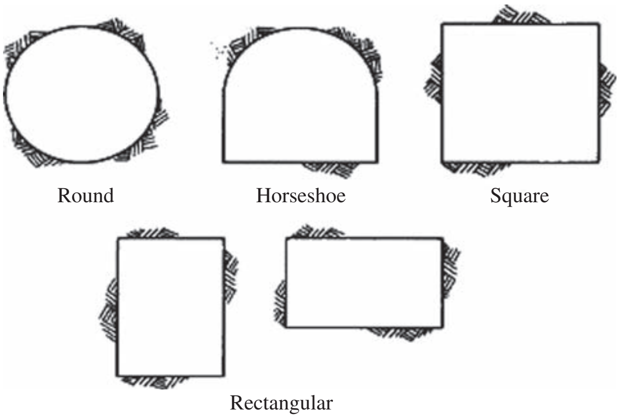

Figure 5.1 Tunnel Cross-Sectional Shapes

FACTORS CONSIDERED WHEN SELECTING TUNNELING METHOD

When choosing the method for penetrating rock, the geologic characteristics, tunnel dimensions, schedule, availability of equipment, availability and level of skilled labor, and costs are primary considerations. Any combination or any one of these can be the deciding factor when selecting the method. The methods discussed here are drilling and blasting, the hydraulic impact hammer, the roadheader, and the TBM. Each method has advantages and disadvantages that should be considered when determining the best way to accomplish the task.

The geologic characteristics include rock strength, nature of discontinuities, water, whether the water is above or below the water table, stand-up time, abrasiveness, and reaction to air.

The tunnel dimensions are tunnel length, cross-sectional area, shape, radius of curves and abrupt changes in direction. Figure 5.1 illustrates various tunnel shapes.

TUNNELING IN ROCK BY DRILLING AND BLASTING

Drilling and blasting are the most frequently used methods for tunneling and underground excavation. The process begins with holes of predetermined diameter, depth, and spacing being drilled into the rock. Explosives are placed in the holes. When detonated the energy generated by the explosive reaction fractures the rock.

The drilling and blasting cycle consists of:

- Drilling—Drilling holes in the rock for the placement of explosives
- Charging/loading—Inserting the explosives in the holes drilled

- Blasting and ventilation—Detonation of the explosives and waiting for the dust and gases generated by the blast to clear
- Scaling—Removing the loose rock in the crown
- Installing support
- Loading and transporting blasted rock (muck)

The order of the last two may be reversed. The ground may be competent enough to allow the mucking prior to the installation of support or the ground may be competent enough to not need ground support. The heading may be partially mucked and then support can be installed from the remaining muck pile. Before discussing the tunnel blasting process it is helpful to discuss blasting theory.

Drilling

The first process involved in a blasting operation is the drilling. The principle of drilling is to obtain maximum penetration to enable placement of the explosive charges.

The drilling system consists of the drill; the drill steel, or rod; and the bit. The force from the drill travels through the drill steel to the rock bit, penetrating the rock. Bits are designed for percussion, rotary, or both. The coordination of these forces with the slope, or the geometric properties of the bit, is what enables the bit to penetrate the rock. The rotation of the bit against the bottom of the borehole creates shear stresses in the rock, causing its separation. The percussion, or hammering, chips the rock by the combination of compressive and shear stresses created by the bit.

The rock strength, hardness, and abrasiveness of the rock affect the production or penetration rate, the bit wear, and operating costs. The operating costs are the time (of labor, equipment rental, etc . . . .) and wear and tear of the drill string. Abrasive rock will wear bits at a greater rate than nonabrasive rock.

The drilling accuracy is important to obtain the desire results. The blast is designed based on the distribution of the explosives within the rock. If a hole is out of alignment, it can cause too much explosive to be in one area and too little in the opposite area.

Types of Drills

Pneumatic Drill Two pneumatic types of drills used both on the surface and underground for drilling benches and secondary blasting are drifter drills and sinkers (jackhammers).

The most economical and common hand-held drills used underground are the jackleg and the sinker. The jackleg, also known as a pusher leg, is powered by compressed air and has the controls on the handle. Attached to the drill is an air-powered cylinder used for providing both thrust and vertical lift. The leg is set to the rear at an angle to the jackleg. The angle has to be balanced

Figure 5.2 Photograph of Jackleg (Photo: Atlas Copco)

both vertically and horizontally to get the most effective vector. The driller controls both the drill speed and air to the cylinder. Through the jackleg feed control lever, the driller uses enough air to hold the drill vertical at the needed horizontal position while applying horizontal force. Figure 5.2 is a photograph of a jackleg.

Like the jackleg, a sinker is for small-diameter holes, 25–50 mm (1–2 in.). Where the jackleg is for drilling horizontal holes, the sinker is limited to vertical holes. It is the primary hand-held drill used for sinking shaft. The sinker gets its name from being used for sinking shaft. Like the jackleg the practical limit for the length of the drill hole is 3 m (10 ft). These drills are only used when larger drills are impractical.

Drifter drills have a hammer or drifter that slides on a boom attached to a chassis. These drills, depending on the carriage configuration, can be used principally for vertical or horizontal drilling. For vertical drilling, the drill is mounted on a track carriage and is commonly known as an airtrac. This configuration provides maximum flexibility in rough terrain. The winch can be mounted on the carriage to allow the drill to work on very steep slopes. Although designed for vertical drilling, the boom can be positioned to drill horizontal holes to a limited extent.

The airtrac is used underground for big openings. Their principal use is drilling benches. As discussed later, sometimes tunnels are drilled in lifts. It may be decided that the lower lifts, called benches, should be drilled vertically. For the lower bench(es) there would probably be enough headroom to use the airtrac. The drill is generally efficient up to a depth of 20 m (66 ft). The external percussion drill has a drifter containing a piston which when subject to sufficient air pressure, usually 6–7 kg/cm^2 (90–100 psi), will start moving reciprocally within the cylinder. On the downward stroke the piston strikes the striking bar, or shank, 1200–2500 times per minute and this in turn transports the energy through the drill steel to the bit penetrating the rock. Figure 5.3 is a photograph of an airtrac.

The air-powered drifter drill will require at least 17 m^3/min (600 ft^3/min) of air at 7 kg/cm^2 (100 psi) pressure to operate. This is generally provided by a portable compressor to which the drill is connected by a hose or a hose attached to a manifold.

The rotation is created by the use of a splined rifle bar with pawls and ratchet or by a motor controlled independently of the movement of the piston. The rifle bar, which causes the rotation of the attached drill steel as it moves vertically through the rifle nut, comes in various ratios, For example, 1:30. This ratio refers to the pitch of the flutes or splines on the rifle bar and indicates the length of rifle bar required for one rotation. For example, a 1:40 ratio means that the rifle bar has to travel 40 in. to make one complete rotation (100 cm). This ratio permits the calculation of the theoretical rotational speed in revolutions per minute of the rifle bar. To illustrate, a 1:50 (travel 127 cm) pitch striking bar receiving 2100 blows a minute with a 6.3-cm (2½-in.) piston stroke will develop $(2100 \times 6.3)/127 = 105$ $[(2100 \times 2.5)50 = 105]$, or approximately 100, rpm. The higher the pitch is, the fewer revolutions per minute; thus, a 1:30 pitch creates a higher rotational speed than 1:50.

As the name implies, independent rotation is independent of the reciprocatory piston, so that the rotation can be regulated without the influence of the hammer. Therefore, in soft material drilling, where greater rotation is needed relative to percussion, the rotation may be increased and the hammer, or percussion, in

Figure 5.3 Photograph of AirROC T25 Top Hammer Surface Drill (Courtesy of Atlas Copco)

blows per minute, may be decreased or not used at all. Also, in hard rocks, where percussion is most needed, the hammer can be greater and the rotation reduced.

Independent rotation is achieved by a gear motor operating independently of the piston. The motor is mounted on the top or side of or adjacent to the hammer.

Because of its carriage, an external-percussion drill is well suited for very rough terrain. In can drill small-diameter holes, 50–150 mm (2–6 in.), in very hard rock. Also, because the piston is larger than the bit (generally it should be at least 25 mm, or 1 in., larger), the drill's penetration rate for holes less than 18 m (60 ft) in hard rock makes it the most efficient of any pneumatic drill.

External-percussion drills are usually mounted on a self-propelled crawler undercarriage. The crawler and boom are either air or hydraulic powered, and the boom controls are generally hydraulic. The thrust comes from the weight of the drill rig and is applied by a chain and sprocket.

The drill rods for this type of drill are 25–50 mm (1–2 in.) in diameter and 2.5–3.5 m (8–12 ft) in length. They have a hollow center for flushing with bits of either chisel or button characteristics.

When a pneumatic drill is configured for horizontal drift, drilling the boom is horizontal instead of vertical and can be mounted on tires, crawlers, or rail. For underground horizontal drilling, the drill can be mounted on a rail chassis, whereby it travels on rail. This permits quick demobilization but is limited by the width of the machine. Outriggers can be used, but they are still limited.

The maintenance of a pneumatic drill is generally restricted to lubrication. The hammering is lubricated by a pressurized oil tank or metal bottle that automatically feeds an adjustable amount of oil to the hammer. Oil is transported to the hammer with air; the air enters the oil bottle, where the oil is atomized in the air and passes to the hammer. With many oil bottles, when the bottle runs out of oil, the air is shut off by a valve within the oil bottle to prevent running the hammer dry and causing overheating. It is good procedure to turn the hammer on briefly before drilling to inspect the air exhaust to make sure it contains oil vapor indicating that the hammer is receiving lubricant. In addition, a tank for holding water is mounted on the crawler frame. This water is added to the drill string for dust suppression and the removal of drill fines.

Hydraulic Drill The hydraulic-powered rock drill generally provides greater production with reduced energy costs than the pneumatic drill. The hydraulic drill is particularly well suited for underground drilling because of the reduced noise levels, even compared to silenced air drills. Like the air jumbo, the hydraulic jumbo can be crawler mounted, on tires, or rail mounted. Crawler mounted, it provides mobility on rough terrain. The hydraulic drill jumbo is self-contained, with its power source mounted on it. This yields greater mobility than an air-powered machine, because the latter must be connected to an air source with piping.

Generally, the drill string has greater life due to the nature of the stress distribution. The piston in a hydraulic drill is smaller in diameter and much longer than the piston in a hammer. The effect of this is that the stresses are in the form of a longer, more extended shock wave that covers a larger area of drill steel. Therefore, the stresses are distributed over more of the drill steel, lessening the effects on any one area and decreasing the life of the drill rod. This improvement can be explained. It is understandable when considering the primary cause of bit wear. Bits are generally worn by rotation against abrasive rock, not the percussion energy. Therefore, the hydraulic drill, with its many blows per minute, can greatly increase the penetration rate without decreasing bit life. Actually, the bit life increases, because there is more footage drilled between grindings.

Hydraulic drills are more sophisticated and have much closer tolerances than those that are air powered. They need the parts to be very clean, thus requiring pristine conditions to repair them. This causes the hydraulic drill to be more difficult to maintain and repair. Often, the driller can repair the air-powered drill because of the tolerance to dirt. If dirt gets into an air hose, it is blown out, whereas a hydraulic hose will require better cleaning. Figures 5.4 and 5.5 are photographs of a crawler-mounted hydraulic rock drill being used for bench blasting and a rubber-tired jumbo used for horizontal drilling in a cavern under Grand Central Station, New York City, respectively.

Figure 5.4 Hydraulic Crawler-Mounted Rock Drill

Figure 5.5 Tire-Mounted Hydraulic Drill Jumbo

Explosives

Possibly the single greatest achievement of the industrial revolution was the development of safe, peaceful uses of explosives. It is believed that black powder was first discovered by Chinese alchemists around 22 BC. While separating gold from silver they obtained potassium nitrate (saltpeter) and sulfur but forgot to add the charcoal. They tried to add the charcoal during the last step and it exploded. It is thought to have been used primarily for fireworks. The first historical record of explosives is when Roger Bacon gave instructions on how to make black powder. In 1320, a German monk, Berthold Schwartz, inventor of the gun, studied Bacon's writings and experimented with black powder.

As an industrial explosive, the principal danger of black powder was when lighting it. Goose quills, wheat straw, wood tubes, and anything conceivable were used as the fuse. Black powder is undependable and dangerous.

While visiting a rope manufacturer, William Bickford wondered if rope could be wrapped around a core of black powder. He placed a continuous core of black powder in a cable of jute and string and added a surface coating to waterproof it. This was the first fuse; this innovation increased miners' safety and saved lives. It still has limited applications today. In addition to saving lives, it has provided more dependable firing and a more accurate time determination, making it safer to light the fuse.

Not being a high explosive, black powder was limited in its strength; something more powerful was needed to efficiently break rock. The first NG was made in 1847 by Ascanio Sobrero, an Italian physician and chemist. He treated glycerin with a mixture of sulfuric acid and nitric acid. While experimenting, a small sample of the yellow-colored oil exploded, spraying him with glass.

He lost his interest in this new explosive. Because of its unpredictability and volatility, little more was done with it.

The method of using primary explosive to detonate secondary explosives as well as the development of high explosives is credited to Alfred Nobel. In 1860, Nobel joined his father, Immanuel, in a small NG manufacturing venture in Sweden. To solve the problem of detonating the volatile NG, Nobel developed a method of detonating one explosive with another. He used a vial of black powder to detonate the NG. He placed the vial of black powder with the NG and he detonated the black powder, which had the explosive power to detonate the NG. In 1863, he replaced the black powder in the vial with mercury fulminate. In 1864, he patented it using lead tubes rather than the vials. He further developed the concept and in 1867, he patented it, using a copper tube for the mercury fulminate. His design of the detonator is still in use today.

The Hoosac Tunnel in Massachusetts, which took from 1851 to 1874 to complete, was built using two innovations of the period. In the tunneling operation, the pneumatic drill invented by George Law was used, and in 1867, the explosive was NG. The quantity of NG needed necessitated that a NG plant be built near the site. About 500,000 kg (550 short tons) of NG was used, and, after extremely poor production for about 17 years, it took 6 more years to complete the project. It took 23 years to drive 7.6 km (4.75 mi) at a cost of 196 lives: 25 deaths/km (41 per mile), or, to put it another way, approximately one life per 40 m (one life per 128 ft).

In addition to the Nitroglycerin Act, which banned the manufacture or importing of NG into Great Britain, the need existed for a safer explosive. Nobel developed a solution by using packing material. He first used sawdust, but when the cartridges leaked, the sawdust would absorb the NG, which could spontaneously combust.

He improved the process by using kieselghur. Kieselghur is diatomaceous earth, that is, the skeletal remains of plants, called “diatoms.” At first it was used as a packing material. However, under examination, Nobel noted that the NG had been completely and safely absorbed. He found that a mass of NG-soaked kieselguhr could be cartridge in waxed paper. He called his new product “guhr dynamite.” He got the word *dynamite* from the Greek word *dynamis*, which means power. The guhr dynamite mixture contained 75% NG by volume. It was found that the mixture was not strongly bound and the NG was easily displaced by water. The NG would separate from the mixture when stored for a period of time. Although kieselguhr stabilized the mixture somewhat, the kieselguhr, being inert, did not add to the effectiveness of the explosive; it actually reduced the amount of energy from the explosive detonation. The problem was solved by adding something to the NG that not only stabilized it but also contributed energy. This was referred to as active-based dynamite.

In 1867, Carl Dittmar added wood meal to the NG. Not only did the wood meal do a better job of holding the NG together but, containing carbon, it actually contributed energy. His discovery was the beginning of the next generation of explosives.

While Dittmar was developing his wood meal additive, Ohlsson and Norbin developed the idea of mixing ammonium nitrate (AN) to improve the explosive properties of dynamite. They first tried a mix of 80% AN and 20% charcoal. They found that the mixture was too insensitive and had a high affinity for moisture. To increase water resistance and sensitivity, they added NG. They sold their patent to Nobel in 1873.

Nobel picked up on their idea and made changes. To no avail, he added wax to try to slow water absorption. Again he tried AN but as an absorbent of NG. From this he filed a patent in 1879 calling the new product extra dynamite. Commercially the product became known as dynamite extra and special dynamite. The new explosive was more powerful and lasted longer with cleaner gases. It still had the problem of water affinity, and when it was wet, noxious gases became a problem. From the commercial side, the market preferred gelatin extra.

The Ammonium was first prepared in 1659 by Glauber but was not considered an explosive until the nineteenth century, when it was used as a replacement for potassium nitrate. When a heated mixture of AN and charcoal exploded in 1849, its explosive properties were reported.

In 1885, Dupont employees formulated an AN-based explosive. The AN was dried and granulated, and for water resistance each particle was coated with a petroleum-based product. It was mixed with known active components, such as sodium nitrate, and all components contributed to the explosive energy.

Because of their effectiveness and lower cost, AN-based explosives became very popular. By 1913, British coal consumed more than 5000 tons of explosives per year, and by 1917, 92% of all explosives used were AN based. It took a couple of disasters caused by AN explosions to realize the potential of AN as an explosive. Noncaking AN fertilizer exploded. It was learned that the wax coating in the fertilizer also acted as a fueling agent. The worst disaster in the United States occurred in Texas City, Texas. A shipload of AN exploded in the harbor after a shipboard fire. More than 500 people died, thousands were injured, a second ship with AN on board exploded, and 4.5-m (15-ft) tidal wave was created in the harbor. The principal flaw with AN is its lack of water resistance.

In the 1960s, the development of water gels negated many of the disadvantages of AN. Dr. Melvin A. Cook patented an explosive composition containing water as an essential ingredient. His original water gel explosive was a large-diameter gel explosive that was used in large wet holes. The gel consisted of water-resistant NG-based explosives, waterproof packages of non-water-resistant AN/fuel mixtures, or non-water-resistant NG-based products. Early water gels were called “slurries”; the term *water gel* did not come into common usage until the 1970s. Water gels are basically a thickened aqueous solution of oxidizer and/or fuel salts, with additional solid oxidizers and/or fuels, as well as sensitizers dispersed in the solution.

Water gels are less sensitive than NG explosives that allow control of the borehole density because they fill the entire cross section of the hole. They provide loading flexibility in that they can be loaded cartridge; they are pumpable and pourable. Less sensitive than NG explosives, the hole-to-hole propagation in water gels is minimized. The amount and toxicity of the fumes are reduced, and a major factor to anyone who has ever had one, NG headaches are eliminated.

Like all high explosives, water gels are comprised of oxidizers, fuel constituents, and sensitizers. Oxidizing salts and fuels are dissolved or dispersed in a continuous liquid phase. By adding gellants and cross-linking agents, the water gel is thickened and made water resistant. The oxidizing salts are usually AN, sodium nitrate (SN), and calcium nitrate (CN). For fuel aluminum, coal, gilsonite, sugar, ethylene glycol, or oil is used.

Chemical sensitizers include nitrate salts of organic amines, nitrate esters of alcohols, perchlorate salts, fine particle sized aluminum, and other explosive that are more sensitive.

Physical sensitizers are entrapped air, chemically produced bubbles, and hollow bubbles of glass or phenolic composition. Physical sensitizers can be used in a combination of chemicals or alone.

In 1964, while trying to make ANFO (ammonium nitrate and fuel oil) waterproof, Egly and Necker received a patent for a blasting agent composed of a blend of water-in-oil emulsion and solid oxidizing agent such as AN. An emulsion is a stable dispersion of one liquid in a second immiscible liquid like milk or oil dispersed in water. Soaps, detergents, and compounds whose basic structure is paraffin wax chain are the largest group of emulsifying agents. It is also a mixture of two immiscible liquids, with one phase dispersed uniformly throughout the second phase. In water-in-oil emulsions, the oil/fuel stage is known as the “continuous or external phase,” because it surrounds and coats the oxidizer droplets. The fuel stage is generally oil and/or wax; No. 2 diesel fuel oil (FO) is common. This mixture is called “ANFO.” The water/oxidizer phase is called the “discontinuous or internal phase,” since the droplets are kept apart because they are surrounded by the oil. The oxidizer phase always contains AN, but it may also contain sodium nitrate, calcium nitrate, or ammonium or sodium perchlorate. The oxidizer remains dispersed in fuel, forming a stable emulsion through the action of a surfactant. This is similar to how oil and vinegar are held dispersed by egg yolks to make mayonnaise. Figure 5.6 illustrates the commercial explosive development chain.

There are three types of explosions: mechanical, nuclear, and chemical. A boiler blowing is an example of a mechanical explosion. A nuclear explosion

Blackpowder -------→ Nitroglycerin -----→ Dynamite ---→
Straight dynamite ----→ Extra dynamite ---→ ANFO ---→
Water gels/slurries ---→ Emulsions

Figure 5.6 Commercial Explosive Development Chain is the result of the rapid release of energy that is caused by an intentionally high-speed nuclear reaction. The reaction may be the result of nuclear fission, nuclear fusion, or a multistage cascading of the two in combination. The well-recognized mushroom cloud is the signature of an air-burst nuclear detonation.

A chemical explosive is a compound or mixture capable of releasing substantial amounts of heat and gas by undergoing extremely rapid decomposition. When a gas is heated and unconfined, it will increase in volume. Therefore, an explosive that creates energy by releasing hot gases will require a space many times the original volume of the explosive, exerting great pressure on the surroundings.

There are four phases to a chemical explosion: release of gas, intense heat, extreme pressure, and the explosion.

A chemical explosive can also be defined as a compound or mixture that decomposes or rearranges very rapidly, generating much heat and gas when heat or shock is applied. To be considered an explosive, it must generate both heat and gas. For example, a mixture of oxygen and nitrogen can be made to react with great rapidity and to produce the gaseous product nitric oxide. However, the mixture is not an explosive because it does not generate heat; it actually absorbs heat.

Primary and secondary explosives are classified principally by their sensitivity. Primary explosives rapidly change from burning to detonation and can transmit the energy to less sensitive explosives, affecting their detonation. Primary explosives must be subjected to heat or shock to detonate. When detonated, the molecules in the explosive separate into two or more fragments, such as atoms (disassociate), and produce tremendous amount of heat and shock. With this tremendous heat and shock, primary explosives can detonate more stable secondary explosives.

Primary explosives are highly sensitive to friction, high temperature, shock, electric spark, and high temperature and will explode whether confined or unconfined. The most common primary explosive is lead azide. Most azides are unstable substances that are highly sensitive to shock. Some inorganic azides and alkyl azides are used for initiating explosives in blasting caps as well as most of the hot-wire detonators

Secondary explosives cannot be detonated easily or dependably by heat or shock. The detonation of secondary explosives requires the shock and heat generated by a primary explosive. Upon initiation, secondary explosive compositions separate quickly into other more stable components.

To be an explosive, a chemical must demonstrate all of the following characteristics: the generation of heat, the formation of gases, a rapid reaction, and the initiation of reaction.

The primary elements in chemical explosives are carbon (C), hydrogen (H), nitrogen (N), and oxygen (O_2). Substances can generate gases in a variety of ways.

When a carbon-based substance, such as wood or coal, is burned in the atmosphere, the carbon and the hydrogen will combine with oxygen to form carbon dioxide (CO_2) and steam with smoke and flame.

If the wood or coal is pulverized, thereby increasing the total surface area that comes in contact with the oxygen, and burned in a furnace or forge where the supply of air can be increased, the speed of burning can be increased and the combustion is more complete. Also, if wood or coal is immersed in liquid oxygen or held in suspension in the air similar to coal dust in a mine, the burning occurs with explosive violence. In each of these cases, the burning combustible forms a gas.

All explosive chemical reactions generate large amounts of heat. The rapid heat generation causes the gases produced by the reaction to rapidly expand, developing extremely high pressures. The rapid development of the high gas pressures is what constitutes the explosion. An explosion will not result if the heat generation lacks sufficient speed. An example of the lack of speed resulting in the generation of an explosion is if a kilogram of coal produces five times the heat as the same weight NG, the coal cannot be used as an explosive because the production of heat is too slow. The factor that separates, or distinguishes, an explosion from a combustion reaction is the speed, or rapidity, of the reaction. The heat generation must be very rapid, or the gas will dissipate before the heat can affect great increases in gas pressure. For example, as wood fire burns, there is the generation of heat and the formation of gases (CO_2 and steam), but neither is generated quickly enough to cause an explosion.

Even if the aforementioned three factors necessary for a chemical explosive are met, the substance cannot be classified as an explosive if the reaction is not capable of being detonated by the application of shock or heat to a small portion of the mass of the explosive substance. That is, the substance must be such that the reaction is initiated to a small portion of the total substance. An example of this is the detonation of a column of explosive in a blasthole, where the firing of the charge can be anywhere along the explosive column and the entire column detonates.

The detonation of explosives produces energy by oxidation. Oxidation is a chemical reaction caused by fuel burning or an explosive detonating. The nature of the end products of an explosion will be dependent on the amount of oxygen that is available during the reaction. This supply of oxygen will be dependent on the quantity of oxidizing atoms present in the explosive molecule. In order to achieve the maximum efficiency (output) from an explosive and reduce the generation of toxic gases, an oxygen balance is necessary. An oxygen-balanced explosive is an explosive in which there is no excess or deficiency of oxygen in the mixture. There is enough oxygen available to produce desired detonation gases rather than noxious fumes, but not an excess of oxygen that leaves excess free oxygen. A fuel-to-oxidizer ratio oxygen balance is called a balanced stoichiometric ratio. It is at this point that an explosive mixture yields its maximum theoretical energy, which is released as heat of detonation and velocity of detonation. For explosives that do not contain sufficient oxygen in the molecule for complete oxidation, the end products will be carbon monoxide, carbon, and hydrogen.

The energy released during the formation of these gases is known as the heat of explosion. If one can isolate these products in an environment with excess oxygen and they are allowed to burn, they will form carbon dioxide, water, and other products. The heat that evolves from the formation of these products is called the heat of combustion. Therefore, with substances that have insufficient oxygen for complete oxidation, the value of the heat of combustion is greater than the value of the heat of explosion. For explosives with positive oxygen balances, such as NG, the values of heat of combustion and heat of explosion are the same. The heat of explosion is the energy, or heat, liberated from a chemical reaction that occurs during the detonation of an explosive. Explosive components can be mixed to achieve a zero oxygen balance.

The volume of gas produced by an explosion is indicative of the amount of work, or energy that the explosion will produce. To measure the volume of gases produced, standard conditions must be used because gas volumes will vary due to temperature and the type of explosives.

Adding other fuels to the explosive composition can increase the heat of explosion. By adding a fuel that has a high temperature of combustion, the total heat of the reaction can be increased, liberating more energy. These chemicals are among the lighter elements. Aluminum is the most common lighter element used in explosives. Although beryllium produces higher heat, aluminum is relatively inexpensive and therefore it is more economical for use in explosives. Aluminum is used in water-based slurry explosives as well as ammonium nitrate. An increase in the output of heat from a reaction with aluminum prolongs the presence of high pressures. However, there is a limit to the amount that aluminum that can be added to an explosive and produce the desired effects.

Its maximum effectiveness is at a quantity of approximately 18% aluminum. Aluminum is ground very fine when it is necessary to increase the sensitivity of the explosive substance. With some emulsions, aluminum is added because of its temperature-increasing capability and does not need to be as finely ground.

Explosives are classified as military when used primarily as propellants and as commercial when used in commerce and industry for the purpose of doing work. The most common types of industrial explosives are dynamite, water gels, and blasting agents.

The military requirements of explosives are quite stringent. Military explosives are more brisant and less sensitive than commercial explosives. They have to be less sensitive because they are used under such adverse conditions. The reduced sensitivity required by military explosives includes resistance, or lack of sensitivity, to impact, friction, and heat. Military explosives must be more stable, less toxic, and less volatile; volatility is the readiness with which a substance vaporizes. Excessive volatility can result in the development of pressure within rounds of ammunition and a separation of mixtures into their constituents. NG is not used in military explosives. Other requirements are availability and the cost. Because of the quantity of explosives required by the military, there is more concern about the availability and costs of explosives than in the commercial sector. Availability refers to not only the adequacy but also the source of supplies. The military has to be more concerned that the explosives are produced from inexpensive, nonstrategic materials.

The most common military explosive is trinitrotoluene (TNT), which has a detonating velocity of 6900 m/sec (22,600 ft/sec) and can be used for demolition charges, bursting charges, and booster charges. All other explosives are measured or compared against TNT.

Commercial explosives are materials that detonate upon introduction of a suitable initiation stimulus, whereby the reaction front moves through the explosive at a velocity greater than the sonic velocity of that material. There are four common types of industrial explosives used in construction today: high explosives, blasting agents, water gels, and emulsions.

High explosives are characterized by high detonating velocities, high density, and high-pressure shock wave and are cap sensitive for initiation. High explosives detonate at a velocity of 1525–7620 m/sec (5000–25,000 ft/sec). These explosives are generally in cartridges made of paper or plastic. High explosives are capable of undergoing a chemical reaction that converts them to gases at very high temperatures under great pressure. High explosives used commercially have two primary bases, NG and AN. Additives are used with these bases to resist freezing and make the explosive less sensitive to shock. In the case of permissible explosives, additives are used to reduce the heat and the duration of the explosion.

Explosive properties include strength, detonating velocity, fume class, water resistance, density, detonating pressure, sensitivity, and critical diameter.

Explosives, dynamites in particular, will generally have a rating such as 40 or 60%. The origin of this rating stems from straight-NG dynamite and it refers to the percent by weight of NG in straight-NG dynamite. In non-straight-NG explosives the percentage rating is a comparison to an equal weight of NG dynamite. That is, 40% ammonia dynamite can produce the same energy as 40% straight-NG dynamite.

Velocity of detonation (VOD) is the velocity at which the detonation wave moves through the explosive. It is expressed in meters per second or feet per second; the higher the VOD, the greater the brisance. Brisance is the shattering power of an explosive. Brisance is the powerful violent mechanical blow generated by an explosive. The higher the brisance, the more suited the explosive is for cutting steel or fracturing very hard rock. Many blasters subscribe to the theory that the velocity of detonation should match the seismic velocity of the rock. Whether one subscribes to this or is inclined to value the heaving effect of lower velocity explosives, a high VOD is very desirable for blasting boulders and cratering charges. Generally speaking, the harder the rock, the higher the seismic velocity and the higher the VOD should be. The greater heave associated with lower VOD is important where material displacement is desired.

Fume class is based on the amount of toxic fumes given off by an explosive when detonated and determines whether or not it is safe to use the explosive in a confined area, such as underground. The toxic gases may include carbon monoxide and oxides of nitrogen.

The explosive will have ability to resist contamination or loss of its ability to detonate when exposed to water. Water can have adverse effects on dynamite in two ways: water pressure may reduce the size and amount of air bubbles, and salts can be dissolved by the water, thereby reducing sensitivity.

The density of an explosive is expressed a grams per cubic centimeter. The power of dynamite generally increases with density. However, with water-based explosives, such as emulsions or water gels, the energy is not as affected by density. The variance in the density of different explosives can aid in blast design; an explosive with higher density permits greater energy in the same area.

The detonating pressure is the measure of the pressure in the reaction zone just behind the detonation front. The detonation pressure, simply put, is the peak borehole pressure. It is often measured in kilobars (14,700 psi).

Sensitivity is the ease with which an explosive is detonated, sometimes referred to as “propagation sensitivity.” It gives an indication of an explosive’s ability to maintain the detonation or the risk of sympathetic detonation.

Relative to the detonation sensitivity of an explosive is the critical diameter. The critical diameter is the smallest diameter at which a column of explosives will detonate. Therefore, as the diameter of an explosive decreases, the propagation ability also decreases.

The “storability” of an explosive product refers to how well the product can be stored without affecting its safety, reliability, and performance. One should follow the manufacturer’s storage recommendation. Although the storage characteristics of explosives have greatly improved, it is still important to turn over supplies to prevent prolonged storage.

An explosive’s resistance to freezing can be important for both safety and performance. When purchasing for cold climates, the resistance to freezing and the effects of the cold on the product should be understood. Although they contain antifreezing additives, NG explosives can become dangerous if frozen. Depending of the product, cap-sensitive water gels can fail to be cap sensitive in typical winter temperatures: -17 to -40° C (1 to -40°F).

Commercial explosives can take four basic forms: granular, gelatin, slurry, and paste textures. The form depends on the formula, and the choice of form depends on the usage required. Slurry explosives can be pumped into a borehole with no container or packaged in polyethylene bags to permit handling in smaller amounts.

The major types of commercial high explosives are:

1. Straight-NG dynamite
2. Gelatin dynamites
3. Ammonia dynamites
4. Semigelatins

Straight-NG dynamite consists of NG and carbonaceous materials such as sawdust and nitrocellulose (guncotton). It is granular and has a high detonating velocity: 5200 m/sec (17,000 ft/sec). Today, it is rarely used because of its sensitivity; however, it is well suited for ditching and mudcapping. It is a Class 3 fume and should not be used underground.

Ammonia dynamites are granular and use the reaction of AN as its primary energy source. The AN is less sensitive to heat and shock and therefore safer to use. It has a lower density and is also less resistant to water. Because of the low water resistance of ammonia dynamites, it is best when loading not to break the cartridge and permit moisture to enter. They are generally not recommended for underground use because of a Class 3 fume classification. Ammonia dynamites come in both high and low density, the low density being less expensive because less NG and more AN are used. The detonating velocity range for ammonia dynamite is approximately 1525–3350 m/sec (5000–11,000 ft/sec). Figure 5.7 is a photograph of NG dynamite.

Gelatin dynamites have a base of water-resistant “gel” made by dissolving nitrocotton in NG. The nitrocotton gel is insoluble in water and tends to bind together other ingredients, making them water resistant and forming a cohesive, plasticlike substance having a confined detonating velocity of 4000 m/sec (13,000 ft/sec).

Ammonia gelatins lack some of the performance characteristics of straight dynamite, but their relative safety and cost considerations make them a viable tool. Ammonia gelatins are lower in cost but also less resistant to water and are Class 1 fumes but do not possess good storage capabilities. The most common strength used is 40%.

Semigelatins are designed to combine the resistance to water and cohesiveness of gelatins with the lower cost of ammonia dynamites. Semigelatins tend to have poor storage and temperature resistance characteristics. They are generally Class 1 fumes and work well as presplit explosives.

A blasting agent is a mixture containing both fuel-producing and fuel-oxidizing elements. None of the elements in a blasting agent is considered an explosive until it is mixed; when not mixed it is not a blasting agent. Blasting agents cannot be detonated by a No. 8 blasting cap when unconfined. The most common blasting agent is nitrocarbonitrate (NCN), which is more generally known as ANFO.

ANFO is a mixture of AN and fuel oil in which AN acts as the oxidizer and fuel oil acts as the fuel. The fuel may be something other than fuel oil, such as carbon, sawdust, hydrocarbons, or other carbonaceous substances with

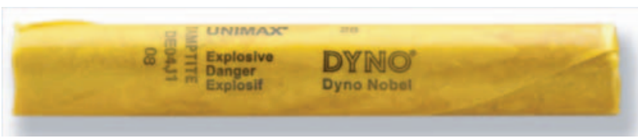

Figure 5.7 Extra Gelatin NG Dynamite (Courtesy of Dyno Nobel)

a carbon base. However, the most commonly used is AN with fuel oil in 94:6 weight ratio (or approximately 6%). The velocity of detonation and the heat of explosion (theoretical energy released) are both at a maximum when the oxygen balance is 5.7% fuel oil. The values reduce as the percentage varies from the optimum, but the values decrease more gradually with an oversupply of oil (fuel) than an oversupply of O_2. AN is an ingredient in a vast majority of commercial explosives; approximately 97% of explosives sold contain AN and 80% of the AN sold is in the form of ANFO; its predominant form is as prills. ANFO offers great economy and safety in modern blasting applications. It generally costs one-quarter to one-half as much as NG explosives and it is considerably safer to handle because of its lack of sensitivity. In many types of blasting situations ANFO will produce better fragmentation due to its high gas-producing properties. Some believe that ANFO is the best type of explosive for blasting dry boreholes in excess of 65 mm ($2\frac{1}{2}$ in.) in diameter, which are conducive to breakage by gaseous expansion. ANFO is poor in small-diameter boreholes and conditions that require very high detonating velocities.

The primary disadvantage of ANFO is its lack of water resistance. Once it comes in contact with water ANFO is no longer dependable and, in fact, will not detonate. Mixing can be a problem with ANFO; that is, sometimes the AN and the fuel oil are not properly mixed.

The velocity of ANFO varies with changes in borehole diameter. Its maximum VOD is about 4400 m/sec (14,400 ft/sec). With borehole diameters of less than 25 mm, the detonation of ANFO is not stable. It is best suited for boreholes with diameters ranging from 75 to 250 mm (3 to 10 in.).

Since using dry prills of AN is the most common method for handling ANFO, if practical, ANFO is loaded in bulk. For nonbulk uses ANFO is packaged in 50-lb heavy paper lined with plastic bags. To avoid water contamination, ANFO is sometimes packaged in plastic bags or cardboard tubes with tough plastic lining and used in holes greater than 150 mm (6 in.). The prills are sometimes crushed, which reduces their particle size and increases their density, and then are packaged.

Heavy ANFO is the combination of prilled ANFO and emulsion. In some cases, the emulsion is mixed with straight AN without fuel oil (FO) if the emulsion has enough fuel in its composition.

Blends are used to increase the density of ANFO, thus increasing the energy in the borehole, to provide water resistance to ANFO, and to reduce blasting costs by improving the performance and water resistance of ANFO. Blends are generally understood to mean mixtures of water-in-oil emulsions and fuel oil. Mixtures consist of water-based explosive material matrix and AN or ANFO. Or the blend can consist of a water-based oxidizer matrix and AN or ANFO. Blends are generally not cap sensitive so they can be treated as a blasting agent for transportation and storage. As the ratio of emulsion to ANFO increases in the blend, the water resistance increases to a ratio of about 40% emulsion to 60% ANFO when the blend is virtually waterproof. At a blend of 60% emulsion to 40% ANFO, the blend is pumpable. ANFO is soluble in water and therefore, unless packaged, cannot be used under wet conditions.

In an effort to take advantage of the low cost and effectiveness of ANFO as an explosive and at the same time overcome the water resistance problem, explosive manufacturers developed slurries/water gels to protect the AN against contamination from water. The detonation velocity of slurries ranges from 3350 to 5500 m/sec (11,000 to 18,000 ft/sec), which can be greater than that for ANFO.

The slurries that were developed to solve the ANFO and water problem are commonly a mixture of an AN base in an aqueous solution of oxidizer and fuel salt, a heat-producing metal, and other ingredients to yield a thick soupy slurry. Water gels are 10–30% water and contain a thickening agent that solidifies the slurry in the borehole to protect the AN from water. Because of a density greater than that of water (1.05–1.8 g/cm^3), the slurry will sink to the bottom of a wet borehole.

Although of like constituents, slurries are fluids which are suitable for pumping, whereas a water gel is a slurry explosive that is dimensionally stable to permit it to be packaged in light plastic cartridges to allow transportation and storage. Even though their origins are of blasting agents, slurries are often cap sensitive, making them an explosive.

Water gels control borehole density; by slitting and tamping cartridges or bulk loading, the explosive borehole density can be increased substantially. Improved flexibility in loading that provides time- and labor-saving techniques for various loading conditions was developed using cartridged, pumpable, and pourable water gels. Water gels also offer excellent fragmentation control, minimized tendency for hole-to-hole propagation, reduced smoke and toxic fumes, and no NG headaches.

Emulsions are composed of separate, very small drops of AN solution and other oxidizers heavily disbursed in a continuous viscous material comprised of wax and mineral oil. The oil and wax mixture contribute as a fuel and, because the AN is disbursed throughout this medium, the oil and wax get to react with a large surface area of the disbursed AN. Therefore the fuel oxygen is well balanced. Figure 5.8 is a photograph of small-diameter emulsion.

An emulsion is a mixture of two liquids dispersed uniformly throughout the second liquid. Emulsions are dispersions of water solutions of oxidizers in an oil medium or a water-in-oil emulsion. Emulsions can be made to detonate

Figure 5.8 DYNO® AP Detonator Sensitive Emulsion (Courtesy of Dyno Nobel)

without the addition of a sensitizing explosive. Instead of a sensitizing explosive, emulsions will have very small cavities, or microballoons, about 0.0001 in. in diameter. These small cavities collapse from the initiating shock wave produced by the cap and cause "hotspots" where the temperature is elevated enough to cause detonation. The density of the emulsion can be modified by the amount of microballs added and the strength can be adjusted by controlling the proportions of additive fuel and aluminum. The ratio of oxidizer to fuel is about 9:1 to maintain an oxygen balance.

The consistency can be modified and controlled by the proportion of oil and wax added. The more wax that is added, the stiffer the mix will be. One can get it to an almost lardlike consistency. However, with the introduction of more oil, the consistency can be that of pumpable grease.

Emulsions have outstanding stability and the detonation properties remain relatively unchanged during long periods of storage under normal conditions. They have a high velocity of detonation and the sensitivity can be varied from detonation by a No. 8 cap to that of a blasting agent. Because the AN is in an emulsion, the water-soluble drops are completely surrounded by the oil/wax substance; emulsions are very water resistant. This oil, or fuel, coating is known as continuous or external because it surrounds and coats the oxidizer droplets with fuel. The oxidizer remains disbursed within to form a stable emulsion through the use of a surfactant (emulsifier). This process is known as "emulsification" in chemistry. *Emulsification* is defined as the process of dispersing one liquid in a second immiscible liquid; this is analogous to milk being dispersed in water.

Aluminum is one of the principal additives used in emulsions to increase its energy. The addition of 5% aluminum theoretically will increase the energy of the emulsion by 25–35%; 10% aluminum will increase the energy to 40–60%. Over 10% aluminum is not cost effective.

Blasting

There are several theories as to what causes rock to break. A brief overview of the breakage theory to which I subscribe follows. To achieve the desired fragmentation and movement, one must be acquainted with breakage theory in order to understand and design the blast for optimum fragmentation. The breaking of rock involves two basic processes, radial cracking and flexural rupture.

Rock is stronger in compression than tension or sheer. Some rocks may have compressive strength of 150 MPa (21,000 psi) and yet be unable to withstand tension of 15 MPa (2100 psi) or less depending on the nature of the discontinuities. Considering this, it is reasonable that the easiest way to break rock is to subject it to tensile stresses greater than its ultimate strength in tension. However, this process can be complicated by the realization that rock formations are rarely homogeneous: The rock formation in one blast may contain different types of rock or, more importantly, the presence of discontinuities in addition to different rock densities. These discontinuities can make it difficult to determine proper location of the explosives to take maximum advantage of tensile stresses.

When the blast first occurs, the explosion creates a sudden application and quick release of high pressure, sending a shock wave through the rock from the borehole, after first crushing a small amount of rock immediately around the borehole. This compressive shock wave travels from the borehole though the entire rock mass as an elastic wave, with its velocity a function of rock density; the denser the rock, the faster the wave travels. The wave travels the distance from the borehole to the nearest free face (this distance is called burden) and is reflected back to the borehole. If the shock wave reaches a change in density, a portion of the wave will return to the borehole and the remainder will continue through the different material in a weakened state.

At every change in density, some of the impulse energy is reflected and refracted back to the source, while the balance of the wave continues through the different material. Particles will begin to spall away from the face if the wave emits energy in excess of the cohesive strength of the rock. This spalling, or breaking away of the rock particles, begins on the free face or at the change in density or discontinuity of the material and moves back toward the source, as does the reflected wave.

Proper fragmentation results when there is enough force in the compression wave to travel to the face and back, overcoming the tensile strengths of the medium through which it travels. It has been estimated that less than 5% of the explosive energy is used in the compression wave in the burden area and the compression wave is responsible for about 15% of the breakage. Along the face, the outermost edge is stretched in tension, which causes cracks (see Figure 5.9).

If the energy release is not great enough to travel to the face and return to the source, boulders will be found in the muck pile around the borehole. However, excessive energy will cause undesirable additional throw and overbreak. Also, boulders on the surface of the muck pile can be caused when the stress wave

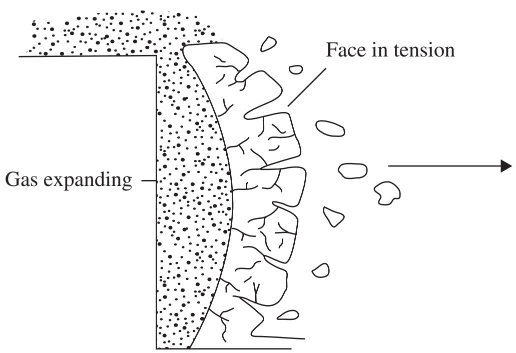

Figure 5.9 Face Being Stretched in Tension

is deflected because of density changes or discontinuities or overbreak from a previous blast. Instead of being fragmented by the blast, these rock fragments are simply pushed or rolled to the face.

The compression wave is short lived and will normally enlarge only short radial cracks that radiate out from the borehole axis. If the rock is homogeneous and therefore lacking many radial cracks, the compression wave will help with these radial cracks. In most rocks, however, the cracks already exist. In actuality, the shock wave primarily helps magnify the existing cracks and directs the energy that follows.

The second process in breaking rock is flexural rupture, bending the rock to the point that the outside edge, the side in tension, breaks. Flexural, or bending, is caused by the rapid expansion of the gases in the borehole. Gas is the major component necessary to cause the face to flexure and therefore the primary component of the fragmentation. This gas expansion exerts pressure against the cylinder walls. The sustained gas pressure drives the radial cracks through the burden to the free face and then causes the rock to displace in the direction of least resistance. This gaseous pressure applies the force that is necessary to cause flexural rupture of the rock and is responsible for the fracture of the rock in a direction perpendicular to the borehole axis. The rock will break following the naturally occurring planes of weakness: joints, cracks, and seams.

Flexural rupture of the rock is analogous to the bending and breaking of a beam. By treating the section of rock as a concrete beam, with depth equal to the burden and length equal to the bench height, or borehole length, it is apparent that to break the beam will require movement, or displacement, in addition to cracking. When explosives have been detonated and the radial cracks have been expanded, the gas starts the movement by putting a compressive pressure against the wall of the borehole. This pressure, as with a beam, will have least resistance to flexure at the center of the span. The action begins with detonation of the primer; therefore, the largest movement will favor the primer side of the center of the explosive column. As the expansive force of the gas moves against the beam, the beam bends. The bending of the beam creates tensile stresses at the side of the beam away from the explosive, the free face of the rock, which breaks the rock beam at weak planes caused by changes in the material, discontinuities, or radial cracks caused by the initial compression wave.

The principle of flexural rupture explains the relationship of the length of the borehole to the burden. Burden is the distance from the borehole to the nearest free face. By understanding the effects of flexural systems and varying movements, it is clear that the deeper the borehole, the greater the permissible burden and borehole spacing to achieve desired results. Like a column, the longer it is, the less resistance there will be to lateral forces.

The terms *geology*, *energy*, and *geometry* (GEG) explain the relationships necessary for blasting. *Geology* refers to the type of rock, the type and frequency of discontinuities, and the material in the discontinuities. *Energy* refers to the energy released by the explosives. Explosive energy will vary primarily in detonating velocity, gas production, and heat generated. *Geometry* is the relationship of the angle of the drill hole relative to the tunnel axis and the strike and dip of the discontinuities. The geology and the geometry are the most critical factors. Provided the geology and geometry are favorable, the rock can be adequately fragmented with high explosives of varying strengths and attributes.

One of the characteristics of the geology that is of interest for blasting is rock strength. Generally, the stronger the rock, the more explosive energy is required to break it. The effect of rock abrasiveness is principally a drilling consideration. Abrasive rock will increase bit wear.

The discontinuities provide the planes of weakness to facilitate breakage. The strike and dip provide the relationship of the bedding and foliation to the direction of the tunnel alignment and blast. The breakage is greatly affected by this relationship. Figures 5.10 and 5.11 illustrate the variations in geometry and their effect on blasting.

The first known industrial explosive was black powder used to blast in the Saxony mines in the seventeenth century. Thus, tunnels may have also been one of the early uses of explosives. It is believed that Stone Age man in Europe sank shafts and drove tunnels to find flint using deer's antlers and flint tools. However, the ability to tunnel in rock was nonexisting with antlers.

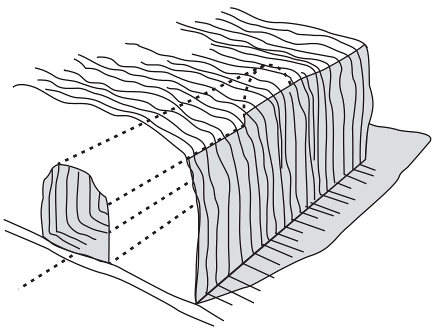

Figure 5.10 Tunnel Driven Perpendicular to the Strike (Széchy, 1973)

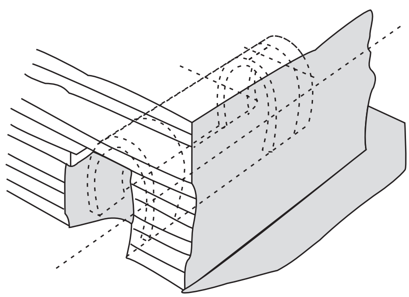

Figure 5.11 Driving Tunnel in Horizontal Bedding (Széchy, 1973)

Air Concussion and Vibrations Although not as critical underground as on the surface, legal ramifications related to blasting cause additional technical problems. Miners must break the rock while limiting the detrimental effects of blasting. Therefore, it is important that all those engaged in blasting have a basic understanding of the effects of blasting.

There a four causes of unwanted effects of blasting:

Air concussion

Airborne shock wave

Vibrations

Flyrock

Air concussion, or air blast, is caused by the movement of a pressure wave through the air; it is generally not a problem with underground blasting. The type of damage created by air concussion is broken windows. However, it must be realized that a properly set window glass can generally tolerate pressures of up to 13.8 kN/m^2 (2 lb/in.^{2}), whereas a wind of 160 kph (100 mph) produces a pressure of only 2.4 kN/m^2 (0.35 lb/in.^{2}).

Air concussion is caused by the movement of a pressure wave generally caused by one or more of three things: a direct surface energy release, a release of inadequately confined gases, and a shock wave from a large free face. Direct surface energy release is caused by detonation of an explosion on the surface. Detonating cord, when exploded, would be a source of energy release. Another cause is the premature release of gases into the atmosphere when a column of explosives is detonated due to poor stemming or mud seams or both. Finally, when there is a very large free face, as is often the case in quarry and coal mine blasting, the face movement can produce an airborne pressure wave.

Air concussion is also referred to as "overpressure," that is, the air pressure over and above the normal air pressure. The difference between noise and concussion is the frequency: Air blast frequencies below 20 Hz, or cycles per second, are concussions because they are inaudible to the human ear, whereas an air blast above 20 Hz is noise because it is audible.

Overpressure can be measured in two ways: kilograms per square centimeter (pounds pressure per square inch) or decibels. The decibel is a measurement of the relative difference of the power of sound waves. Pounds per square inch and decibels are related by the following equation:

$$\text{Overpressure (dB)} = 20 \log \frac{p}{p^\circ}$$

where

$$\begin{aligned} \text{overpressure (dB)} &= \text{overpressure in decibels} \\ \log &= \text{the common logarithm} \\ p &= \text{overpressure in lb/in}^2 \\ p^\circ &= 3 \times 10^{-9}\ \text{lb/in.}^2 \end{aligned}$$

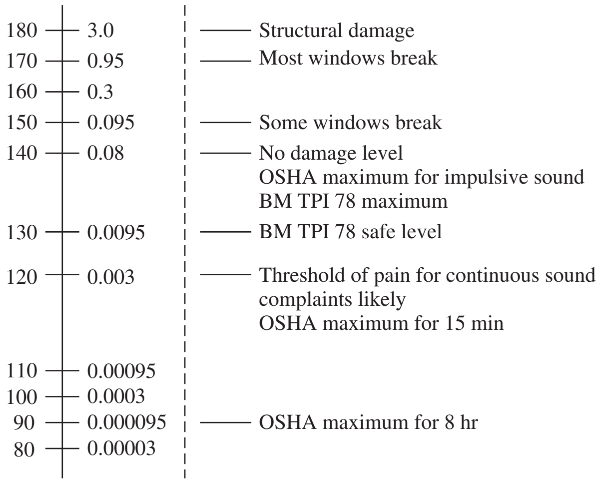

Figure 5.12 Selected Values of Overpressure as Ratios and in Decibels (From *Explosives and Rock Blasting*, Atlas Powder Company, Dallas, TX, 1987)

For example, if we have an overpressure of 0.5 lb/in.^{2}, the figure in decibels is

$$\text{Overpressure (dB)} = 20 \log \frac{0.5}{3 \times 10^{-9}}$$
$$= 164.4 \text{ dB}$$

Figure 5.12 is a chart of the equivalent of several values of overpressure numerical ratios and in decibels.

Unlike air concussion, airborne shock waves generally travel considerable distances from the site of the blast at the speed of sound. These shock waves are caused by an explosion on the surface that produces a pressure wave moving at supersonic speed for 20–50 diameters of the explosive before slowing to the speed of sound. Its total distance and intensity are governed by terrain, atmospheric conditions, and obstacles. Generally these pressure waves achieve the greatest distance traveling upward until they are dissipated in the atmosphere. Of course, the terrain and atmospheric conditions can affect the behavior of these pressure waves. Such was the case in San Antonio, Texas, when 50,800 kg (112,000 lb) of explosives was accidentally detonated. Most of the damage was 11–16 km (7–10 miles) from the explosion. In this case it was caused by an inversion layer at an altitude of 1500 m (5000 ft). As with air concussion, glass breakage is the first damage to occur; one will find no damage without glass breakage.

The most common type of blasting complaint is due to earth movement, or vibrations. Although noise is often what produces the complaints, vibration generally is the cause of damage, not flying debris.

When using explosives to break rock there are a number of effects: noise (which can produce complaints but generally not damage), total displacement in the immediate area around the explosive, plastic deformation, and elastic motion. Total displacement in the immediate area around the explosive is what one wishes to accomplish; it is intended that the affected rock change position or location and a permanent differential deformation occur, changing the size and shape of the rock.

The plastic zone is the area just a few feet beyond the planned displacement. Researchers still do not understand exactly what happens in this region; however, it is of no consequence to the usual blasting complaint.

Elastic motion refers to the back-and-forth vibration in which frequency and amplitude are predicable from the variables, terrain, and amount of explosives. This is the cause of the majority of blasting complaints. However, as one investigates further, it will be obvious that these complaints of damage are infrequently accurate.

When an explosive that is buried in the ground is detonated, most of its energy is spent shattering the rock or other materials around it. However, since an explosion is an imperfect use of energy, there is a loss of some energy transmitted through the earth in the form of waves or vibration. Some waves will still escape in the form of noise or concussion. There are two principal classes of waves: one that travels through the interior of the earth and another that travels on the surface. Approximately 95% of the earthborne energy reaches the surface a few feet from the total displacement area and acts as surface waves. These surface waves are analogous to the ripples that develop when a stone is cast into a pond. Acoustical waves are often referred to as seismic because their propagation characteristics are similar to earthquakes. They have much lower peak amplitudes and higher dominant frequencies than earthquake vibrations, primarily because of lower energy and smaller propagation distances.

The propagation velocities, amplitudes, and frequencies of both earthquakes and blasting are related to the elastic properties of the materials through which they travel.

The terms that describe the characteristics of vibrations are *particle velocity*, *amplitude*, and *frequency*. Particle velocity is the measurement of a point or particle on the ground resulting from vibratory motion. The peak is the maximum amplitude. The particle velocity is still the primary measurement to determine the potential for damage due to blasting operations. The peak particle velocity (PPV) is the number of complete waves that pass a given point per unit of time, usually a second.

The PPV is particularly sensitive to distance and the transmitting media. Vibrations in rock retain higher frequencies than in soil. Nondesign features affecting vibrations are topography and geology of the transmitting media and measurement site.

Amplitude is half the distance between the crest and trough (see Figure 5.13), measured in inches or millimeters. The most important design parameters for amplitude are explosives detonating in a time interval and distance from the

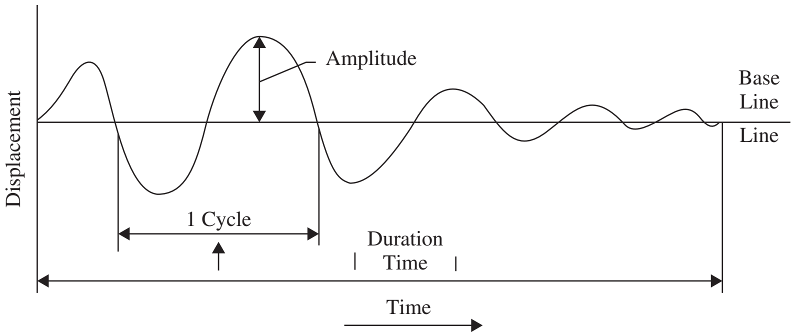

Figure 5.13 Vibration Wave Trace (Atlas Powder Company, 1987)

shot. Amplitude (displacement) is affected by four factors: the amount of explosives used per blast, the geologic material through which the wave passes, the distance between the structure and the blast, and the type of material under the structure.

Frequency is the number of complete waves that pass a given point per unit of time, usually a second. It is particularly sensitive to distance and the transmitting media. Vibrations in rock retain higher frequencies than in soil and the nondesign features affecting vibrations are topography and geology of the transmitting media and measurement site.

An easy way to visualize amplitude and frequency is to picture a small boat on water. As it moves up and down on the waves, the highest point the boat reaches is the crest and the lowest is the trough. Half the distance between the crest and the trough is the amplitude. The number of times the boat moves up and down in one second is called the frequency. In a structure, we find that the amplitude (displacement) is affected by four factors: the amount (weight) of explosives used per blast, the geologic material that the waves pass through, the distance between the structure and the blast, and the type of material under the structure. The most important design parameters for amplitude are explosives detonating in a time interval and the distance from the shot.

Vibration causes damage by differential displacement. As waves pass under a structure, they will lift the structure up and down, from side to side, and back and forth. However, if this movement of the structure could be in its entirety, there would be no damage. It is the differential movement that causes the damage. Theoretically, you could turn the structure upside down and set it back down in the original position without damage provided the structure moved monolithically. However, in practice, structures subjected to ground movement generally resist the movement, creating differential loading and therefore stress. Generally, while the lower portion is in motion due to vibration, the top of the structure is in its original position at rest. (See Figure 5.14.)

Vibration damage usually first appears as extensions of old cracks. Because plaster or locations where sheet rock has been taped are usually the weakest in a building, this is where new cracks first appear.

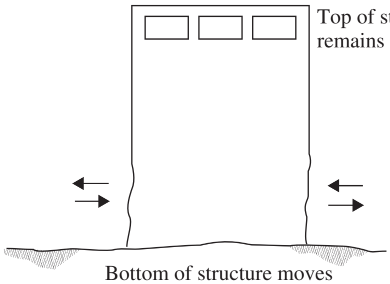

Figure 5.14 Differential Movement of Structure

There are common misconceptions about blasting and the damages caused by vibration. Usually people believe the louder the noise is, the greater the damage. There is not necessarily a relationship between the two. The main reason for peoples' concern about blasting damage is that the human body can readily feel the effects of vibration. Some people have been tested and have been found able to detect vibration at a level one one-hundredth of the level necessary to damage structures. Most of the time, normal activities will produce more vibration than blasting. (See Table 5.1.)

The scaled distance formula is used to estimate the amount of explosives that may be used without causing unwanted vibrations.

It correlates ground motion levels at various distances from the blasts. A scaling factor based on a dimensionless parameter is used. The total energy varies with the amount of explosives used. As wave generates out from the blast and more rock is exposed, distributing the energy over more surface and in turn reducing the peak ground motions.

The peak level of ground motion at any given point is inversely proportional to the square of the distance from the shot. An empirical scaling relating peak particle velocity to scale distance has been developed.

The scaled distance combines the effects of the total charge weight per delay relative to increasing distance influencing the generation of seismic or air blast energy. Based on monitoring large numbers of blast, safe scale distance parameters were established. The formula correlates ground motion levels at various distances from the blasts. The scaling factor based on a dimensionless parameter is used. The total energy varies with the amount of explosives used.

Scaled Distance Formula For a site where no instrumentation readings were made, the scaled distance formula the scaling factor used is 50:

$$\frac{D}{\sqrt{W}} \geq 50 \text{ ft/lb}$$

where D = distance from blast to nearest structure (ft)
W = weight of explosives in pounds per delay

Table 5.1 Vibrations Caused by Some Normal Activities

	Particle Velocity in Room, in./sec[a]			Particle Velocity in Adjacent Room. in./sec		
Activity	Radial	Vertical	Transverse	Radial	Vertical	Transverse
Walking	0.00914	0.167	0.372	0.00129	—	0.00102
	—	0.0578	0.0155	0.00167	0.0281	0.00227
	—	0.00770	0.00210	0.00229	0.0626	0.00462
	0.0600	0.120	0.0300	—	—	—
	0.0100	0.0600	0.007	—	—	—
	0.00600	0.0110	0.00400	—	—	—
	0.00800	0.0200	0.00700	—	—	—
Door closing	0.0110	0.0558	0.0149	0.00170	—	0.00193
	—	0.0150	0.00500	0.0125	0.0970	0.00953
	0.008	0.0100	0.00800	—	—	—
Jumping	0.0524	4.03	1.05	0.120	0.219	0.551
	0.120	0.219	0.551	0.0153	0.0239	0.0101
	1.00	2.500	1.70	0.00450	0.0100	0.0045
	0.500	5.00	1.10	—	—	—
Automatic washer	0.00340	0.00400	0.00340	—	—	—
Clothes dryer	0.00500	0.00500	0.00500	—	—	—
Heel drops	0.0100	0.0100	0.0100	—	—	—
	0.0800	0.600	0.0300	—	—	—
	0.0200	0.200	0.0200	0.006	0.0100	0.006
	0.900	3.500	0.400	—	—	—
	0.0500	0.450	0.0700	0.009	0.014	0.008
	0.0100	0.200	0.00900	—	—	—

Source: From "Blasting vibrations and their effects on structures," *Bureau of Mines Bulletin* 656, 1971, p. 21.
[a] 1 in./sec = 0.024 in./sec.

For sites that were instrumented with peak particle velocities less than 2 in./sec, the equation uses a scaling factor of 20:

$$\frac{D}{\sqrt{W}} \geq 20 \text{ ft/lb}$$

Converted to tabular form, we get Table 5.2.

In some cases a scale distance factor of 60 is used. At many locations the maximum peak particle velocity permitted is now 1.0 in./sec whereas it used to be 2.0 in./sec. In some cases contractors and miners are allowed only 0.5 in./sec.

The U.S. Bureau of Mines did a study (USBM 1980) to determine the causes of blasting damage. The study concluded that particle velocity is still

Table 5.2 Difference in Maximum Allowable Charge Weights for Scale Distance Factors of 20 and 50

Distance to Structure		Maximum Weight of Charge Permitted for: $D/W^{1/2} = 50$		$D/W^{1/2} = 20$	
m	ft	kg	lb	kg	lb
33	100	1.8	4	11.4	25
150	500	45.4	100	2,955	6,500
305	1,000	182	400	1,136	2,500
610	2,000	727	1,600	4,545	10,000

Source: Adapted from Atlas Powder Company (1987).

the best descriptor of the ground. It is the most practical descriptor for regulating damage potential for a class of structures with well-defined response characteristics. A peak particle velocity of 0.75 in./sec is the practical safe criterion for low-frequency blasts for sheet rock houses and 0.5 in./sec for plaster. For frequencies of 40 Hz and above, a peak particle velocity of 2 in./sec is recommended.

For uninstrumented shots, a scale distance factor of 70 should be used. This should yield a PPV of 0.08–0.15 in./sec.

Frequencies lower than 40 Hz have a higher potential to cause damage than higher frequency blasts; the high frequencies are generally associated with construction blasts.

The type of home construction is a factor in blast damage. Also, all homes eventually crack from various environmental factors.

The chance of a peak particle velocity below 0.5 in./sec causing damage is not only small but also decreases more rapidly than the entire range of vibration levels.

The graph in Figure 5.15 illustrates how lower frequencies, generally below 40 Hz, require lower peak particle velocities to prevent damage.

The U.S. Office of Surface Mining (OSM) established a regulation regarding the use of explosives for controlling ground vibrations and air blasts. Although intended for surface coal mining industry operations, it is also used for other blasting. The OSM provided three options for controlling the effects of blasting regarding vibrations and air blasts. The operator was permitted to select any one of the options. The three methods are:

Method 1—Limiting particle velocity criterion

Method 2—Scaled distance criterion

Method 3—Blast level chart criterion

Method 1 Method 1 requires that the operator monitor each blast with a seismograph. If the peak particle velocity does not exceed the limits in Table 5.3

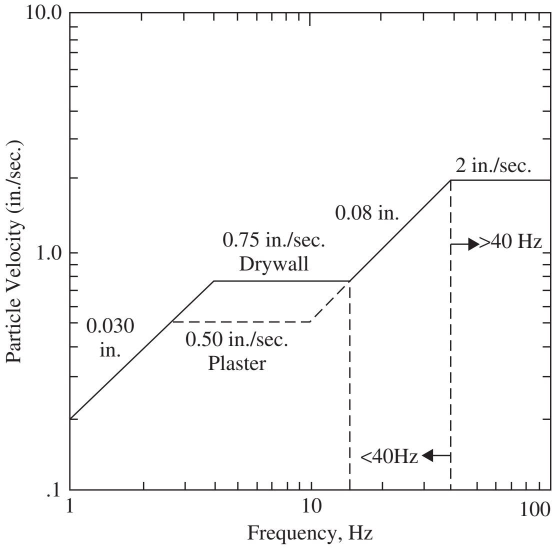

Figure 5.15 Safe Levels of Blasting Vibration for Houses Using Combination of Velocity and Displacement (USBM Recommendation RI 8507, 1980)

Table 5.3 Maximum Permitted Particle Velocities and Scaled Distance Factors According to Distance

Distance from Blast (ft)	Scale Distance Factor without Seismic Monitoring	Maximum Peak Particle Velocity
0–118 m (0–300 ft)	50	1.25
119–1970 (300–5000 ft)	55	1.00
Greater than 1970 m (greater than 5000 ft)	65	0.75

with regard to maximum peak particle velocity based on the distance of the structure from the blast site, the operator is considered in compliance with the regulation.

Method 2 Method 2 requires that the shot be designed in accordance with the values given in Table 5.3, which specifies a scaled distance factor to be used for various distances between the structure and the blast site. Scaled distance varies with distance to reflect varying limits of peak particle velocities. No seismic recording is required provided scaled distance factors are used. If the scaled distance factors of the OSM are used,

$$\frac{D}{\sqrt{W}} = \mathrm{SD}$$

where SD = scaled distance factor from Table 5.3
D = distance from dwelling to blast site in feet
W = weight of explosives in pounds per delay

The safe distance equation is

$$D = \text{SD} \times \sqrt{W}$$

For example, using a scaled distance factor of 50 and 30 lb/delay, the distance the blast must be from the house is

$$D = 50 \times \sqrt{30} = 273 \text{ ft } (107 \text{ m})$$

The equation to determine the maximum permissible explosives for a certain distance is

$$W = \left(\frac{D}{\text{SD}}\right)^2$$

Assume a home is 500 ft from the blast site and the scaled distance factor is 55. Then the maximum permissible amount of explosives is derived as

$$W = \left(\frac{500 \times 500}{55 \times 55}\right) = 82 \text{ lb } (37 \text{ kg})$$

Method 3 If the design method parameters in Table 5.3 are too restrictive, method 3 allows the operator to use peak particle velocity limits that are relative to frequency as represented in Figure 5.16. The lower the frequency, the lower the peak particle velocity must be. Conversely, higher frequencies permit higher peak particle velocities. However, the method requires a frequency analysis of the ground vibrations generated by the blast as well as a measurement of the peak particle velocity after each shot.

Although there are some differences between the two figures, peak particle velocity that is below the solid line is considered safe and will not cause damage to the structure.

Studies of the effects of blasting on tunnels indicate that the tolerance of tunnels to vibrations is high. Langefors et al. (1948) asserted that rock falls underground will occur when the peak particle velocity exceeds 12 in./sec and fractured the rock requires a peak particle velocity of 24 in./sec.

Table 5.4 from work of Dr. Peter Calder Professor of Mining, Queen's University at Kingston, 1970 -1996 and Professor Alan Bauer, also of Queen's University represents the predicted damage criteria for rock masses based on stresses from peak particle velocities in ground motion due to blasting.

There are things that can be done in the blast plan to reduce adverse vibrations when blasting. The most common is to reduce the amount of explosives

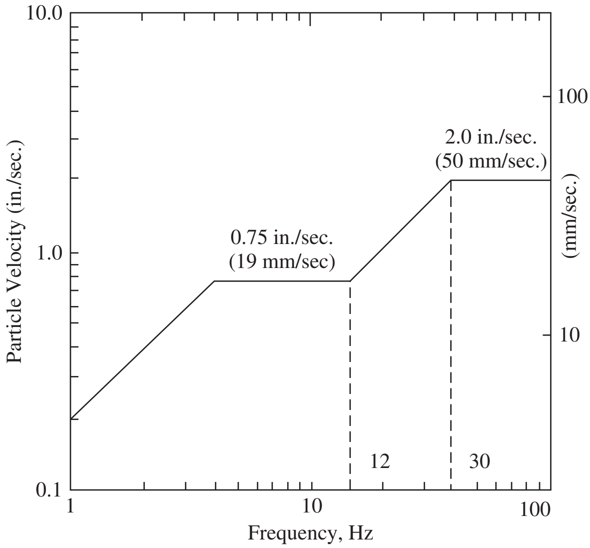

Figure 5.16 Particle Velocity versus Frequency as a Blast Damage Indicator (USBM, 1980) (Note that the OSM regulation using method 3 is slightly different than USBM (1980) RI 8507. For example, the threshold for permitting a peak particle velocity of 2.0 in./sec is 40 Hz for the USBM and 30 Hz for the OSM.)

Table 5.4 Damage Criteria for Rock Masses

Peak Particle Velocity (in./sec)	Effects on Rock Mass
Less than 10	No fracturing of intact rock
10–25	Minor tensile slabbing will occur
25–100	Strong tensile and some radical cracking
Greater than 100	Complete breakup of rock masses

Source: From Bauer and Calder (1978).

per delay. This can be done by increasing the number of delays or reducing the length of the round. Also, one can implement to OSM criteria.

The PPV can be predicted using the graphical relationship of the historic PPV and SD and the slope of the line that illustrates the relationship.

Flyrock Flyrock is the undesirable throw or movement of rock or debris from the blast area. Good blast design is the primary method of avoiding flyrock. Although one generally thinks of flyrock in surface blasting, it is still a consideration in tunnel blasting. Unplanned throw has the potential to damage your mining plant, for example, by destroying the vent line or waterline. An important consideration is the effect in ground that requires extensive support.

Although, as we learned earlier, the resistance to vibration damage is high in a tunnel, flyrock can hit steel sets near the face, tearing them out and leaving a difficult, if not dangerous, situation.

Tunnel Driving by Blasting

Tunnels can be in all shapes and sizes, including round, horseshoe, square, and rectangular, just about any geometric shape (see Figure 5.1). The size and shape of the tunnel will sometimes be the determining factors in the tunneling methodology. The most common drill blast tunnel is horseshoe shaped. The reasons for this are threefold: first, the roof, or crown, forms an arch facilitating self-support; second, the flat invert allows better mobility for equipment and muck haulage; and third, it is easier to muck a flat invert. Tunnel shape and length are the two primary considerations on whether to use blasting as the tunneling method.

Tunnels generally require holes drilled perpendicular to the face or at sub-angles to the tunnel axis. The shot design must create a free face to achieve adequate fragmentation. It requires a higher powder factor than surface blasting and is greatly influenced by GEG. Do to environmental and space constraints there are practical maximum cuts.

The theory involved in tunnel drilling and blasting is basically the same as for surface blasting. However, because of the limited geometry and confining underground conditions, some of the blasting problems are increased. When blasting, installation of utilities should be during drill or support cycle for safety and advance. These include hanging of ventilation pipe, waterlines, and air lines; laying track; and resupplying consumables to the heading. If the project is large enough, there will be a separate crew, called the “bull gang,” to do much of this support work and to install the utilities with protection.

A tunnel may be advanced by the full-face method, the top-heading and bench method, or the pilot tunnel method. The full-face method is advanced with a full round, that is, the entire cross section is drilled and blasted with each round. Therefore, the heading, or face, advances one complete round with each production cycle. The full-faced method is best for small- or medium-size noncircular hard rock tunnels.

Full-Face Method With a full-face advance most of the holes are drilled at angles to the face. Because the tunnel face is the only face, a method must be devised to create a free face, a second face, a face that the rock can break to. Methods may be employed to create a free face artificially to enable a reduction in the powder factor and improve breakage.

The methods employed require techniques involving the placement of drill holes to develop the second face. The names of the various types of drill holes in a tunnel round are related to their location (see Figure 5.17). The cut holes are made at the center of the face and are primarily responsible for the breaking and movement of rock in the center of the face. This is the initial step in creating

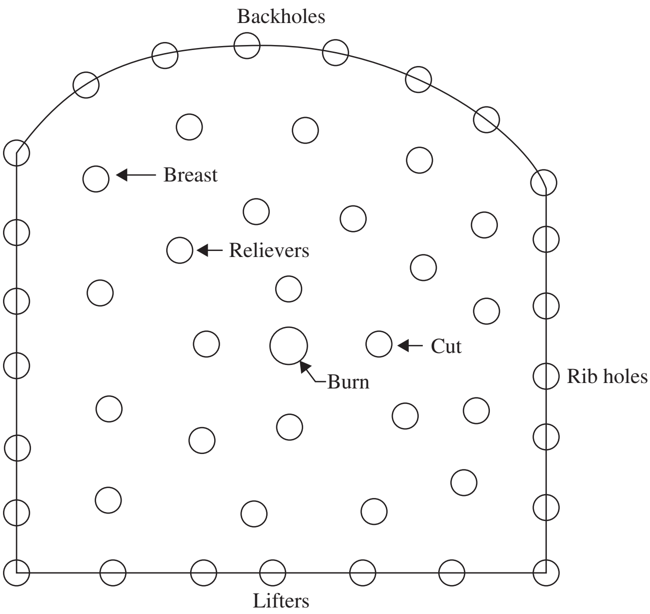

Figure 5.17 Blasthole Nomenclature by Location

the free face. The reliever holes are those that help or aid the breakage of rock after following the cut holes. They detonate just after the cut holes and further increase the free face. The trim holes are on the perimeter of the tunnel.

The names of the trim holes are related to their location in the round. The trim holes at the top of the round are called "back holes" or "crown holes." The holes on the side of the tunnel are called "rib holes," and the holes located at the invert are called "lifter holes."

Breast, or enlarger, holes are those blast holes that are between the reliever and the trim holes. They are drilled parallel to the tunnel axis, and generally there are fewer breast holes than other types of drill holes.

For smaller tunnels, the best way to advance the face by blasting is the full-face method. With the full-face method, the round consists of the entire face being drilled and blasted, thus advancing the entire face with one shot. The small tunnels are often drilled with jacklegs. As the tunnels get bigger, there is a point that drill jumbos become more effective. Depending on ground stability, vibration considerations, and tunnel mucking and support, larger tunnels can be driven by the full-face method.

For medium size tunnels using the full-face, it will probably require that the ground support be installed from the muck pile or a platform. Using full-face in medium size tunnels will very much depend on the equipment available and the rock quality. Medium tunnel equipment may require additional capital. It may be much better to save capital by top heading and benching.

With the full-face advance, most of the holes are drilled at right angles to the face. However, because of conditions, a method must be devised to create a free face to break toward. As we have seen, a confined blast such as a tunnel or shaft round requires a very large powder factor because of the lack of area to which to break. Methods may be employed to create a free face artificially to enable a reduction in the powder factor and improve breakage.

Advanced Heading An advance heading is when the tunnel is not driven full faced; that is, one portion is driven in advance of the rest of the face. This will be discussed more in Chapter 13.

Top Heading The most common advance heading method is top heading. The major reason for using the advance heading method instead of the full-face method is that the crown might be too wide or not stable enough to support itself. When top heading is used, the installation of the roof support can begin immediately. (See Figure 5.18.)

In bad ground situations, it may be necessary to keep the bench as close as possible to the top heading. The bench may be drilled from the top with vertical holes or from the invert with horizontal holes. Drilling from the bench requires subdrilling and thus will leave a rough invert that may require additional concrete. However, because of additional work area, it often allows more drilling, thereby increasing production. Also, if the bench is long enough, shots can be predrilled leaving a space between to be completed when the shot is ready for loading. In this way, much of the drilling can be done prior to needing the shot, whereas horizontal drilling requires that most blast holes must be detonated with the shot at once. Figure 5.19 illustrates top heading and horizontal drilling.

Although drilling the invert may be slower, sometimes the extra time is compensated for because the side trimming is easier, having fewer "tights."

Generally, the top heading is driven first, then the bench follows a workable distance behind; however, the entire top heading may be driven before the bench

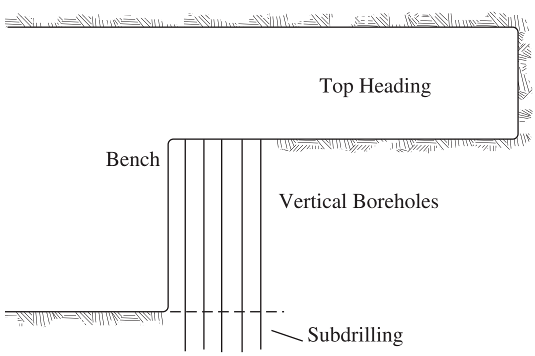

Figure 5.18 Top Heading and Bench with Vertical Bench Holes (Atlas Powder Company, 1987)

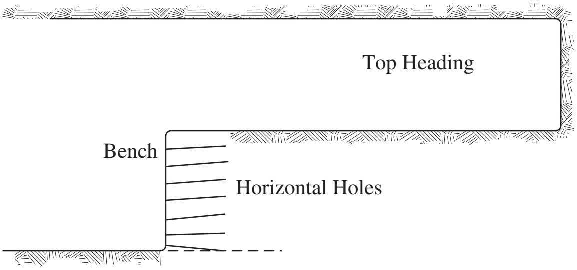

Figure 5.19 Top Heading and Bench with Horizontal Bench Cut (Atlas Powder Company, 1987)

is started. The decision of whether to drive them simultaneously is dependent on the following:

1. Length of tunnel
2. Muck disposal
3. Type of lining
4. Comparative cost

Pilot-Tunnel Method Another method of partial advance is a pilot tunnel. A smaller pilot tunnel is driven down the centerline of the larger tunnel, generally the entire distance or from shaft to shaft; then the tunnel is enlarged using the pilot tunnel as a large burnhole. Generally, the enlargement is drilled from the pilot tunnel with holes perpendicular to the tunnel axis. However, when overbreak or tights are a problem, it may be more desirable to drill holes parallel to the tunnel axis from the face. (See Figure 5.20.)

Parallel (Burnhole) Method The parallel-cut method of blasting a round first developed as a method utilizing a series of loaded and unloaded straight holes

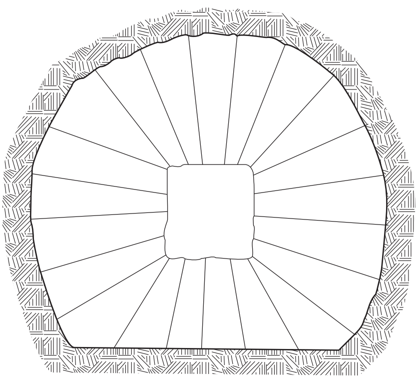

Figure 5.20 Pilot Tunnel

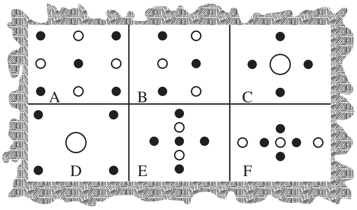

Figure 5.21 E, F: Burn Cuts; A, B, C: Line Cuts; D: Box Cuts (Atlas Powder Company, 1987)

parallel to the direction of the advance or tunnel axis drilled on 100–150-mm (4–6-in.) spacings (see Figure 5.21). These closely spaced holes offer a plane of weakness that results in the loaded holes breaking. In this way, the centerpiece, the area between the holes, becomes a cavity for the rest of the round to break toward when it fires after the empty holes. Note in Figure 5.21 that the empty holes provide an area to which the loaded holes can break. For the cut to be successful, it is important to have appropriate timing (delays) between holes. The holes closest to the parallel holes detonate first, with the detonation of subsequent holes firing after the one before it. Should the timing be incorrect or too many holes around the burnhole fire simultaneously, the movement of the broken rock causes the cylinder to have so much rock moving into it that it clogs, causing the broken rock to be bound against itself, restricting the movement of the other rocks. Likewise, being the first holes to be detonated, if the empty holes were loaded, the rock surrounding them would push against the rock surrounding the charged holes and prevent the rock from moving away from the face. The primary consideration to allow sufficient time and sequencing to prevent the rock from plugging the opening and freezing in place is a well-designed delay pattern. This lack of movement is called "freezing." To minimize freezing of the burn cut, additional empty relief holes or large-diameter holes can be used to increase the void area for greater swell relief. Make certain that the hole is completely loaded. However, one has to be concerned with air blast from lack of borehole pressure retaining time due to lack of stemming. The number of explosives per foot can be reduced by using a smaller diameter explosive cartridge. Both the spacing of the burn cut holes can be changed and the delay timing can be extended to permit more time for rock to clear prior to the next detonation.

The burn cut can also be accomplished by drilling one larger borehole to serve as the burnhole. This drilled cavity serves as a free face for the surrounding to break toward. This method is advantageous, because the advance is not as restricted as with small burnholes. In other words, the advance may be longer with a larger diameter burnholes. There are two classifications of burn cuts: line cuts and box cuts. In larger headings box cuts are more common; line cuts are preferred in small headings where jacklegs are used. Figure 5.21 illustrates various burn-cut patterns.

Sometimes if a drilled and properly loaded round does not pull to the planned depth there are a few potential causes. It could be that the holes are out of alignment and thus the drilling must be improved to maintain alignment. Sometimes the geology can change considerably. To allow for the change in geology, one may increase the void space provided in the burn and increase spacing. To eliminate cross-propagation when using NG-based explosive, which can eliminate the delay sequence in the cut, freezing the burn, non-NG dynamite should be used.

However, if water-based explosives are used, movement of the rock may have squeezed the explosive cartridge below its critical diameter, or the density of the explosive may have increased due to the shock wave from a hole with an earlier delay reducing its sensitivity. The phenomenon is referred to as "precompression" or "dead pressing." This can be reduced by using a burn/crater cut, in which the lead holes are drilled closer to the burnhole than is done for a regular burn and is fired instantly or with a delay sequence of 1–2, rather than the usual 1–2–3–4. See Figure 5.22.

For drilling with hand-held drills, such as jacklegs, suppliers sell specially adapted larger bits for burnholes. When drilling 38–51-mm (1½–2-in.) drill holes, one can drill 76-mm (3-in.) burnholes using the same type of drill steel. With jumbo drills, burn-cut holes can be 200 mm (8 in.) in diameter. Keeping the drill holes straight and parallel is important for success. Using modern computer-controlled drills helps to achieve this with relatively little difficulty. In smaller drifts, when using a jackleg, placing a loading pole in an adjacent hole will help the driller to maintain alignment.

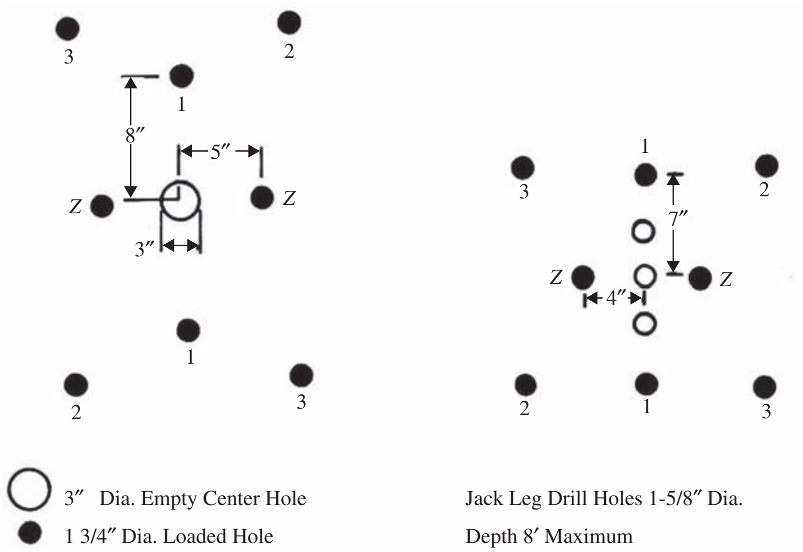

Figure 5.22 Burn/Crater Cut

Advantages of Burn Cuts The burn cut offers several advantages in small- and medium-sized headings:

1. Because of the direction of throw with a burn cut there tends to be less flyrock than with a V or wedge cut.
2. The burn cut is drilled straight, permitting more drills at the heading. By the same token a wedge cut restricts advancement because of the inability to get greater depth and a proper angle.
3. One can use one total length of steel for all holes.
4. The geometry of blasthole placement is less critical for the burn cut than for the angle cut.

Coromant Cut The coromant cut consists of a drill hole pattern: two overlapping holes about 5.5 cm ($2\frac{1}{4}$ in.) in diameter are drilled in the center of the face and left uncharged; they form a slot roughly 10 cm × 5 cm (4 in. × 2 in.) looking much like Figure 5.23. The advantage of this type of burn cut is that the area of the free face is greatly increased. When the various burn-cut holes are being drilled, care must be taken to drill the holes straight. With the spacing of the loaded holes so close, the chance of sympathetic detonation is quite high (sympathetic detonation is the detonation of one hole by a blast in an adjacent hole). Also, if the holes are drilled inaccurately, the chance of having bootlegs—of not having the rock break the entire length of the borehole—is greater. It must be stated that it is quite difficult to achieve the drilling accuracy required for the coromant cut.

Angle Cuts Angle cuts are those cuts that have holes drilled at an angle to the face. The wedge cut, also called "V" or "plow," is drilled symmetrically at approximately 60°. Because of the geometry breakage, the free face is longer, lending to some bending action; thus the wedge cut requires fewer holes and a lower powder factor than the burn cut (see Figure 5.24).

The wedge cut is particularly well suited for well-laminated or fissured rock. It is generally used on wide tunnel faces or in underground rooms where the holes can be aligned with the axis of the tunnel.

A problem with the wedge cut is that, being somewhat larger than the other fragments from the blast, the rock from the wedge can cause considerable

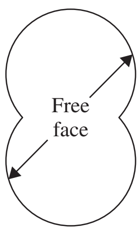

Figure 5.23 Coromant Cut

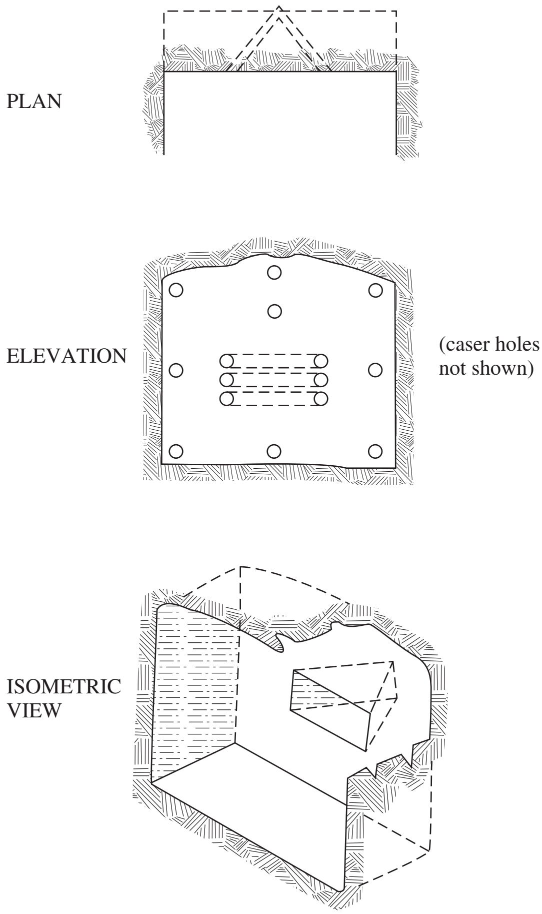

Figure 5.24 Wedge Cut (Gregory, 1984)

damage because, as it is catapulted from the face, it has enough mass to inflict damage to steel of timber sets. A method to avoid this is to place a small shallow borehole (a buster hole) in the center of the wedge so it is broken down into smaller fragments as the face is detonated.

The wedge cut will generally pull 2.5–3.6 m (8–12 ft) round for a face area of approximately 10–20 m^2 (100–215 ft^2) This can vary depending on rock type, number and orientation of discontinuities, drill pattern, and size of the drill holes; however, the drilling accuracy is quite important. The wedge cut can be made with two or three rows of holes using the V, or wedge, at any angle to the centerline of the face. The V can be horizontal or vertical, depending on which direction allows the greatest angle and, more importantly, the direction and angle of the foliation or bedding planes.

The fan cut is approximately one-half of a wedge cut. Because the angle of drilling restricts drill space, it is not used often. It is usually limited to pulling

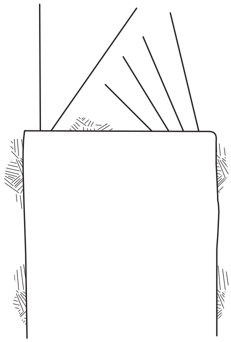

Figure 5.25 Fan Cut

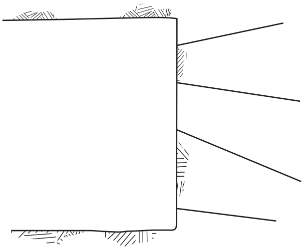

Figure 5.26 Drift Round Using Bottom Draw

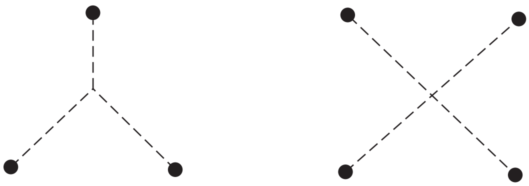

Figure 5.27 Pyramid Wedge (Center Cut) (Gregory, 1984)

a 1.2–1.5-m (4–5-ft) round and only where one drill is used in the heading. All the holes are drilled at different angles from one location near either rib. (See Figure 5.25.)

The draw-hammer cut is another modified V-cut in which the holes are drilled away from the center of the face. It is best suited in small tunnels and raises, where there is not much room for drilling. (See Figure 5.26.)

The pyramid cut is similar to the wedge cut, only it has three or four sides. The pyramid cut is used primarily in sinking circular shafts. (See Figure 5.27.)

When sinking shaft, pyramid cut holes should be deeper than the other holes to provide a sump when drilling the next round. Because of the size of the round, another pyramid cut, called a “baby pyramid,” is required within the perimeter of the larger pyramid cut to aid the blast geometry and reduce the size of the pyramid wedge rock fragment. (See Figure 5.27.)

Patterns for Tunnel Headings The boreholes must be deeper than the desired round pull length. Depending of the round length, the holes should be drilled one half to one times the burden length deeper. That is, if the round is 3 m (10 ft) long and the burden is 550 mm (22 in.), the total length of the borehole drilled will be approximately 3.28 m (11.0 ft) to 3.55 m (12.0 ft). Figure 5.28

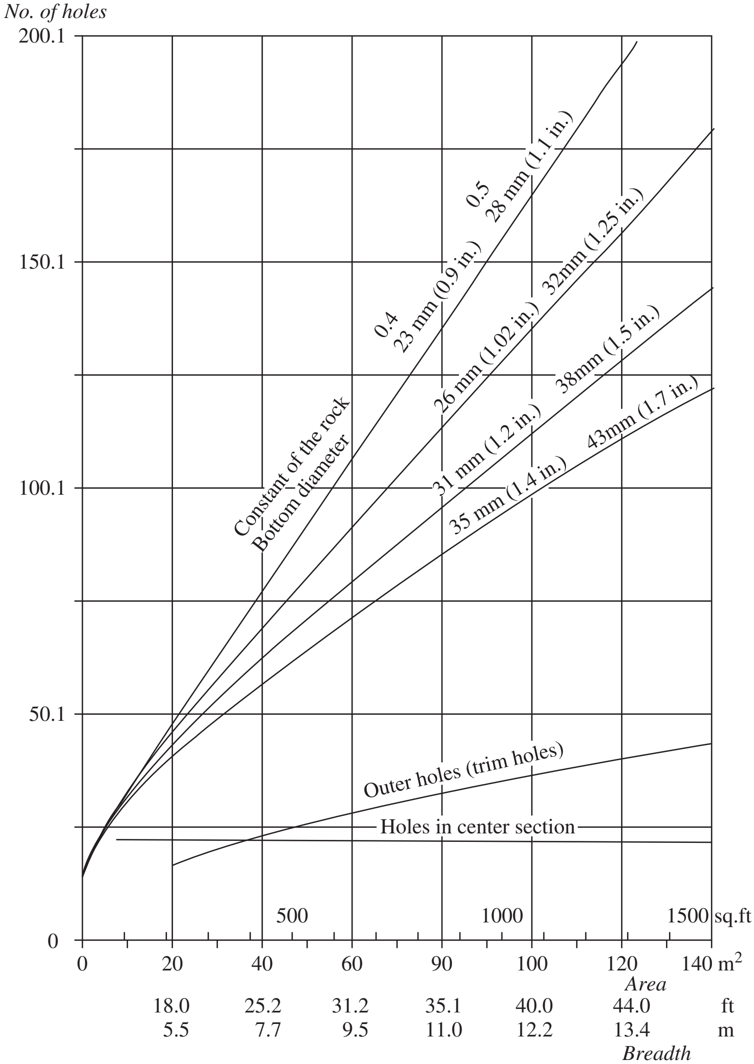

Figure 5.28 Least Number of Holes as Function of Face Area (Langefors and Kihlström, 1978)

is a graphical representation of the least number of holes as a function of the tunnel face area with parallel hole rounds for different bottom diameters. The curves include holes for smooth wall blasting.

The depth of cut relative to the width of the tunnel varies with requirements. The optimum breakage will occur when the length of the round is between 0.5 and 0.7 times the tunnel width and it should not exceed the tunnel width. Using 0.6 for a tunnel 5 m (16 ft) wide, the round length can be around 3 m (10 ft). The optimum breakage may not be the blasting requirement. If the mucking equipment can handle less than optimum breakage, perhaps having the round length equal the tunnel width may provide marginal breakage in which there are boulders but, for example, because of reduced drilling setups, there is an increase in production. Vibration consideration may necessitate shorter rounds. Ground stability may require a shorter round. However, in larger tunnels sequential excavation may be required. Figure 5.29 is a graphical representation

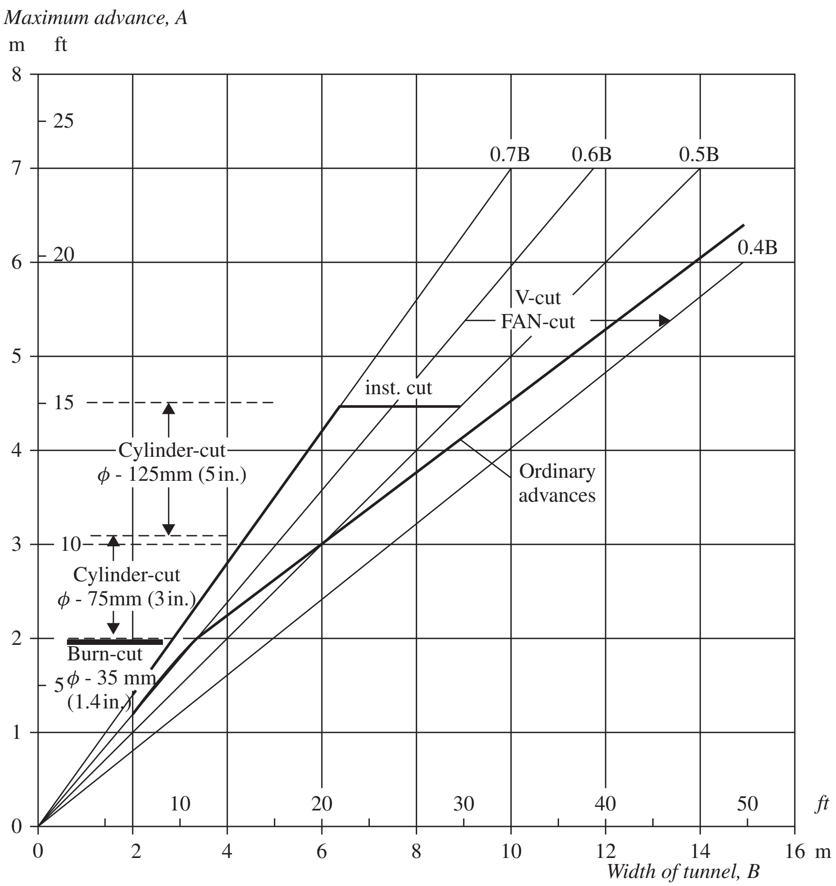

Figure 5.29 Maximum Advance per Round as Function of Width of Tunnel (Langefors and Kihlström, 1978)

of the maximum advance per round as a function of width of the tunnel for various types of cuts.

For V-cuts and fan cuts the values are between 40 and 60% of the width (shaded area). The percentage for the "ordinary advances" falls with increased depth due to increased influence of the deviation in the drilling. "Inst. cut" indicates WP-cuts and pyramid cuts (Langefors and Kihlström, 1978).

The drill pattern selected is determined by the size of the face to be mined and the GEG. Each drilling situation is varied, so hard-and-fast rules of thumb for borehole spacing are not practical. However, rules of thumb for space determination can be made to give an estimate. By no means should these rules of thumb be considered appropriate to be put into practice. The principal benefit for the rules of thumb is estimating the work involved for scheduling and cost estimating. Borehole spacing should be determined based on the experience of a qualified individual. These techniques we are about to discuss are for demonstration purposes only; they are empirical formulas and should not be taken literally.

In some cases with a burn cut, the burden is equal to the borehole diameter in millimeters (inches), D, multiplied by the constant b, where b can range from about 12 to 16 or a wider range depending on the conditions. For example, to estimate the burden and spacing, the following equation can be used:

$$\text{Burden} = Db$$

where $D = 44\,\text{mm}$ (1.75 in.)
$b = 13$

Therefore,

$$44 \times 14 = 616\ \text{mm}\ (24\ \text{in.})$$

Obviously, if the borehole diameter is smaller, the burden and spacing will be less. Conversely, a larger borehole will yield larger burden and spacing.

Using the results of the equation above and applying a spacing-to-burden ratio (SB) of 1.4 yields

$$\text{SB} = 1.4 \times 616\ \text{mm} = 862\ \text{mm}\ (34\ \text{in.})$$

These values, or rules of thumb, are generalizations and should not be construed as exact results. They are used to demonstrate how the burden and spacing may be determined in the stated examples and can assist in cost estimating. This approach is good for cost estimating, but it should not be considered an accurate method of drill pattern design.

The preceding example is for medium-hard to hard rock and provides a small burden which may be less than actually needed. The constant b can be adjusted to reflect the conditions. For softer rock or if the geology and geometry are very favorable, the constant be may be increased. It is possible that in soft rock the burden and spacing ratio may be 20% greater than determined in the example above. For example, if the constant b is increased to 15.6 and using the same burden and spacing (SB), 1.4, the burden will be 740 and 1036 mm (29 and 41 in.), respectively. This and higher values of b may be more suitable for larger tunnels with large face cross sections, longer holes, and more favorable geometry. It can also reflect a larger borehole diameter.

Stemming It is necessary to confine the charge to localize the effects of the gases produced by the explosive reaction. Stemming refers to an inert material that is placed in the borehole between the top of the explosive column and the collar of the hole. Stemming material can consist of sand, drill fines, gravel, or pea stone; sandy clay is the most effective. It is important that all holes be stemmed to help seal or delay the escape of expansive gases through the top of the borehole, so that the explosive's efficiency is increased. Increasing the explosive's efficiency reduces the amount of explosives required. If the gases are not properly confined, the results can be flyrock, increased ground vibration, and air blast. The reduction in efficiency causes poor fragmentation and boulders.

Stemming material can also be such manufactured products as Squelcher™, which seals the borehole. It is an inert, nonexplosive gel that is available in diameters ranging from 25 to 50 mm. An advantage to these is they are pliable and can be held in the borehole by tamping with a loading pole. These are particularly useful for underground blasting.

When a borehole is improperly stemmed or not stemmed at all, the resulting action is called "rifling." Rifling gets its name from the action of blowing the stemming material, much like a rifle. The action desired in blasting is the opposite of that with a rifle. A rifle is designed to eject a projectile when the gunpowder explodes. However, if the round had a very strong explosive, say NG dynamite, instead of gunpowder, the projectile would be ejected from the barrel but the rifle would blow apart. If the barrel was plugged, the projectile would not be ejected; instead, the only action would be the destruction of the rifle. This is the effect desired in a borehole—the destruction of the borehole, not the launching of projectiles. If the stemming material is insufficient, a projectile will be launched in the form of either some or all of the stemming material and possibly some of the rock at the collar. When no stemming material is used, pieces of rock break off from around the collar hole.

Stemming material should be clean and free flowing for vertical holes. Combustible materials should not be used. The stemming materials must be inert. When loading above or horizontally, tamping bags, tamping plugs, or clay dummies will hold the stemming material in the hole. Sand can be poured into cylindrical paper bags that will fit into the borehole. The bags containing the sand are tamped in the hole. This is rarely used anymore. It is time consuming, and there are better stemming materials.

The best shape of stemming material to use in larger holes is angular-type stemming, such as crushed peastone. Not only is the peastone convenient to use because of its availability, but also its density causes it to settle faster than sand in wet holes. Because of its angular shape, it is also best at resisting ejection of the borehole. Angular stemming material tends to bridge across the borehole as the gas expansion pushes against it. This pushing by the expansive gases causes the stemming material to move slightly, wedging itself against the wall of the borehole, with high gas pressure causing the angular points to gouge deep into the wall of the borehole, restricting further movement. Because the high gas pressure is short lived and pushes against the stemming, there is little gas leakage, even though the stemming material is quite porous.

Tests have suggested that the size of the stemming particle affects the performance of the stemming (Konya, 1978). A borehole-to-particle-size ratio of 17:1 appears to be the best. That is, 6.3-mm (0.25-in.) peastone would be the optimum size for a 100–115-mm (4–4.5-in.) borehole.

Table 5.5 gives the ideal crushed rock sizes based on borehole diameter.

Generally, the amount of stemming material required will range from $0.7B$ to $1B$, where B represents the burden. Therefore, in the case of burdens of 2.4 m (8 ft), the amount of stemming material will be between 1.7 m (5.6 ft) and 2.4 m (8 ft). That is the ratio of stemming material to burden can vary through this range, depending on existing conditions. At times, when blasting a tunnel face or shaft, one may consider that the stemming material is reduced to $0.5B$. However, this would be a very rare instance and is not recommended.

The geology and geometry are what will most likely affect the amount of stemming material. It is the same as the rock and rock mass characteristics considered for borehole burden and spacings, that is, the type and strength are used in the amount and nature of discontinuities, strike and dip, the distance between joints, and the RQD.

Delay Detonators The amount of vibration caused by blasting is related to the amount of explosives detonated at any one time. Through the use of delay detonators, the amount of explosives fired at one instant may be reduced by using a different delay period for different holes. The standard requires 9 msec between blast holes for each to be considered an independent blast with regards to vibration. For example, if a blast is to be composed of four holes each containing 11 kg (25 lb) of explosive, when detonated simultaneously, a total of 45 kg (100 lb) of explosive is detonated at one instant. However, if each

Table 5.5 Common Stemming Sizes Based on Hole Diameter

Hole Diameter (mm)	Hole Diameter (in.)	Size of Stemming	Size of Stemming
38	1.5	10-mm minus chips	3/8- in. minus chips
50–90	2–3.5	10–13-mm chips	3/8– 1/2-in. chips
100–127	4–5	16-mm chips	5/8-in. chips
127 and above	5 and above	19-mm minus chips	3/4-in. minus chips

Source: From Atlas Powder Company (1987).

hole is loaded with the same amount of explosive but has a different delay from the others, then only 11 kg (25 lb) of explosives is being detonated at one instant. Thus, the use of delays permits a larger total blast, because the amount of explosives being detonated at one time is reduced.

By permitting the movement of rock at various time intervals, the delay detonator aids fragmentation. In tunnel blasting, the tunnel support systems can be damaged if the rock is thrown too far from the face. This unwanted excessive movement of the blasted rock is called "flyrock." The flyrock can be reduced using proper delays by permitting the rock to move in the planned direction rather than ricocheting off of other moving rock.

Delay blasting is necessary in all blasting. However, it is essential in tunnel blasting. In tunnel blasting, due to the need to obtain adequate breakage, delay detonators must be used. The geometry of the breakage will indicate the correct delay sequence. That is, the holes involved in a wedge or V-cut must be the first detonated, creating a free face. Then the holes surrounding these holes will detonate in sequence until finally the trim holes detonate. The holes around the empty burnholes will be the first to be detonated. The sequence in which the trim holes detonate relative to each other determines the shape and location of the muck pile. If the lifters (the holes at the invert) fire last, the muck pile will build up against the face. Also, by having one of the ribs fire last you can have the muck pile favor one side of the tunnel. (See Figure 5.30.)

The time period between delays varies according to the burden, the geology, the energy used (amount of explosives), and geometry. The general available range of detonator timing is from instantaneous to 8 sec. However, delays above 1 sec are not often used.

Basically, the delayed detonation allows the rock in front to move out after it is detonated. This, in turn, gives a space and free face for the later boreholes

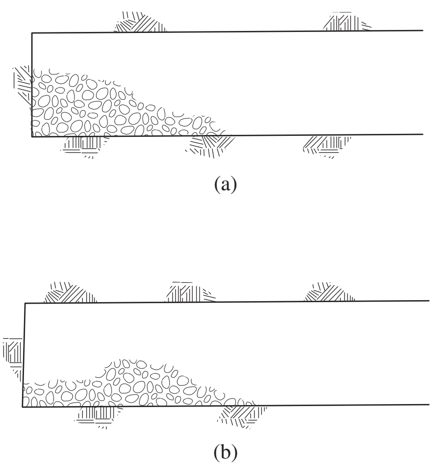

Figure 5.30 Muck Pile Placement Based on Blast Timing: (a) Back Holes Fired Last, (b) Lifters Fired Last

to detonate. It is like people walking through a door. If they do not wait for the other person to go through the door, no one will get through the door. However, if people take turns, they will all pass through the door.

Controlled Blasting Controlled blasting techniques are used to control overbreak and aid in the stability of the remaining rock formation. Generally, overbreak is replaced with concrete, and helping to stabilize the rock formation reduces the need or amount of ground support. On the surface, when highways are built through rock cuts, controlled blasting reduces the falling of rock on the road. For these reasons, many tunneling specifications require some form of controlled blasting. Surface-controlled blasting techniques are also discussed, because they may be appropriate for bench blasting.

Line Drilling Line drilling provides a plane of weakness where the rock can break. The line-drilled holes help to reflect the shock wave, reducing the shattering effect of the rock outside of the blast perimeter. This is often used when overbreak is a big problem. It can be less expensive to drill the line-drilling holes than the additional excavation and having to place more concrete. In a tunnel round, line-drilling holes are spaced about four diameters apart, depending on the properties of the rock. Rock masses that are highly jointed require closer spacing between the line-drilling holes. In firm homogeneous rocks, the line-drilling holes can be spread further apart; four diameters is common. Line-drilling holes are not loaded.

Line drilling requires considerably more drilling than the other controlled blasting methods. Also, the line-drilling method is not very effective in nonhomogeneous formations. In formations that have bedding planes, jointing, and other discontinuities, the line-drilling method is not effective in preventing the existing planes of weakness from extending into the wall. Line drilling is more effective when there are large delays, thereby decreasing the mass at the perimeter. This will reduce back pressure from the shot, thereby cutting reflective pressure waves.

Presplitting Presplitting works by creating a plane of shear in solid rows along the desired excavation before the production blast. The presplit holes are fired on an earlier delay than the production shot. By providing this sheared plane before the production shot, one reduces not only overbreak but also the vibration.

Presplit holes are 50–100 mm (2–4 in.) in diameter at a spacing equal to one-half the production burden. All holes are loaded with a trunk line and can be delayed provided it is before the production shot.

The presplit theory is that two simultaneous fire holes emit shock waves which, when they meet within the web, place the web in tension, causing cracks and shearing it. (See Figure 5.31.)

Presplit holes must be stemmed with an increasing bottom charge to move the toe. The maximum effective depth is approximately 15 m (50 ft) because

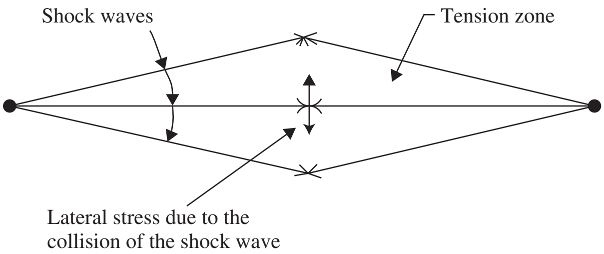

Figure 5.31 Presplit Principle

of hole alignment, and when blasting is done in tough formations, it may be necessary to use uncharged guide holes between the presplit holes.

Cushion Blasting Like line drilling, cushion blasting requires a single row of holes ranging from 50 to 90 mm (2 to 3½ in.) in diameter. Unlike line-drilling holes, the cushion holes are loaded with light, well-distributed charges. These holes are fully stemmed between charges, allowing no air gap, and are fired after the production shot has been excavated. The charges should be placed against the production side of the borehole, because stemming acts as a cushion to protect the finished wall from the shock of the charges when detonated; the larger the borehole is, the greater the cushion.

For best results, the holes should be detonated simultaneously to achieve a shearing effect in the web; however, if vibration is a problem, small delays can be used.

The spacing, generally, is nominally the hole diameter times 12 + 300 (English units is diameter in inches times 12 + 12 in. = spacing in inches):

$$D_h = \text{hole diameter in mm}$$

$$S = \text{spacing between holes in mm}$$

Therefore:

$$S = 12D_h + 300$$

Thus a hole with a diameter of 50 mm would have a spacing of 12(50 mm) + 300 or 900 mm. English units yield 12(2 in.) + 12 in. = 36 in.

The holes are string loaded with detonating cord used as a downline. The holes are stemmed as they are loaded to maintain the charge distribution; the charges are taped to the downline or inserted into spacing tubes ready to be used to maintain charge distribution, thus allowing the stemming to be done after the hole is loaded.

The charge distribution is one cartridge every 600 mm (2 ft), with cartridge size a function of borehole diameter. Therefore, a 75-mm (3-in.) hole would have a cartridge of explosives 38 mm (1½ in.) in diameter every 600 mm

(a)

(b)

Figure 5.32 (a and b) Photographs of Dynosplit Ridged Continuous and Cartridge Perimeter Control Explosives (Courtesy of DynoNobel)

(2 ft). The bottom of the hole should be loaded approximately three times more heavily to move the toe. The spacing between the cushion holes should be less than the production shot burden, preferably 0.8 times the burden distance. As the diameter of the hole increases, the diameter of the explosive packaging, and thus loading density, increases.

The easiest explosive to load is one that is premade for such use, as illustrated in Figure 5.32. From this manufacturer the sizes available range from 22 to 50 mm ($^7/_8$ to 2 in.) in diameter. Cushion blasting is not suited for underground applications because the horizontal holes have a tough stemming requirement. However, it can be used for both incline and vertical holes. Cushion blasting permits a reduction in the number of holes required by line drilling and also performs better in nonhomogeneous rock than line drilling. Deep cuts may be taken because the larger diameter holes result in a reduced alignment problem. Cushion blasting requires the removal of the excavated material before firing. This can be costly; there will be production delays, because the excavation of the entire muck pile cannot take place at once. One must partially excavate and then return. Another problem with cushion blasting is that sometimes the production shot can break back to the cushion holes, creating redrilling problems and causing loading changes.

Perimeter control explosives, which are used for cushion and smooth wall blasting can be ridged cartridged or continuous. The ridged cartridge has couples that allow the cartridges to be attached, providing a continuous column of low-density explosives. Figure 5.32 provides photographs of cartridge and continuous packaging.

Smooth-Wall (Contour) Blasting Similar to cushion blasting, smooth-wall blasting requires stemming at the collar, but not at the entire length of the hole. Intended for underground use, smooth-wall blasting consists of horizontal holes charged with small-diameter powder cartridges fired last with as short a time as possible between holes. If vibration is not a problem, the holes can be fired simultaneously. The burden range is about 1.2–1.5 times the spacing depending on the rock quality, and the collar of the hole must be stemmed to equalize the explosive energy along the borehole, prevent, or reduce, the hole from rifling, and prevent the charges from being pulled from the holes by rock movement caused by earlier delays. Like other methods, it works best in homogeneous rock, but smooth-wall blasting is especially important for poor-quality rock. Not only does the better perimeter reduce overbreak, thus reducing ground support and decreasing the amount of concrete or shotcrete, but also the reduction of cracking will reduce the amount of water inflow because of the small cracks.

Hydraulic Impact Hammer

Rock is broken, or fractured, by an impact hammer usually equipped with a chisel impact tool. Breaker hammering, as well as boom movements, are carried out by hydraulic power. The machine is either mounted on a track undercarriage or as a backhoe attachment. Operating similar to the pneumatic pavement breakers used for breaking up sidewalks, the hydraulic impact hammer strikes with an impact tool having the power to break hard rocks in situ.

The hydraulic impact hammer works on the same principle as a hammer hitting a chisel. The chisel receives energy from blows from the hammer. The chisel of the impact hammer receives energy to break the rock from the movement of the hydraulic piston in the impact hammer. See Figure 5.33.

Hydraulic impact hammers eliminate the difficulties of blasting by not requiring drilling, explosives are not used, and thus hazards and vibration are reduced. In medium or larger size tunnels it allows mucking operation to be performed simultaneously. Unlike blasting, secondary fragmentation can be done

Figure 5.33 Impact Hammer Mounted on Excavator Boom

immediately. That is, if a fragmented rock is too large to be efficiently mucked out, the impact hammer can immediately break it. If this occurs with blasting, the rock has to be drilled and blasted or, if small enough, additional equipment will be required to break it up. The impact hammer offers the ability to excavate the full face with minimal overbreak.

There are two principles used when breaking rock with an impact hammer: primary and secondary breakage.

Primary breakage describes when an impact hammer breaks intact material. Examples of this are concrete slabs and intact rock mass without discontinuities. Secondary breakage is when the impact hammer is used to reduce the size of existing blocks.

The properties of the rock to be fragmented will determine the success of the operation. The efficiency of primary breakage is determined by the rock intact properties. Secondary breakage depends on the rock mass properties. Intact rock properties are related to the strength and toughness of the rock. These include unconfined compressive strength, tensile strength, total hardness, and sonic velocity. All of these characteristics are related to the strength of the rock mass and its ability to resist breakage.

Where primary breakage depends on rock intact strength, secondary breakage is dependent on the rock mass properties. These properties are related to rock discontinuities and their properties. Desired information to evaluate secondary breakage is the nature of the discontinuities, the spacing, distance between them, the orientation, strike and dip, and RQD. The RQD will provide an indication of the condition of the mass, that is, how fractured it is. The seismic velocity is the best indicator of the toughness and resistance of the rock mass to excavation by an impact hammer.

Generally hydraulic impact breakers are readily available and mobilization is similar to other heavy construction equipment. There is considerable flexibility to different shapes for headings and these are adaptable to various rock types. Because of its flexibility, the impact hammer can work in mixed rock conditions. Being self-contained, the hydraulic impact breaker can be introduced to any equipment mix on the job. The overbreak is minimized, and there is no break in the progress of the tunnel to blast and the rock excavation cycle can be continuous because of the ability to muck while excavating.

When precise excavation lines are required, the hydraulic impact hammer can excavate to those lines without blasting or special equipment. Hydraulic impact hammers are much slower than blasting. However, if blasting is prohibited, using the hydraulic impact hammer to excavate can be the most efficient and cost-effective replacement for blasting.

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6 Roadheaders

Roadheaders are mobile mining units that cut rock formations by the rotation of a pick equipped cutterhead mounted on the end of a boom. The roadheader was originally developed by the British coal industry to mine roadways in mines and was further developed by the Russians, increasing the ability for rock cutting. With an operation principle similar to mechanical miners, it is best for softer formations. The boom is hydraulically operated and can be placed in a full range of vertical locations and can be used as a telescope. Horizontal movement is provided by a turret. A loading apparatus is usually an apron with gathering arms. The arms pull the muck on to the apron, where it is picked up by a chain or belt conveyor that carries it from the loading device to the rear of the machine, where it is then deposited on another conveyor or dumped into a muck train or an earth-moving machine bucket for transportation to the surface and final disposal.

The base frame, which is sometimes equipped with jacks or outriggers for stabilization, has electric over hydraulic controls in the operator's cab. From the cab, the operator can control horizontal and vertical movement, thrust, cutter speed, the loading device, the propelling drive, usually a crawler track assembly, and the conveyor. Many roadheaders now come with microprocessor guidance systems, which automatically correct deviations from line and grade.

The system stiffness, the ability of cutting tools to endure the large normal forces, the carbide inserts' lack of ability to resist the heat generated, and the force from the impact in high-silica-content rock limit the ability of boom-type roadheaders to penetrate effectively in hard rock.

OPERATING PRINCIPLE

One of the most popular excavation methods in soft rock is use of a roadheader. Roadheaders can cut a variety of face configurations and cross sections and have the flexibility of full-face machines. Among the advantages of the roadheader are its face shape and ability to change directions at any time. Because of limited stiffness and rock strength, the use of roadheaders has generally been limited to rocks with an unconfined compressive strength less than 100 MPa (15,000 psi). Bit wear is also a limiting factor. Minerals like silica can cause considerable wear. This wear can be mitigated with a head design suited for the rock being cut. Even so, bit wear reduces production and increases costs.

There are two roadheader cutting approaches: axial and transverse. The axial approach can be referred to as milling, radial auger, or inline. With the most common cutting method used, the inline, the roadheader rotates in line with the cutter axis and, applying the cutter forces sideways, reduces the utilization of the machine's weight in the cutting process. However, it provides the optimum position for maximizing cutting force forward. Because of lower cutting speeds, there is less pick consumption. Telescopic booms provide the force for sumping; less time is spent on breaking the face and sumping. In harder rock, the machine is supported by hydraulic jacks, or stelling rams. Stelling rams act like outriggers on a crane; that is, they offer stability from the resultant forces. The stelling rams may not be effective in soft formations, because the rock may not have the strength to carry the imposed load. Also, because of the obvious finite length of the rams, the rams lack effectiveness in wider tunnels. The axial roadheader throws the muck to the side, making gathering it to the apron a little more difficult. It is best suited for small-diameter tunnels.

The transverse approach is often referred to as "ripping." The ripping principle is adapted from continuous mining machines. Its best cutting performance is in weak rocks. The slewing force is at a right angle to the circumferential force. Cutting in the direction of the face makes it more stable. It is more adaptable to a wider range of conditions. It is less affected by changing rock conditions, including hard rock bands, and it is highly maneuverable. It can cut rock up to 100 MPa (15,000 psi). The most powerful roadheaders can cut rock up to 150 MPa (22,000 psi). However, the optimum rock strength for cutting is 30 MPa (5000 psi). The transverse roadheader can cut large cross sections; a large machine can cut 60 m^2 (650 ft^2). Figure 6.1 is a photograph of a transverse roadheader; note the star gathering arms.

The roadheader offers several advantages. It has versatility and mobility. That is, it can mine varied cross sections, there is easy access to the heading, providing good access for ground support, and large faces can be subdivided simultaneously.

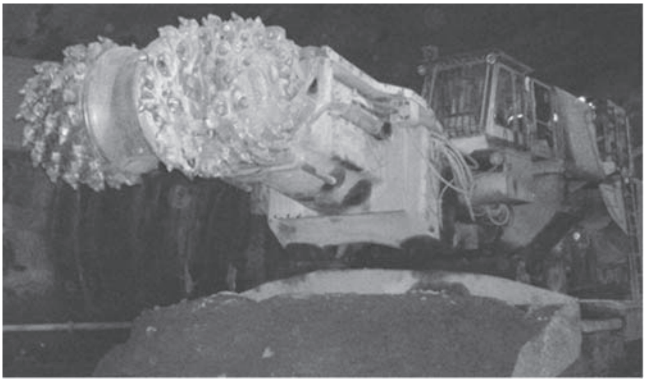


Figure 6.1 Transverse Roadheader

The cost is relatively low, approximately 20%, and can vary by 10%, depending on the size of the roadheader and the TBM. Roadheaders are often available for renting, which makes it ideal for small projects.

The roadheader is relatively easily mobilized. The delivery time is less than half of a TBM and it can be put into operation as soon as it arrives at the site.

The cutting advance rate depends on the penetration per cut and the rotary speed. Cutter speed and torque determine the power transmitted to the head. When driving a tunnel with a roadheader, about a quarter of the available time is used for maintenance. It has a shorter cycle in that it consists of cutting and ground support. The roadheader causes little overbreak and requires less ground support.

The recommended procedure for maximizing production is to first cut the sump forming a cavity the depth of the cutterhead. Rather than move it, the boom is held stationary and the machine is advanced. This places the machine at a good location for attacking the face. The sump is then enlarged by tracking the cutterhead across the face to the perimeter of the face. At the perimeter of the face, the cutterhead is moved up and down, cutting the face. The boom is then moved across the face to the other side and the process is repeated. The cutting should move to the central part of the face if strata are friable. The cutting pattern following the direction of the head rotation should maximize advance rates. If the strata vary with different strengths, the soft one should be cut first. This facilitates cutting the harder strata by being able to cut from the gap remaining after the removal of the soft rock. It acts similarly to a free face in blasting.

MUCKING

As the rock is cut from the face by the tools on the rotating head, it falls on to the apron. The muck is then collected by the gathering arms to the conveyor. Figure 6.2 is a photograph of a gathering arm loader. Note that the arms pull the muck toward the conveyor in the center of the apron. The conveyor transports the muck to the rear of the machine and drops it onto another conveyor or into a muck car, train, or excavator bucket.

The star disc loader rotates, depositing the muck on the conveyor similar to that of the disc loader that uses disks to load the conveyor rather than a rotating arm.

Rock and machine parameters are the factors that affect machine performance. The rock parameters include intact properties, rock mass properties, and the environment. The intact properties are strength, cutting ability/resistance, impact resistance, abrasiveness, and thermal properties. Strength is quantified using the unconfined compressive strength (UCS) and is one of the most important parameters in penetrating the rock. As stated earlier, the largest machines are not used for rock strength above 150 MPa (22,000 psi). The rock strength is used for the determination of the ability and the capacity of the roadheader.

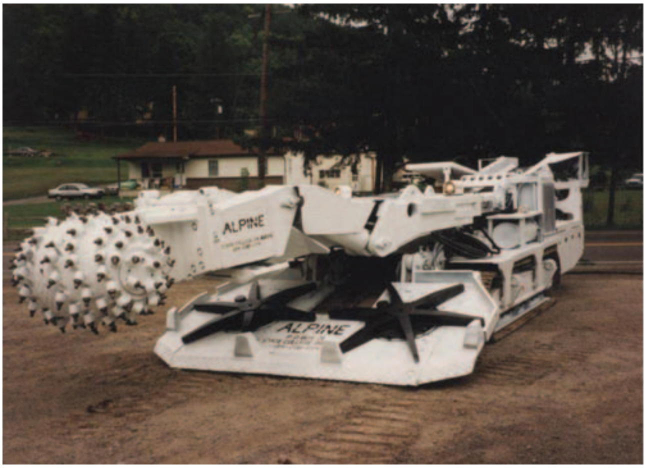


Figure 6.2 Roadheader with Gathering Arm Loader (Courtesy of Alpine Equipment)

Also, tensile strength and shear strength indicate the toughness of the rock fabric. The density or specific gravity provides an indication of the muck and ability of the excavator.

Cutting ability is the property that indicates if the rock can be effectively drilled. Stronger rock results in lower penetration rates. Impact resistance describes the resistance of the rock to penetration. This can be obtained from the point load test results. Abrasiveness of the rock is the most important indicator of bit wear and life. Young's modulus and Poisson's ratio indicate the competence and brittleness of the rock. The ultrasonic pulse velocity (acoustic velocity) indicates the competency of the rock and its brittleness, which strongly affects its resistance to excavation. Abrasion provides a strong indication to bit wear. Tests to determine abrasion are discussed earlier in this chapter. The ratio of the compressive to tensile strength (UCS:T), which is the compressive strength divided by the tensile strength, is an indication of the toughness of the rock fabric. Point load and punch strengths indicate the forces needed for the cutters and bits to penetrate the rock. Testing to determine cutterhead design is available at the Earth Mechanics Institute at the Colorado School of Mines. Figure 6.3 is a graph, showing roadheader performance relative to cutting performance.

The rock mass properties provide the face conditions and structural descriptions that affect tunneling. Mixed-face conditions are the degree of stratification along the tunnel for predicting the effects on tunneling and selecting mining methods. As mentioned earlier, in mixed-face conditions the roadheader should cut the weakest strata first.

The discontinuities are those rock attributes that keep rock from being a massive formation—anything that causes the formation to be discontinuous. Very simply put, discontinuities are cracks most of which are referred to as

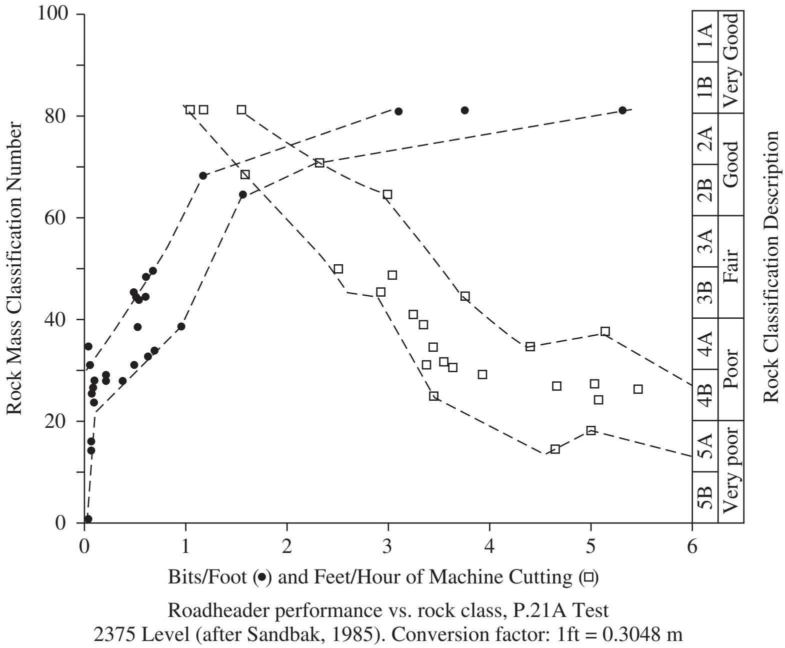


Figure 6.3 Roadheader Performance vs. Rock Class (After Sandbak, 1985)

joints. There are different types of discontinuities, depending on the origin. However, it is the spacing, orientation, shear strength, width, condition of the joints, and type of fill material in the joints that are indicators of machine production as well as ground support needed.

Orientation refers to the direction of the joint. If the joint runs east and west and we are mining north, the ease with which the rock chips can be pulled from the face is affected. Although compressive strength determines whether or not a roadheader can be used, shear strength is a greater factor in the mining. The spacing and joint condition are indications of the shear strength of the joints.

Discontinuities can reduce the effective shear strength of the rock. High-strength rock with many discontinuities can be mined with a roadheader. When a roadheader tool is dragged across the face, it chips and pulls the rock from the face. If the rock has many discontinuities, it is easier for the tools to pull rock pieces from the face.

The environment includes both water added for dust suppression and water make. The geometry refers to the size of the tunnel, the shape, and the gradient. The geometry is an element in the roadheader utilization. The tunnel crown may be beyond the reach of the machine or so wide that the machine will not perform at its peak because of a lack of stability. The gradient is an issue with mobility of the machine. In situ stress plays a major role in determining the quality of rock with regard to discontinuities.

MACHINE PARAMETERS

The machine parameters that need to be considered are the type of cutting head, stability, and operational characteristics. The cutterhead selected depends on the mining conditions. The cutterhead can be transverse or axial. The type of head influences the lacing pattern. The location and spacing of the tools will be different for the cutterheads. The different rotational direction will determine its cutting capacity in different rocks. The tool used versus the rock hardness and abrasiveness is an important factor in roadheader productivity. The type of tip geometry and the grade of the carbide tip affect production. The geometry affects the angle at which the pick strikes the rock. The more the angle takes advantage of weak planes in the rock, such as foliation or discontinuities, the better the advance. Obviously, more pick wear will require more stops to replace the picks, reducing roadheader availability, and thus decreasing production; it also makes it harder to penetrate the rock.

Cutting Bits

Bits are drag or point of attack. The tools are inserted into tool holders, boxes, or housings and held in place by a clip or spring. The points of attack are often free to rotate in the holders, providing even wear. As a result, they have long cutting life. Some claim that this acts as a self-sharpening feature; there is no research to confirm this. Point-of-attack bits are probably stronger than radial picks because of the high rate of impact loading.

The large radial picks permit increased penetration efficiency while decreasing the amount of dust. Because they are larger, they are well suited for weak to moderately strong rock. However, in very strong rocks, the picks can become damaged, which will decrease efficiency and cause vibration problems.

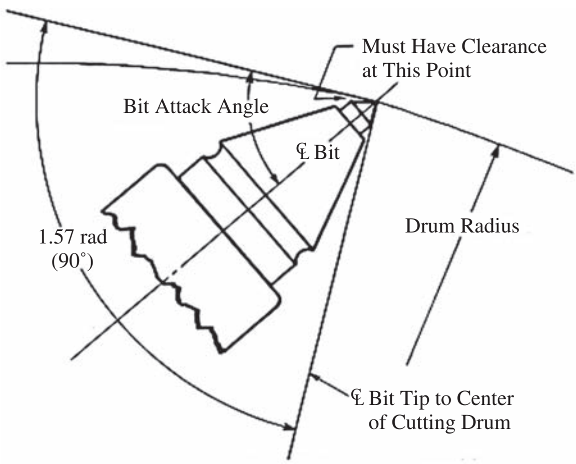
Forward-attack picks are designed to permit greater axial thrust during cutting than is generally possible with radial-type picks. The forward-attack pickbox is better aligned for accepting the loads. Because of the better alignment, the shank is less exposed to radial damage in the forward-attack cutting mode and is more able to seat point attack picks.

During cutting, bits are forced against the rock, developing cutting forces that are parallel to the direction of head rotation and normal thrust (see Figure 6.3). As the head rotates, the tip of the pick strikes the rock at a set penetration angle. The penetration of the rock by the tool causes a zone of crushed rock, which transfers the load to the surrounding rock. A pressure bubble is formed in this zone, and a hydraulic effect of the fines propagates fractures into the fabric of the rock. Because of the limited forces available with the tool, the fractures are not very deep, but a relatively small crushed zone with low pressure is created. The rock is relieved from the face by subsequent passes of the bits at a set spacing, exploiting these fractures. Water jet cutting heads are available with most roadheaders. They can increase the strength of rock that can be cut in addition to reducing dust and increasing pick life.

As with TBM cutters (to be discussed later), the spacing between the crushed zones has an effect on the cutting forces and mining efficiency. If the spacing is too close, there will be an overcrushing that will consume more energy and generate more dust. As the spacings widen, they will reach an optimum spacing for productivity and energy consumption. If the spaces get too far apart, a point is reached where the cutter marks no longer extend to the adjacent cutter marks, thus ending the interaction, reducing penetration, and increasing energy consumption. The main indicator of cutting efficiency is specific energy, which is the energy required for a unit volume of rock. Therefore, the lower the specific energy is, the more efficient the mining. The spacing-to-penetration ratio (S/P), which is critical to cutterhead design and cutter geometry selection, is dependent on the rock type. The optimum S/P for conical point attack bits is usually of 2–4 times the penetration. This ratio can be greater than 4 for brittle rocks. Figure 6.4 illustrates the bit angle relative to the face.

Although the system stiffness limits the roadheader performance in harder rock, the cutting bits have the most effect. The limited torque and the inability of the tools to withstand high normal forces and impact loads with heat and high bit tip speed can cause premature failure of carbide inserts. Therefore, the bit tip speed is limited by the ability of the bit to dissipate heat.

The bit life is the major player in roadheader performance. However, hard rock with silica tends to break carbide inserts and wear the bits because of their abrasiveness. The aforementioned causes contribute to the principal cause of bit failure. However, heat is the single most significant cause of tool wear. The faster the head goes and the more the thrust is put on the face combined with the hard silica-containing rock, the higher the temperature. Bit temperatures can exceed 1100°C (2000°F), causing the metals to lose their strength and resilience.



Bit attack angle for cone-point bits.

Figure 6.4 Cutter Head Pick

The bit life can be increased if the head is kept in good condition. Also, slower rotation of the head will reduce the heat. In addition to increasing cutting ability, water can mitigate the temperature rise in the tools.

Slewing and lifting forces are greater with a transverse machine. It uses its weight to counter the forces against the face and when lifting the weight of the machine provides stability. However, for penetrating hard rock the milling head is more effective because, due to its conical shape and axial direction of head movement, it thrusts the head into the rock. Although meant for weaker rocks than the axial roadheader, the ripping-type head has more power. The strength of the rock is directly related to the weight of the machine. The heavier the roadheader is, the stronger the rock that it can penetrate. Even though the milling-type machine has greater thrust against the face, the ripping-type roadheader can be smaller because of the direction of movement of the boom.

OPERATIONAL PARAMETERS

The operational parameters are shape, size, length of excavation, inclination, turns or cross cuts, sequence of cutting and enlargement operation, number of variations in the rock type/formations, ground support approach, and work schedule.

Roadheader Cutting Capacity

Historically, the maximum-strength rock that can be cut by a roadheader has been below 100 MPa (15,000 psi). Although, as Table 6.1 illustrates, the capacity has increased, they are still most effective in rocks that are less than 30 MPa (400 psi). The silica content is still a major consideration when using a roadheader. The silica content may reach a level such that the cost of bit wear makes using a roadheader uneconomical.

The UCS/*T* (ratio of unconfined compressive strength to tensile strength) indicates the toughness of the rock fabric. A low UCS/*T* means a high tensile strength relative to compressive strength of the rock. Thus, the rock is harder to penetrate. Generally, the use of a road header is not recommended if UCS/*T* is less than 10.

Table 6.1 Classes of Roadheaders

Roadheader Class	Maximum UCS	Weight (tons)
Light duty	60–80 MPa (8700–11,600 psi)	40
Medium duty	80–100 MPa (10,000–14,500 psi)	0–80
Heavy	100–120 MPa (14,500–17,500 psi)	0–100
Extra heavy	120–150 MPa (17,500–21,700 psi)	100

Performance Prediction

The operational cutting rate (OCR) is a basic factor in predicting roadheader performance. The overall roadheader performance is assessed based on the advance rate. The advance rate is determined by the relationship of the OCR, the face area, and the utilization. Utilization is the time that the machine is available for mining. Time available for mining can be affected by shift change and meal time, maintenance, both planned and unplanned, installation of ground control, surveying, mucking delays, or loss of operating needs, such as electric power.

Mealtime and shift change are known disruptions and can be predicted when planning roadheader performance. The time required and the timing for planned maintenance can be predicted prior to performing it based on experience. For unplanned maintenance, this is not so. A good complement of spare parts can mitigate any downtime caused by mechanical failure. The further the job is from the source of the parts—that is, the distributor—the better the stock of spare parts should be. The installation of ground control may be cycled with mining. To do this, the roadheader mines the length of the support spacing. That is, if the support is ribs on 1.2-m (4-ft) spacing, the roadheader mines the spacing and is pulled back to permit the installation of the support. In some cases, planned maintenance can be done while waiting for ground support installation or the crew that is not involved in ground support can use this time to eat their meal. Table 6.2 shows the amount of time available for mining based on the type of ground support being installed.

The time for surveying can be mitigated by having it performed during another activity such as ground support installation or meal time. Mucking delays are generally the result of a part of the mucking trail not performing. This could be due to conveyor breakdown, trouble at the shaft, and so on. In lesser developed areas, the potential of loss of power is very real. During job planning, the risk of loss of power has to be assessed and necessary actions taken to mitigate the risk and the result of power loss. Water handling may become a delay issue in water-bearing ground. Generally, pumping of the water from the heading can be conducted concurrently with mining.

Table 6.2 Time Spent for Ground Support

Support Type	% of Cutting Time per Available Face Time
None	60–80
Rock bolts	40–50
Shotcrete	40–50
Shotcrete and rock	30–35
Steel sets	30–35
Steel sets with full lagging	20–25

Source: Adapted from Kogelmann (1988).

Advantages of Roadheader

Roadheaders offer certain advantages over TBMs and drill blast methods. Unlike TBMs, roadheaders can mine any face configuration. They can also be selective in the mining. For example, in mining applications roadheader cutting can be limited to following ore. The need for ground support can be reduced 40% or more than blasting because of minimal ground disturbance. In addition, there are no complaint-causing vibrations as from blasting.

The uniform muck size aids with muck handling. The roadheader can change directions at any time and is able to make 90° or more turns. It is a continuous operation. That is, if it is not necessary to install ground support, the operation is continuous, unlike drilling and blasting, which necessitate the mobilization and remobilization of workers and equipment in the heading.

Multiple activities can be performed while mining. Utilities and, in rare cases, ground support can be installed while the roadheader is operating. Roadheaders are conducive to automation providing more accurate line and grade. In softer rock, they can have production rates 50% greater than blasting, thereby reducing mining costs.

Roadheader Shortcomings

The rate of penetration of the roadheader is very dependent on the rock strength, more so than with blasting or TBMs. Depending on the rock hardness, the cost of cutters can be very high, adding to overall mining costs. The dust generated by roadheader operation can cause severe environmental problems. Dust can be mitigated by the use of a dust collector. Figure 6.5 is a photograph of a roadheader dust collection system.

The roadheader can be affected by changing ground. That is, if a very hard layer of rock is encountered, the roadheader may not be able to cut it or will have severe cutter wear and costs.

Figure 6.5 Roadheader Dust Collection System

Although the roadheader can address most face cross sections, certain cross sections that can be easily done by blasting may not be within the capability of the roadheader. For example, a tall narrow cross section may not be conducive to its reach.

Although they are less expensive than a TBM, roadheaders may cost considerably more than drilling and blasting equipment.

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7 Tunnel-Boring Machines

HISTORY OF TUNNEL-BORING MACHINE

The first record of a working TBM was a machine patented by Charles Wilson in 1856, called "Wilson's Patented Stone-Cutting Machine." Wilson's second, "improved" machine (Figure 7.1) was built in 1857 and was used to build the 7645-m (25,081-ft) Hoosac tunnel in western Massachusetts.

In the 1857 TBM, the cutter wheel was attached to an open-ended cylindrical drum. The cylinder and wheel were advanced against the face by the use of screw jacks. When operating, the cutter wheel cut the circumference of the tunnel and the turning drum rotated the cutter wheel and indented the cutters clockwise. When the drum turned one complete revolution, a layer of rock was removed from the hemispherical face.

The muck generated by the mining fell into a drum and was passed out through various holes in the drum's surface. Scraper blades were attached to the drum in a helical pattern that shoved the muck to the rear. Documents from the Hoosac tunnel project indicate that Wilson's 1857 patented machine was used at the Hoosac tunnel (Stack, 1982). The excavation plan was to bore both an annular groove at periphery and a center hole. The machine would then be removed from the face and the remaining core of the face would be blasted using gunpowder.

Figure 7.1 is a drawing of Wilson's 1857 patent. The patent narrative describes a short, large-diameter hollow cylinder (h) at the forward end of the machine, the direction of advance, with a series of disc cutters mounted at the perimeter. This cylinder was attached to a sliding horizontal upper frame (j) that was mounted on a rail carriage. The rolling disc cutters were set in groups of two or three, and two or three additional rolling disc cutters were centrally mounted to cut the core of the face.

The groove cut at the Hoosac tunnel was 330 mm (13 in.) wide and 7.3 m (24 ft) in diameter. Penetration rates of 254–610 mm (10–24 in.) per hour were achieved but the machine cut only 3 m (10 ft) of tunnel and proved to be a commercial failure. The tunnel was unique for its first use of a TBM, the introduction of the pneumatic rock drill, and the extensive use of NG dynamite.

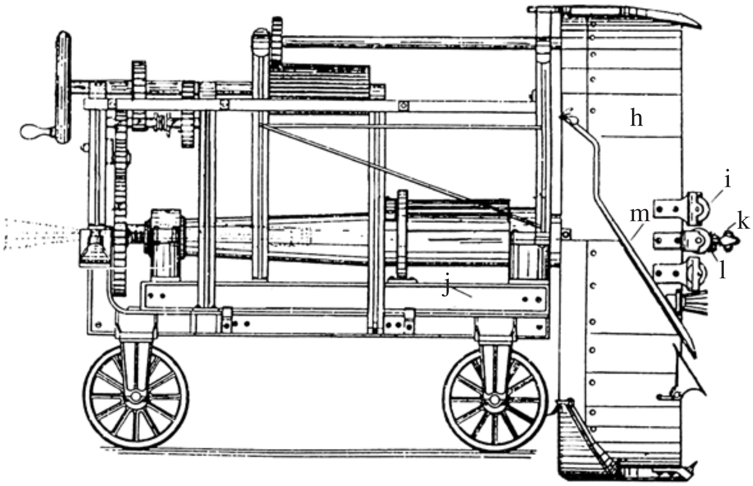


Figure 7.1 Wilson's Improved Machine (Stack, 1982)

Channel Tunnel

The channel tunnel is a rail tunnel linking Folkestone, Kent, in England with Coquelles, Pas-de-Calais, in northern France. Completed in 1994, the 50.5-km (31.4-mi) tunnel crosses the English Channel beginning at the Strait of Dover and at its deepest point is 75 m (250 ft) below the English Channel. It was first conceived in the early 1800s, and construction began in the 1880s. In 1882, the French Channel Tunnel Company completed the sinking of a 5.4-m-(17.75-ft-) diameter shaft at Sangette, France, and was ready to test two different machines for tunneling under the English Channel: one was invented by John D. Brunton; the other, the Beaumont/English machine, was already being tested by the British.

The Brunton machine, an improved version of a machine patented in 1868, operated by means of two "chucks" or "face-plates" each mounted with six disc cutters. The chucks were mounted on an eccentrically located shaft, or spindle, which was attached at opposite ends of the crosshead and revolved around the central shaft. By the use of a worm gear and the eccentrically placed spindles, the chucks were adjustable to compensate for cutter wear. Figure 7.2 is a drawing of the Brunton machine and is similar to the model displayed at the Conversazione of the Institution of Civil Engineers in Paris in May 1868. The Brunton machine failed to perform well and was dismantled by the French in favor of the Beaumont/English machine.

Beaumont/English Machine

Most late nineteenth and early twentieth century writers credited Fredrick Beaumont as the inventor of the machine that was successfully tested in

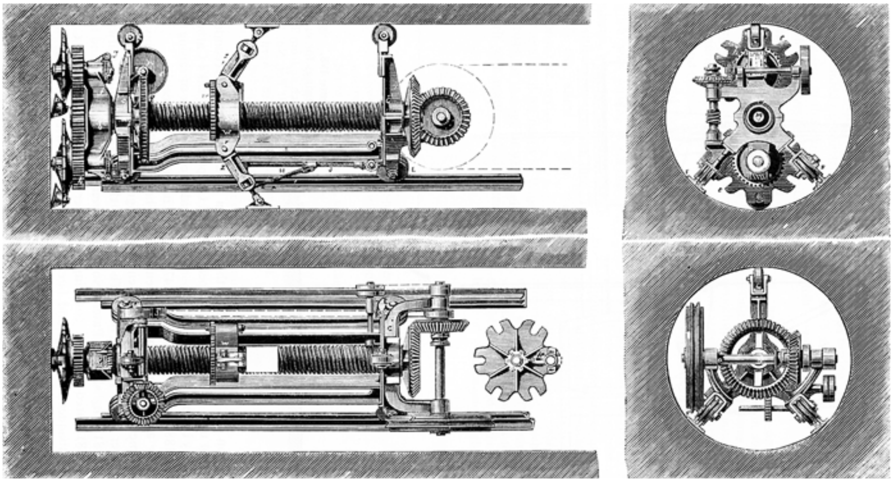


Figure 7.2 Brunton Machine, U.S. Patent No. 80,056 (Stack, 1982)

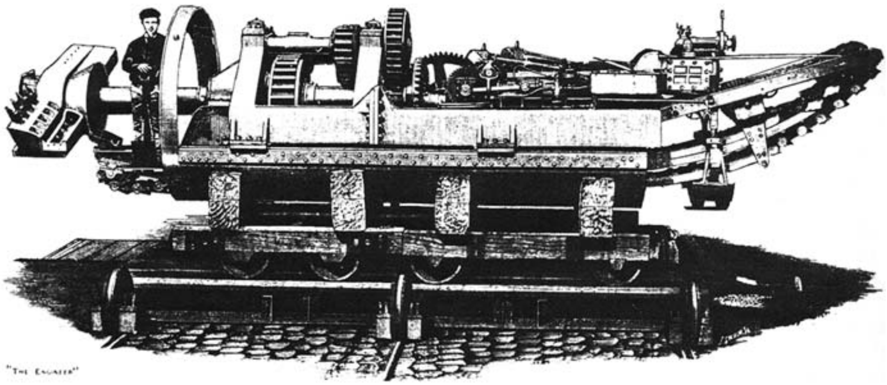


Figure 7.3 Beaumont/English TBM (Stack, 1982)

the channel chalk. Recently, researchers (Stack, 1982) have determined that Colonel Thomas English should be included in that credit. English's proposed modifications to Beaumont's 1864 patent made the machine more effective. English proposed to build the machine in two parts to permit the boring head to be advanced against the face slowly and evenly by using hydraulic power and compressed air.

A sketch of the Beaumont/English machine is shown in Figure 7.3. The machine had two cutter arms projecting from the boring shaft. Chisel-edged or pick cutting tools were spaced along each arm. The muck cut from the face was directed into small buckets on a conveyor by the mouth of the trough that was formed by the hollowed section on the rear of the arms. On the left side of the figure, the front of the machine, one can see the arms with picks.

Preliminary trials of the Beaumont/English machine were conducted from a 22.6-m-(74-ft-) deep shaft near the western end of the Abbotscliffe tunnel of the Southern Railway between Folkestone and Dover. The machine was powered by a compressed air motor and mined a 2.1-m-(7-ft-) diameter tunnel for a distance of 804 m (2600 ft).

On July 19, 1882, the Beaumont/English machine began tunneling beneath the English Channel from the French side. Between January 13, 1883, and March 18, 1863, a period of 53 days, the machine advanced an impressive 810.3 m (2659 ft), 15.3 m/day (50.1 ft/day). From the time the machine began working in July of 1882 until operations ceased on March 18, 1883, the Beaumont/English machine mined a total of 1839.6 m (6036 ft). On the British side, the machine mined a total of 1815 m (5955 ft) until the project was suspended due to political pressure regarding concerns about English national defense; the Board of Trade stopped all work. The official reason given was that the British government wanted to study the feasibility of the project. The Gladstone government relented due to fears that the tunnel would make it easier for Europe to attack England.

Following this three-decade TBM development in mechanized tunneling, no practical advances were achieved until James Robbins, in 1953, built an 8-m-(26.25-ft-) diameter machine for the Oahe Dam project in South Dakota. The machine, referred to as the "Mittry mole," was 27.4 m (90 ft) long and the cutterhead was comprised of two counter-rotating heads. The interior section had three radially arranged cutting arms and an outer section with six cutting arms. The machine was powered by two 149-kW (200-hp) electric motors.

For either head, each arm had a row of radially arranged, fixed tungsten carbide drag bit cutters mounted parallel to a row of free-rolling mild steel disc cutters. The row of disc cutters was positioned slightly behind the drag bit cutters, with the leading edge of each disc cutter positioned so that it could travel over the center of the ridge left by two adjacent kerfs.

On the Oahe Dam project, this machine set a world record for tunnel advance, 18.5 m (61 ft) in an 8-hr shift and 49 m (161 ft) in 24 hr. Although the experience with the machine was excellent, it was boring in soft Pierre shale and could not be considered rock; however, it demonstrated the feasibility of using a TBM to mine harder rocks.

The first successful application of a TBM in medium hard rock was at the Humber River sewer project. In 1956, for this project in Toronto, a 3.3-m-(10.75-ft-) diameter machine was built for the Foundation Company of Canada. The machine used both fixed and disc roller cutters, but because of trouble maintaining the fixed cutters, the contractor tried to tunnel with just the roller discs. The contractor realized he was able to maintain his previous production rate and the disc cutters held up better in the harder material. The machine drove tunnel through 4510 m (14,800 ft) of sandstone, shale, and crystalline rock with compressive strengths ranging from 5.5 to 186 MPa (800 to 27,000 psi).

In 1960, the Hydro-Electric Commission of Tasmania contracted with The Robbins Company for a TBM to drive the Poatina Tunnel. The 4.9-m-(16.1-ft-) diameter machine was basically the same as the TBM used on the Humber River project; however, the steering had been improved. The TBM was 12.8 m (41 ft) long and, if jammed, the rotation direction of the single cutterhead could be reversed. The head was rotated at approximately 75 kW (5.3 rpm by six 100-hp) electric motors.

The machine was delivered to Tasmania on March 18, 1961. The cutterhead was advanced by the main hydraulic propulsion rams until it touched the face. As the cutterhead was turned, the forward motion of the rams slowly increased pressure until the cutters started to cut the face. The cut rock would drop to the invert, the tunnel floor, where it was picked up by buckets on the cutterhead. There were four buckets on the periphery of the head that scooped up the muck from the tunnel invert and deposited the muck on the 762-mm (30-in.) conveyor belt, which transported the muck to an auxiliary conveyor behind the machine. The machine advance continued for the limit of the hydraulic ram stroke, which was 1.2 m (4 ft). When the end of the stroke was reached, the cutterhead and conveyor were stopped. A vertical supporting device, support shoes, at the rear of the machine was lowered to the tunnel invert. This relieved the gripper assemblies that pushed against the tunnel walls to provide bracing from which the machine cutterhead could push against the tunnel face. With the support shoes stabilizing the TBM, the grippers could be retracted from the tunnel walls, moved forward, and pushed against the tunnel walls ready to begin mining again.

The Poatina machine was modified after finishing the 4450-m (14,600-ft) tailrace tunnel to incorporate lessons learned. The front support shoe was replaced by a large articulating front shoe that provided full contact along the tunnel invert. The six drive motors were rewound, reducing the rpm from 5.3 to 3.6 rpm, lessening problems experienced with the gear box. Finally, the number of buckets on the periphery of the cutterhead was changed from the original 4 to 16 smaller buckets. When the number of buckets was increased, the number of disc cutters on the periphery was increased from 12 to 16.

A world tunnel boring record was achieved during the mining of the second section of the headrace tunnel on the Poatina project. During a six-day work week, the face was advanced 229 m (751 ft). Beginning in January 1963, 1.6 km (1 mile) of tunnel was dug in less than 11 weeks. The best advance for a shift was 18.2 m (67 ft) and a maximum penetration rate of 93 mm/min (4 in./min) was achieved.

The Poatina TBM was later modified to fit on a specially designed skid plate used to catch the cut material produced as the TBM reamed an existing tunnel. The TBM was the first to use a full floating gripper that permitted the operator to steer the TBM during a mining push while the grippers remained extended against the tunnel sidewalls. Also, this was the first occasion where permanently sealed oil-lubricated bearings were used.

As Robbins continued to improve and modify its TBMs, other equipment manufacturers were trying to improve the technology introduced by Robbins and, in some cases, to develop new technologies.

OPERATING PRINCIPLE

The TBM is usually fabricated to match the rock conditions. If the TBM is not powerful enough for the rock, the machine will be overstressed and be prone to breakdowns. Similar to an electric drill, the TBM operates by using thrust and torque. Like the drill, the motor rotates the bit (cutterhead), whereas the thrust for the hand electric drill is provided by the person operating the drill; the TBM receives its thrust by cylinders that push the cutterhead against the rock face. As with an electric drill, there is an optimum thrust and torque combination. The machine gets its forward thrust by pushing against the precast segmental lining. Figure 7.4 is a photo of hydraulic push cylinders. Note that the push cylinders are extended.

Figure 7.5 is a schematic of four systems on a TBM. The boring system (1) includes the cutterhead and disc cutters. The thrust and clamping system (2) includes the thrust cylinders that provide forward movement and the gripper shoes that are pushed against the sidewall. In addition, the front shoe, side-steering shoe, and supporting invert shoe and rear support hold the TBM off the invert. Using the friction between the grippers and the side wall provides the stability to push forward. The muck removal system (3) removes the muck from the bottom of the cutterhead and, using the conveyor, transports the muck to the rear of the machine, where the muck can be further transported by other

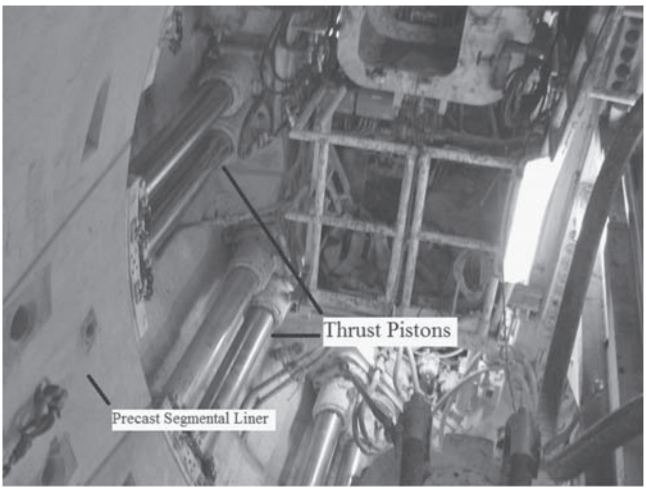


Figure 7.4 TBM Push Cylinders

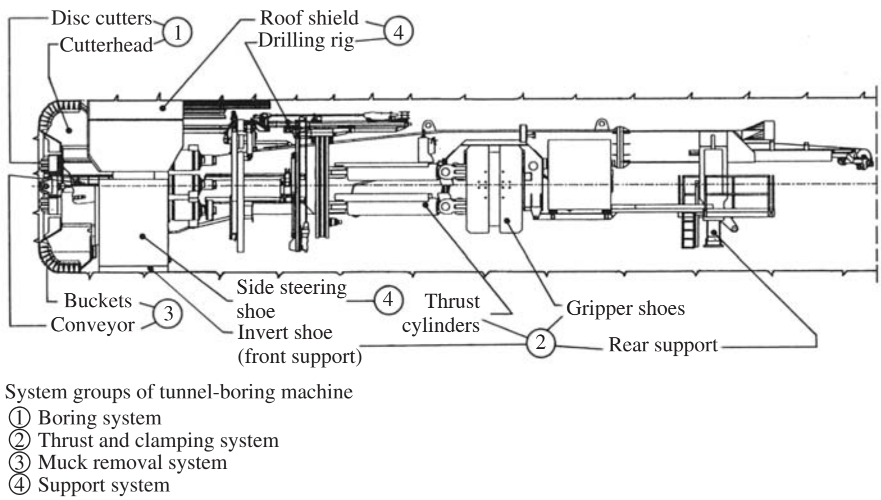


Figure 7.5 TBM Schematic (Maidl, 2008)

means. The support system (4) consists of the roof shield and drills for installing root bolts.

The operation and effectiveness of a TBM are influenced by a number of factors, including the geology, rock properties, water, tunnel design, local practices, labor regulations, working hours, management policies, and machine capabilities.

The principal nonmechanical factors of TBM performance are the geology and rock properties. Physical testing of the rock can provide indicators as to the rock properties and their effect on tunneling with a TBM. The rock density affects the muck-handling properties of the excavator; that is, the greater the density, the heavier the rock is per unit.

The uniaxial compressive strength (USC) indicates the rock's resistance to penetration. It has been the primary limiting factor of TBM development. Not only does strong rock require more energy, but also machine durability is a factor.

The tensile strength and shear strength of the rock indicate the toughness of the rock fabric. Thus, rock with a high tensile or shear strength will be more difficult to fracture.

Seismic velocity indicates the penetration, or excavation, difficulty. The higher the seismic velocity, the more difficult it is to penetrate the rock. The elastic constants provide the potential competency and brittleness.

Abrasiveness of the rock it a good indicator of bit and tool wear. The most common abrasive rock encountered is quartz. A rock containing quartz will cause high wear of the cutting tools.

The compressive-to-tensile-strength ratio indicates the toughness of the rock, that is, the ability of the rock to resist the separation of pieces, or fragments, from it. The rock can endure considerable plastic deformation prior to separating.

The point load test indicates the rock's resistance to penetration. The punch strength, similar to the point load test, also introduces indenters.

The following rock properties are reviewed below: plasticity, porosity, moisture, in situ stress, bedding/foliation, and discontinuities

Plasticity refers to the brittleness of the rock. If the rock is brittle, cutting efficiency is increased and the brittle rock allows for larger spacing between the cutters. However, for initial efficient chipping, greater forces will be needed, and cut interaction may not occur with plastic rocks, leading to a coring action.

Porosity reduces the penetrating force and may prevent effective formation of the pressure bulb. It may also interfere with crack initiation and propagation.

Moisture content affects the strength of the rock. In hard rock, moisture can have inconsistent effects. In soft/porous rock, strength may be reduced. Although it appears that moisture content has little or no known effect on the cuttability of hard rock, in soft porous rock it has an inconsistent effect on drag bits and reduces cutting forces of disc cutters.

Although there are no known effects on cuttability, in situ stress has significant effects on ground conditions.

Depending on orientation, foliation can have significant effects on the cutting rate, whereas the degree of the effects that the bedding has on the advance rate is dependent on the thickness of the bedding and the orientation.

Discontinuities can be joints or other cracks in the rock, regardless of type or origin. The effects of discontinuities on TBM mining are dependent on the orientation, frequency, and type of discontinuity. They can have a significant effect on overall production due to ground support issues.

The TBM's capacity or ability to do work is principally dependent on the thrust, torque, and type and size of cutters. There are ancillary factors, such as capably mucking excreta. A study of TBM performance on a particular project found that the TBM was the most reliable tunnel-related equipment on the project and delays in mining were the result of other factors, such as train derailment excreta (Hemphill, 1990).

Thrust is that force that pushes the TBM cutters against the rock face in order to penetrate the rock. The disc penetration of the rock is affected by the thrust. The total thrust is the pressure on the face exerted by the cutters, the friction between the TBM and ground, and the towing of the trailing gear behind the TBM.

The thrust imposed on the face per cutter can be determined by the following equation:

$$F_n = \frac{N_c p_c \pi r_c^2}{4N} - (t + f)$$

where F_n = normal force
p_c = applied hydraulic force
r_c = radius of the cylinder/piston

N = number of cutters, N_c = number of cylinders
t = hydraulic force to pull trailing gear
f = friction between rock and TBM

The thrust is exerted axially, and the TBM needs a mechanism from which to thrust, that is, something to push against. Grip clamping is the most common type of mechanism in rock tunnels. The clamping system is to use grippers that are curved to the radius of the tunnel and are pressed against the ribs of the tunnel. The gripping unit is braced against the rib using hydraulic cylinders. The line and grade of the TBM can be adjusted while in the clamping unit. The thrust cylinders expand by pushing the forward machine body and sliding on one invert shoe. This is often in the form of a partial shield. Once the TBM/cutterhead has been advanced, the cylinder stroke length is usually 1.2–1.5 m (4–5 ft). The cylinders retract, and the grippers are moved forward or segments are installed ready for the next push.

Cutting Tools

Cutting tools are attached to the TBM cutterhead and are used to penetrate the rock. The type of tool is determined by the type, or hardness, of the rock. In soils or very weak rock, drag-type tools are generally used. For hard rocks, disc cutters are used. Hard rocks require free-rolling cutters because of the high wear on stationary pick type of cutters, and pick cutters do not get the penetration into the rock.

Drag cutters (picks) resemble chisels that during cutting, the bits are forced against the rock, developing cutting forces parallel to the direction of head rotation and normal to the thrust. As they are dragged across the face, they develop microscopic fractures that spall away. These bits are generally used on TBMs only in soft ground.

Although not often used today, with the concepts taken from the petroleum industry, mill-tooth cutters and button cutters are part of the TBM cutting tool evolution. There are two types of roller cutters, the mill-tooth cutter and tungsten-carbide-insert cutters. Roller or mill-tooth cutting is similar to disk cutting except that, instead of a tapered disc edge, the cutter is equipped with circumferential teeth. As the cutter moves in response to rolling forces, each tooth is pushed into the rock and acts like a wedge, causing local failure. The milled tooth cutter is made from heat-treated alloy steels. The cutter is truncated and has a cone-shaped base with rows of teeth on the outside. In abrasive rock, tungsten carbide inserts may be used. The inserts may be shaped as a chisel similar to a tooth. They may have a rounded apex conical nose with a cone angle of 0°–120° or they may be hemispherical.

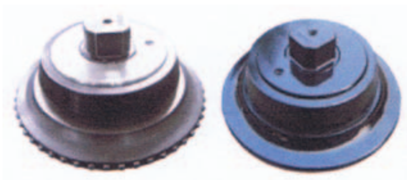
Button cutters have cylindrical or conical tool bodies that are inset with tungsten carbide buttons. They are mounted in a bearing similar to disc cutters and are free to roll, reacting to applied forces acting parallel to the rock face. Thrust forces cause high concentrations of stress under each button as they roll across the face, causing localized failure of the rock. The influenced area is small, resulting in small muck size, and the specific energy is high, resulting in higher costs. It is the least effective method and is used where high rock strength and abrasiveness preclude other methods.

Disc Cutters As the standard TBM cutter used today, the disc cutter generally consists of solid steel alloy discs with a tapered cutting edge, with a replaceable cutting edge of hardened steel around the perimeter. It is mounted on a bearing, making it free to roll over the surface in response to applied forces acting parallel to the rock face. It must be secured in a certain position attached to the cutterhead or in the cutterhead to prevent a swing motion when subjected to a heavy load. It must also be fairly easy to change out. For cutterheads greater than 4 m (13 ft), the discs can generally be changed from behind the cutterhead.

Thrust and drag forces are applied to the disc through the bearing, causing the bearing to be extremely important for cutting the face. The forces are both normal and parallel to the face. They may be single or multiedge discs and may have carbide inserts. (See Figure 7.6.)

They range from small up to 508 mm (20 in.) in diameter; the harder the rock is, the bigger the disc. Disc cutters are used principally on TBMs but are also mounted on other types of machines, for example, raise bore machines and continuous miners. Disc cutters can penetrate rock with strength exceeding 170 MPa (25,000 psi); in abrasive rocks, the capacity is reduced.

Discs roll in concentric tracks on the face as the cutterhead rotates. Single discs roll in their own tracks. Depending on the geologic conditions, the average disc loading is 100–250 kN (10–25 ton force). Cutters are available is sizes ranging from 280 to 508 mm (11 to 20 in.) in diameter.



(a)

(b)

(c)

Figure 7.6 Disc Cutters: (a) Single-Disc Cutters, (b) Twin-Disc Cutters; (c) Center-Disc Cutters

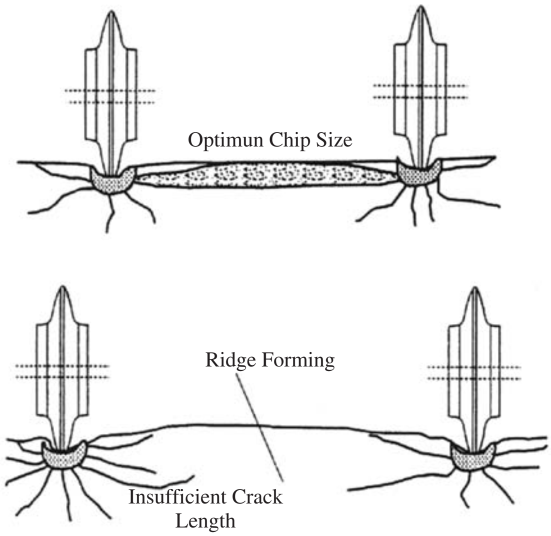


Figure 7.7 Disc Cutter Chipping Process (Adapted from Atlas Copco, 1982)

Kerf Principle

With the kerf principle, the cutterhead rotates with cutters being pushed against the rock face. The discs penetrate the rock, creating concentric grooves. As illustrated in Figure 7.7, the kerf spalls and chips rock using sheer stress and tensile stress caused by the penetration of the rock.

The basic principle of rock mining is that the rock is broken in shear and in tension. It is generally assumed that penetration is within 4–15 mm (0.16–0.60 in.), and in softer rock it is up to 20 mm (0.79 in.) or more.

Disc Cutter Spacing

The cutter spacing should be such that the adjacent grooves are close enough for the radial cracks to interact and extend, creating a chip. With larger cutters, the spacing may be in the range of 80–95 mm (2.5–3 in.). The optimum spacing will permit the largest chip and the maximum penetration rate.

Cutterhead The cutterhead is the component on which the discs are mounted and the excavated muck is accumulated by the bucket as the cutterhead rotates and is dropped on to the conveyor for transport to the rear of the machine. The buckets are located on the periphery of the cutterhead. The cutterhead supports the face when the TBM is stationary; in the case of cave-ins, it supports the face until the situation can be remedied, and it can carry overcutters. (See Figure 7.8.)

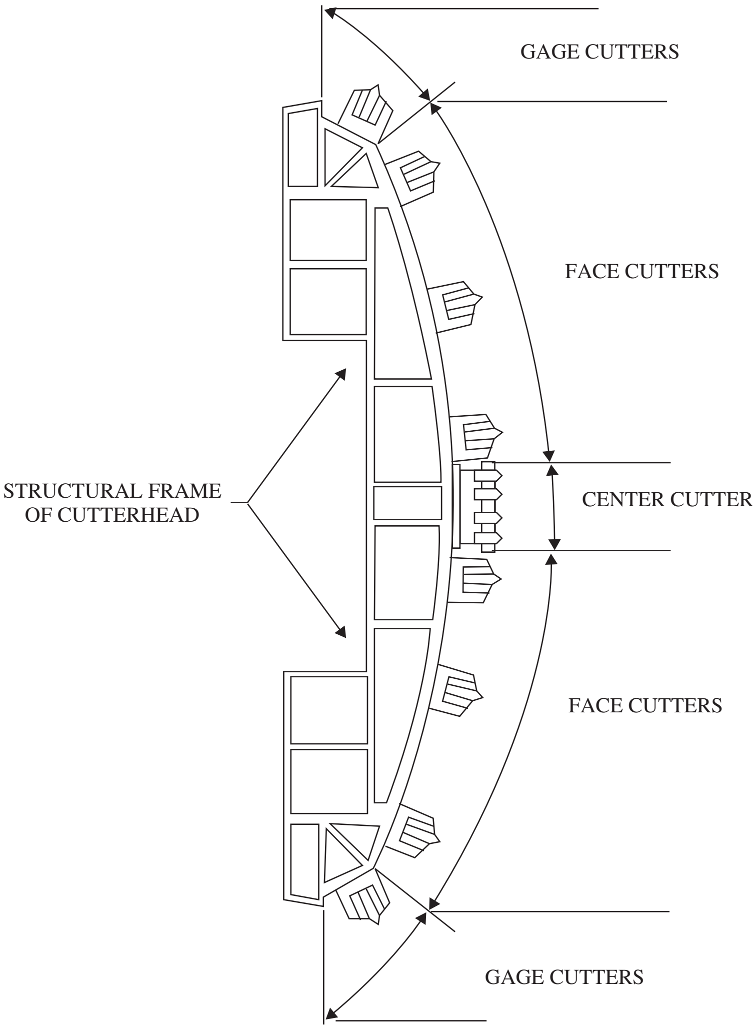


Figure 7.8 Cutterhead with Disc Locations

Backup Equipment

The backup gear, also referred to as "trailing gear," is towed behind the TBM and contains most systems and supplies needed by the TBM. The TBM tows a sledlike structure that carries the systems needed to support the TBM operation. As indicated in Figure 7.9, this includes the energy supply, rock support equipment, ventilation equipment, steering, the materials handling and transportation system, and various other components. The main transport system for materials on the TBM is the rail that runs the center of the trailing gear. Men and materials are transported to the TBM on this rail. Whether the materials are

Energy supply TBM

High-voltage cable drum
Transformers
Electrical cabinets
Hydraulic systems
Compressed air

Rock support

Segment erector
Segment storage
(invert segment or complete rings)
Grouting of annular gap
Drill equipment
Ring erector
Wire mesh erector
Installation meterial
(transport, storage, installation)

Ventilation

Main ventilation, air ducts
Ventilation TBM area
Dust suppression system

TBM steering

Steering cabin

Material transport

Conveyor belt
Meterial transfer station
Belt assembly and extension
Temporary muck storage
(stacker)

Components

Invert segments
Waterproofing materials
De-watering system
Cable canal
Invert fill
Transport roads

Figure 7.9 Backup Equipment List (Maidl et al., 2008)

Figure 7.10 Trailing Gear

transported for immediate use, such as chemical grout for ground conditioning or concrete segments for the lining, it will travel by rail. The grout kept on the TBM is generally limited to backfilling the annual space between the lining and the ground (Figure 7.10).

The segments are delivered to the face on a car specially designed for car segments. The segments are picked up by either mechanical or vacuum systems.

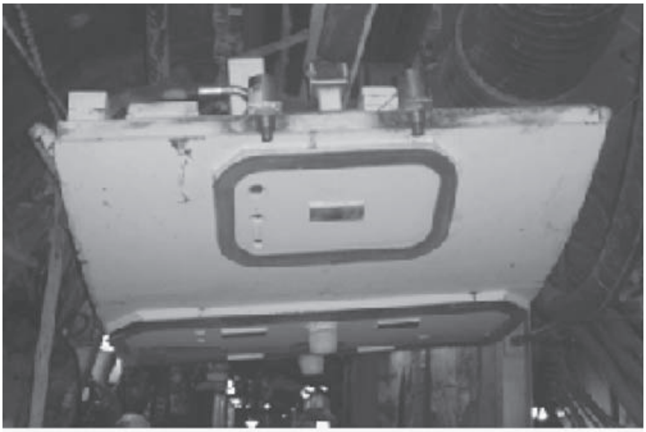


Figure 7.11 Vacuum Erector

The mechanical system consists of a rapid screw in the center of the segment where a large spherical headed screw sufficiently large enough to be hitched is inserted. The vacuum system uses suction to attach the segment through a large suction cup. For safety, there are two or three cups.

Figure 7.11 is a photograph of a segment handler. Note the large vacuum pad on the lower part of the device and a smaller pad closer to the camera. There is another smaller pad at the other end of the device. This handler uses a vacuum to lift and hold the segment. The segment is lifted from the segment car or from a trough at the invert. The segment is then placed at the appropriate location on the tunnel perimeter.

TBMs have several advantages over drilling and blasting. They can achieve greater production rates with less labor. They are safer for workers because of the lack of blasting, and explosives are not needed. Because there is no blasting, there is generally no concern with vibration and there is less overbreak. When backfilling with concrete or shotcrete is required because of overbreak, the lack of overbreak provided by a TBM is a cost saving.

However, using TBMs has a high capital cost, and the mobilization time is relatively long. The level of skill required is higher, there is a lack of versatility with regard to tunnel shape, and in many cases TBMs are not adaptable to changes in geology. There is high power consumption, and it has limited reuse.

The issue of reuse has improved. When considering a tunnel design, many owners are trying to size their tunnels to take into account available TBM sizes. When it does work, the construction cost can be considerably less, because the TBM is used and scheduling is improved because the lead/mobilization time is reduced.

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8 Ground Control

When a void is created by excavating, whether it is a tunnel, shaft, or cavern, and unless the ground is extremely massive and lacks discontinuities, it is necessary to keep the rock from caving into the void. The selection of the amount and type of ground support to use on a job can be the most important decision made, especially with regard to safety. In this chapter, we will discuss how to determine the ground control needed and the types of support available. The support may include steel sets or some type of reinforcement, which is support that penetrates the rock. Timber is still used in mining but will not be discussed here, and wood blocks are used to introduce the rock load to the steel set. Rock reinforcement provides tensile, shear, and/or frictional strength across discontinuities. It involves the installation of rock bolts, untensioned rock dowels, prestressed rock anchors, or wire tendons in the rock mass.

SUPPORT SYSTEM OBJECTIVES

The principal objective of the ground control system is to keep the ground from moving into the excavation, thus maintaining the utilization of the excavation. There are two primary approaches: active and passive. Active support is placed against the rock to be supported without penetrating the rock. The best example of this is steel sets placed against the rock to be supported; the steel sets carry the weight of the rock that, if left unsupported, would fall into the excavation.

Passive rock support becomes part of the rock mass and is used to help the rock support itself. The passive support, such as a rock bolt, reinforces and mobilizes the inherent strength of the rock mass by preventing blocks and other fragments from loosening and falling from the mass. This initial movement of the rock can cause the rock to peel away as the rock fragment or block ahead of it falls. Often, the full strength of the rock bolt is not being used because, rather than acting only as a rod or cable carrying the load of the rock, it holds only a fragment or small block of rock that is preventing other blocks from moving. Passive reinforcement provides tensile, frictional strength, and shear across the discontinuities. When tensioned, it provides shear through the shear strength of the reinforcement and the rock mass, being in compression across the discontinuities, and increases the shear strength where it would not exist.

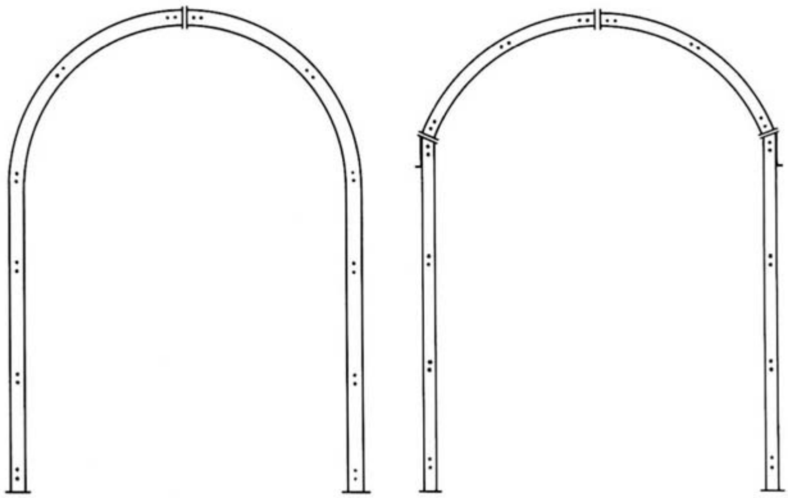
TYPES OF ROCK SUPPORT

Types of rock reinforcement include active support, that is, steel sets and lattice girders, and passive support, that is, rock bolts, which are tensioned reinforcement rods inserted into drilled holes that can have a mechanical or grouted anchorage and utilize a plate and a nut to retain the tension resulting from directly pulling or torquing the bolt.

Active Support

Steel sets are mostly used when sinking shaft in ragged blasted rock, starter tunnels, and building portals. Also, in some TBM situations, especially in soft ground, steel sets are used with lagging both for initial support and to thrust against for forward movement. Steel ribs consist of a structural shape, generally an H-section; the H-sections provide support in both directions or I-beams are used. Plates are welded at the ends for bolting them together. They generally consist of straight pieces that act as legs and curved sections for the crown. In TBM tunnels and in some shafts, all the steel is curved. In addition there can be struts at the invert. Ribs are typically spaced from 1 to 1.5 m (3 to 5 ft) apart, depending on the loading and operational factors. When being installed behind a TBM, the spacing is related to the thrust piston length.

Types of Steel Sets There are five types of rib support: continuous, rib and post, rib and wall plate, rib, wall plate and post, and circular. The continuous type generally (Figure 8.1a) consists of two pieces that are joined at the center of the crown forming the radius or curve for the crown shape.



(a) (b)

Figure 8.1 (a) Continuous- and (b) Rib-and-Post-Type Supports (Lewis and Clark, 1964)

Because there is generally only one connection, the continuous-type steel set usually is installed faster. The rib-and-post type (Figure 8.1b) has four pieces—two straight posts and two arched pieces. The fewer the number of pieces of steel per set, the better it is. Sometimes the set can consist of two pieces joined at the top by bolting the end plates.

Lateral loading on the posts can result in a loss of all the support provided by the steel ribs. Since the posts are attached only at the crown, the horizontal loading can cause an intolerable moment load on the top connection.

To increase the horizontal load capacity of the steel, a structural member called an invert strut is used. Invert struts are generally made of the same section as the structural member and extend across the invert and connect the posts at opposite walls, providing lateral support by forming a load-carrying capability similar to a circle. Figure 8.2 illustrates a typical invert strut connection.

Today, the most common use for circular ribs is for ground support and to act as a thrust block behind a TBM and for shaft sinking. The section lengths depend on the diameter of the tunnel and the weight of the beam. Commonly, circular ribs contain three sections. (See Figure 8.3.)

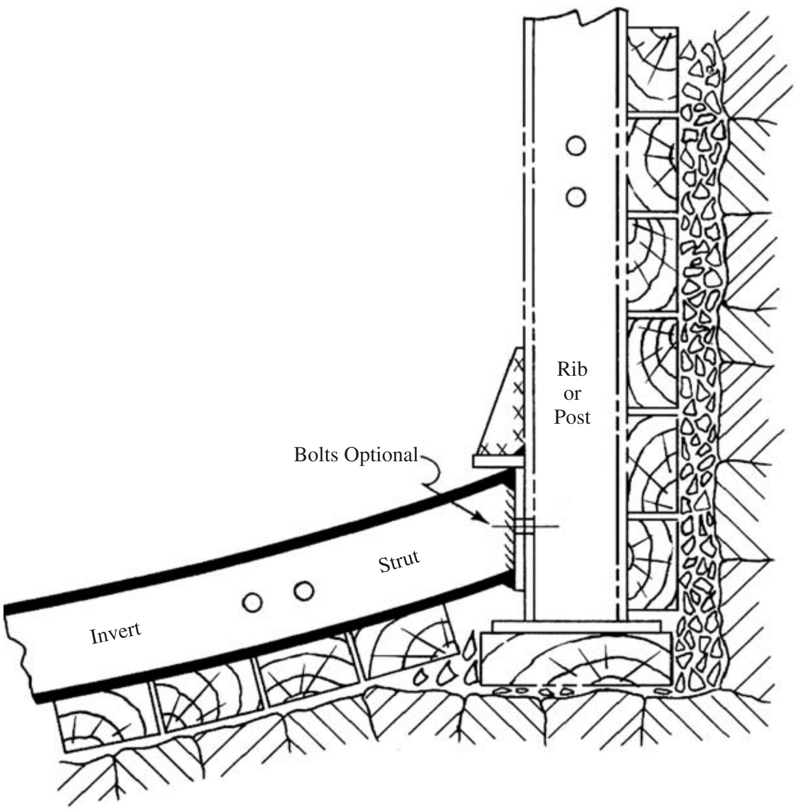


Figure 8.2 Typical Invert Strut Connection Detail (Proctor and White, 1977)

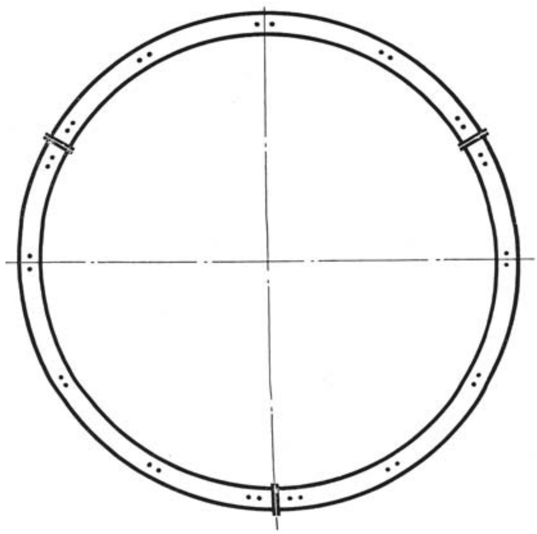


Figure 8.3 Circular Rib Type (Lewis and Clark, 1964)

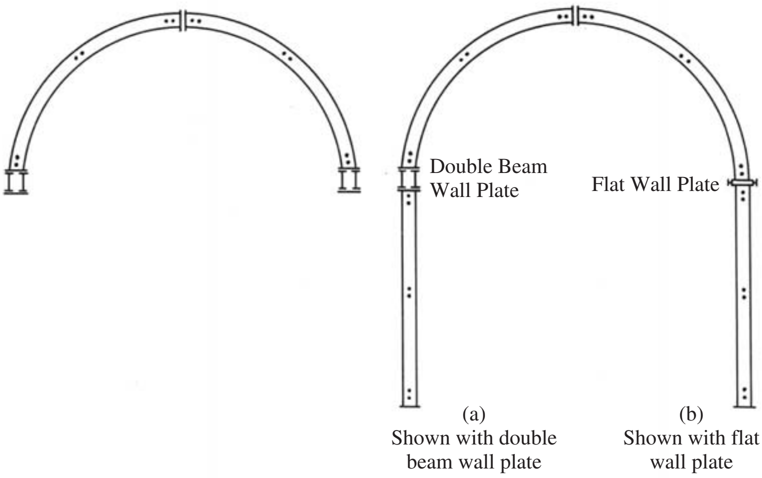


Figure 8.4 Rib and Wall Plate, Rib, and Wall Plate and Post Types (Lewis and Clark, 1964)

Steel sets can also be used with wall plates if it is impractical or is more efficient to do so. The wall plate permits the installation of the leg, or post, in sections. It is often used when the height of the tunnel is too great to efficiently install the steel sets, permitting working off the muck pile or for top heading and bench drifting. (See Figure 8.4.)

Wall Plates Rib and wall plates are assembled by first mounting a device, usually a small structural shape that is reinforced. Wall plates are selected based on available materials and loads. For normal installation, the entire cross section has to be mucked, or at least at the post locations. The wall plate allows part of the muck pile to remain to serve as a work platform. By installing wall plates, the crown and part of the leg post can be assembled. The wall plate has to be able to carry the load of the rib it is supporting, that is, not only the steel member but also the attachment to the wall.

The three types of wall plates are the double beam, single beam, and flat wall plate. The wall plate is mounted on the wall on which the rib is mounted. This acts as a shelf on which to set the post or occasionally the end of the crown rib where it intersects the vertical wall.

The double-beam wall plate consists of two beams, usually I-beams but also channel beams, connected by a diaphragm. The diaphragm creates a box shape which reduces twisting and rolling. In addition, because of their flat broad surfaces, double-beam plates are good for blocking. Figure 8.5 is an illustration of a double-beam wall plate.

The single-beam wall plate consists of a structure, generally an H-beam, reinforced by welding plates to the web as stiffeners. They are not as resistant to twisting and rolling as double-beam wall plates. (See Figure 8.6.)

The flat wall plate is usually the minimum wide-flanged beam into which the rib and post fit. Figure 8.7 illustrates ribs fitting in the flange of the wall plate.

Usually, miners need some type of mechanical aid to set the rib, at least the crown rib, in place. It can be done by machine or by use of a chain fall or other lifting aid, depending on the size opening, access to the spot in which the rib is to be installed, and availability of equipment. Equipment that can be used to lift the crown rib to the crown for installation can be a small crane, forklift, or excavator. Care must be taken that all safety regulations are followed during the process.

Installing Steel Sets Unless the beams are light enough for workers to handle, lifting the ribs into place will require some kind of mechanical advantage, for example, using a chain hoist or come-along mounted to the crown; of course, the rock will have to be stable enough to take the load. To gain mechanical advantage without using the crown rock, the ribs that have already been installed can be used. The crown bar is a beam or double channel that act as a cantilever. It acts as an aid or expedient for construction. Figure 8.8 illustrates the crown bar. The beam is suspended from ribs that have been installed. Then the crown rib can be placed on the crown bar and advanced and lifted into position.

Using a crown bar is similar to forepoling and can provide immediate ground support (see Figure 8.9) or support the ribs, eliminating the need for wall plates.

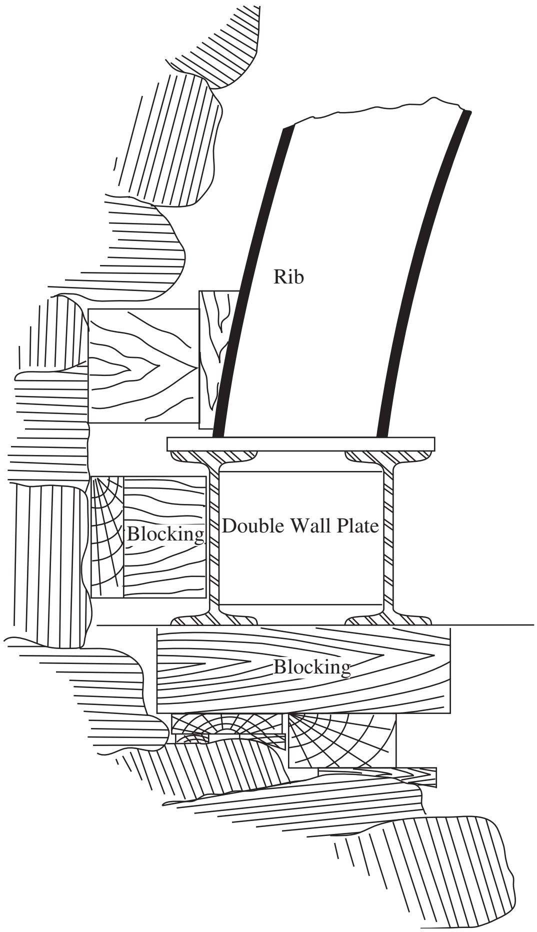


Figure 8.5 Double-Beam Wall Plate in Hitch (Adapted from Proctor and White, 1977)

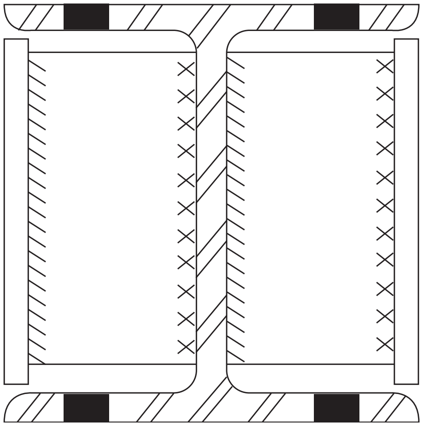


Figure 8.6 Single-Beam Wall Plate (Proctor and White, 1977)

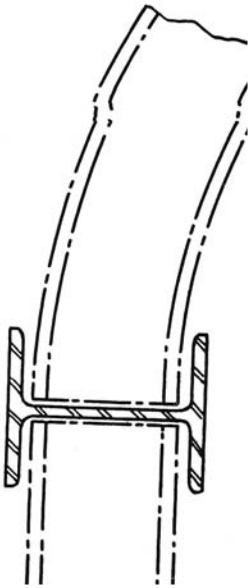


Figure 8.7 Flat Wall Plate (Proctor and White, 1977)

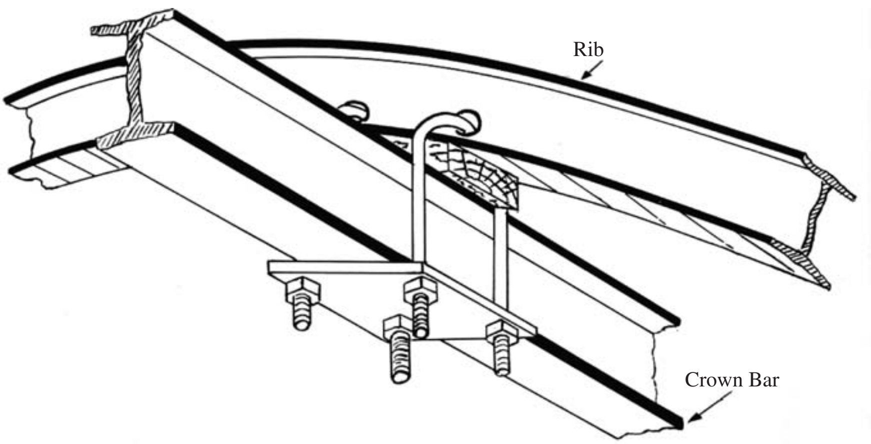


Figure 8.8 Crown Bar (Proctor and White, 1977)

Often, a short bridging action is needed to provide support during the heading and bench method of mining. Sometimes, a load that is greater than wall plates can carry needs the additional support provided by the crown beam.

Lattice girders are used in the same way as steel sets. They are lighter and thus easier to install and their strength-to-weight ratio is higher. They provide similar moment capacity. They are often used with shotcrete. Because

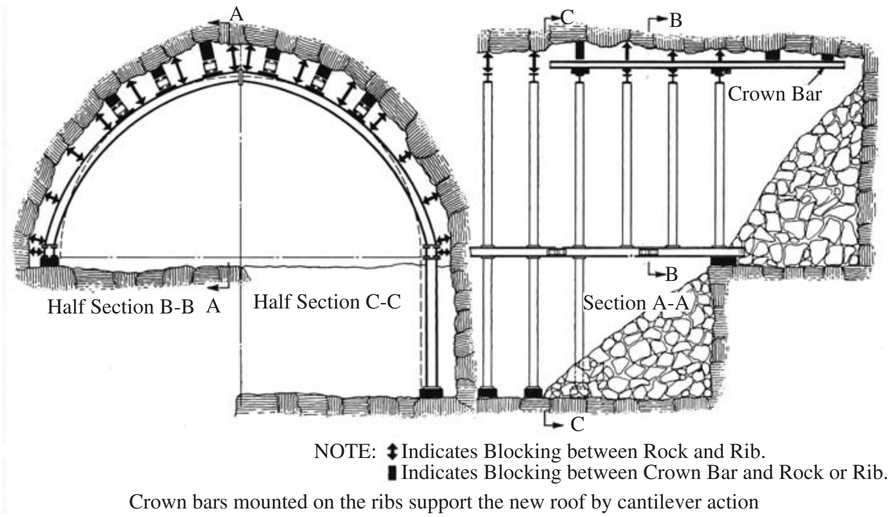


Figure 8.9 Crown Bar Used as Immediate Ground Support (Proctor and White, 1977)

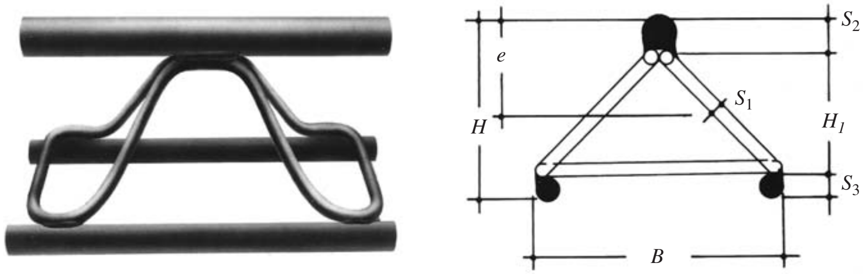


Figure 8.10 Three-Bar Lattice Girder (DSI)

of the open lattice, they can be covered with shotcrete with little or no voids. They are available with three or four bars. A girder is formed using three bars (Figure 8.10) with spaced diagonals. Based on the design geometry, the single bar can be located toward the inside of the tunnel or against the ground. Reduction of the local buckling length of the bars is achieved by the spaced diagonals. This assures high normal and bending moment resistance, securing the transfer of the lateral forces in the lining before the shotcrete has reached its design strength.

Construction of the four-bar lattice girder is the same as for the three-bar Girder; that is, with spaced diagonals, the bars are assembled to form a girder. Like the three-bar girder, it can be fabricated to the design geometry. The four-bar girder has the same moment resistance ability and can transfer lateral forces until the shotcrete has reached its design strength. (See Figure 8.11.)

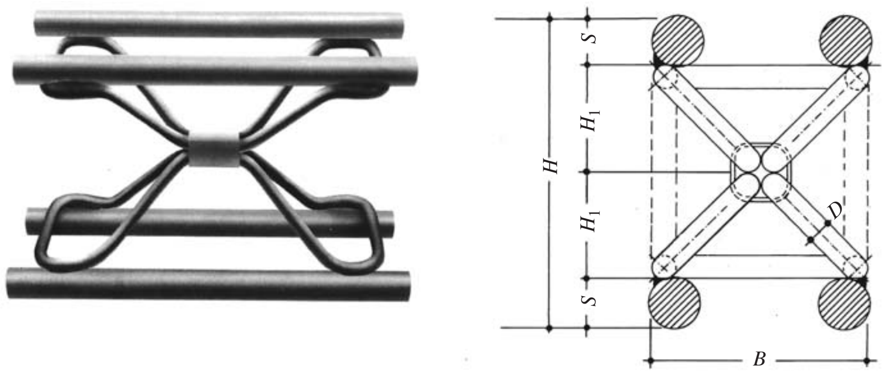


Figure 8.11 Cross Section of Four-Bar Lattice Girder (DSI)

In addition to providing immediate crown support over the excavated section, the lattice girder can act as a template to assure the shotcrete thickness. With its open construction, the girder becomes part of the reinforcement in the shotcrete lining. The lattice beam construction allows shotcrete to pass through it, easily reducing the possibility of shadows and unconsolidated shotcrete areas behind the beam. The quality of the shotcrete can be improved because it can be applied evenly, producing the superior, integral lining. The open fabrication also provides spaces to use to support spilling if it is required to advance the face. Lattice girders can be erected quickly by a small crew without special handling equipment.

Lattice girders are particularly well suited on projects where shotcrete is being used for temporary or permanent support. They can be fabricated to any size and can be used for driving through soft, hard, and mixed-face ground.

Blocking Steel sets are held in place by the use of wood blocks and wedges. Blocks are placed between the rib and the rock. The blocking is used to distribute the load of the rock across the set to mitigate differential loading. Blocking should be distributed as evenly as possible to ensure even distribution of the load. Vertical loads are transferred to the rib by the blocking. The rib will then tend to deform, or bow out, causing the load to be transferred through the rib to the blocking to the rock. It occurs until the loading is basically uniform on the set. Loads that occur at an angle to the vertical have the same effect in that the combined loads result in a thrust that is uniform throughout the rib.

Lagging The members of the tunnel support that span the spaces between the primary supporting ribs are called "lagging." Lagging provides protection from spalling or falling rock, and transfers the ground load to the ribs. Lagging provides a convenient surface to which to place blocking and a place to stuff back packing. If concrete is not to be formed against the ground, the lagging can serve as an outside form. It will also help to divert water, preventing leaching.

Lagging materials can be wood or steel. Wood lagging consists of hardwood boards 75–100 mm (3–4 in.) thick by 150–250 mm (6–10 in.) thick. The wood lagging is generally placed on the outside flange if there is space. If not space, it is placed on the inside flange. Depending on the ground conditions, the lagging may be solid in a granular running material or can be spaced such as for rock. (See Figure 8.12.)

Channel lagging consists of 100-, 125-, or 150-mm- (4-, 5-, or 6-in.-) wide rolled steel channels placed flatwise on the ribs, with the flanges outward. In addition, pressed steel channels of varied dimensions may be used. These pressed steel channels are the equivalent to wood lagging. Figure 8.13 illustrates the various types of lagging.

Figure 8.12 Ribs Using Wood Lagging (Note conveyor on lower right and rail at lower left.) (The National Archives, Her Majesty's Stationery Office, United Kingdom)

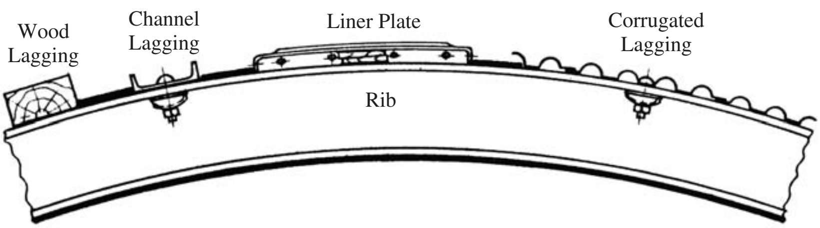


Figure 8.13 Types of Lagging (Proctor and White, 1977)

Rolled steel H-beams 100 or 125 mm (4 or 5 in.) deep attached an appropriate distance apart and attached to either flange are referred to as “beam lagging.”

Pressed channel lagging refers to channels pressed from a plate or strip with high strength developed by corrugations created. The ends are reduced in depth and thus require less head room. Generally, they are 300 mm (12 in.) wide and 50 mm (2 in.) deep to match the conditions.

A steel liner plate is a small steel rectangular plate that can be flanged on two or four sides. It is curved to the radius of the tunnel in the longitudinal direction. It is strengthened by deforming it with corrugations that provide a deeper section, thus increasing its section modulus. Also, some will have holes for grouting.

The plate dimensions can vary based on utility, that is, primarily by ease of handling, weight and size, and size of the cavity dug to install the plate. Liner plates are generally the same length, 957 mm (37.7 in.), and although the width may vary, 406 mm (16 in.) is the norm. Note that the length is 304 mm (12 in.) times π (3.14159). Thus, in English units a tunnel 8 ft in diameter will use eight plates.

Passive Support

Passive support is that which reinforces the rock, that is, it uses the rock’s strength. The passive support merely keeps the blocks from moving. It is similar to an ore pass or silo not flowing because the material is bound, preventing it from flowing. Rock reinforcement causes the binding of the rock to prevent it from moving into the cavity.

There are three methods for securing the rock bolt in the borehole: mechanically anchored, friction anchored, and grouted bars. The common types of mechanical anchors are slot and wedge and expansion. The slot-and-wedge type is achieved by inserting a wedge into the slotted end of the bolt. The slot is expanded by driving the wedge against the bottom of the drill hole. To get good anchorage, the rock should be hard and the hole must be within 75 mm (3 in.) of the length required. Because of the equipment required and the fact that the expansion anchor is much easier to install, the slot-and-wedge bolt is rarely used.

Mechanical The expansion shell anchored rock bolt is the most common form of mechanically anchored rock bolt. A wedge attached to the bolt shank is pulled into a conical expansion shell as the bolt is rotated, forcing the shell to expand against the wall of the borehole. (See Figure 8.14.) Once the bolt is rotated and the threads have forced the ridges on the wedge into the borehole wall rock, the support is in place. As the bolt is rotated, a torque is applied to the bolt head and tension accumulates in the bolt, permitting installation and tensioning in one step. The bolt can have a forged head, much like a standard bolt or thread that can tension from the head and wedge ends.

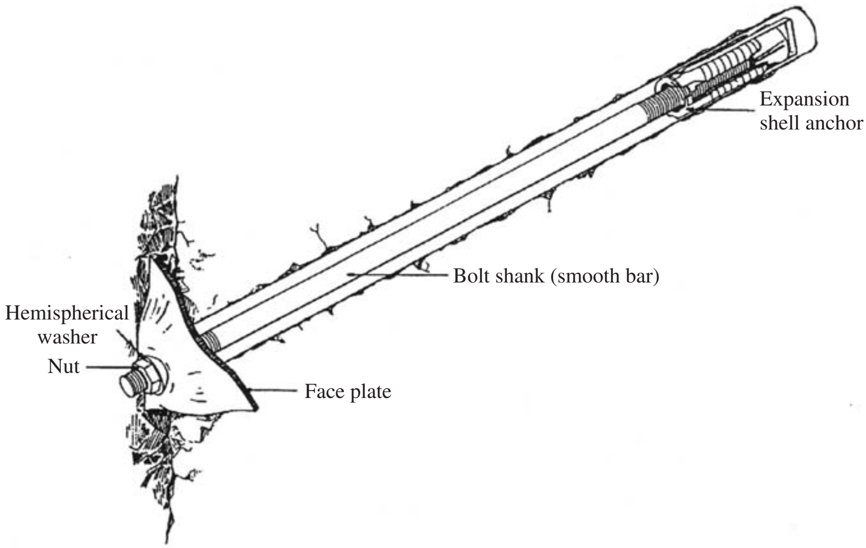


Figure 8.14 Expansion Shell Anchor Bolt (Department of the Army Engineer Manual 1110–1–2907)

It is a relatively inexpensive method of support but its use is limited to moderately hard to hard rock. In hard rock conditions it is a versatile system of rock reinforcement and can achieve high bolt loads. Some bolts are hollow and can be grouted. With an expansion shell approach, the grout acts more as corrosion protection, because the bolt is already tensioned prior to the grout being added. The bolt can be postgrouted, thereby being able to be viewed as permanent reinforcement. Figure 8.15 illustrates the types of rock bolts.

The bolts can be difficult to install reliably and the process must be monitored and checked for proper tensioning. Another difficulty is that blasting occurring nearby can cause the bolt to lose bearing capacity as a result of the vibrations or when rock spalls around the borehole collar due to high rock stresses. Fractured rock can spall between the bolts, leaving the bolts in place but without benefit for support. This type of rock bolt is the most commonly used system for rock mass stabilization, particularly in mining. The system is versatile and can be used in any excavation geometry. It is usually easy to apply and lends itself to mechanized installation.

Friction Friction rock bolts (split set) are rock stabilizers that rely on the friction between the bolt and the rock as support. (See Figure 8.16.) They consist of a high-strength steel tube that is slotted along its length and a matching domed bearing plate. One end has a welded ring flange to hold the bearing plate and the other end is tapered for easy insertion into a drill hole. Once the bearing plate is in place, the tube is driven into a slightly smaller hole using the

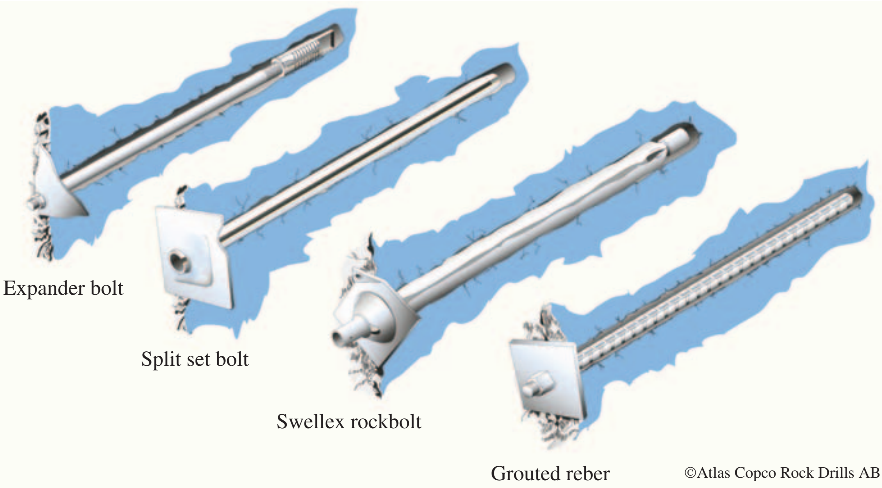


Figure 8.15 Types of Rock Bolts (Courtesy of Atlas Copco)

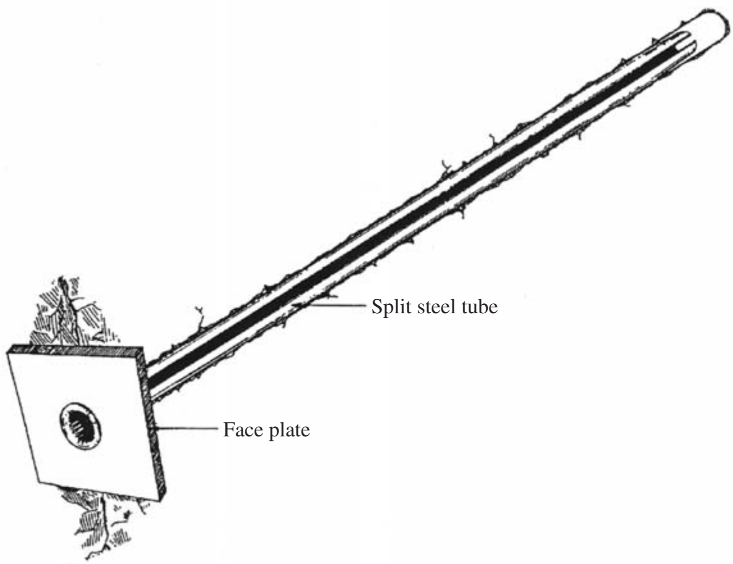


Figure 8.16 Friction Bolts—Split-Set Bolt (Department of the Army Engineer Manual 1110–1–2907)

same standard percussion drill that created the hole, such as a jackleg. Radial pressure is exerted against the rock over its full contact length because as the rock bolt slides into the hole the full length of the slot narrows. The rigidity of the metal being forced against the side of the hole as it is pushed into it causes a radial load against the rock, resulting in friction between the wall of the hole and the bolt. The bolt provides an immediate plate load support while exerting a radial pressure against the rock over the entire contact length.

Friction bolts are easy to install. The hole is drilled and the bolt is inserted with the drill. Friction bolts are designed for a specific diameter; therefore, it is important that the drill hole be accurate. After the hole is drilled, it should be cleaned with air or water. Then the bolt should be installed quickly.

Friction bolts are easy to install and therefore save labor and money. They have hangers to which welded wire mesh can be attached, facilitating mesh installation. Because of their ease of installation, shorter ones are often used with a hanger attachment in tunnels for hanging utilities. The bolts are short, 46–61 cm (18–24 in.), and are not used for ground support. They have loops on the bearing plates and are effective for hanging such lightweight items as ventilation pipe.

Swellex® bolts function on the same principle as split-set bolts, that is, by the friction of the bolt against the rock. Where the split-set bolt uses the stiffness of the metal to provide friction, the Swellex® bolt uses the deformation of the bolt caused by high-pressure water. The Swellex® bolt is made from a folded thin wall tube of steel. Bushings on both ends of the bolt are sealed by welding. High-pressure water is injected through a small hole in the lower bushing to expand the bolt. As it expands, the Swellex® bolt compresses the rock surrounding the hole and adapts its shape to fit the irregularities of the borehole. Figure 8.17 illustrates the ground support principle of the bolts.

Figure 8.18 illustrates the expansion principle of the Swellex® bolt. The upper part of the figure illustrates the folded shape of the thin steel tube. The miner drills a hole in the rock, inserts the bolt, and then inflates it to a predetermined pressure using a dedicated inflation system.

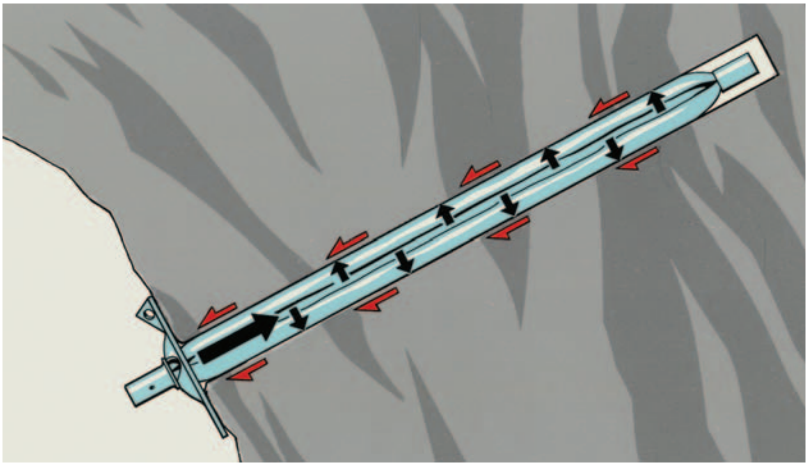


Figure 8.17 Swellex® Bolt Ground Support Stresses (Courtesy of Atlas Copco)

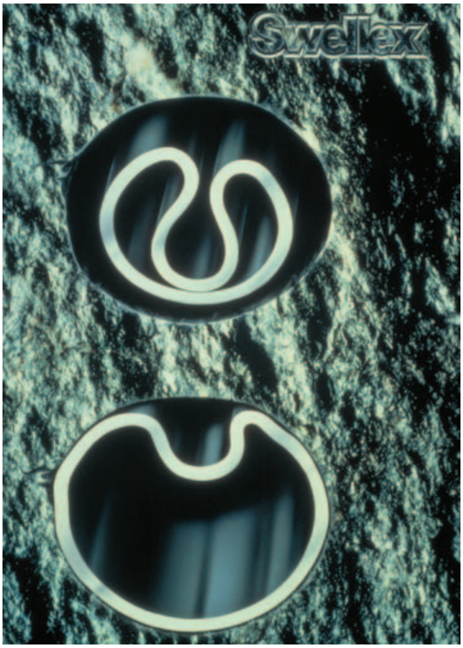


Figure 8.18 Swellex® Bolt Working Principle (Courtesy of Atlas Copco)

Figure 8.19 Swellex Pump (Courtesy of Atlas Copco)

When the water pressure increases, the walls of the bolt expand to the shape illustrated in the lower part of the figure. The Swellex® pumps stop when the recommended inflation pressure is reached. The expanded bolt puts stress against the wall of the hole. This stress provides friction between the bolt and rock. Figure 8.19 is a pump used for Swellex® bolt expansion.

There are two versions of the Swellex® bolt: the Swellex® Premium line, which is a relatively stiff rock bolt used for tunneling and mining in moderate stress conditions, and the Swellex® Manganese line, which is a highly deformable rock bolt for large rock deformation. The Swellex® strengthens the rock mass by a combination of mechanical interlock at the rock and bolt interface and friction. Because of the tight interlock with the borehole, corrosion can develop in an aggressive environment. To counter this, Swellex® can come with bitumen or plastic, significantly extending its life. In soft rock, Swellex® provides instant reinforcement. Another application of the Swellex® is in soft rock underground openings, where immediate support is required following the excavation. Due to its unique anchorage mechanism, Swellex® bolts can adapt to a wide variety of ground conditions. In fact, Swellex® bolts perform better in holes that have rough or uneven walls. The bolt will deform into the rough and uneven areas. After installation, the bolt is held in the hole by the contact stress between the bolt and the borehole due to the elastic recovery of the rock material, as well as the mechanical interlock due to the roughness of the borehole.

The primary contact stress plays a significant role in the anchorage of the bolt in soft rock. The anchorage of the bolt is determined mainly by the secondary contact stress due to the roughness of the hole. The irregular surface of the borehole wall must be rough enough and the bolt tube intimately matched to get satisfactory results in hard rock.

The Swellex® bolts are less sensitive to blasting or slight rock movements than mechanical bolts. Figure 8.20 illustrates the bolt's deformation with the borehole and its ability to deform to adjust to the direction of the load. Swellex® is best used in tensile strength situations. Due to the thin wall, the bolt has little strength in shear.

As with other bolts, wire mesh and other surface protection can be attached to Swellex®. (See Figure 8.21.)

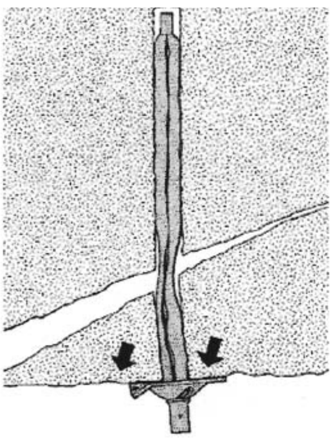


Figure 8.20 Swellex® Bolt Deformed to Match Borehole Wall and Load (Courtesy of Atlas Copco)

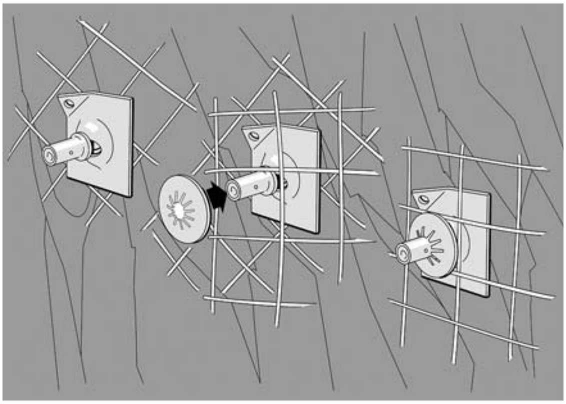


Figure 8.21 Attaching Wire Mesh to Swellex® Bolts (Courtesy of Atlas Copco)

Grouted Rock Bolts Tensioned bolts are used to prevent movement along the axis of the bolt, that is, normal to the rock surface. Although a rock dowel is not tensioned, it still has to have something to anchor it to the bottom of the hole. The anchorage is important in that it permits the tensioning of the rod or cable. If the anchorage fails, the support the bolt can be useless, because the ground reinforcement is based on the element being tensioned, and thus tensile strength is required. The anchorage strength should be greater than the ultimate strength of the bolt. The strength of the bolt is limited by the strength of the anchorage. The anchorage allows tensioning to the desired prestress, putting the rock in the vicinity of the hole in compression by applying force against the rock that tightens and increases the shear strength of the discontinuities. This is similar to what happens when several books are stacked and then, using one's hands, are compressed by squeezing them together. The force from squeezing the ends toward each other increases the friction, and thus the shear strength, between the book covers, permitting one to carry them in the vertical position. The books remain in place, because the shear strength is greater than the force of gravity acting on the books. Also, when in compression, the rock resists tensile loading; that is, the bolts also hold up the rock if necessary.

Most commonly used grouted rock bolt is the fully grouted rebar or threaded rebar made of steel. Cement grout or resin is used as the grouting agent individually or in combination. Rebar used with resin creates a system commonly used for tensioned rock bolts. Rebar or threaded bar with cement grout can also be used for untensioned bolts. Both systems are used for temporary as well as permanent support under various rock conditions. Threaded rock bolt is mainly used in civil engineering applications for permanent installation.

Rebar is used when relatively low strength rock bolt and rock dowel are required. Prestressing high-strength steel is done to allow the maximum design load in each borehole. A reduction in the number of bolts in the pattern may be allowed by prestressing steel, and thus it will usually reduce the cost of drilling.

In preparation for installation of the bolt, each borehole must be cleaned with either air or water prior to inserting the resin cartridge. Although standing or flowing water does not affect the resin, it may cause deterioration of the hole. Therefore, it is a good idea to make sure all standing water is removed from the hole, usually with air.

It is important that the borehole and cartridge diameter be compatible with the diameter of the threaded bar specified; that is, to gain required bonding strength, the manufacturer's recommendations must be followed. Borehole diameters other than those recommended may be used under special conditions. However, testing may be necessary to determine that methods and tools used to mix the resin with the catalyst meet requirements. Accessories such as anchor plates will vary in size, both area and thickness, to match the bearing conditions and utility. Bevel washers, flat washers, or anchor nuts are required for rebar bolts. When prestressing thread bars where the plate is not perpendicular to the rod, a wedge washer may be used.

When installing the bar, based on the manufacturer's recommendation, one should drill the smallest diameter borehole which is compatible with the bolt and cartridge diameter selected. Fast-setting resin should be used for bond length. For tensioning and when full encapsulation is required, after the bottom of the borehole, the anchor portion, slow-setting resin or cement grout should be used in the upper length to accommodate stressing. In this way, the hole can be fully encapsulated. After the grout in the anchor has set and prior to the remaining, slower setting cement or resin grout have set, the bolt can be tensioned and the bolt is protected from corrosion.

Generally, the bar should be rotated with a drill at about 100 rpm for 30–60 secs once the bar reaches the bottom of the hole, being careful not to exceed the gel time. The bearing plate should be mounted and secured with an anchor plate. If the plate is not perpendicular to the bar, wedge washers should be used. After the resin has not set, the bar can be stressed.

Depending on the type of resin and the temperature, the setting time will vary from 1 to 20 min.

The bolt tension is monitored by reading the pressure gauge where the hydraulic jack is utilized. Where torque or air wrench is utilized, the bolt tension is monitored by developing a tension–torque relationship curve for the specific bolt.

Generally, threaded bar can have the permanent load induced on it or be stressed to proof load. Because of the threadlike deformations, stressing is simplified. A torque wrench, an air wrench, or a center hole hydraulic jack can be used for stressing. The rock is a determinant of the anchorage length and will vary.

Unlike mechanically anchored bolts, resin is not affected by blasting and the resin provides corrosion protection. It is free from deterioration from water, fresh or salt, or mild acid and alkalis. Like any adhesive, the resin gel time and

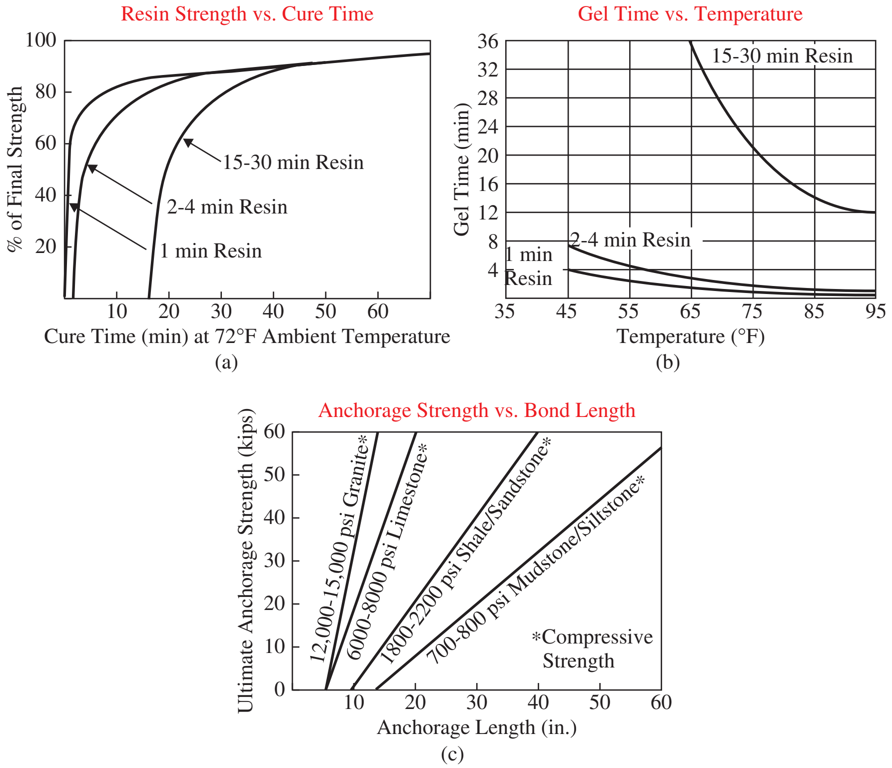


Figure 8.22 Effects of Temperature on Resin (DSI)

cure time are sensitive to temperature. Therefore, the ambient temperature of the rock must be monitored.

The graphs in Figure 8.22 illustrate the effects of temperature on gel time, percentage of final strength, ultimate anchor strength, and resin performance.

When a grouted rebar is used, the bolt gives rapid support action after the installation because of the “fast-setting” resin. When resin is used to bottom anchor the bar, the fully grouted rock bolt can be tensioned, providing not only support but also high corrosion resistance in permanent installations. Figure 8.23 is an illustration of a bolt tensioning pump.

Resin has a limited shelf life, and with the difficulties that resin cartridges have in underground environments, installation reliability can be affected and the cartridges can be wasteful, messy, and hazardous. Figure 8.24 illustrates the three bolts used for grouting. Figure 8.24a shows a rebar with bottom resin anchorage. Unless the bar is coated to prevent corrosion, the ungrouted length of the bar is not protected from corrosion by grout and thus cannot be considered a permanent support. Figures 8.24b and c

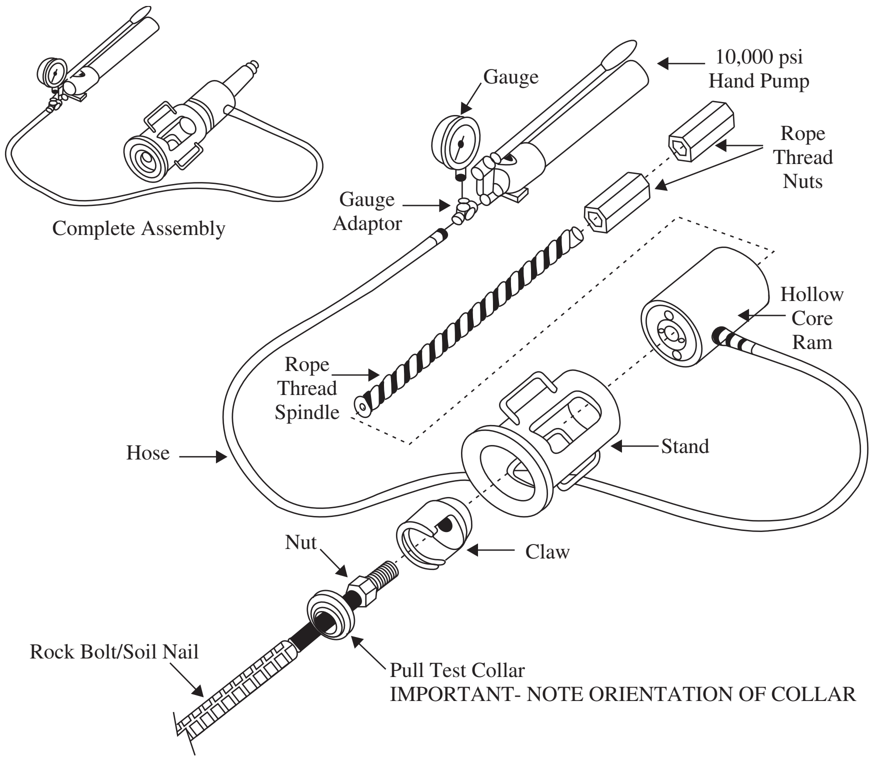


Figure 8.23 Bolt Tensioning/Testing Device (DSI)

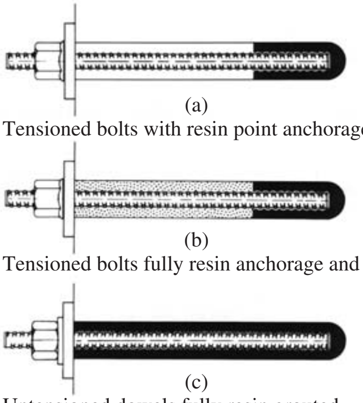


Figure 8.24 (a) Rod with Bottom Anchor Resin Only, (b) Bottom Resin Grout with the Remainder of the Borehole Being Grouted with Cement Grout, (c) Entire Bar Grouted with Resin (DSI)

represent bars with resin anchor grout, with cement and resin covering the remainder of the bar, respectively, permitting them to be used for permanent support.

At the rock face, the bar is anchored using a threaded nut, which, unlike a wedge-type anchor, will not slip when the stress in the anchor is reduced due to possible ground movements, such as blasting. In addition, the threaded nut anchor has an overload capacity that without utilization of elaborate and expensive details is not available with a wedge-type anchor.

When properly installed, a threaded bar has good corrosion resistance and is a competent and durable reinforcement system. In various rock conditions, high bolt loads are provided by the system. Threadbars are expensive, and tensioning of the rock bolt is possible only if special installation procedures are followed. The use of standard cement grout requires several days curing before the bolt can take the load and the quality of grout is difficult to check and maintain.

Prestressed rock anchors or tendons are similar to rock bolts in that they are tensioned but have a greater capacity for tensioning because they are made of higher strength material. They consist of high-strength steel rods or one or more wires, strands, or bars.

A prestressed rock anchor is a high-strength steel tendon fitted with a stressing anchor at one end and a way to permit the force to be transfer to the grout and rock on the other end. The rock anchor tendon is inserted into a prepared hole of suitable length and diameter, fixed to the rock and prestressed to a specified force.

Glass-Reinforced Polymer (GRP) Rod The GRP composite rebar is made from high-strength glass fibers with a durable vinyl ester resin. Resistance to corrosion in harsh chemical and alkaline environments is provided by the vinyl ester resin, while the strength is provided the glass fibers.

GRP rebar is nonmetallic, noncorrosive, and nonconductive. Its tensile strength is twice that of steel and has a consistent modulus of elasticity (about 43). The deformation pattern is the same as steel, and it is four times lighter. It is well suited for use in tunnels and mines. It can be severed by mining machines, permitting the equipment to mine through areas that are supported with the bolts. It is particularly useful when expanding an existing supported opening.

The GRP rebar has a surface roughened by helical winding or a textured surface and enhanced shear bonding between the grout and the rod. It has a high injection molded thread and composite nut. The bolts can be installed by existing methods, and because they are relatively lightweight, they are easy to transport and handle. They have high strength and are tough against impact. They provide a noncorrosive and nonconductive excellent bonding capacity. They can be cut without the danger of sparking, are fire resistant, and have an antistatic rating.

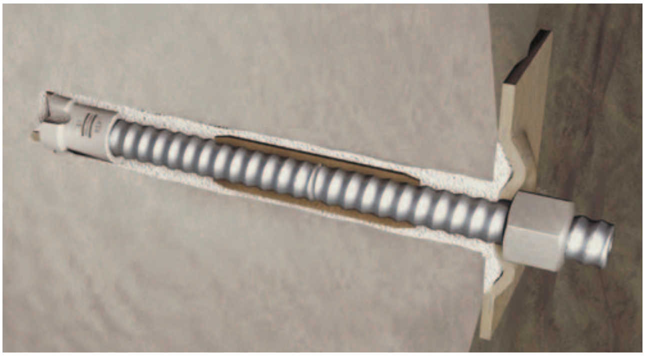


Figure 8.25 The MAI Self-Drilling Anchor (Courtesy of Atlas Copco)

Self-Drilling The MAI self-drilling anchor (SDA) is used for unstable ground conditions such as sand, gravel, silt, and clays and in soft to medium fractured rock formations. These systems are used to stabilize weak rock and soil in tunneling, mining, and other ground engineering applications. When drilling in collapsing soil or loose or decomposed rock, a sacrificial drill bit and hollow rod are used. After the drilling is done, cementitious or resin grout is injected into the hollow rod and the surrounding cavity.

The most crucial part of the anchoring system is the sacrificial drill bit. It is responsible for the productivity of the installation. The selection of the most suitable drill bit is required for a successful installation of the system. Self-drilling hollow core anchor systems use a hollow drill rod for injection of the grout. Figure 8.25 is a photo of the Atlas Copco MAI SDA.

For flushing or simultaneous drilling and grouting, the anchor rod has a hollow bore. In addition, it has a left-hand thread for connection to standard drill tooling. The standard rope thread of the anchor rod produces an excellent bond between the rod and grout, and enables connection to other drilling rigs.

The type of support depends on the material to be supported, strength required, speed of installation, durability, cost, and ease of installation.

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9 Mucking and Haulage

Mucking involves removing or collecting the excavated material at the face and preparing it for transport out of the tunnel. Muck can be of many shapes, sizes, and moisture content. Its engineering properties can be abrasive, weak, or strong. It can be of various specific gravities and forms. Simply put, muck can be any geologic material with varying water content. It can be dense abrasive rock that can cause wear to the mucking equipment, or it can be soil with water content, which results in increased water handling. The muck can be sticky, which creates problems in dumping the material. The stickiness also will reduce capacities and require more labor and time to clean out the bucket. When dumping a car or muck bucket, some of the sticky material will adhere to the bucket, thus preventing it from being completely emptied. If material stuck to the bottom of the bucket is not removed, the capacity of the bucket is reduced. Cleaning the muck from the bottom of the bucket will require time and additional labor, but if it is not cleaned, productivity will decrease. Having an additional bucket can mitigate the time lost, but the labor is still required.

Bank quantity is the measurement of the material in its undisturbed state. Swell is the measurement of the material after it has been disturbed by blasting or digging. The swell of the excavated material must be considered. When the material is dug or blasted from the face, its loading density is decreased. That is, a ton of material will require more space once it is disturbed. In surface work, it is referred as "bank quantity, in cubic meters or yards, and loose volume." The difference in the bank quantity and loose quantity is the swell factor. The swell factor can vary from about 15% for sand and gravel to 80% and higher for blasted rock.

When mucking blasted rock, the rock should be fragmented sufficiently to permit effective mucking productivity. If the rock fragments are too large to be handled by the mucking system, secondary breakage will be required—generally mechanical, but for larger fragments, it may require blasting.

MUCKING METHODS

Mucking methods may vary from a miner shoveling muck into a wheelbarrow to highly automated conveyor or slurry systems. For non-TBM tunnels, rails, crawlers, or rubber-mounted muckers are used. Rails offer an easily maintained roadway that is less affected by a poor invert.

For long haulage distances and inverts that are difficult to maintain, rail-mounted loaders are preferred. Rubber-tire and crawler loaders are used where the ability to move laterally is required. Rail-bound loaders are suited for tunnels with a tunnel width of 4 m (13 ft) and needs at least 4 m (13 ft) height for the overthrow action of the machine and cross section less than 4×4 m (13×13 ft). From a single track the workable width of a large rail-bound loader is about 4 m (13 ft) and the height of the machine extended determine the height required. The Eimco 630 overshot mucker has been a staple of both rail and crawler muckers in smaller headings. (See Figure 9.1.) The muck is scooped up by the forward movement of the mucker and the vertical lift of the bucket. The bucket arm assembly rotates vertically to the rear of the machine and the muck is thrown into a nearby muck car or bucket, in the case of shafts, to its rear.

The crawler version of the Eimco 630 can operate in a larger area and has the flexibility of being able to turn at any angle. The operating principle is the same but mobility is different. Also, where a rail-mounted loader is restricted to a grade of about 2%, the crawler version can work at grades up to 40%. It is not considered good for haulage much more than a few unit lengths and requires the same headroom as the rail-mounted 630. The crawler-mounted version is used sinking large shafts. Being air operated, the ventilation requirements for the unit are not an issue. The overshot mucker has 0.13–0.6 m^3 (4.6 –21.0 ft^3) bucket capacity.

In addition to overshot rail-mounted muckers, there are rail-mounted mucking loaders that have a backhoe-type pulling arm with a centrally located conveyor. The boom arm pulls the muck into its conveyor, which transports the muck to the rear for dumping into a rail car or onto a conveyor.

Figure 9.1 Eimco 630 Overshot Rail-Mounted Mucker (Courtesy of the Society for Mining, Metallurgy, and Exploration)

Crawler and rubber tire excavators can be used for surface excavation or modified for underground excavation. The use of surface equipment is dictated by the dimensions of the tunnel or underground excavation. Space permitting, surface-excavating equipment seen underground can be front-end loaders (FELs), backhoes of various sizes, and trucks (diesel).

Load–Haul–Dump Machines

load–haul–dump (LHD) machines are capable, as the name indicates, of loading, hauling, and dumping the muck; they function similar to front-end loaders and trucks combined. The machines are narrower and not as tall as a FEL, and generally longer. The narrower width and loader profile permit working in areas denied a FEL. In a wide tunnel, it is possible to locate the LHD to the side of the tunnel, allowing LHD's to pass each other at will. Generally powered by diesel engines, there are ventilation considerations, but electrically powered motors are available in smaller units.

As opposed to the FEL, which requires an operator for the machine and a driver for the truck to operate, since the LHD also hauls the muck, only one miner is needed to operate the LHD. They are self-loading and function much like a FEL. They force their bucket into the muck and with a prying motion the bucket is tilted back to the carrying position. The muck is trammed (hauled) to the dump site. The boom rests on the machine frame rather than on hydraulic cylinders, adding to its durability over FELs. The machines are generally more durable than surface equipment.

On the LHD machine, the driver's seat is generally mounted sideways, permitting good visibility in both directions. However, some smaller machines may face forward. Also, the machine is bidirectional in that it has the same number of travel gears in reverse allowing it to travel in reverse at near the same speed as forward. Figure 9.2 is a photograph of a LHD machine.

Although they can be as small as 0.76 m^3 (1 yd^3), standard bucket capacities range generally from approximately 3.1 m^3 (4.1 yd^3) to 8.6 m^3 (11.3 yd^3) and optional bucket capacities of about 11.6 m^3 (15.2 ft^3) are available. Load weight

Figure 9.2 The LHD Scoop Tram (Courtesy of Atlas Copco)

capacities can range between 6800 kg (14991 lb) and 20,000 kg (44100 lb). Resting the boom on the frame can reduce the digging depth to approximately 64 mm (2.5 in.).

The LHD machine will operate on any roadway on which it can fit. Thus, there is maximum flexibility in operation and the roadway extension is not rigidly determined to heading advance. Because of its narrow width and low profile, the LHD machines can maneuver in smaller spaces than surface equipment and a central articulation using power steering and hinged pins permits low-radius turns. The roadbed is generally hard to maintain, but the vehicle can negotiate rough roads. It can maneuver on grades up to 20–25%. Using trucks for haulage increases the ventilation requirements. Because of the potential tramming speed of the LHD machine and the ventilation considerations when using trucks, LHD machines can be used to tram greater distances.

Figure 9.3 presents rates of productivity versus bucket size and distance. Note that the difference in productivity between various bucket sizes decreases with length of the haul. LHD machines can haul materials back to the heading, satisfying materials-handling needs. In the event of a vehicle accident, it is less likely that the tunneling operation will be interrupted, as it would be with rail haulage.

Mine Trucks

For longer runs of drill and blast tunnels, the use of mine trucks can be warranted. Figure 9.4 is a photo of a MT 42 Mine truck. Generally, the muck is initially loaded by a scoop tram and is hauled to a muck bay. Muck bays, or niches, are constructed to store the muck from the round. (See Figure 9.5) When the face is ready to be mucked, a scoop tram will muck out the face by tramming the muck to a niche. The LHD machine will continue to tram

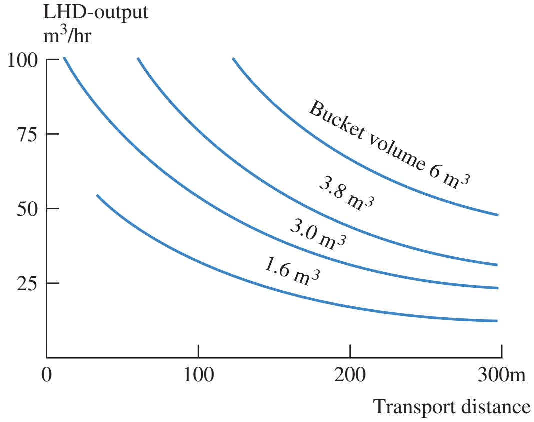

Figure 9.3 LHD Productivity Graph (From Atlas Copco Manual, 1982)

Figure 9.4 MT 42 Mine Truck (Courtesy of Atlas Copco)

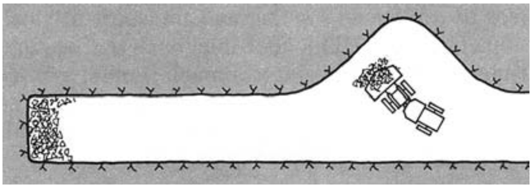

Figure 9.5 Muck Niche (Adapted from *Atlas Copco Manual*, 1982)

the muck until the face is ready to be drilled. During the drilling cycle, the scoop tram will load muck from the niche into a mine truck for haulage to the disposal area. Figure 9.5 is a sketch illustrating a LHD machine tramming muck from the face to the muck bay.

Muck bays are generally located about 150 m (500 ft) apart along the tunnel. Although the muck is being handled twice, the heading is cleared much faster, permitting the drilling to begin sooner than mucking and hauling in one operation. The muck is hauled while the face is being drilled. Therefore, the hauling portion of the mucking operation is done off of the critical path.

Continuous Mucking Machine

Continuous mucking machines lift the muck from the floor directly onto a conveyor belt which then loads it into a truck or onto another conveyor. High-speed electric trolley trucks also have the ability of being used to bring the muck to the surface. The use of conveyors for transporting muck from tunnels is increasing.

Rail Transport

Of the numerous factors to be required for tractive effort to move a train, only four need to be considered here. The first factor to be considered is that to move a train on a level track a force of 2–5 lb/ton of train weight is needed. At high speeds about 5 lb is needed, whereas 2–3 lb will suffice for yard speeds. Overcoming bearing friction, rail deflection, minor flange contact, and other resisters to movement requires this force. Although the bearing resistance used to require more energy to start the operation, with the advent of roller bearing journals, this is no longer a consideration when determining tractive effort. Although air resistance at speeds above 30–40 miles/hr is significant, due to the aerodynamics differences between trains, air resistance is not included here.

Track curvature is the second factor to consider. Since the wheels are mounted on solid axles, cars in a curve require considerable tractive effort. Therefore, the wheels must slip and slide through the curve caused by the difference in the radius of the outside and inside rails. Also, contact between the wheel flange and the rail causes additional friction. The tractive effort for cars in a curve is determined using 0.8 lb per ton per degree of curvature where curvature is defined as the number of degrees the track curves per 100 ft. To calculate the degree of curve radius, if the radius is known, divide 5730 ft by the radius. As an example, a track with a 700-ft radius would have 8% curvature. Another method for determining the tractive effort is to stretch a cord or tape tight across two points on the rail that are 62 ft apart. The degree of curvature is determined by measuring the distance between the rail and the midpoint of the cord.

It is considered prudent to limit the curvature to 12°; however, in an industrial yard where space is limited, the curvature can reach 20°. However, wheel and rail wear will be excessive.

Perhaps the leading factor governing tractive effort is the percentage of grade. For every ton of train weight at grade, 20 lb of tractive effort is needed per 1% of slope of train weight.

The equation for this is

$$T = 20(W_t \beta)$$

where T = tractive effort in pounds
W_t = weight of train in tons
β = percent of grade (1% grade is the vertical increase in elevation measured as number of feet of vertical increase pet 100 ft of horizontal distance. An increase of 1% is considered steep and an increase of 2% is unusually steep).

Therefore, for a 10-ton train on a 1% grade, the tractive effort required is

$$\begin{aligned} T &= 20(10 \times 1) \\ &= 200 \end{aligned}$$

Figure 9.6 Brookville Locomotive

The tractive effort required for acceleration of the train is the final factor and one that is frequently overlooked. A reasonable rate for a heavy train to accelerate to a speed of 6 mph in 1 min or 12 mph in 2 min is about 10 lb/ton. If one increases this tractive effort, the acceleration rate increases proportionately.

Summing the factors will give the total tractive effort required. Each factor will vary for any point on the rail system, depending on the load, grade, or curve and whether acceleration is required at that location. A train may be able to obtain a "running start" at a hill, and the kinetic energy of the train can be converted to elevation using the equation

$$S^2/30 = H$$

where S = speed in miles per hour
H = elevation in feet

The following formula is used to determine the power required to move the train:

$$P = TS/375$$

where P = power in horsepower at the rails
T = tractive effort in pounds
S = speed in miles per hour

Auxiliaries such as the air compressor, cooling fan, charging alternator, and traction generator and losses such as motor losses, drive losses, and gearing losses reduce the engine horsepower at the flywheel. The reduction in power to the rail caused by these losses can be 20–30%. The rated engine power available at the rail for a modern alternating current (AC) traction locomotive is about 80% versus about 70% for the older direct current (DC) units.

As an example, assume a DC muck train on a straight track weighs 70 tons and has a slope of 3%. To determine the tractive effort and the power required, the following equations are used:

$$\begin{aligned} T &= \text{on level 350 lb} \\ &= 4200 \text{ lb (3\% grade)} \\ &= 4550 \text{ lb } (4200 + 350) \end{aligned}$$

Then,

$$\begin{aligned} P &= (4550 \times 20)/375 \\ &= 242 \text{ hp} \end{aligned}$$

Efficiency factor is 70% (diesel)

$$\begin{aligned} 242/0.7 &= 345 \\ P &= 257 \text{ hp (308 kW)} \end{aligned}$$

The selection of rail as opposed to conveyor or rubber tire haulage is based on the tunnel dimensions, including length; it is easier to mobilize a rubber tire loader for short tunnels. Cost and productivity, which can be closely related, can play a major role in determining the haulage system. Although one method provides greater productivity, the capital costs may not be justified.

Rail offers an easily maintained roadway that is less affected by a poor invert provided the track is maintained. It has many functions in tunnel construction. It transports muck from the heading to the dumping point, either a shaft or dumping facility outside the portal. It is also instrumental in moving personnel and materials. Personnel are moved in specially designed cars that provide seats and are enclosed to prevent personnel being injured by an obstruction fouling the tunnel.

For transporting materials, various cars may be employed. Some are modified flat cars, but some are specially fabricated cars for a particular purpose. Cars can be used for transporting explosives in accordance with safety regulations. These are often an explosive magazine mounted onto a car undercarriage. Cars may be designed for carrying concrete. In addition, in some cases, slightly modified flatcars may be used for carrying vent pipe, grouting and shotcreting equipment, segmental lining segments, or rail, that is, whatever is needed at the heading. It is the only haulage system that offers such synergism. Rail can also be combined with conveyors to provide the materials handling.

There are various types of muck cars that can be used for transporting the muck. Figure 9.7 shows a train of muck cars. The principal difference is in the way they are emptied. Muck can be loaded at the heading by rail-mounted, crawler-mounted, -rubber-tire-mounted, or conveyor loaders. For optimum productivity, generally muck cars should be as large as possible.

Figure 9.7 Locomotive Positioning to Couple to Muck Cars

Bottom-dump muck cars, as the name implies, are emptied by the opening of doors located at the bottom of the car. The muck drops into a chute, bin, or muck pile and is transported generally by truck. The Granby side-dump car has a roller attached to the side of the car. The roller travels up the cam and the car is tipped. A modification of this type of car uses a hydraulic cylinder that lifts the car box to dump it to the side and then returns it to its frame.

Other cars are dumped by being lifted by a hoist or crane. The box is lifted from the underframe and is turned over to empty. This is often done with an additional hook on the crane. Thus, one hook holds the car while the other turns it over, dumping it.

Some tunnels are large enough to have two parallel tracks in the tunnel to provide transporting of men and materials. However, some tunnels are too narrow, and if there is limited space in a tunnel and more than one train cannot operate in the tunnel at the same time, there is considerable reduction in production. To solve this problem, structures are fabricated to allow the trains to pass each other. There are two common types, the California switch and the Jacobs sliding floor.

The California switch is a structure that is placed on a single track. (See Figure 9.8.) The switch it is a steel deck with additional tracks. This is a section of track that is parallel to the tunnel track and is connected to it at both ends by switches. It is made of two jump-type Y-switches, with rails all welded onto steel ties. It acts like a typical railroad, allowing trains traveling in opposite directions to pass. However, standard railroad sidings are at grade.

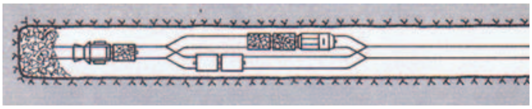

Figure 9.8 Detail of an End of the California Switch (From *Atlas Copco Manual,* 1982)

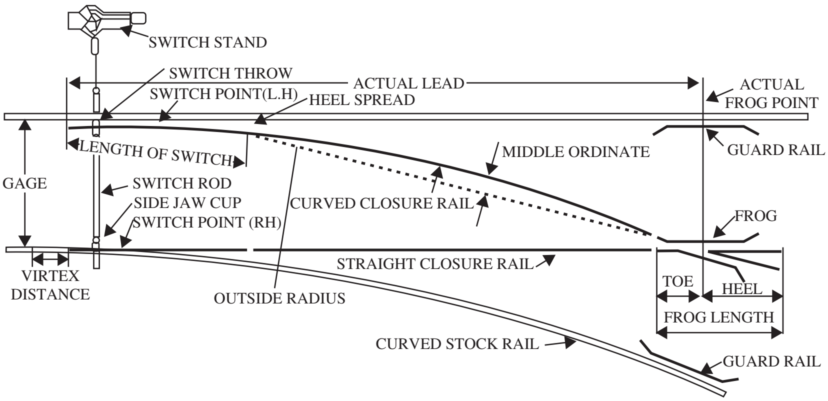

Figure 9.9 Typical Turnout with Parts (Courtesy of Bethlehem Steel)

The California switch has a ramp and switch at each end. The ramp is so constructed that the tracks on the ramp align with the lower principal track. The deck floor, depending on the width, will have at last one additional track. It has to be longer than the trains so that one train can locate on one of the tracks, allowing another train to pass.

The Jacobs sliding floor is used in drill-blast tunnels wider than 10 ft and generally longer than 2 miles It consists of a deck of steel extending from within 3 m (10 ft) of the excavation face for a distance ranging from 75 to 135 m (250 to 450 ft) from the face and wide enough to carry railroad and jumbo tracks. A facility for passing cars (built-in California switch) that is central to the sliding floor extends the full length of the floor. There is an increase in advance rates because the sliding floor advancement is incorporated into the mucking cycle, and the car-passing switch facilitates much faster mucking because muck car switching is possible. An electric-over-hydraulic system makes the sliding floor self-propelled, allowing the entire assembly to move forward as the tunnel is advanced. Advanced by hydraulic cylinders the sliding floor will typically consist of three or more sections. To move the floor forward requires from 2 to 4 min. This can be incorporated into the tunneling cycle and accounted for in the schedule. Permanent track is laid behind the floor from a storage magazine at its trailing end. This eliminates the need for laying short sections of railroad track at the face, thus taking the laying of track away from the face. As the floor advances, ties are placed beneath these rails on a smooth, uniform, and firm roadbed. During mucking operations, the jumbo can be “parked” on the rear end of the sliding floor on a special section of track provided for that purpose. The length of the structure is long enough to provide storage space for a loaded muck train and a train of empty cars on separate parallel tracks. To prevent damage from falling muck, the forward unit of the floor is reinforced. Because a large portion of the muck pile rests on the steel deck after the blast, mucking is facilitated, because the steel floor provides the same advantage as

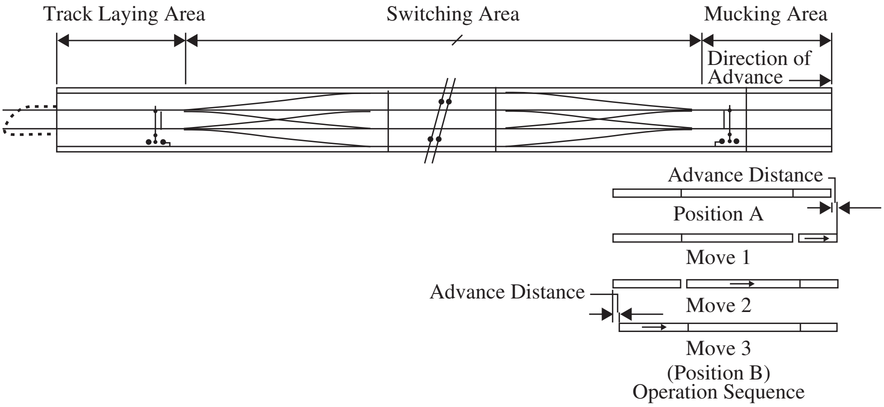

Figure 9.10 Jacobs Sliding Floor (Courtesy of Jacobs Associates)

"slick plates" for hand mucking. The Jacobs sliding floor has proven to be useful and can provide substantial savings in time and cost when driving a tunnel by drill and blast methods. Figure 9.10 is an illustration of the Jacobs sliding floor.

The Jacobs sliding floor offers several advantages over using just a single tunnel rail or laying track near the face:

It provides faster mucking.

Car switching takes less time (because the passing switch is located immediately behind the mucking machine it only takes up to 1 min to change each car).

Since much of the muck is scooped from a steel deck, the mucking machine loads faster and more efficiently.

The problem of derailment of the mucking machine is eliminated because the muck track is straight, level, and firmly positioned.

Straight and accurately positioned rails for gantry type, or mainline-type drilling jumbos, reduce setup time.

Well-placed track way reduces the maintenance costs of mucking machines and other heading equipment.

There are improved maintenance requirements of locomotives and muck cars.

Equipment can be stored near the face.

Most sliding floor track arrangements permit both the drill jumbo and the mucking machine to be parked within 300 or 400 ft of the face.

The sliding floor is from 90 to 150 ft in length and is usually made with three or more sections. In order to facilitate transportation, the sections are fabricated a maximum of 35 ft in length. They are connected together through vertical flexible joints at the final assembly. Hydraulic cylinders are mounted horizontally and longitudinally, and are attached to adjoining sections so that, when the cylinders are expanded, one section moves away from the other, with the result that the floor increases in length. When a floor system consists of more than two sections, the system is propelled forward by operating the cylinders in sequence so that the reaction from the movement of a single section is against the weight of the two or more remaining stationary sections.

The front of the sliding floor is kept within one round length of the face during all of the activities of the drill blast cycle. The railroad tracks are permanently attached to the sliding floor for the locomotives, cars, and mucking machines. If the width of the tunnel permits, wide-gauge tracks are provided for a gantry-type drill jumbo. When operating in small tunnels that do not permit the use of gantry equipment, the drill jumbo operates on the same track as the mucking equipment. The sliding floor is most advantageously used in combination with rail-type equipment, and it is necessary to use a mucking machine that can excavate ahead of and below its own track level. The U.S.-built Conway muckers are ideally suited for this type of operation. To ensure proper utilization of the sliding floor, it is imperative the invert be in excellent condition or the advantages provided for equipment setup will be moot.

Generally, mechanical conveyors are short so they can remove the muck that they are producing from the immediate face area. The conveyors dump into trucks, muck cars, or another conveyor. Conveyors are often used to transport muck from the heading to the surface. They have the advantage that production does not stop to wait for muck transport. It is considerably faster than rail or rubber tire, and it functions independently of other transport equipment, allowing the other equipment to service the tunnel with workers, materials, and equipment.

The speed with which a conveyor can transport muck depends on the width of the conveyor belt and the material being transported. Table 9.1 illustrates the speed with which material can be moved for various belt widths.

Table 9.1 Normal and Maximum Recommended Belt Speeds

Belt Width	Lump or Moderately Abrasive Material (such as well-shot gravel sand)	Heavy Sharp or Very Abrasive Materials (such as poorly shot rock)
450 mm (18 in.)	91–122 m/min (300–400 ft/min)	76–106 m/min (250–350 ft/min)
600 mm (24 in.)	152–183 m/min (500–600 ft/min)	122–152 ft/min (400–500 ft/min)
914 mm (36 in.)	152–183 m/min (500–600 ft/min)	122–152 ft/min (400–500 ft/min)
1219 mm (48 in.)	213 m (700 ft/min)	189 m/min (600 ft/min)

Source: Adapted from Continental Conveyor and Equipment Company.

The principal factors when determining what type of mucking transport systems to use are:

Size of tunnel

Length of tunnel

Type of material

Availability of equipment

Capital cost

The size of the tunnel will have a direct effect on the size of equipment used for mucking. Obviously, a larger tunnel cross section will permit larger equipment. Therefore, one might be able to use a scoop tram instead of a 630 mucker. The length of the tunnel does not necessarily affect the choice of mucking equipment, but it does affect the choice of muck transport. That is, if it is a very long distance from the heading to the portal, a scoop tram would not be efficient for hauling. It could still be used for mucking and dumping the muck into a muck car or a conveyor hopper. The type of material refers to the abrasiveness of the material. Very abrasive material may cause undue damage to conveyor belts. (See Figure 9.11.)

Is the equipment available? This can be paired with capital costs. In a short tunnel, contractors may use equipment they already have rather than purchasing new equipment that may be more suitable for the task. The newer or more modern equipment may be more efficient, but is the additional cost worth the increased capital cost? Whatever equipment is chosen, it is probably going to have to provide the lowest project cost. The bottom line is the bottom line.

Figure 9.11 Conveyor for Underground Muck Transport

Figure 9.12 Vertical Shaft Conveyor (Courtesy of D'Angelo International)

Vertical conveyors have come into use for transporting muck up the shaft. (See Figure 9.12.) Vertical conveyors can bring the rock up a shaft to the surface whether the tunnel is 20 or 200 m deep. Vertical conveyors are so designed that they will reliably deliver the rock to the surface with minimal spillage. Generally, the vertical conveyors are custom designed to suit the project requirements, and the systems may include muck transfer from the horizontal conveyor to the vertical conveyor or from the vertical conveyor to an overland or stacker conveyor.

Freeing the shaft from being used for hoisting muck has benefits in addition to faster muck disposal. The shaft is now clear for the hoisting of workers and materials, thus eliminating shaft muck haulage delays. Also, not having to raise and lower muck buckets in the shaft provides a safer work environment.

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10 Grout

A good way to start a debate on a tunnel project is a discussion about grout. Two tunnelers may have had grouting experience in the same ground and have different results with the same method; one had success and the other did not. There may be several reasons for this, such as the means and methods used or the wrong type of grout; however, the primary reason may be that there is no such thing as the "same" ground; there is only similar ground. Even in the same tunnel, ground characteristics can vary, thus affecting not only excavation and ground support but also grouting. Simply put, in underground excavations grouting is done to improve the ground, with the grout acting like cement, binding the soil particles together, filling cracks in rock to reduce water inflow, preventing movement and increasing shear strength, filling voids, or replacing the undesirable soil with the grout or a soil cement-type material.

Grout is a mixture of cementitious and noncementitious material that may or may not have some type of aggregate to which water or some other type of fluid is added producing grout with a flowing consistency. The process of injecting suspensions, emulsions, and/or solutions under pressure into rock, soil, voids, or cavities for the purpose of stabilizing or strengthening the ground, filling voids, or sealing the ground is called grouting.

OBJECTIVES OF GROUTING

The objective of grouting is to reduce the permeability of the rock, reduce the compressibility of the ground, compress the ground to provide stability, and seal the rock. The effectiveness of the grouting program is dependent on the drilling method(s), the ground characteristics, the type of grout, and the grouting procedure.

The types of grouting include jet grouting, permeation grouting, consolidation grouting, compaction grouting, pregrouting, postgrouting, backfill grouting, and contact grouting; each has its purposes and techniques.

GEOTECHNICAL

The type of ground and what one is trying to accomplish are the two principal factors in determining what and how to grout. For example, the type and method of grouting a rock formation to block water and the method to strengthen clay it will likely vary.

Key characteristics to consider when determining the method and type of grout to be used for sealing off water are permeability and porosity. Permeability and porosity are two of the physical factors that determine the movement of fluids in rocks and soil. These characteristics are intrinsic to soil and rock. The movement of water through soil and rock is partly dependent on the properties of porosity and permeability. Most rocks are considered to be impermeable. The flow of water through rock is the result of discontinuities such as joints and cracks.

Porosity is the measurement of the volume of voids to the total volume of soil or rock. Expressed as a percentage, it is the space between the grains of soil and the grains and/or discontinuities in rock. It represents the ability of the material to hold fluids. In rock the porosity consists of the spaces between the grains that make up that material. The lower the porosity, the more tightly packed are the rock gains. In sedimentary rocks, formed by the buildup of rock fragments, grains, or shells, the porosity can be as low as zero to as much as 90% with 15% being the typical value (Goodman, 1989). Well-sorted sands can have a high porosity of 25–50%, whereas the porosity of silt and clay can range between 30 and 60%.

The size, mixture, and shape of the particles that compose soil and rock determine the porosity. That is, clays, which have same particles, are able to compact more tightly together, increasing the density and reducing the amount of porosity. Sand and gravel, which have larger particles, will have more spaces available between them. If the particles are angular, the porosity will be lower than sand and gravel with round particles. That is, the porosity of soils is determined by the shape of the grains, soil particle size, distribution of particles, sorting, fabric, degree of compaction, and range of grain sizes present. If the grains are poorly sorted, in soils with a larger range of grain sizes, the porosity will be greater because the spaces between the large grain sizes will be larger. However, when soils have a larger size distribution, the finer grains tend to fill the spaces between the larger grains, resulting in lower porosity. Porosity can range from about 1% for granite to 55% in some soils. In some rock, the porosity is increased through fractures or matrix of the material itself. This is known as “secondary porosity.” Secondary porosity is a subsequent or separate porosity system in a rock that often increases the overall porosity of a rock. This can be caused by chemical leeching of minerals or the generation of a fracture system. This porosity can be of a magnitude to replace the primary porosity or coexist with it.

Permeability is the measurement of the ease with which fluids (i.e., water) will flow though rock or soil. Like porosity, the degree of packing, shape, and distribution of granular materials is what controls their permeability. A rock can be highly porous, but if the voids are not interconnected, the fluids within the pores are closed. That is, fluids cannot move between the isolated pores. Effective porosity is the degree to which pores within the material are interconnected. Rocks that can have high porosity include pumice and shale, but they can be nearly impermeable if the voids are poorly interconnected. Well-sorted sandstone that contains rounded sand grains can provide significant permeability. The rounded grains afford unrestricted void spaces that are free from smaller grains and are very well linked. Therefore, sandstone of this type is rock that may have both high porosity and high permeability.

The permeability of geologic materials has a very large range of values. The most conductive materials have permeability values that are millions of times greater than those that are less conductive. The permeability of the geologic material can often be directional in nature. The way that the particles intersect is a characteristic that may cause the permeability to be significantly greater in one direction. The permeability of the material often can be significantly impacted by such secondary porosity features as fractures. The viscosity and pressure of the fluid also affect the rate at which the fluid will flow. The coefficient of permeability is the rate of water flow expressed as square centimeters or feet per day.

Table 10.1 gives types of soil and rock and their respective coefficients of permeability. As the soils become finer and more unconsolidated, the coefficient of permeability is reduced. For example, the unconsolidated sand and gravel become less pervious as the particles become smaller and less graded. The rocks are affected by the discontinuities. Note the highly fractured rock is highly permeable, but as the rock becomes more massive, for example, fresh granite, the permeability deceases. With the loss of discontinuities, the conduits for water flow reduce.

Part of the permeability of rock is related to the number and size of openings. These openings, or discontinuities, are fractures, joints, cracks, and faults in the rock mass. The water inflow can be predicted by doing pump testing, but this may have limited results. Many scientific and engineering methods for determining potential water inflow have improved predicting grout take. However, it should be realized that, like all geotechnical testing and predictions, predictions

Table 10.1 Permeability of Selected Soil and Rock

Permeability	Pervious			Semi-Pervious				Impervious					
Unconsolidated Sand and Gravel	Well Sorted Gravel	Well Sorted Sand or Sand & Gravel				Very Fine Sand, Silt, Loess, Loam							
Unconsolidated Clay & Organic					Peat		Layered Clay			Unweathered Clay			
Consolidated Rocks	Highly Fracture Rock				Oil Reservoir Rocks			Fresh Sandstone		Fresh Limestone, Dolomite	Fresh Granite		
(cm²)	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}	10^{-11}	10^{-12}	10^{-13}	10^{-14}	10^{-15}

Source: Adapted from Bear (1972).

of the probable inflow, or grout take, of a geologic material are still estimates at best. The degree of difficulty of grouting is dependent on the number of joints that have to be treated: single or few joints are easier to treat than sugar cube rocks. Of course, the aperture size of joints and any filling in them are important considerations. Hydraulic conductivity may be dry or saturated. That is, if the geologic material is dry, the fluid will be absorbed by the soil or rock until it is saturated. Saturated conductivity is the maximum rate of water movement through the geologic material.

Table 10.2 presents possible estimates of water inflow based on fissure width (aperture) and water pressure head of 20 and 100 m (65 and 328 ft). The inflow presented from fissures can vary considerably with apertures that are only a fraction of a millimeter.

In hard rock the sealing is accomplished by filling the fissures with grout, that is, permeation grouting. In fault zones and decomposed rock, pores have to be permeation grouted, while bands and zones of clay, that is, fault gouge, need to be washed out, replaced, and/or compacted. Silty materials need to be both permeation grouted and compacted, whereas sandy/gravelly materials need to be permeation grouted. A high degree of tightness is normally required in fault zones and decomposed rock to avoid erosion of sandy and silty layers.

Preconstruction geotechnical site investigations give a general idea of rock conditions. However, the need for grouting is mainly decided by the size of the fissure and whether it is open or closed. Generally such details are not often obtained from the preconstruction investigations. Water pressure testing (falling head, raising head, and pump testing) may give some indications, but to decide on the need for pretreatment of the ground, the area ahead of the tunnel face has to be probed. Water inflows from holes and/or water pressure tests give an indication of conditions ahead of the excavation and the need for pregrouting.

Grouting of rock is difficult and unpredictable, as the grout may go in any direction, giving an incomplete treatment. Treatment may have to be repeated to get any degree of protection against inrush of water.

Table 10.2 Predicted Water Inflow Based on Aperture of Fracture and Head of Water

Aperture of Fissure		Water Pressure (Head)			
		20 m (65 ft), 28 psi		100 m (328 ft), 142 psi	
mm	In.	l/min	gal/min	l/min	gal/min
0.1	0.004	≈1	≈0.26	≈5	1.3
0.2	0.008	5–10	1.3–2.6	25–50	6.6–13.2
0.4	0.16	50–100	13.2–26.4	500	132

Source: Adapted from Littlejohn (1975).

Table 10.3 Determination of Grout and Methods Based on Rock Mass Classifications

Rock Mass Jointed Rock	Q	RMR	Grouting Required	Type of Grout Material
Massive, no joints	1000	I	Not required	—
Very few joints <0.1 joints/m^3 (0.13/yd^3)	100	I	Spot/target grouting	MFC, if joints >0.5 mm used OPC
Few Joints, <1/m^3 (1.3/yd^3), ≤ 2 joint sets	10	II	Limited continuous	MFC
Jointed rock, <10/m^3 (13/yd^3), >2 joint	1	III	Continuous	MFC
Very jointed rock, ≥ 10 joints/m^3		III, IV	Continuous, closer spacing, in stages	MFC/UFC
Fault Zones				
Zones with clay	<0.1	V	Displace, wash out/replace, compact	OPC/possibly MFC
Silty zones	<0.1	V	Penetrate, very close spacing, stages	UFC, chemicals
Sandy zones	<0.1	V	Penetrate, close spacing, in stages	MFC, UFC
Gravel zones/sugar cube rock	<0.1	IV, V	Penetrate, quick set, in stages	OPC, MFC
Mixed material	<0.1	IV, V	Penetrate, displace, compact, replace, in stages, close spacing	OPC/MFC → UFC/chemicals
Regional Structural Zones	The grout materials and techniques depend on the size and composition of the zone. Often it consists of penetrate, displace, compact, replace, in stages, close spacing. Using OPC/MFC→UFC/chemicals			

OPC = ordinary portland cement, maximum grain size range 120–14 μm
MFC = microfine cemen, range of particle size 0–30 μm
UFC = ultrafine cement, grain size range 0–15 μm
Chemical grout contains no particles
Source: From Sjöström 1980.

When estimating required grouting prior to construction, rock mass classifications may assist with determining the type of cement to use for grouting. Table 10.3 presents the RMR and Q values of rock. It should be noted that the rock class grouting required and the type of grouting materials are indicated only. In rock of Classes III and V, repeated grouting and more than one grout cover may be needed. The table attempts to summarize and classify treatments needed for different types of rock. In rocks with a few joints, the grout pattern may be more open, while in heavily fissured rock, a regular drill pattern giving a good grout cover is needed.

GROUTING MATERIALS

A goal of determining the appropriate grouting material is to find an ideal grout that has excellent flow characteristics and penetrates in the fine fissures and yet the traveling speed slows with distance from the injection hole and thickens resisting further movement. The grout permits quick grouting of the hole or stage of grouting without traveling too far to areas not in the planned grouting area. The grouting zone should extend about 5–10 m (15–30 ft) outside the tunnel area and have a high early gelling time and hardening (setting) time. This allows the packers to be removed early facilitating a rapid grouting and tunneling cycle. The grout should have high early strength, be durable, and effectively fill fissures. The principal consideration when determining the grout and grouting method is to efficiently achieve the desired results at the least cost possible.

The size of the cement particle, the viscosity, the grout's stability, and the gelling (hardening) time determine the ability of the grout to penetrate a rock's fissure or the pores of a soil. The required sealing effect, the in situ water pressure, and the predominant rock conditions determine the optimum type of grout to use.

Most chemical grouts have no particles and thus can penetrate areas through which water can flow. Because of the limited viscosity and strength of chemical grout and the limitations of pressure that can be used, chemical grouts are not effective in all applications.

The most widely used material for grouting is OPC grout. A major determinant of the effectiveness of cement grout is particle size. Generally, for a grout to have good flowing characteristics, a cement particle should be from 3 to 5 times smaller than the aperture of the fissure (Sjöström, 1980).

The principal types of cement grout are OPC, Types I and III portland cement, and UFC. The type used depends on the grouting requirements. Type I is a general-purpose cement used in most grouting situations, unless there are special needs. Type III cement is similar to Type I, except that it has high early strength and, because of its smaller particle size, is used when Type I cement particles are too big. Ultrafine cement has the smallest particle size of the three. There is no standard yet that is established for microfine and ultrafine cement. Some suggest there is no difference between what are labeled as "microfine" and "ultrafine," and both are called "ultrafine" (Henn and Soule, 2010).

Henn and Soule (2010) compared three international and U.S. standards definitions for ultrafine cement: The International Society for Rock Mechanics (ISRM, 1995, p. 5) defines it as: "Superfine cement is made of the same materials as ordinary cement. It is characterized by greater fineness ($d_{95} < 16$ microns) and an even, steep particle size distribution." Committee 552 of the American Concrete Institute (ACI) states that the particles must be less than 15 μm (Henn, 1996). "The cement particles are less than 10 micrometer in diameter with 50% of particles less than 5 micrometers" is the Portland Cement Association definition provided in Kosmatka et al. (2002).

Sjöström (1980) suggests that there are two divisions of microfine and ultrafine, with maximum particle sizes of 30 and 15 μm, respectively. OPC has a maximum particle size of 100–140 μm. Irrespective of what it is called, the maximum particle size of ultrafine cement varies from 16 to 10 μm, depending on the industry association.

The size of aperture penetrable by the different types of portland cements is about 0.4 mm for OPC, whereas for ultrafine, it is 0.05 mm. Figure 10.1 is a graphical representation of the gradations of cement of Type I, Type III, and ultrafine cement.

The lack of stability of a grout results in the particles dropping from suspension and clogging the lines. Also, the fissures may not be adequately filled from the bleeding of an unstable grout. Bleed is the phenomenon such that, when a solution is at rest, the particles tend to settle out. This leaves an excess of mixing water above the settled particles. The amount of bleed can be reduced by developing good distribution of the cement particles by means of high shear mixing. To maintain the mix, it can be continuously mixed prior to being injected. To be considered stable, a grout should bleed less than 5%; in some cases, it may be limited to 2%. Additives such as bentonite, thixotropic agents, and stabilizers can improve grout stability; however, bentonite should not be

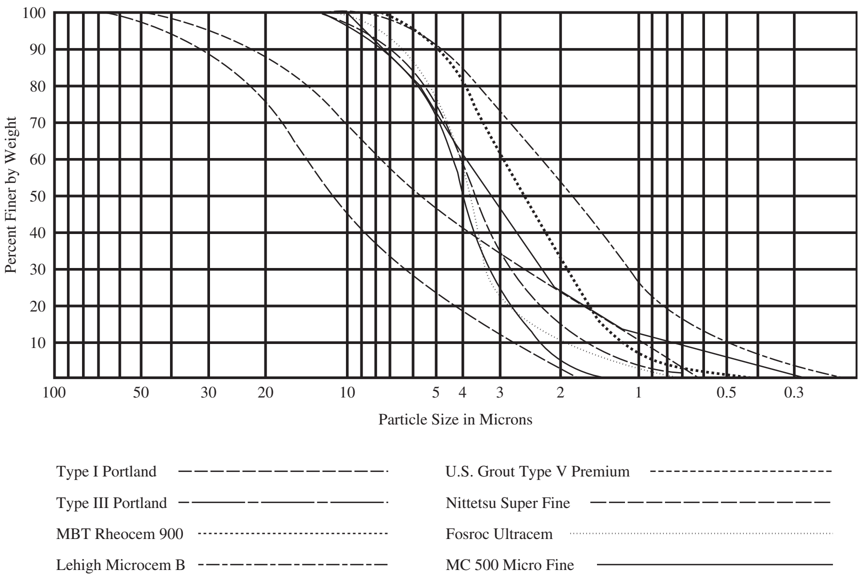

Figure 10.1 Grain Size Distribution of Various Ultrafine Cements (Warner, 2004)

used with finer cements, because the grain size of the bentonite is larger and it can also cause the grout to be less stable under pressure. As with concrete, a lower water–cement ratio will increase stability. When injected, it fills the fissures or the pores in the soil, forming a waterproof bond and strengthening the geologic matrix.

The viscosity of the grout should be less than 35 sec when measured by the Marsh cone to permit penetration into the fine features. For the capability of penetrating fissures without segregating about 1–3% plasticizers should be added. This should achieve an appropriate viscosity for a 1:1 mix.

A Marsh cone is a cone 152 mm (6 in.) in diameter at the largest point and 305 mm (12 in.) in height to the apex and consists of a 60-mm (2.4-in.) fixed tube with an internal diameter of 10 mm (0.4 in.). The top end of the cone that is open is covered with mesh to prevent particles that are too large from clogging the tube. (See Figure 10.2.)

The funnel is held vertical, with a finger blocking and sealing the tube. The funnel is filled to the mesh, which is 1.5 l (0.4 gal). The finger is removed, and the time (in seconds) it takes to empty the funnel into a measuring container is the viscosity. To differentiate it from other cones used, the Marsh cone has a volume of about 1.5 l and an aspect ratio of 2:1.

Chemical Grout

Commonly used in granular soils with a high particle distribution of fine sand, chemical grout will harden the ground, preventing excessive movement and lowering the permeability of the soil. It can also be used to underpin adjacent structures to prevent movement due to tunneling operations.

Figure 10.2 Marsh Funnel (Courtesy of ChemGrout)

The ability to put grout into the pores of the soil without any appreciable change in the original soil structure or increasing the volume affecting the change in the support capability of the soil without disturbing the soil structure is a significant advantage of grouting with chemicals. Also, being less disruptive to the soil structure it can permit tunneling with less overexcavation. Many kinds of chemical grouts are available, the most common being sodium silicate, acrylate, lignin, urethane, and resin grouts.

The geologic material being grouted, the injecting methods used, and the type of grout are what determine the penetration of the grout. Closely spaced holes and rapid injection are generally necessary for grouts that gel quickly. Low-shear-strength grouts can often be useful if the time after initial gelation is needed to extend the range of treatment. In situations where groundwater may displace grout during injection, rapid-time setting grouts are particularly useful in an assortment of different strata with different permeabilities (Bowen, 1981). If the gelling occurs before injection is completed, the last grout injected typically moves to the outside of the grouted area and voids are filled. As long as the grout is moving, gelling will be retarded and the penetration will be greater than quicker gelling grout.

Although it acts as an accelerator, portland cement can be used as a filler in silicate grouts. Because of its extremely short gel times, portland cement has been used to cut off flowing water or water under pressure. When silicates were combined with portland cement, strong bonding properties to the in situ materials were reported. If not allowed to dry out, this system produces a high-strength, permanent grout and has been used in grouting below the water table. By increasing the amount of cement, gel or set times in the range of 10 to approximately 600 sec, with strengths as high as 7000 kPa (1000 psi), have been reported. Finely ground cement is generally better with sodium silicates. Sodium silicate grout is generally easier to inject than a silicate–portland cement grout, and apparently, because the cement particles are lubricated by the silicate, silicate–portland cement grout can be injected more easily than portland cement grouts.

When determining the type of grout to use for a treatment, there are mechanical and chemical properties that have to be evaluated. These include viscosity, strength, and durability. Viscosity is the attribute of a fluid that describes a fluid's internal resistance to flow and may be thought of as a measure of the resistance of a fluid which is being deformed by either shear stress or tensile stress. The common unit of measure for viscosity is the centipoise (cP). To relate it to water, water at 20°C (70°F) has a viscosity of 1.0 cP. It is an important factor when describing a grout because it determines the capacity of the grout to flow through the pore spaces in the soil. The viscosity of the fluid must be measured against the permeability of the soil. A rule of thumb to use for this determination is, for a soil having a hydraulic conductivity (permeability) of 10^{-4} cm/sec, the viscosity of the grout should be less than 2 cP, whereas grouts with viscosities of 5 cP can be used for soils with a hydraulic conductivity greater than 10^{-3} cm/sec. For a viscosity of 10 cP, the hydraulic conductivity should be greater than 10^{-2} cm/sec (U.S. Army Corps of Engineers, January 31, 1995).

The time that it takes for the grout to gel has a significant effect on the performance of the grout. Gel time is the period of time between the initial mixing of the grout and when a gel forms. The gel is controlled by the time and has great significance with regard to pumping. The gel time is the result of the elements of the activator, the inhibitor, and the catalyst. The gel time can be changed by varying the proportion of the elements relative to each other. Depending on the grout, the viscosity of the grout may change during the gel time or can remain constant. The deviations of the gel times are important because of the pumping requirements of grouts with high viscosity. As the grout gels, it continues to gain strength. The period of time it takes the grout to reach its planned properties is called the cure time.

Because of their safety and environmental compatibility, sodium silicate grouts are the most popular. They have been developed into many grouting systems. A silicate solution can form a colloid which polymerizes; it further forms a gel that binds particles of soil or sediment together and fills voids, and this applies to basically all sodium silicate systems.

Sodium-silicate-based grouts are most regularly used for ground strengthening. Sodium silicate grouts have been used to stabilize or strengthen foundations composed of granular material and fractured rock and to seal off water flowing through permeable foundations. If the gel is not allowed to dry out and shrink, granular materials that have been saturated with silicate grout develop quite low permeability. In spite of the occurrence of shrinkage, low permeability is generally achieved. Any groutable granular material coarser than the 75-μm sieve will have improved strength and load-bearing capacity if treated with sodium silicate grout. The strength of the grouted material is influenced by absorption, particle shape, moisture content grain size, particle size, distribution, capacity of the grout to adhere to particle surfaces, curing environment, and injection technique.

Sodium silicate mixtures, which are alkaline, act as a reactant because when the solution is neutralized, colloidal silica will combine to form a gel. This occurs if the sodium silicate is present in concentrations, by volume, above 1 or 2%. There are three kinds of grouts that, based on reactants used with silicate solutions, are recognized as alkaline silicate grouts (Yonekura and Kaga 1992):

An acid reactant, which includes phosphoric acid, sodium hydrogen sulfate, sodium phosphate, and carbon dioxide solution

Alkaline earth and aluminum salts, which include calcium chloride, magnesium sulfate, magnesium chloride, and aluminum sulfate

Organic compounds, such as glyoxal, acetic, ester, ethylene, and carbonate formamide

These chemicals have been included here for general reference, but it is not necessary to know how they work for them to perform an effective grouting job.

The reactant and sodium silicate can be injected as separate solutions or the sodium silicate can be premixed with the reactant forming a single solution that can be injected. This two-solution process is sometimes referred to as the "Jooste two-shot technique" (Bowen, 1981; Karol, 1990). In this methodology, the sodium silicate is injected into the target material. Usually, the reactant, a solution of calcium chloride, is added as a second step in the process. It has been stated that the two-solution approach produces the highest strength increase in injected soils, but it is the most expensive method used. Because of the near-instantaneous hardening, forming a gel very rapidly, the two-component technique has the ability to cut off water and the hardened grout also has a permanent nature, that is, it is durable and resists deterioration. However, because of the rapid hardening characteristic of the two-component technique, the volume of sediment of soil that can be grouted from a single injection point is restricted. Generally, it is not possible to mix the reactant and the silicate on the surface. When the two-component technique is used, one should anticipate some unreacted grout components.

In the one-solution technique, a mixture of sodium silicate and a reactant, or reactants, is injected, causing the silicate to form a gel. The individual solutions are completely mixed and the mixture depends on the timing of the beginning of gelation. Improved control of gel distribution, a more uniform gel formation, a stronger grout, and improved control of gel distribution during injection are considered to be the advantages of this process.

Similar to the way that the two-solution process reactants are used, the one-solution system neutralizes the alkalinity of sodium silicate; however, the reactants are diluted and materials that react slowly, such as organic reagents, are used. Common reactants are sodium bicarbonate and formamide. A mixture of formamide, sodium aluminate, and sodium silicate is a standard preparation. In the formulation the formamide causes gelation and after the initiation of the gel the sodium aluminate accelerates gel formulation.

The percentage of silicate in the grout is related to the viscosity of the grout. Therefore, a higher level of silicate concentration produces a grout with a greater viscosity. Thus, with a higher concentration of sodium silicate in the mix, the ability to enter smaller fissures is reduced. Table 10.4 illustrates the relationship of sodium silicate concentration to viscosity.

Types of Chemical Grout By using one of the many chemical grouts available, various types of grouting projects can be achieved. Each grout has its own characteristics that can be matched to the application required. The most common chemical grouts are sodium silicate (discussed above), acrylate, lignin, urethane, and resin grouts.

Table 10.4 Relationship of Sodium Silicate Concentration to Viscosity of Grout

Sodium Silicate Concentration, %	Viscosity (Compared to Water)
10	2.5
20	3.2
30	3.5–4.5
40	4.0–6.0
50	5.2–12
60	8.0–20
70	92

Source: From U.S. Army Corps of Engineers, January 31, 1995.

The medium being injected and the procedures implemented affect a grout's ability to penetrate a medium. Because of their limited range of treatment, close spacing of injection holes and a rapid injection rate are necessary for quick gelling. Low-shear-strength grouts are often used to extend the range of treatment. Rapid times of setting are useful when a variety of different strata with different permeabilities are being treated and in situations where groundwater flows may displace the grout during injection (Bowen, 1981).

The effectiveness of a grout penetrating the ground is a function of the permeability of the ground being injected and the method used for injecting the grout. Generally speaking, quick-gelling grouts are limited in the distance they can travel and require closer spacing of the holes and a rapid injection rate. Frequently, low-shear-strength grouts will extend the treatment range past the initial gel time. In situations where the groundwater flow may displace the grout during injection in different strata and different permeabilities, rapid-setting-time grouts are useful (Bowen, 1981). The last injected grout typically moves to the outside of the grouted mass, and both large and small openings are filled when gelling occurs before pumping is halted. Grouts that continually move will have more penetration using the same volume than with batch injection and the grout gels less quickly.

The strength and durability of a grout will vary according to its function. The primary function of grout injected into a granular soil is to strengthen the soil, and it must be strong enough to meet its intended purpose. After pumping, grout can be exposed to freezing and thawing cycles, chemicals in the ground water that can attack it, and wetting and drying cycles. The measurement of the ability of a grout to deal with these is called durability.

Because they generally contain no particles and have low viscosities, chemical grouts can be injected and travel into smaller crevices than cementitious grouts. They are more expensive than cementitious grouts, so their use is generally limited to those grouting projects that cannot be done with cementitious grout.

If other constituents remain constant, increasing the concentration of silicate will increase the gel time. However, an increase in the reactant decreases gel time. When the temperature is increased, the gel times will decrease; generally special precautions are not necessary with temperatures up to 48°C (120°F) and the optimum temperature range is about 7–27°C (45–80°F) (Warner, 2005). Also, increasing the concentration of the accelerator decreases the gel time, but the greatest effect of temperature is below 37°C (100°F) and it increases as the temperature lowers. Direct sunlight has little effect on gel time and the silicate grout is affected very little by freezing. However, freezing must be avoided during placement. Except for areas where large amounts of acid exist, the pH of the material to be grouted has little effect. Silicate grout containing aluminate should be used if acid is present. If soluble salts such as chlorides, sulfates, and phosphates are present in the material to be grouted, depending of their concentration, they can have an effect on gel time. To determine the gel time in water that contains dissolved salts or other impurities, one should test it using a sample of the water. Although fillers such as bentonite and clays have little effect on gel time, moderate to high concentrations of fillers will cause the temperature to vary, changing the gel time. When using reactive materials such as portland cement, the gel time should be checked.

The compatibility of portland cement and sodium silicate can generate various results depending on other additives and the relative amounts of both chemicals. In addition to acting as an accelerator in silicate grouts, portland cement can also be used as a filler. The system is useful for cutting off flowing water or water under pressure because of portland cement's extremely short gel time. When silicate is combined with portland cement, strong bonding properties in the in situ material have been experienced (U.S. Army Corps of Engineers, January 31, 1995). When used below the water table, it produces a high-strength, permanent grout provided it does not dry out.

As a less toxic alternative to toxic acrylamide compounds, acrylate grouts were introduced. The polymerization of acrylates results in the acrylate grout gel. The addition of triethanolamine and ammonium or sodium persulfate to a metal acrylate (usually magnesium acrylate) catalyzes the gelling reaction. If long setting times are required, potassium ferricyanide is used as an inhibitor.

Acrylate grouts generally form soft gels and are useful in fine sediments because of its low viscosity and long gel times, as long as 2 hr. Using acrylate grout in a two-part injection technique with each injected solution a monomer (e.g., silicate or acrylate) and the catalyst for the other monomer, special grouts have been developed that are restricted to use at temperatures between 5 and 30°C (40 and 86°F).

Depending on reactions involving the cross-linking of isocyanates to form a rubbery polymer, urethane grouts are available in several different forms. Prepolymers formed by partly reacting the isocyanate with a cross-linking compound producing a prepolymer with unreacted isocyanate groups result in one-part polyurethane grouts. To complete polymerization, the one-part grouts react with water. Depending on the amount of water available, grouts will typically gel or foam, yielding viscosities ranging from 50 to 100 cP. Depending on the formulation, two-component grouts employ a direct reaction between an isocyanate liquid and a polyol, producing a hard or flexible foam.

Depending on the exact formulation, isocyanates typically are toxic to varying degrees. In groundwater the solvents used to dilute and control the viscosity of the urethane prepolymers are pollutants. Also, some grouts are highly flammable before and after setting. A very versatile grout, hydrophobic water reactive prepolymers can be injected directly into flowing water as a water stop and can be used to seal openings as small as 0.01 mm and in distributing loads in underground structures with ridged foams.

A by-product of the sulfite paper-making process, when combined with an oxidizer such as sodium dichromate, lignin forms an insoluble gel. Because of the range of viscosities of various lignin solutions that can be obtained, the lignins are capable of being injected into voids formed by fine sands and possibly coarse silts. The reactant or reactants being premixed in the lignin-based material, lignin-based grouts are injected as a one-solution single-component system by changing the quantity of water; gel times with the precatalyzed ignosulfonate system are easily adjusted. Also available commercially are the two components of lignosulfonates. In this system, the reactants are mixed separately as with a proportioning system, and the total chemical grout is not blended until immediately prior to injection.

Among the advantages of using this system are the closer control of gel time and a wider range of gel times coupled with elimination of the risk of premature gelling. Although mechanical agitation is recommended, the materials used in lignin grouts are rapidly soluble in water. The lignin gel has a slightly rubbery consistency, a low permeability to water, and normal grout concentrations and is irreversible. Intended primarily for use in fine granular material, lignin grout is used to decrease the flow of water within the material or increase its load-bearing capacity. Lignin grout has been used effectively in sealing fine fissures in fractured rock or concrete. It is generally unsatisfactory and is not recommended for use in soils containing an appreciable amount of material finer than the 75-μm sieve because material this fine will not allow satisfactory penetration. Injected at moderately high pressures, lignin grout of low viscosity may be effective in fine materials.

Sodium bichromate, potassium bichromate, ferric chloride, sulfuric acid, aluminum sulfate (alum), aluminum chloride, ammonium persulfate, and copper sulfate are among the various reactants used with lignin-based grouts. Now considered a potential groundwater pollutant, the bichromates have been the most widely used and apparently are the most satisfactory. With an ultimate strength of approximately 40% that of a similar grout mixture in which sodium bichromate is used as a reactant, ammonium persulfate has also been used as a reactant in the lignin grout system.

Consisting essentially of solutions of resin-forming chemicals, upon adding a catalyst or hardener, resin grouts combine to form a hard resin. Epoxy and polyester resins are the principal resins used as grouts. Resin compounds that are similar but have some different properties are epoxy and polyester resins. Various types of both epoxy and polyester are available and each can be altered by changing the components. Although viscosities of resins are generally higher than those of chemical solution grouts, resin can be formulated to have a low velocity. Resins generally give off a lot of heat when curing. During the course of the greater part of their fluid life they maintain their initial viscosity and just before they harden they pass through the gel stage. The time from the mixing to the gel to the hardened stage can be adjusted by varying the amount of hardening reactant, by adding or deleting filler material, and by controlling the temperature, especially the initial temperature. Generally supplied as two components, each component of an epoxy grout is an organic chemical. The two components to a resin grout are usually a resin base and a catalyst or hardeners. Sometimes, to increase the ability of the hardened grout to accommodate movement, a flexibilizer is incorporated in one of the components. In both filled and unfilled systems tensile strengths generally range in excess of 28 MPa (4000 psi). When another ingredient, generally material such as sand, has been added, it is called a filled system, whereas an unfilled system refers to the original mixture of components. Elongation can reach 15%. Both filled and unfilled systems can generally achieve flexural strength more than 40 MPa (6000 psi) and there have been considerably higher strengths achieved. In a filled system with water absorption of approximately 0.2% or less and shrinkage, by volume, of 0.01% or less, the compressive strength may be greater than 70 MPa (10,000 psi) and can reach 270 MPa (40,000 psi).

Due to its resistance to acids, alkalies, and organic chemicals, epoxy grout can cure without volatile by-products (therefore, no bubbles or voids are formed); has the ability to cure without the application of external heat; can accept various thixotropic or thickening agents such as special silica, bentonite, mica, and short fibers such as asbestos or chopped glass fiber; and is capable of being used in combination with various fillers to yield desired properties in both the hardened and unhardened states. Examples of epoxy fillers are aluminum silicate, barium sulfate, calcium carbonate, calcium sulfate, and kaolin clay, which act as extenders; graphite, which aids in lubricating the mixture; and lead for radiation shielding.

Generally, epoxy resins are easier to use than polyesters, because they exhibit less shrinkage, develop a tighter bond, and are tougher and stronger than polyesters. Although they may soften, being thermosetting resins, epoxies, once they have hardened, will not again liquefy even when heated. For grouting fractured rock to give it strength, in particular, rock bolting, the use of epoxies has become very common.

Types of Grouting

Permeation Grouting The term *permeation grouting*, also applying to rock and ultrafine cements, is used to permeate the ground generally using, because of pore size, a low-viscosity grout, sodium silicate or polyurethane. Without displacing the soil or changing the soil structure, the grout flows into the pores of the soil. The grout hardens, or gels, and depending on the ground characteristic, can be permanent or temporary. It increases the strength and cohesion of granular soils and, in tunneling, increases standup time and inhibits the movement of structures above or adjacent to the tunnel or shaft.

Compensation Grouting When a tunnel is being driven under structures or a shaft is being sunk near a structure and there is a danger of causing surface settlement, grout is injected between the tunnel or shaft being excavated and the foundations of structures on the surface. The quantities are calculated to offset the settlement caused by material removed by the tunnel or shaft. This is generally used in soft ground. The settlement can often be propagated from the gap over the tail shield. The zone of disturbed soil extends up and horizontally until the ground surface is reached. Depending on the depth of the tunnel or the distance from the shaft to the structure, this settlement can occur quickly or over an extended period of time. This settlement can affect not only foundations but also buried utilities.

Jet Grouting Jet grouting, sometimes referred to as "jet mixing," is a technique that uses high pressure of about 300–600 bars (4350–8700 psi) and high-velocity jets of grout to hydraulically erode, mix, and partially replace the in situ soil or weak rock with cementitious grout slurry. Figure 10.3 illustrates columns made by jet grouting. The columns were constructed to strengthen soil through which a shallow tunnel was driven. This mixing creates an engineered soil–cement material of high strength and low permeability.

Jet grouting has three processes called "single fluid," "double fluid," and "triple fluid." The single-fluid process uses grout to erode as well as mix with the soil. It is the most simple of the three. With the single system neat cement is injected through a small nozzle with high pressure and mixed with the soil.

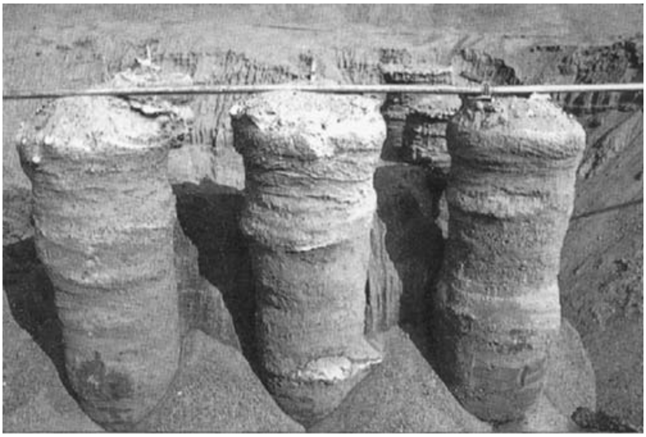

Figure 10.3 Photograph Illustrating Three Jet Grout Columns in Granular Material

With this method, the most homogeneous soil–cement element with the highest strength is produced with the least amount of grout spoil return.

With the double-fluid method, there are two fluids: cement and air. The double system uses a combination of the air and grout to erode the soil and mix the grout with the soil. As the lower pressure cement grout is injected, the compressed air shrouds the grout injecting and reduces friction loss. Although the presence of the air reduces overall strength compared to the single-fluid method and produces more spoil, it permits the cement to travel farther from the point of injection.

A soil replacement method rather than a soil mixing method, the three fluids use air and water to erode the soil and a separate nozzle to inject the grout for mixing with the soil. With the three-fluid system water is added and injected with high-pressure compressed air. The air-lifting effect of this method actually removes the soil within the intended column diameter. To fill the void created by the air-lifting process, the grout is injected through a separate nozzle below the water and air nozzles.

The grouting can be performed above or below the water table and in most subsurface highly fractured weak rock and soils from cohesionless soils to highly plastic clays. When both excavation support and underpinning of structures abutting the excavation are required, this technique can be very cost-effective.

Figure 10.4 illustrates the jet grouting process. The hole is first drilled to a depth. As the drill is retracted from the hole, the jet spray erodes and partially removes and replaces the soil.

For tunneling and shaft sinking, in addition to stabilizing soils, jet grouting can be used to support surface structures to prevent movement caused by

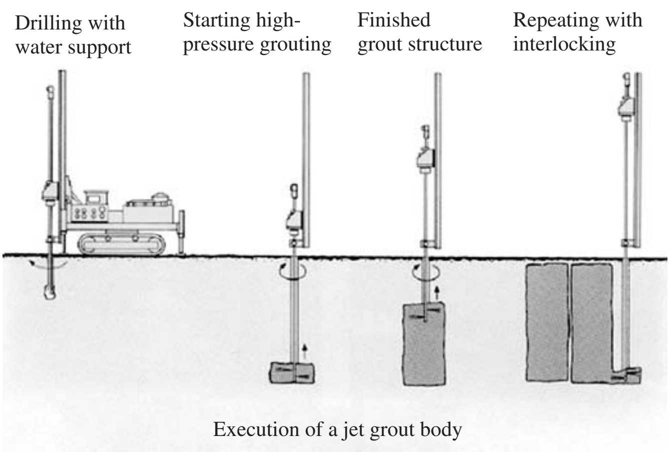

Figure 10.4 Distribution of Compaction Around Compaction Grouting Injections (Top) for Isolated Injections and (Bottom) for Close Split-Spaced Injections (Warner, 2004)

tunneling below or sinking shaft adjacent to the structure. Structures can include buildings, roads, and utilities to name a few. It can also be used underground to strengthen unstable ground.

An example of the successful use of jet grouting was a section of a tunnel driven in Taiwan that had very poor tunneling conditions, namely poor ground and high groundwater. The contractor decided to mitigate the effects of very weak material by having the area jet grouted. The jet grouting was a single fluid producing 2500 columns approximately 8 m (26 ft) in length and 1.5 m (5 ft) in diameter. The jet grouting was successful for improving the ground conditions. To control the water inflow grouting was done from the tunnel.

On the same project, another tunnel that was about 80 m (260 ft) below the surface suffered a collapse of about 50 m (150 ft) of tunnel. Fortunately, the supervisor sensed the problem and removed equipment and personnel prior to the collapse; also, it was in a rural area because there was resultant subsidence on the surface measuring approximately 28 m (92 ft) in diameter and 12.1 m (40 ft) deep at its lowest point. Jet grouting was one of the methods employed to safely mine through the collapsed zone. Horizontal jet grouting was used to install a series of circular columns around the periphery of the required tunnel arch with steeply dipping jet grouted columns for the bench section (Figure 10.5a). Three levels of slightly ascending horizontal jet grouted columns formed a triple canopy over the area to be remined (Figure 10.5b). Note that in the plan, the grout columns bottom out the same distance from the tunnel centerline.

Figure 10.6 is a photo of jet grout columns drilled nearly horizontally to provide support of the ground in a previously collapsed section. The circle shapes are the grout columns.

Permeation grouting is done to prevent water inflows, water from having adverse effects on ground stability, flooding of the work area, contamination by groundwater, subsidence, and the increase of sewer flows. When the goal of the seal grouting is to prevent water outflow, it is done to prevent the loss of fluids from the final conduit and contamination of the surrounding ground/groundwater from the conduit flow.

For tunneling, the grouting sequence is pregrouting, postgrouting, backfill grouting, and contact grouting. Pregrouting can be conducted from the surface or from the face prior to excavating the round, postgrouting is conducted after excavation of the round, backfill grouting is done subsequent to the installation of the lining to fill known voids, and, similar to backfill grouting, contact grouting is done after placement of the concrete lining and backfill grouting to fill voids that remain after the backfill grouting. Figure 10.7 is an illustration of pregrouting from the surface.

Sealing and reducing the permeability of rock in tunnels to control water inflow to the tunnel are done by pregrouting and by postgrouting. Pregrouting is conducted prior to the excavation cycle, that is, prior to mining; the ground is injected with grout with the purpose of sealing off water and/or increasing the stability of the rock. When tunneling, it is sometimes necessary to drill ahead

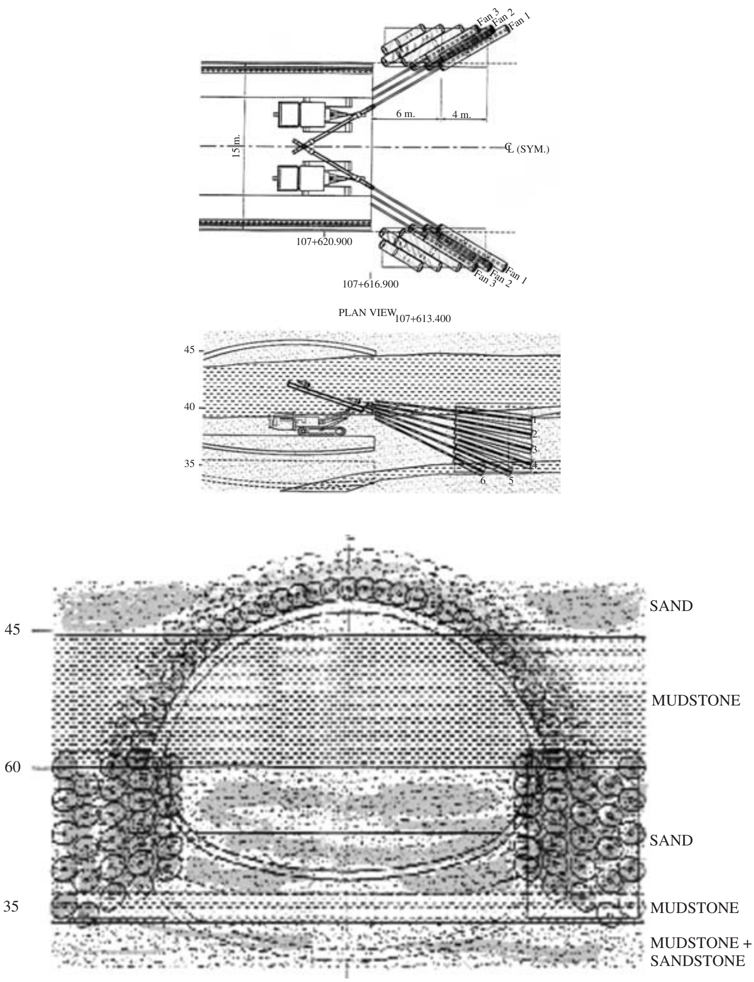

Figure 10.5 Underground Jet Grouting

(probe) to locate any water source to permit grouting it off prior to tunneling into it. Figure 10.8 illustrates investigatory and grout holes drilled ahead of the face. Generally, because of the lack of ability to control the drill bit location in long holes, the length of the holes is limited to about 20–25 m (60–80 ft). The decision as to what length to drill is dependent on the rock, drilling equipment, and skill of the drill operator. The lengths of the probe holes may be shortened or lengthened.

The location, number, and disposition of the holes depend on the equipment, space, and type of ground. Pregrouting for drill and blast or roadheader tunnels permits drilling holes anywhere in the heading that is determined to be necessary. When tunneling with a TBM, the grout hole spacing is much

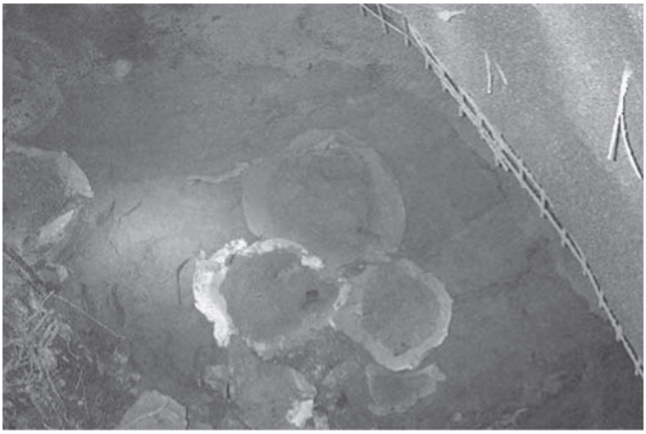

Figure 10.6 Jet Grout Columns Used as Ground Support Tunnel

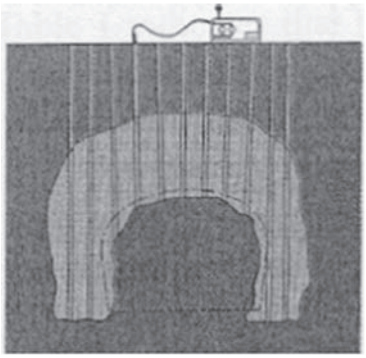

Figure 10.7 Pregrouting from Surface

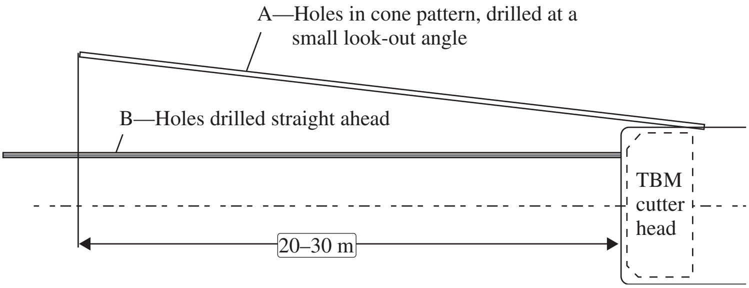

Figure 10.8 Grout Hole Locations

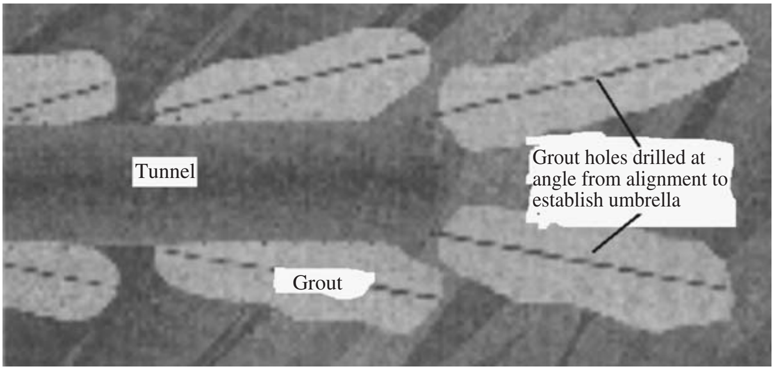

Figure 10.9 Illustration of a Pregrouting Single Cover

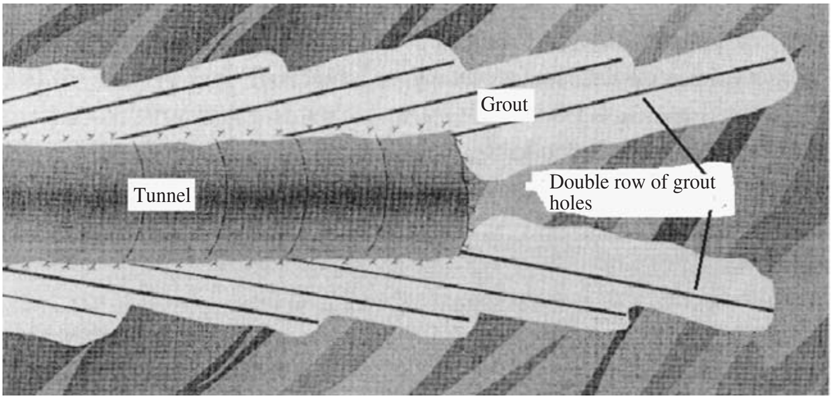

Figure 10.10 Double Coverage

more restricted. The holes have to be drilled through openings in or near the cutterhead and the shield and generally at an angle; thus the access for drilling in the cutterhead controls the grouting hole locations.

Figure 10.9 illustrates single-canopy grouting, and Figure 10.10 illustrates double-canopy grouting. There is one layer of grouting around the tunnel. Although this can be effective for stabilizing the ground, it will generally not stop the inflow of water.

Pregrouting will have an effect on the schedule, being unable to mine and grout simultaneously, whereas postgrouting can generally be done simultaneously. Controlling water inflow after opening the tunnel is very difficult because, when trying to grout it off, one ends up “chasing” the water along the tunnel. That is, there is no natural way to stop the water. One might successfully stop the water 50 m (160 ft) behind the TBM, but it may reappear 75 m (250 ft) or 25 m (80 ft) behind the TBM. These locations merely illustrate what is required to seal off the water. For water to flow into the tunnel there must be a conduit for the flow. If not, the water will find another route, around the grouted area. Water will flow to a point of lower pressure according to the laws of gravity. With water under head, the problem is increased. When pregrouting, the water is grouted ahead of the machine, and this may be far enough from the rear of the TBM to prevent water from entering the tunnel through the already mined tunnel, behind the TBM.

If not pregrouted and the TBM mines into a water-bearing area, the water inflow may be to such a degree that the tunnel can flood quickly with greater flow than the pumps can handle. Also, the water inflow can contribute to ground loss by removing material in the fissures of the rock or, with enough flow, it can actually cause small pieces of rock to erode. Without the use of slurry of EPM TBMs, the effects of water inflow in soil tunnels can be devastating. Reducing even moderate inflows by postgrouting is a tedious, expensive, and frustrating job. Although one can probably continue mining, it has been estimated that in water-bearing rock the cost of postgrouting can be as much as 20 times the cost of pregrouting.

As pregrouting often is on the critical path, it might be tempting to minimize the number of holes. However, if the number of holes drilled is insufficient, the resulting inadequate grout performance will cost more and take more time

Normally, it is not possible to get a completely watertight tunnel, and grouting generally will only reduce the water inflow to varying degrees of tightness, depending on the circumstances.

The permissible amount of water inflow depends on the ability to mine with it, the ground stability, and the sensitivity of the overlying ground and structures. In nonsensitive ground, it depends on the amount of water inflow that can be tolerated without disturbing the advance and the cost of pumping and disposing the water; the water may require treatment before discharging. Also, during use of the tunnel, the access for maintenance and the maintenance cost have to be considered. Another consideration is the potential effect of continuous water inflow into a tunnel on the groundwater balance and the environment.

The degree of water tightness that will be achieved depends mainly on the type of ground and frequency and size of fissures, grout material used, grout hole patterns, water pressure head, characteristics of the rock, and of course knowledge and skill of the personnel doing the work.

Often, the volume of fissures in hard rock is less than 1%; occasionally, it is higher. Grout takes in fault zones and decomposed rock varies within wide ranges. Limited pregrouting may be successful in rock masses with few joints, while in heavily fissured rock, pregrouting giving a continuous cover has to be carried out.

As a general rule, when grout cover all around the tunnel periphery is needed, grouting in holes not more than 2.5–3.0 m (8–10 ft) apart at the distal end, in boreholes no longer than 20–2 5m (65–80 ft), and using stable MFC grouts with plasticizers will give a high probability of an adequate treatment. However, sometimes the fissures are too small to permit the penetration by cement grout and require the use of chemical grout. It can be advantageous to drill grout holes on the face parallel with the tunnel alignment. Often it is not possible because of the cutterhead, requiring more grout and grout holes per linear meter of tunnel to dry up the face.

In water-bearing ground grouting and mining may be conducted in cycles to better control the water. For example, it is determined that 20 m (65 ft) can be effectively grouted. However, grouting and then mining 20 m (65 ft) means one is into water-bearing problem ground. It is recommended that once 20 m (65 ft) is grouted, the face is advanced about 12–15 m (40–50 ft). Then mining is halted and another grouting series a distance of 20 m (65 ft) is performed; this process is repeated. This not only provides a water stop ahead of the mining, thus mitigating water-caused problems, but also mitigates drilling problems because the ground ahead of the TBM has been permeated with grout, thereby causing it to be less likely to have raveling or difficulty maintaining the drill hole. This will help when installing grouting equipment in the hole.

A problem that can occur when drilling grout holes is the breaking of drill steel in the hole. Retrieving the steel from the hole may not be possible because of the location of the break. Outside of the tunnel alignment this is not a problem. However, if the steel is within the tunnel alignment, hitting the steel with the TBM can create problems. A way to ensure that a drill steel that breaks is retrievable is to engineer the break near the hammer on the drill string. The coupling that connects the striking bar to the drill steel can be grooved, making it the weakest part of the drill string. This will function like a shear pin on a prop. Thus, if the drill string does break, the break will be reachable and facilitate drill steel removal. The groove should be engineered and made by the coupling manufacturer or qualified machine shop. The down side is that purposely reducing or providing a weak link will cause more breaks; however, the increase in drill string breaks may be more than offset by the cost of leaving a drill string in the tunnel alignment. Figure 10.11 is a sketch of a grooved coupling.

Often the grouting of the boreholes is done in one stage, but in difficult rock that is heavily fissured and with big variation in fissure widths, treatment has to be carried out in 5–25-m- (15–80-ft-) long stages or even less.

Grouting of fault zones and decomposed rock can be very demanding. Often the treatment has to be repeated a number of times to give an adequate water

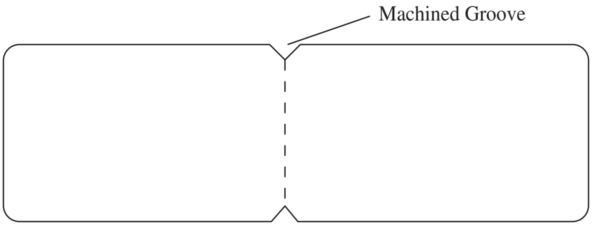

Figure 10.11 Machined Drill String Coupling

exclusion, making zones of soil less prone to erosion. Normally, grouting for water exclusion in weak or highly fractured rock also stabilizes the rock and improves the standup time.

For ground stability and water inflow control of an underground excavation, there are primarily two types of drilling methods: rotary and percussive.

Rotary drilling is exclusively done from the surface. The penetration is achieved by the rotation of the drill bit while it rests against the rock at the bottom of the hole. Percussive drilling relies on the rotation of the drill bit and the percussion, that is, impact, caused by energy transmitted by the drill string from the hammer. Drilling for pregrouting in tunnels is usually carried out with percussion drills.

Grout borehole drilling, like any activity, should be done as inexpensively as possible. For grouting to be effective, the quality of the drilling method is important. The boreholes should be drilled straight and the borehole walls should be protected from caving. The cuttings from drilling should be of a size that will not block the fissures while being flushed from the borehole.

Experience has taught us that boreholes deviate to a certain extent. Up to a drill length of about 20 m (65 ft), boreholes are normally within 1 m (3 ft) of target, depending on the rock structure, drilling equipment, and skill of the driller. Beyond that length of drilling, the deviation accelerates. This explains the rule of thumb not to drill more than 20–25-m- (65–80-ft-) long boreholes for successful pregrouting. In broken rock, the deviation has a tendency to increase. Sometimes, if drill boreholes collapse, drilling and grouting have to be carried out in successive, descending stages to enable drilling onward. Where possible, grout boreholes should be drilled at right angles to the main fissures in order to intercept as many as possible. This is particularly important when postgrouting in tunnels but is more difficult to do.

The borehole diameter has little or no influence on the grouting results. Larger boreholes are generally straighter because of the stiffness of the drill string. But larger boreholes are more expensive, because they require bigger equipment, the packers are more expensive, and the borehole will require more grout. A 3 in. hole requires more than twice the grout of a 2 in. hole.

Grouting Pressures

For good penetration into fissures, it is beneficial to use high pressure. For pregrouting in hard rock the pressures applied vary from 1 to 6 MPa (10–60 bars; 150–900 psi), depending on the thickness of the overlaying rock and the strength of the rock; however, the pressure for postgrouting from within the opened tunnel normally has to be limited to 0.5–1 MPa (5–10 bars; 75–150 psi).

Packers Packers are plugs that are placed in a borehole to seal, or block, material, usually grout, from flowing from the borehole. They allow an area of the borehole to be isolated. In this way, when grouting, if the location that requires grouting is known, the packer(s) can be placed in the borehole on either side of the area to be grouted. Once the area is isolated, grout can be pumped into the area. The diameter of the packer is determined by the size of the borehole. The packer has to be small enough to fit in the borehole yet be able to expand to seal the borehole. There are basically two ways to seal rock boreholes for grouting with a packer: mechanical or pneumatic.

Typically used for polyurethane injection and high-pressure epoxy injection, mechanical packers have limited expansion and are used principally near the surface in relatively firm round holes. They are constructed of a short length of pipe with a rubber seal around it. Normally, a Manchette tube is not used, but one can be connected if necessary. The packer is inserted directly into a hole slightly larger than the packer; to expand the packer and seal the hole, a threaded screw on top of the seal is then turned against the rubber seal, compressing it and causing it to expand tightly against the walls of the bored hole. When the sleeve is compressed longitudinally by mechanical means, the diameter increases, sealing the hole. The most common design for mechanical packers involves external threads on the top portion of a ridged central tube where a nut can be screwed down to compress the elastomeric sleeve. The amount of expansion is related to the thickness and elasticity of the sleeve. The sleeve is generally some type of hose and is limited to an expansion ratio of 1.2 (Warner, 2004). (See Figure 10.12.)

Figure 10.12 Mechanical Packer (Courtesy of Chemgrout)

With the elastomer restrained at the base, usually an outer steel sleeve is placed between the top of the elastomer and the turning nut. It is usually provided with handles. The weight of the packer increases as the length of the sleeve gets longer, restricting the limit of the depth to which it can be practically used.

Typically of fixed length, they can be purchased in different lengths. A result of the elastomeric sleeve being restricted in length is the possibility of the grout bypassing the packer. Because of this, mechanical packers are generally limited to shorter holes in competent formations with hard borehole walls. A practical depth limit for a mechanical packer is about 6 m (20 ft) (Warner, 2004).

Pneumatic packers consist of an expandable sleeve that, when subjected to internal pressure, expands, providing a seal. See Figure 10.13. The packer can be pressurized with water, air, or a gas, generally nitrogen. These types of packers are generally more effective in rough or irregular holes because of the amount of expansion along the length and the ability to deform slightly to match the contour of the hole. There is no restriction in the length of the expanded sleeve. The longer the sleeve, the more effective it is because of the increased contact area. The fixed-end packer, also called the balloon, has the simplest construction. It has a flexible hose clamp on each end over a rigid central tube. The hose is specially reinforced to match the unique use of the tube. Because of its simplicity, it is lower in cost. Fixed-end packers are sometimes the best choice and the diameter expansion ratio is 1.2 (Warner, 2004).

The sliding end pneumatic packer has one end fixed with the other free to move as the rubber sleeve tends to shorten when inflated, allowing greater expansion. The O-ring seal on the free-end and special rubber element construction is combined, providing a 1.9 expansion ratio (Warner, 2004). Because it has moving parts, it is more expensive and is likely to have more operating problems. To maintain satisfactory operation, the sliding end must be able to continually move freely and the internal seal must remain tight.

When selecting a pneumatic packer, the borehole size and the differential pressure to be restrained must be known. The differential pressure to be restrained is defined as the pressure of the grout being pumped. Also, the amount of unit expansion and the contact length need to be considered. The frictional resistance is directly related to the contact area. The key is to hold the pressure with the minimum expansion, that is, the ability to restrain

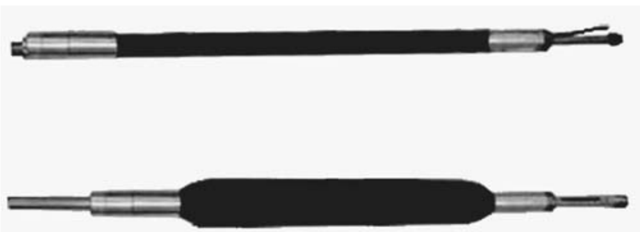

Figure 10.13 Inflatable Packer (Courtesy of ChemGrout)

the pressure with the minimal amount of expansion of the unit. This ability reduces as the diameter increases. The best results are obtained by using the largest packer that will fit into the hole, and the ability to hold pressure increases with the flexible length. The possibility of grout bypassing the packer because of the fissures adjacent to it is reduced with a longer packer. Figure 10.13 shows an inflatable packer.

When the hole is uniform in diameter and the walls are firm, minimal annular space is needed. Therefore, the packer can be only slightly smaller than the hole. This will permit the packer to accept higher inflation pressures and provide maximum friction resistance. If the holes contain wall fallout or other areas that are significantly larger than the drilled diameter, the unit can expand to its maximum diameter, but its ability to withstand high pressure is reduced. The packer has a central tube for injecting the grout. The tube should be 19 mm (3/4 in.) in diameter (Warner, 2004)

The two main methods for inflatable packers are single-packer grouting and double-packer grouting. In single-packer grouting, grout is injected into the ground below a single-inflated packer. The technique is extremely simple in that the packer is lowered into the borehole and inflated to the required pressure, and grout is pumped in under pressure below the packer. If multiple levels are to be grouted, the lower level is treated first; the packer is deflated, retreated up the hole to the next level, and reinflated; and grout is pumped in. This process is repeated as required.

Double-packer grouting is normally limited to the Tube à Manchette (TAM) system pressure grouting technique. (See Figure 10.14.) It offers an efficient,

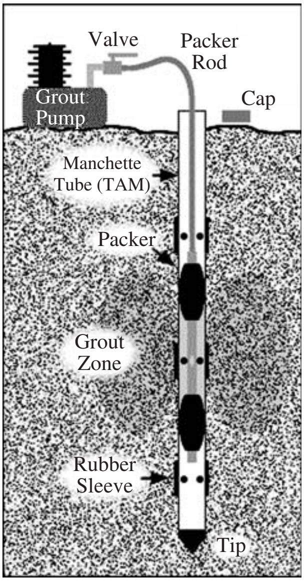

Figure 10.14 Tube à Manchette (Courtesy of Strata-tech)

cost-effective means of grouting in tunnels. The TAM system consists of a length of pipe with small holes drilled around the circumference and at equal intervals along the length of the pipe. A TAM is a PVC or metal pipe around which rubber sleeves cover holes that are drilled in the pipe at specific intervals. The tubes are inserted into the drilled holes. The TAM is located in the hole at the area that is to be grouted. This is known as the grout zone. Grout is pumped into a packer that is located in the tube. The grout is pumped through the holes in the tube, past the flexible rubber sleeve, and into the area to be grouted. The tubes are often located every 380 mm (15 in.) along its length and four holes are drilled around its circumference. All rubbers are held in place with a light adhesive and are taped down at both ends with electrical tape. Each set of holes is covered by a rubber sleeve (or manchette), which allows the whole arrangement to act as a series of one-way valves. That is, acting as a one-way valve, flow out through the holes is permitted by expansion of the rubber sleeve but flow into the holes is prevented by the sleeve collapsing onto the pipe. Custom lengths of tubing can be assembled via couplings. Double packers are used so a short section of grout hole can be sealed off from the remainder of the hole.

This is the most common use for double packers when the grout is injected through sleeve port pipes, such as the TAM. The TAM valve is accessed individually by a set of inflatable packers that are run inside the TAM pipe and inflated so that they straddle the valve. The sequence of operations normally consists of two or more separate stages of grouting. In the first stage of the process, the TAM is installed in a predrilled hole and, using a light “sleeve” grout or a gravel packing for low-pressure applications, it is grouted in place. This is necessary to seal the TAM into the borehole and to prevent grout from flowing, under pressure, up the borehole/TAM interface. The level to be grouted is isolated with the double packers, referred to here as straddle packers, which are run on a separate string inside the TAM during the actual pressure grouting operation. The grout is pumped through the isolation valve after the sleeve grout around the valve is fractured by pumping high-pressure water down the packer string. The pumping through the isolation valve stops when the maximum injection pressure or volume has been reached. This is repeated at each valve, generally from the bottom up. Ground treatment may require more stages of grouting. Figure 10.14 is an illustration of an inflatable grout double packer with TAM.

For holes that are longer than approximately 10 m (30 ft), it is generally appropriate to divide them into shorter lengths, called “stages,” and grout each of these stages separately. Staging the grouting pressures can provide better control while the upper stages are exposed to less pressure if not needed. Also, connections and leaks in the system can be more easily handled.

If problems arise in the borehole, such as loss of drilling water because the drill bit has broken into a wide crack or void, the drilling can be halted and a special stage made. This will permit the grouting of the feature individually providing better control. Once the feature is grouted, the drilling and grouting can continue without adverse effects of the feature. Also, if there is an area or section of the hole where soft or broken rock collapses and fouls the hole, drilling can be stopped; a special stage can be started, improving the stability of the area by grouting. But, if the length of the hole that collapsed is extensive, a special grouting program may be necessary just to repair the hole to allow subsequent grouting.

Often relatively short lengths of grout staging start near the collar of the hole and get longer as the hole deepens. The reason for this is related to the effects of pressure. The longer the stage is, the greater the pressures at the top. When possible, stages should be equal to drill steel lengths, thus optimizing drill rod changes. Figure 10.15 is an illustration of the assembly for a typical packer use for this application.

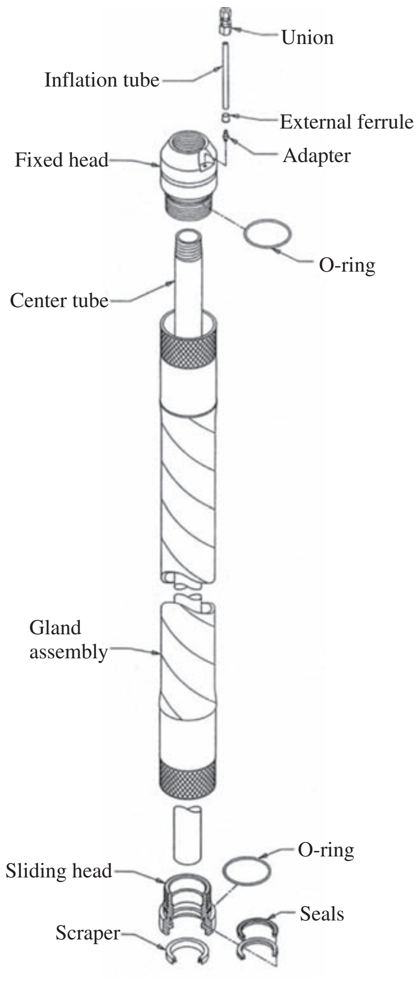

Figure 10.15 Inflatable Grout Packer Assembly (Aardvark Packers)

Stage Grouting

The methods for stage grouting are downstage without a packer, downstage with a packer, and upstage. In the downstage-without-a-packer process a hole is drilled to a predetermined depth. The hole is then connected to the grout source and the grout is injected the length of the hole. When the grout reaches the initial set, the hole is washed out to reduce the amount of drilling for the next stage. Once the grout is fully set up, the hole is again drilled through the grout remaining in the hole for the second stage. This process is repeated until the hole grouting is completed to the predetermined depth. To reduce, or minimize, the amount of drilling when the hole is deepened for the next stage, when the grout has reached the initial set, the hole is flushed out removing some of the residual grout in the hole. Drilling for the next stage requires drilling through the grout that remains in the hole and deepening the hole to the next required depth. As with the first stage, the grout is pumped to the extended hole for the next stage. This process will continue for each subsequent stage. The top-down technique reduces the risk of the bit being trapped or bound by small rocks or fragments, falling to the hole behind the bit. It is a time-consuming method but making the grout connection at the collar of the hole reduces the risk of the hole caving.

The downstage-with-a-packer method is also called “stage down,” “stage grouting with a packer,” and “stage grouting.” The difference between this method and the one without a packer is that here a packer is used between stages and the injected grout passes through the packer. Therefore, the packer is removed and reinserted in the hole at the next stage level, which is time consuming. However, an advantage is that the lower stages can be injected at greater pressures.

Upstage grouting is also known as “packer grouting,” “stage up,” “stop grouting,” “stage grouting,” and “ascending stage.” In sound rock, where borehole stability is not an issue, the hole is drilled to a depth and grout is injected in stages, starting at the bottom and working upward. The packer may be removed as soon as the grouting pressure has dissipated in the stage just completed, and then the next stage is started immediately; however, some wait at least 6 hr before beginning the next stage.

Grouting techniques employed earlier used thin grouts, starting with water–cement ratios of 5:1 and 3:1 by weight, successively going down to mixes below 1:1, and continuing grouting to refusal. When the flow of grout into the hole at a given pressure falls below a determined take, over a given time, it is considered that the flow has reached refusal. With the advent of stable grouts using plasticizers to provide high flowability, batch grouting has come into use. A predecided amount of grout is injected into the hole; once it has reached the determined quantity, the grouting is stopped. It has been found that this normally gives the required sealing effect, and only in certain conditions does the grouting need to be repeated. This method is very beneficial as far as the cycle time using a stable grout—one-shot mix—with a limited quantity. It is also considered beneficial to use speeds and pressures as high as possible from the beginning, which further reduces the time spent on any one hole. The technique can be used under fairly homogenous conditions in rocks with little variation in fissure widths.

A refusal criterion is the determination of sufficient injection. That is, a criterion is established for the length of time that a hole must hold a predetermined pressure. The time may be 1–5 min (Warner, 2004).

Test of Grouting Results At the end of a grouting cycle, check holes are often drilled to verify the result of the grouting. When the inflows from check holes are less than the preset limit, the grouting is considered satisfactory and excavation may resume. However, if the test does not meet the requirements, another hole is drilled and grouted. This procedure of doing additional holes when the test holes fail continues until the holes tested meet the requirements. Generally, there is a predetermined selection of borehole locations. That is, if a grout hole fails to meet the requirements, the additional hole to be grouted has already been determined and is designated in the grouting procedure. To measure the result of the grouting over any excavated length of tunnel to verify the inflow of water, it is normally done by isolating a section of the tunnel and establishing a cofferdam or a stank board with an outlet to act as a weir from which the water flow is collected and measured.

Test of Grouts The grout is pumped into the rock and is normally not accessible afterward to sample or check its quality. Because of the inability to check the quality of the grout in place, it is important to take samples and test the grouts. Some simple tests that should be performed on each grouting site include cup tests for gelling and hardening, marsh funnel tests, mud Balance tests, and bleed tests. All are simple and can be performed onsite and should be conducted routinely at each grouting location.

The cup test is conducted by taking a sample of the grout, placing it in a simple cup, and noting the time when the grout thickens, that is, the gel time; this provides the time when the pumpabilty is reduced. The test is also used to indicate the start of hardening, indicating when packers may be removed. Although cup tests can also be used as an indication of bleed, the preferred method is to use graduated measuring glasses to determine bleed.

The viscosity of the grout is determined using Marsh funnel tests and is given in seconds; a Marsh viscosity of 30–35 sec for cement grout is considered good.

Mud balance tests are carried out to check the volume weight of the grout as well as indirectly check the mix proportions.

Backfilling

Among the most important reasons for backfill and concrete grouting is to stabilize the lining by transferring the load from the lining to the adjacent ground or the load from the adjacent ground to the lining (Henn, 2003). When mining with a TBM there is a gap between the TBM and the walls of the excavation. The gap is the result of oversizing the peripheral cutters of the cutterhead to permit space for steering and the conical shape of the shield diameter from the face toward the rear of the shield to provide for forward movement and reduce the chances of the shield binding. The outside diameter of the segment ring must be smaller than the inside diameter of the tail shield to enable the assembly of the segments within the tail shield and the wire brush seal between the segments and the tail shield. This annular space has to be filled.

The purpose of backfill grouting during tunnel excavation by a TBM with precast segmental lining erected in the tail shield is to make the lining of a tunnel tight enough against the surrounding ground to ensure stability. The way this is accomplished is by backfilling the annulus between the tunnel lining and the ground. Backfilling will reduce the deformation of the rock around the tunnel and will help reduce settling above the tunnel. Filling the annulus puts the liner in contact with the surrounding ground, which helps to stabilize the liner during construction, thus securing the lining and making it more stable for thrusting during TBM advance. In addition to providing some corrosion protection, backfilling will reduce the flow of groundwater around the tunnel lining, reducing the potential for void formation. (See Figure 10.16.)

The timing of the backfilling depends on the standup time of the material. When TBMs in soft ground use nonexpansive precast concrete segmental lining systems mining in soil below the water table or with low standup time, the annulus must be backfilled simultaneously with the mining. If there is little or

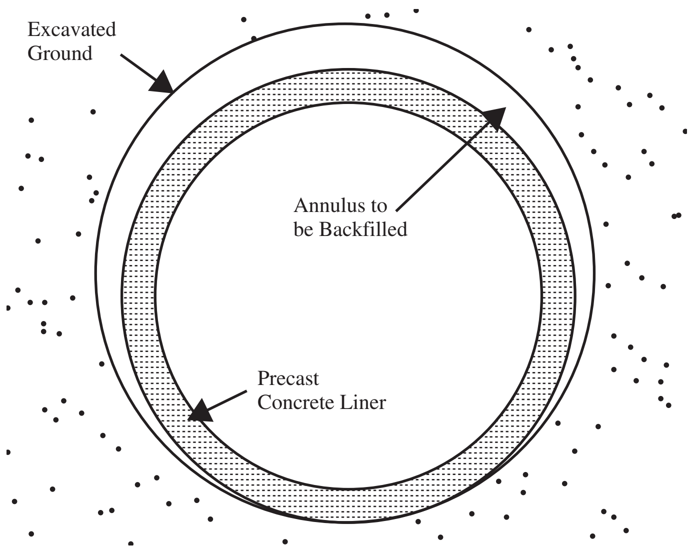

Figure 10.16 Example of Backfill Grouting Behind Nonexpandable Precast Concrete Segmental Tunnel Liner (Warner, 2005)

no standup time, the grout will prevent the geologic material from falling on the segments, resulting in uneven grout thickness.

For stable rock tunnels with longer standup time rapid backfill grouting may not be necessary for ground stability, but early grouting is still preferred, because it stabilizes the liner. The reaction to the thrust force advancing the TBM can cause the segments to come out of alignment. Not only can this cause alignment problems and difficulty installing subsequent rings, but also the movement of the ring may transfer the stress on the ring, causing an eccentric loading and possibly damaging segments.

The grout can be pumped into the annulus through prefabricated ports in the segments or through the tail shield. Grouting through the tail shield provides immediate grout protection. The pump senses the movement of the TBM as it is advanced by the thrusting jacks. The grout is pumped at a predetermined pressure through a piping system that is integral to the tail shield.

In Figure 10.17, the grout line passes through the wire brush seal in the tail shield spaced around the perimeter of the tail shield. This allows the grout to be injected at the edge of the tail shield into the annulus at a point where the tail shield and ring separate.

Generally, the grout is pumped simultaneously with the advance of the machine through the ports in the tail skin. The grouting can be done with a two-component system which uses a grout and an accelerator. The components are pumped through separate lines from the batch plant at the surface or portal to the heading and are stored separately in holding tanks located on the TBM trailing gear. When necessary, both components are pumped simultaneously through separate lines to a mixing nozzle at the tail shield. The components are mixed and pumped through the tail shield to the annulus. The grout gels and solidifies after mixing, usually taking 12–15 sec. Often systems have spare grout pumps and pump spare parts available onsite to reduce the downtime in the event that a pump breaks down.

The grout is usually batched on the surface batch plant. The batch plant is computer controlled and has a preprogrammed mixed design and controls the quantities of the constituents by weight using load cells on the batch plant mixer. It can typically take about 4–5 min to batch approximately a cubic meter

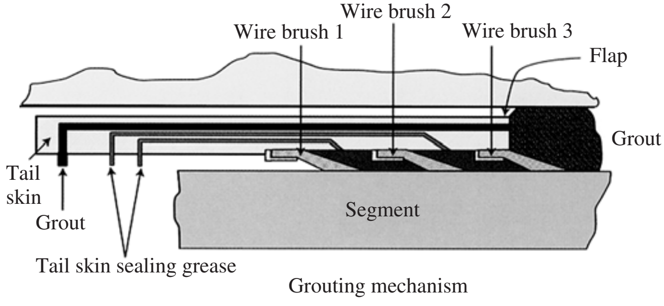

Figure 10.17 Tail Skin Wire Brush Seal (Loganathan, 2011)

of grout. Once a batch is completed, the grout is automatically discharged into an agitator tank. The agitator tank generally has a volume of about 3 m^3 of grout. The levels of grout, both high and low levels, are indicated by a level sensor. When the low level is reached, a signal is sent to the batch plant. The plant will batch grout until the agitator tank is full.

Generally, grout constituents that are in powder form, such as cement, fly ash, and bentonite, are stored in silos and fed to the mixer by screw conveyors. Liquid constituents, such as sodium silicate and stabilizers, are stored in tanks. The stabilizer is pumped to the mixer, whereas the sodium silicate is pumped directly to be the TBM.

Generally, 50-mm (2-in.) pipes transport the grout to the TBM from the surface. The grout is stored on the gantry until use (in some applications, the grout is mixed on the trailing gear). The accelerator is pumped to the TBM from the batch plant. The accelerator and grout storage tanks on the TBM are instrumented to request more grout or accelerator from the batch plant when the tanks reach the low point. Upon receipt of the request, the grout plant will send more of the grout or accelerator to the holding tanks on the TBM. Once the high level is reached, the transfer will stop.

As the TBM advances, accelerated grout is pumped automatically into the annular gap through the ports in the tail shield. Generally, the grout lines leading to the ports are embedded into the skin of the tail shield so as not to protrude and interfere with the ring build. A spare line is often provided for each of the grout lines; in the event there is blockage, the lines can be quickly changed. Figure 10.18 is a schematic of a TBM backfilling system.

Grout ports generally have an automated back-flush system to prevent blockage should the TBM advance stop. High-pressure water will be pumped through the grout lines in the tail shield to a tank in the trailing gear.

Generally, the grout system will be controlled by the advance speed of the TBM and the grout pressure. As the shield advances, the grout is automatically injected. To achieve the theoretical volume, the rate of the grout injection is related to the TBM advance. The target injection is generally set more than 100% of the theoretical volume of the tail shield void.

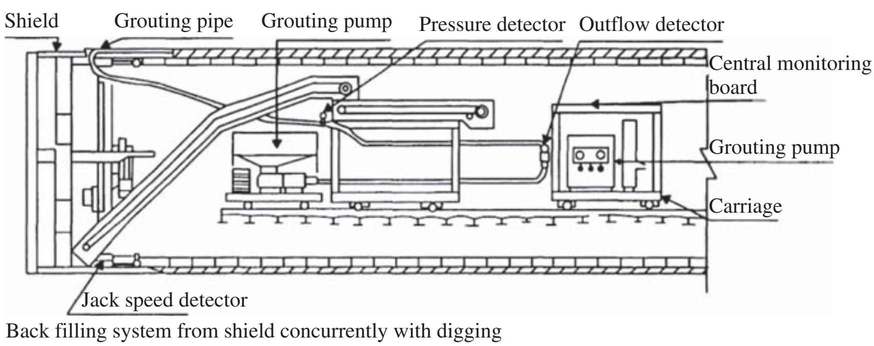

Figure 10.18 TBM Backfilling System (Adapted from Tatiya, 2005, p. 279)

Grouting pressures are generally determined in accordance with the limits determined by the confinement pressure and backfill grouting pressure analysis. The grouting pressure should be greater than the confinement pressure in the working chamber but less than the overburden pressure.

Sufficient standup time of the ground will allow a gap in the time of mining until the injection of grout. The grout can be injected through the precast ports in the segments, as is often the case with hard rock TBM.

Contact Grouting

Contact grouting is required to fill voids that are unknown or of unknown size. Like backfill grouting, contact grouting is done to fill voids between the lining and surrounding ground to prevent ground subsidence, prevent seepage paths along the contact of the ground and lining, and eliminate point loads on the lining. Grout is pumped into a small-diameter hole in a tunnel lining, ports in the lining, or tubes that are fixed prior to the liner installation. Figure 10.19 illustrates holes drilled through a concrete lining into rock.

Generally, contact grouting is done with a neat or sanded grout mix, and the quantity is much less than consumed by backfill grouting. To fully fill the void, it is important to provide an escape for the air in the void. This is usually done by positioning a pipe or tube at the apex of the void so that, as the grout rises, the air will be pushed out of the void through the tube as it is replaced by the grout. (See Figure 10.20.)

There are many reasons for performing contact grouting. Henn (2003 pp. 128–129) provides a comprehensive list of typical examples of contact grouting:

> In shaft excavations in soil, filling any voids that may exist behind soldier piles and horizontal lagging, ring steel and vertical lagging, or liner plate initial support systems; in tunnel excavations, filling any voids that may exist behind steel rib and lagging, liner plate, or expandable concrete segmental initial support system; filling any remaining voids that may exist at the top or crown area, usually located between the 10 o'clock and 2 o'clock positions, after cast-in-place concrete tunnel lining has been placed and cured, or after backfilling of non-expandable precast concrete segments, various types of pipe, penstock, or other types of pre-formed tunnel linings; after backfill has been placed and cured, filling any skin (outer surface) voids that may exist between the final liner and backfill of an installed pipe of penstock to account for shrinkage of backfill material away from the liner; after backfilling or placing cast-in-place lining has been placed and cured, filling any voids that may exist around embedments within the backfill or cast-in-place concrete, such as stiffener rings, shear lugs, reinforcing steel, electrical conduit, small pipes, and other embedments; filling voids associated with sheeting, panning, and invert gravel or piping dewatering systems installed to protect fresh backfill and cast-in-place concrete liners during placement from running water.

Table 10.5 provides the reasons for contact grouting relative to the type of lining.

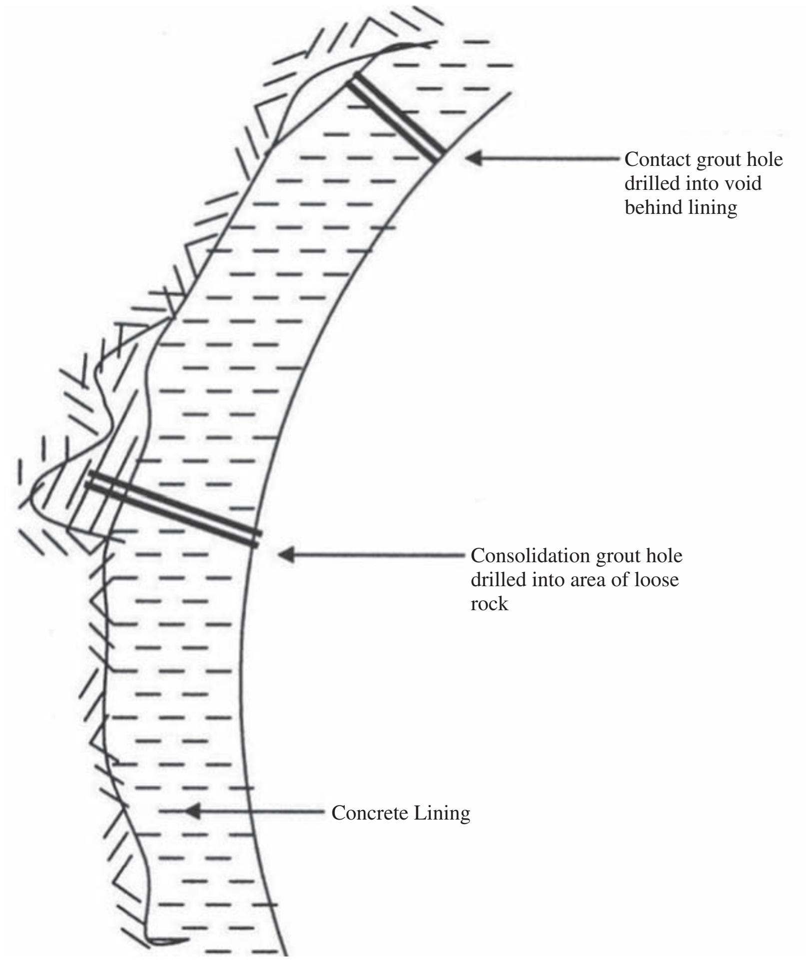

Figure 10.19 Grout Holes Drilled through Lining (Adapted from Henn, 2003)

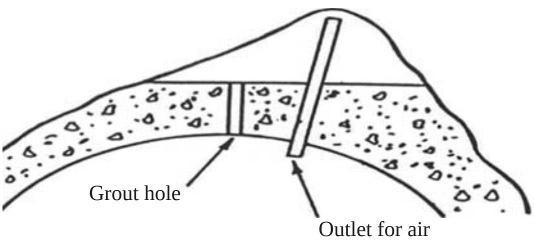

Figure 10.20 Air Relief Hole

Table 10.5 Contact Grouting Applications

Contact Grout Application	Reason to Contact Grout	Initial Support or Final Lining System
Filling any voids that may exist between the shaft initial support system and the excavated ground	To help mitigate surface settlement, to enhance the effectiveness of the ground/liner interaction for load transfer, and to help reduce groundwater flow into the excavation during construction	- Soldier pile and horizontal lagging - Soldier pile and steel plate sheathing - Ring steel and vertical lagging. - Steel liner plate - Steel or CMP pipe installed as initial support or in close contact with shaft walls
Filling any voids that may exist between the tunnel initial support system and excavated ground	To help mitigate surface settlement, to enhance the effectiveness of the ground/liner interaction for load transfer, and to help reduce groundwater flow into the excavation during construction	- Expandable precast concrete segments - Steel ribs and lagging - Steel liner plates -Timber supports
Filling any tunnel crown top voids that may exist after backfill placement and cure. Voids are usually located between the 10 o'clock and 2 o'clock positions.	To help mitigate surface settlement, to enhance the effectiveness of the ground/liner interaction for load transfer, and to help reduce groundwater flow around the final liner	- Nonexpandable precast concrete segments - Reinforced-concrete pipe - Fiberglass pipe - Concrete cylinder pipe - Steel pipe - Steel penstock
Filling any tunnel crown with voids that may exist after cast-in-place concrete placement and cure. Voids are usually located between the 10 o'clock and 2 o'clock positions.	To help mitigate surface settlement, to enhance the effectiveness of the ground/liner interaction for load transfer, and to help reduce groundwater flow around the final liner	Cast-in-place concrete
Filling any skin voids that may exist between a pipe or penstock liner and backfill material; usually only required for penstock and pipe with high internal operating pressure	To enhance the effectiveness of the liner/backfill interaction for load transfer. Voids could be located anywhere on the circumference of the pipe or penstock.	- Steel pipe - Steel penstock
Filling voids that may exist around embedded items such as stiffener rings, shear lugs, electrical conduit, small pipes, and heavy concentration of reinforcing steel	To enhance the effectiveness of the liner/backfill interaction for load transfer, to stabilize the embedded items, and to help reduce water flow around the final liner	- Embedments in cast-in-place concrete liners and in other types of lining systems

(*continued overleaf*)

Table 10.5 (*Continued*)

Contact Grout Application	Reason to Contact Grout	Initial Support or Final Lining System
Filling voids associated with sheeting, panning, and invert gravel or piping dewatering systems installed to protect fresh backfill and cast-in-place concrete during placement from running water	To seal intentionally installed water passageways behind and under the final lining and to help reduce flow around the final liner	- Cast-in-place concrete - Can be used with any two pass lining systems that require backfilling

Source: From Henn (2003, pp. 130–133.)

Grouting Equipment

The basic function of equipment in the grouting process is to mix the grout and pump it to the intended location. However, to perform that function there are several components to the process. Mixing by creating high shear is the primary reason for mixing backfill and contact grout.

Paddle Mixers The most common type of mixers is the tub mixer. Tub mixers consist of a tub with an open top with the height of the tub being approximately 1.5 times the diameter. They are usually driven by air-powered motors mounted on top and often connected directly to the paddle shaft with various capacities and arrangements of mixing blades. The grout is mixed using several horizontal blades mounted on a vertical spindle. They may be used individually, but usually two or more are used in parallel or series. The paddle blades are arranged so as to force grout to the lower section of the tub where the grout is discharged through a valve to a sump. This approach can be easily charged, observed, and cleaned. The paddle mixer slowly stirs or blends the mixture. The shearing forces are very small and unmixed cementitious lumps are usually suggestive of an unstable heterogeneous mixture. These clogs are at times orders of magnitude larger than the apertures of the voids that they are intended to penetrate.

These clumps can cause problems with the delivery system, such as plugging pumps, valves, and grout lines or any of the injecting equipment. The effects of clumps for backfill or contact grouting is not as serious as with grouting operations requiring the penetration of small apertures, for example, with ground stabilization and water control. Being unstable, the clumps are more susceptible to being diluted by groundwater.

Although there are no known standards, the rotational speed has to be below 60 rpm to avoid grout being thrown out of the tank. Although the relatively low cost of these types of mixers makes them popular with contractors, they only slightly meet requirements for bulk placements such as backfill grouting; they are not adequate for most grouting needs.

Despite their low cost, paddle mixers have many disadvantages that negate the simplicity and cost savings. Chiefly, the disadvantages are the poor quality of grout produced and the slow mixing.

Modifications to the mixer can improve the mixer at minimal cost. The grout can be forced to the bottom of the tank and up the tank wall in a vortex by increasing the speed to about 300 rpm and providing a paddle about 100 mm (4 in.) wide at the base and inclined upward about 30° in the direction of rotation. Fins inclined downward are requisite to contain the grout on the top of the tank by causing the grout to return toward the center of the tank with constant circulation. Notched fins on the side of the tank will further increase the shear energy provided to the grout. This modification requires a more powerful drive motor than that used for low-speed mixing. Also, the shaft will need to be restrained to prevent drifting on the tank bottom because of the high forces acting on the blades (Warner, 2005).

High-Speed/High-Shear Mixers High-speed/high-shear mixers are often referred to as "colloidal mixers" when, in reality, they are not. A more appropriate term to describe them would be "semicolloidal" or "near colloidal" (Reschke, 2000). A colloid is a substance that is microscopically dispersed evenly throughout another substance. A colloidal system may be in any form: solid, liquid, or gas. Examples of everyday colloids are shaving cream, milk, smoke, and mayonnaise.

Colloidal mixers can be single- or double-drum types. Often referred to as "colloidal pumps," they use centrifugal pumps to circulate highly fluid grout mixtures at high speeds through the drum system. They are superior to paddle mixers, because this action produces grout with a greater uniformity, resulting in better penetrability and pumpability.

The process consists of water being injected at the base of the drum cone bottom from which it is sucked into a chamber that houses a rapid spinning rotor. The rotor is in the form of centrifugal pump. The rotor throws the material against the close-fitting case. The process can produce grouts with water cement ratios as low as 0.32 (Henn, 2003).

The cement and other dry components are deposited at the top of the drum over a cone-shaped cap. To prevent clumping from interfering with flow into the chamber, the dry components are distributed fairly evenly over the cone. To form a vortex in the tank, as the materials are batched, the mix is circulated, entering the mixer drum tangentially at the top and upper portion. Occurring rapidly, the circulation continues until the mix is thoroughly blended. This usually occurs much faster than the mixer can be fed with bagged cement. When mixed the product is diverted, using a two-way valve, from the mixer tank to an agitator. To collect any waste or oversized material in the grout, a depressed trap is located directly ahead of the rotor.

High-shear mixers must operate at speeds of 1400–2000 rpm; most electric motors run at 1450 rpm at 50 Hz. The size of the mixing pump should be of a capacity to circulate the whole tank content at least three times per minute (Henn, 2003).

High shear is created by close tolerances between the impeller and the casing or high turbulence in the pump housing. To ensure sufficient turbulence, the pump must have recessed vortex-type impellers. As opposed to close-tolerance pumps, high-turbulence mixers are less subject to wear or damage from large grout particles and they are more suitable for mixing sandy grouts without needing a sand-making drum. Because of the heat generated by close-tolerance mixers, the mix should be conveyed as soon as possible to the agitator holding tank.

Before the mix can be used, a full mixing cycle has to be completed; thus these mixers are referred to as batch mixers. Because the mixing cycle has to be completed, to maintain a steady feed of grout, either the agitator acts as a holding tank or a second mixer is required. With this method, the fillers and admixtures used and the water–cement ratio can be controlled. Water is measured and controlled through a water flowmeter with a preset cutoff control or a calibrated holding tank. Other ingredients are measured in premeasured volumes or by bulk delivery methods. By mounting the mixers on load cells, the weight of all of the ingredients that are placed in the mixer can be measured and accurately known (Henn, 2003).

High-speed/high-shear grout plants generally are compact and are used for underground and heavy construction jobs. The colloidal mixers can easily mix cement slurries, fly ash, bentonite, microfine cements, and other fine-grained materials. Many pumps are designed to provide a simultaneous mixing and pumping operation. Colloidal mixers can be equipped with a high-shear, high-speed diffuser-type pump through which water and solid materials are sucked to achieve complete particle wetness, rotating at speeds up to 2000 rpm that disperse fine particles of dry material to achieve complete particle wetness and prevent flocculation. To assist the induction of dry materials into the colloidal mixer, mixing tanks are also equipped with a powered bridge breaker. Once mixed, the material is conveyed to the agitated holding tank to maintain uniform consistency in preparation for pumping.

Henn (2003) writes that contemporary high-shear mixers have the following features:

- Mixing pump speed of 1400–2000 rpm (Most contemporary electric motors run at 1450 rpm at 50 Hz and are directly connected.)
- Mixing pump capacity such that the whole tank content is circulated a minimum of three times per minute. The pressure generated by the pump is of little importance but typically reaches 0.2 MPa (30 psi).
- The mixing pump should create high shear either through close tolerances between impeller and casing or by creating high turbulence in the pump housing. The latter must have recessed vortex-type impellers to ensure adequate turbulence. High-turbulence pumps have much less wear than

those close-tolerance types and are less prone to damage by larger grout particles. They also are suitable for mixing sanded grouts without the need for separate sand-mixing drum. Sand for backfill grouting is typically specified to have 100% finer particles than the No. 8 sieve.

The shape of the mixing tank should be such that the grout vortex in the tank is broken up, near the base, to void the intake of air. Peripheral baffles are not suitable as they lead to a buildup of grout and are difficult to keep clean.

High-speed mixers had a rotor spinning at approximately 2000 rpm with a smooth surface that was efficient with neat cement and was unable to handle sand. To remedy this, the discar was developed and is still the standard. The discar can deal with mixes containing standard concrete sand with proportions up to four times the cement and can combine neat cement grouts with water yielding water–cement ratios as low as 0.36. Sufficient force is generated by the rotor to feed the mixed grout into the top of the agitator or to transport it a minimum of 45 m (150 ft) horizontally (Warner, 2005).

The colloidal mill is the key component of the colloidal mixer. It involves a high-speed rotor (or discar) operating at 2000 rpm combined with a close-fitting chamber housing. With the internal fluid pressures centralized in the housing, the discar may float freely on its horizontal mounting shaft. The clearance between the discar and the housing walls is approximately 3 mm (0.12 in.) (Reschke, 2000). (See Figure 10.21.)

Acting both as a mixer and a centrifugal pump, it can discharge a mixed slurry either into a holding hopper or into the transporter. It can generate a

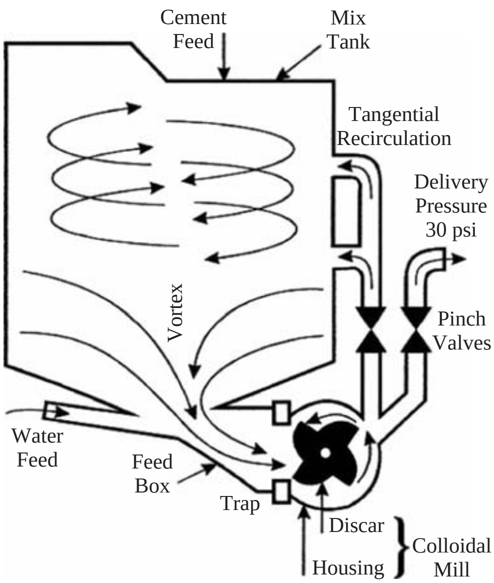

Figure 10.21 Components of a High-Shear Mixer (Reschke, 2000)

Figure 10.22 CEMIX Mixer (Photograph courtesy of Atlas Copco)

maximum discharge pressure and flow rate up to 200 kPa (29 psi) and 850 l/min (225 gpm), respectively.

The centrifugal action resulting from circulation of the material spins the heavy, unmixed, thicker grout toward the tank wall where the water, partly mixed grout, and other lighter materials move inward toward the throat of the tank where the vortex feeds into the mixer housing. (See Figure 10.22.) After the material goes through the high-shear mixer, it is tangentially ejected into the outer part of the vortex among the relatively thicker grout where it migrates inward to the vortex throat under the influence of the strong centrifugal separation process. The mix becomes increasingly thicker with multiple passes through the rotor until the entire mix becomes uniform and the centrifugal force can no longer separate materials with different densities. Once this has occurred, the appearance of the vortex surface is smooth and uniform. This action inside the tank helps to quickly blend any admixtures when first added to the mixer. The entire mixing process can last as little as 15 sec, depending on the mixer size.

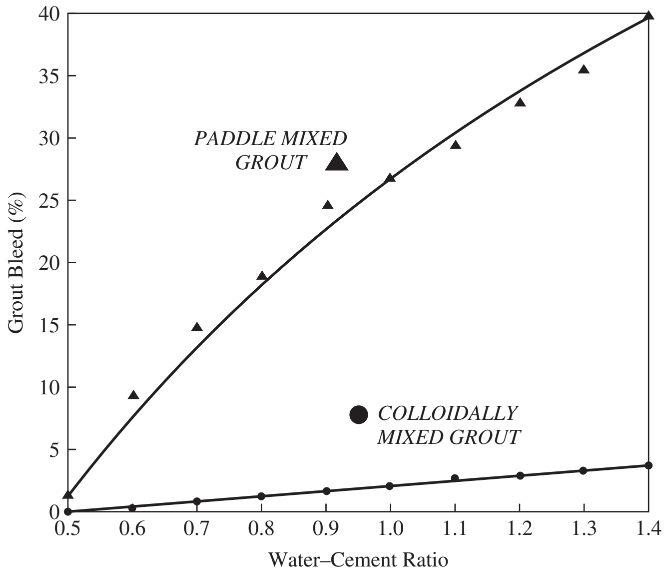

Figure 10.23 Comparison of Bleed for Paddle and Colloidal Mixers with Various Water−Cement Ratios (Reschke, 2000)

Colloidal mixes of grouts or slurries provide benefits that make them superior to paddle-mixed grouts. Since the mix is nearly immiscible in water, washout or contamination resistance is good. Because of its stability and fluidity, it can be pumped substantial distances. If the mix contains sand, segregation is nearly eliminated, which increases the ability of the mix to permeate voids uniformly. There is less bleed in the mixture. This is illustrated in Figure 10.23. Grout mixes were compared using paddle-mixed and colloidal-mixed grouts with water−cement ratios ranging from 0.5:1 to 1.4:1. The graph illustrates the difference in bleed. As discussed earlier, bleed is the characteristic of a grout whereby individual particles in suspension in a fluid settle out of the solution, leaving excess water on top of the settled solids.

An important benefit of colloidal mixes is their ability to accept admixtures and sand. As with any cement product each cement particle should be surrounded by a film of water to facilitate the complete process of hydration in order to achieve the durability and strength required. This can be achieved by every particle being thoroughly moistened with no clumps or particles stuck together. This can be assured by the high speed shearing action of the colloidal mixer. Due to the thorough mixing of the ingredients, the grout is uniform.

Pumps

Piston Pump A piston pump is a positive-displacement pump that uses a cylindrical mechanism to create a reciprocating motion along an axis which then builds pressure in a cylinder or working barrel to force the grout through the pump. The piston reciprocates back and forth in the cylinder, pushing the material, whether grout or gas, to discharge lines. The volume of the fluid discharged is equal to the area of the piston multiplied by its stroke length. Piston pumps can be used to move liquids or compress gases. Although piston pumps are more expensive, maintenance costs are lower. They are also not as bulky as plunger pumps.

Progressive Cavity Pump Also called “eccentric screw pump,” “helical screw,” “progressive helical cavity,” “worm,” “Mono pump,” “Moyno pump,” or just “cavity pump,” the progressing cavity pump is widely used in rock and geotechnical grouting for ground improvement and water control. Figure 10.24 is an example of such a pump. The progressive cavity pump is a positive-displacement pump that transfers fluid by means of the progress, through the pump, of a sequence of small, fixed-shape, discrete cavities as its rotor is turned and provides essentially a continual pressure output. It operates by an eccentric rotation of a helical metal rotor inside a molded double internal helix, which is double the pitch length. Therefore, the volumetric flow rate is proportional to the rotation rate and to low levels of shearing being applied to the pumped fluid. Thus, it has application in fluid metering and pumping of viscous or shear sensitive materials. Other than that caused by compression of the fluid or pump, no flow pulsing is caused by the arrival of cavities at the outlet components, because the cavities taper down toward their ends and overlap with the adjacent cavities, and thus the flow is smooth and free of pulsations.

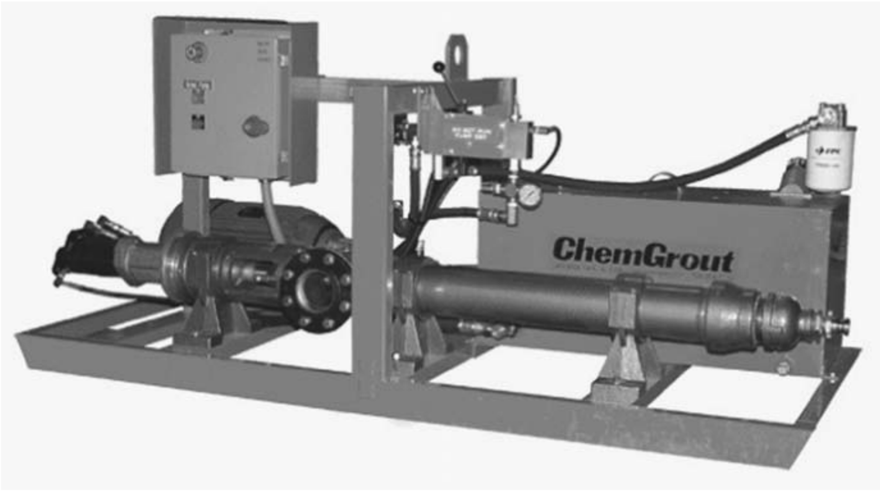

Figure 10.24 ChemGrout CG-L8 Progressive-Cavity Grout Pump (Photograph courtesy of Atlas Copco)

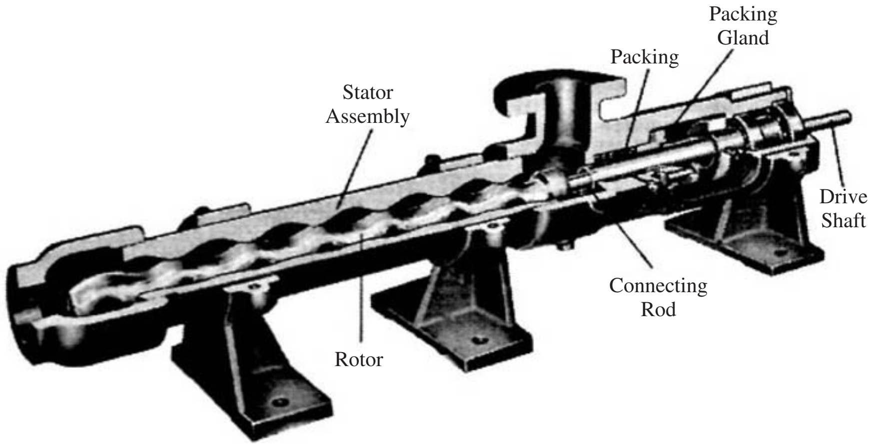

Figure 10.25 Cutaway of Moyno Pump with a Helical Rotor (Warner, 2005, p. 618).

Progressive-cavity pumps are comparatively inexpensive, and although normally these pumps offer long life and reliable service, transporting thick or lumpy fluids and abrasive fluids such as sanded grout will significantly shorten the life of the stator, resulting in higher maintenance costs. Although it can have high injection rates of greater than 90 m^3/hr (120 yd^3/hr) using a four-stage pump, its maximum pressure is a moderate, 2 MPa (284 psi). Because some models do not have pressure and flow valves, it may be necessary to have a valve system at the injection point working with a return line system. Figure 10.25 is a cutaway of the Moyno pump showing a helical rotor.

Plunger Pumps Plunger pumps, also known as ram pumps, are similar to piston pumps in that they both involve a reciprocating plunger within a cylinder. The primary difference between is that the plunger pump does not have a tight seal between the piston and the cylinder wall, that is, whereas the piston has contact with the cylinder, the plunger maintains a distance from the cylinder wall. (See Figure 10.26.)

In the plunger pump, there is a gap between the piston and the cylinder. To prevent fluids from passing around the piston rod, a seal, called a “packing gland,” is instated at the cap end of the housing. As the material in the pump is compressed, the pressure in the pump increases. The packing gland, seal, is compressed, which tightens the gland around the plunger. A pressure of 55 bars (800 psi) is required to compress the gland enough to be an effective sealant. Plunger pumps are well suited for high-pressure pumping and can be used in applications that can range from 70 to 2070 bars (1000–30,000 psi). (See Figure 10.27.)

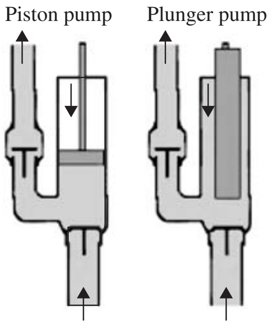

Figure 10.26 Piston and Plunger Pump Operating Differences (From Wikipedia, http://en.wikipedia.org/wiki/Plunger_pump)

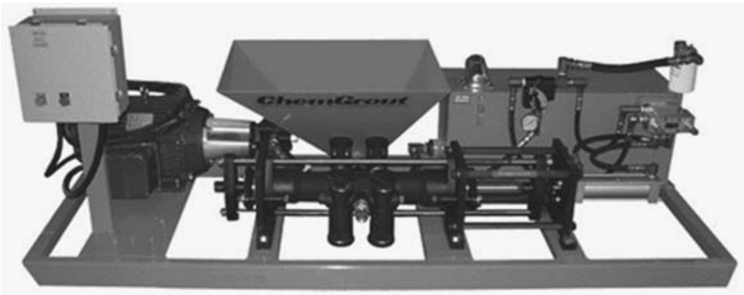

Figure 10.27 ChemGrout CG-3X8 Electric Hydraulic Double-Acting Positive-Displacement Plunger Pump (Photo courtesy of Atlas Copco)

Gauges As with any activity involving pressure and flow quantity, gauges are essential to grouting activities. Their sizes, ranges of pressure, accuracy, and overall quality vary. The top and bottom 25% of gauge range are the least accurate sections on the gauge. Therefore it is prudent to select a gauge where the center range, 25–75%, of the gauge is consistent with the planned grouting working range of pressure. For example, if the planned grouting pressure is 100 psi, the gauge may have a range of 50 to 200 psi (0 to 200 psi). Therefore, various gauges will be needed for the different grouting applications.

Gauges can be purchased in dial sizes from 38 mm (1½ in.) to 406 mm (16 in.). Of course, the smaller gauges are less expensive and more available, but because it is critical to accurately measure the pressure, the gauge should have a minimum diameter of 75 mm (3 in.), preferably 102 mm (4 in.) (Warner, 1984).

Gauge Savers Gauge savers are used to protect the gauges from damage by the material being pumped. Because it will destroy the gauge, contact or backfill grout should not be allowed to come in direct contact with gauges. To accomplish this, a protective medium, or insulator, must be used to separate the grout from the gauge. The implement that is used for this purpose is called a "gauge saver." In the gauge saver, the upper section is separated from the grout by a diaphragm. The cavity between the top of the diaphragm and the gauge is generally filled with glycerin or light oil. The liquid, being noncompressible, is displaced by movement of the diaphragm and conveys pressure changes to the pressure gauge.

Operating Tips

The following suggestions are intended to assist the operator in identifying some of the factors that may need to be considered when mixing and pumping cementitious grouts. Given the wide variety of materials available for many different applications, it is a good idea for the operator to become familiar with the specific characteristics of the materials that may be used, for example, the manufacturers description of the product, the material safety data sheet (MSDS), and the manufacturer's recommendation for optimum performance of its product.

Knowledge of the designed flow of the material is necessary. An analogy is the slump with concrete. Grouts are materials that are to be pumped. The proper consistency must be known and adhered to. The knowledge of the flow may be predicted based on the amounts of the different constituents of the grout. Also, there are field tests that can be conducted to determine the pumpability of the grout.

When conducting grouting operations, knowledge of the grout set time is a necessity. Materials may contain accelerators or retarders to control the set time. In addition, other environmental factors may affect the set time. The temperature of the grout can be a factor. The grout temperature may be affected by the mixing method of the grout, for example, longer circulation than necessary may affect the grout properties.

It is prudent to rinse the mixer and charge the pump hopper with sufficient water to thoroughly flush the pump and all grout lines before attempting to mix and pump production materials. By doing this, you will purge the grouting system of any residual materials or scale that may exist. After the purging is completed, the grout hose should be removed from the pump and, by elevating one end or starting at one end, the entire hose progressively elevated, proceeding to the other end to drain out all water.

To remove any residual water from the hose and lubricate it for the production material to follow, a slurry comprised of portland cement in approximate proportions of 25–28 l (6.5–10.5 gal) of water to one 42.6-kg (94-lb) bag of cement should be mixed and pumped through the grouting system. After flushing and lubricating the line with cement slurry, the production grout may be mixed and pumped immediately behind the slurry mix, which is thus evacuated from the hose, and may be retrieved in a bucket. Trying to pump production material through a dry hose must not be done.

Sometimes a hose will get plugged. The correct way to clear the plug is to empty the material from the hose. Running over the hose or beating it with a hammer is not the correct way to clear the hose. One should have a sufficient length of small-diameter stiff tubing, hose, or plastic pipe that can be quickly attached to a water source and flush the hose using the water. However, experience teaches us that there will be times that the grout cannot be flushed out. If the hose cannot be cleared, it will have to be replaced. That is why it is good to have extra hoses so that the loss of time is minimal.

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11 Portals and Shafts

There are generally two ways to enter a tunnel: through a shaft or through a portal. Whether to use portals, shafts, or a combination of the two is determined by the elevation of the tunnel, utility during construction, and whether the ingress is temporary or permanent. This chapter will discuss the construction of portals and the various ways of sinking a shaft. Shaft sinking is more applicable to mining, but it may also be the best method in civil tunneling.

Generally speaking, transportation tunnels rely on portals because of the relative elevation. Usually, tunneling through a mountain will use portals, and sometimes congestion may require that the portal to the tunnel be below the surface, as is often the case with urban vehicular and rail tunnels. When this is the situation, a ramp is constructed by the cut-and-cover method.

PORTALS

The portal needs to be constructed to protect the entrance to the tunnel from rock or other objects falling on it. Portals provide easier access to the tunnel because one does not have to wait for hoisting equipment, thus eliminating bottlenecks that occur regularly with shafts. Structurally, the portal protects and supports the tunnel entrance and approach from the earth and rock above it. It prevents surface water from entering the tunnel and provides a means to pan, or drain, the water running down above the portal and the invert of the tunnel approach.

Often acting as a retaining wall, the portal is more critical than often realized and should be designed and constructed based on that criticality. The loads that the portal has to support will sometimes require the utilization of geometric shapes for strength. Figure 11.1 is a drawing of a portal that is curved to use arch support. This portal is approximately 20 m (65 ft) high and provides an area to launch four TBMs. On the right side, note the railroad yard. Maintaining the stability of the track is the principal reason for the creative portal design.

If there is only one portal, which is often the case, every worker, every piece of equipment, every utility, every bit of materials, and every ton of muck passes through the portal.

If the tunnel begins at a soil slope, the slope should be cut back so that the ground above it is one to two diameters above the portal minimum. The ground may consist of competent rock or talus on a mountain side and everything in between. If the ground is stable rock, a portal structure may not be necessary.

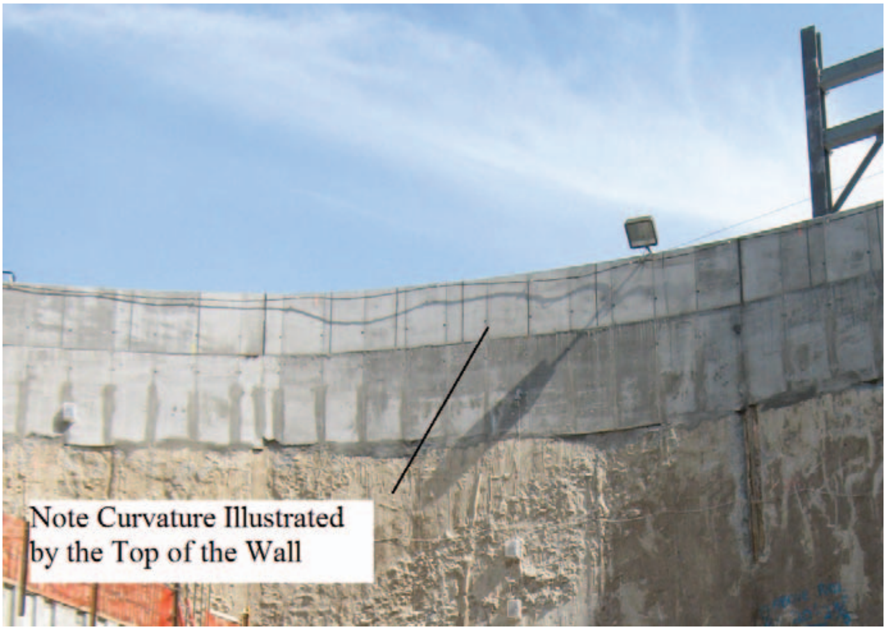


Figure 11.1 Arch-Shaped Portal

Using rock bolts or rock bolts with wire mesh or adding shotcrete to the bolts and mesh may suffice. If blastholes can be drilled vertically from the surface, it is recommended to blast the slope so that there is a vertical wall of rock and where appropriate to clean the surface using controlled blasting, such as presplitting to provide a wall with limited blasting damage. Figure 11.2 is a photograph of a portal in Taiwan. Note that the steep slope has been benched and supported with wire mesh, shotcrete, and tie backs. Not seen in the photograph are the ditches on the benches to collect water and drain it away from the portal.

Portal construction involves excavating and supporting the tunnel entrance. If support is required to stabilize, or hold, the crown, spiling can be installed. Spiling can be pipes that are driven and/or drilled above the tunnel crown, such as in Figure 11.3. These pipes support the roof of the excavation. Note that the ends of the pipes in this soft ground have angular cuts in the field, making the pipes self-drilling. The pipes are then grouted to provide a solid shell above the portal.

Other shapes can be used. The goal is to support the crown. If the portal is to be permanent, extra attention must be given to the portal support. Once the tunnel is complete and the final stages of construction are underway, the portal may be built for its final purpose. Steel beams can be used to stabilize the roof. Figure 11.4 illustrates the portal construction for a rapid transit tunnel. This is

Figure 11.2 Installation of Pipes Above Portal

Figure 11.3 Pipes Cut To Facilitate Installation

a permanent structure. Note the use of both steel beams and pipe for supporting the crown.

The criticality of the portal requires that it be totally supported. Rock should be completely bolted in the portal area. Generally, depending on the quality of the rock, bolts can be spaced anywhere from 1.25 m (4 ft) to 3 m (10 ft). In soft ground, that is, soil, it may be necessary to use forepoling with pipe or one of the methods previously discussed.

Figure 11.4 Using Steel Beams as Spiling for Portal Construction

The portal can be constructed and supported using ribs and lagging, shotcreted ribs and lagging, and liner plate, to name a few. The portal must be constructed to facilitate water handling; on a slope, berms can be built to control the water, as in Figure 11.2.

Irrespective of the portal's surroundings, there must be a means to handle water, whether it is from the tunnel, groundwater, or runoff. The portal should be graded to create a low spot with a sump and a pump to handle water that enters the portal area. When planning this, one should check local regulations regarding the disposal of the water.

The portal should extend from the tunnel entrance far enough to prevent falling or rolling material from landing on the approach invert. To facilitate this, it is desirable to construct a façade wall on the roof of the portal. If the portal is below the surface and a ramp is necessary to get to it, the banks of the ramp may need to be supported. Using shotcrete and mesh is a common method.

Figure 11.5 is a photograph of portal construction using a liner plate and steel beams. Note that the footings are constructed so as to provide long-term stability. Also, because the portal is on a slope, it is being built toward the slope to protect it from rocks rolling down the slope. The slope has been cleaned to minimize anything from rolling down onto the portal. This should be shotcreted once the liner plate is tied into the slope. The shotcrete will prevent erosion of the slope onto the portal roof and strengthen the portal. Because of the slope material, they may have to begin mining using spiling or some sequential excavation approach whether with shotcrete or liner plate.

In addition to considering ground and water control requirements, it must be kept in mind that the portal serves as the focal center for the tunneling operation. Since everything goes through it, it must be designed and equipped to facilitate movement through it. Generally, electrical panels are either in the portal area or, when below grade, above the portal elevation on the top of the bank. If that is the case, there should be easy access from the portal to the panel, for example, using stairs.

Figure 11.5 Portal Construction Using Ribs and Liner Plate (Proctor and White, 1977)

The workers, materials, and muck will be traveling into and out of the portal area. The area will probably have more than one track, more than likely at least three. If trains are being used for muck haulage, the train will have to pass through the portal area to dump the muck cars. All supplies, utilities, equipment such as grout pumps, workers, and possibly liner segments will be loaded in the portal area.

There should be a ventilation fan in the portal area. It should be mounted on a solid frame, elevated out of the way, and should be as quiet as possible. It is dangerous for workers in the portal area to not be able to hear. Storage for general-use small tools and materials should be readily available in the portal area.

In summary, the portal should be well planned and constructed to endure the usage it will get. It will be a very active area, which means safety considerations must be taken. If trains are moving in the area, walkways should be easily seen by the locomotive operator.

SHAFTS

Shafts are vertical or inclined openings connecting the surface and the underground structure and when used during construction serve the same purpose as portals. Many shafts, although used for construction, have uses as permanent shafts. They can be used for deep foundations such as for structures that must be set on competent rock. This would require the sinking of the shaft through the overburden. Shafts have been used as wet wells for the storage of wastewater and pump stations. They can function as ventilation shafts for automobile and subway tunnels. Also, once the tunnel is driven, the shaft can be used as a station at the tunnel level and for access to the station by using the shaft for elevators, stairways, or both. In the mining industry, shafts have a multitude of uses. Mines are required to have more than one entrance. Shafts can be used for the hoisting of muck or ore, transporting of materials, ventilation, hoisting of men, and emergency exits. It is thus required that a mine have at least two shafts if there are no portals that may be used as a means of escape.

Locating Shafts

When selecting the location for a shaft, one must consider the purpose of the shaft during construction, the end use, the proximity to utilities, the proximity to potential laydown and storage areas, and the underground needs. If the shaft is to serve as a ventilation shaft for the vehicular tunnel, then the shaft location alignment is fixed. The availability of power may assist in the determination of the shaft location. If it is located on vacant land, there will be an area for service buildings and muck storage. If the shaft is to be used for mucking, it may be appropriate to locate the shaft in the middle of the tunnel, allowing a double heading driving in both directions from the shaft. By mining from a centrally located shaft, the need to have duplicate plants for mining from both ends is eliminated. It must be remembered that the muck-hoisting capacity must be considerably higher than for a single heading; also, if the shaft becomes out of service, the hoisting equipment breaks down, or some other problem stops the shaft activity, two faces are adversely affected. Another major consideration is the effects on the environment. If the shaft is located on a street, traffic and local businesses will be affected, resulting in both traffic congestion and loss of business income. In addition, it may be prohibited to locate a shaft near an environmentally sensitive area such as a stream or it may create environmental problems and require facilities to treat the water, for example.

Shape of Shafts

Theoretically, shafts can be in any geometric shape. However, the most common shapes are the circle, rectangle, square, and ellipse. The shape of the shaft is dependent on the use and ground conditions.

Rectangular and square shafts are common in mines and generally are internal shafts, for example, a winze, supported with timber. The most common surface application is as jacking pits. For pipe jacking and horizontal boring, the shaft lengths should be longer than the length of a pipe joint and the jacking equipment; often, more is needed for connecting and/or welding lengths of pipe together. As a start, the width of the pit should be wide enough to easily handle the pipe and to work in the area. Rules of thumb can be helpful for cost estimating and a starting point, but the shaft dimensions should be determined by the person on the ground. Their responsibility will be to provide a safe pit requiring a minimum of excavation. Generally, jacking shafts/pits are relatively shallow when compared to shaft depths for tunnels.

The circular shaft is generally preferred for construction shafts. Because of the geometric shape, the circular shaft is much stronger and can be more efficiently supported. Therefore, in ground with high lateral stresses or heavy loading on the shaft ground support, a circular shaft is preferred. Miners often prefer circular shafts to the surface because of their inherent strength and ventilation characteristics.

Construction

Which shaft-sinking method to use is dependent on the size and shape of the shaft, type of ground, ingress of water, and availability of options. Sinking a shaft in soft ground can be done using a clam loader, mini-excavator, or loader. Mini-excavators are on tracks generally with a blade and an excavator boom and bucket. The mini-excavator will generally be used to load a skip or muck bucket for hoisting to the surface. The miniskid steer has a bucket similar to the one on a front-end loader. For working room, the shaft may have to exceed a cross section of 35 m^2 (400 ft^2). The mini-excavator is used in small-diameter shafts. As the diameter of the shaft increases, the size of the excavating equipment can increase. Figure 11.6 is a photograph of a mini-excavator.

Figure 11.6 Mini-Excavator

Generally, the most difficult shaft sinking is through the overburden, because the overburden is generally not as stable as rock. It can be particularly difficult at the interface of the overburden and the rock. This is often an area of water ingress.

As with any construction, if the activity is not started correctly, there will be problems for the entire activity that could have been avoided. The same is true with sinking a shaft. The starting point in the construction of a shaft is the collar.

Collar

The collar is the uppermost section of the shaft. It is where the shaft's stability begins and it protects the shaft from soil and water falling into the shaft. The depth of the collar depends on the depth to the rock, the sinking method, and the groundwater. It is generally considered that the collar extend to bedrock; in some shallow shafts used for civil tunneling applications, that can be the entire depth. For mining applications with deep shafts, the first 30 m (100 ft) is often referred to as the collar. The collar structure is constructed prior to shaft sinking. The shaft perimeter area is excavated, generally as a trench. The shaft provides a secure work area and maintains the shaft's shape. For small shafts, the collar can be made simple but should be no smaller than a minimum of 1 m (3 ft) wide and 1.5 m (5 ft) deep and should be constructed of reinforced concrete and extend below the frost line. It is recommended that the shaft collar beam be shaped like half of a trapezoid (see Figure 11.7), with the sloping side on the outside of the shaft beam. This will reduce some of the shear loading on the edge of the soil below the collar beam. Figure 11.7 represents a

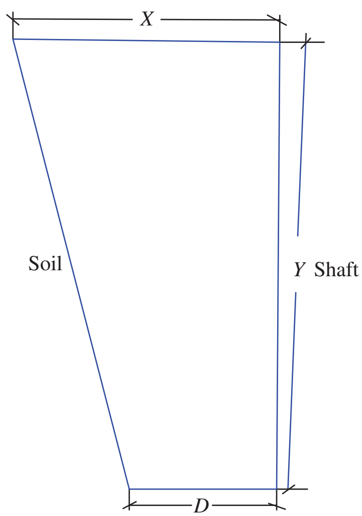


Figure 11.7 Section of Collar Beam (conceptual only)

possible shape without indicating reinforcement; the reinforcing will vary based on calculations and should be designed by a qualified professional engineer.

The collar beam should be 0.3 m (1 ft) above the ground surface to prevent things being inadvertently kicked into the shaft and to prevent the inflow of surface water at a minimum. In addition, the shaft area should be graded to drain all surface water away from the shaft. It is a good idea to place gravel or stone (concrete is even better) on the surface area to maintain a neat and organized work area. If, to save money, the shaft area is not organized well, in the long run this decision may cost much more than proper preparation would have. The collar is where the survey is established and transferred to the tunnel. It should be remembered that a shaft collar is a load-bearing structure. Therefore, a complete analysis of the loads must be conducted. The size of the collar area may be increases or modified to suit operations. For example, if a gantry crane is to be used for shaft transportation, a concrete slab will need to be appended to the collar for the crane's rail and other equipment.

The shaft collar should be constructed based on an engineered solution. That is, it should be designed based on the engineering properties of the soil and loading of the collar area.

The materials used for the collar should be the same as for the shaft lining. The lining immediately follows below the collar. The transition from collar to shaft should not be apparent.

Calculations of lining thickness are done according to structural loads and stresses to which the collar will be subjected. The stresses are calculated based on the loading. In addition, the design must include required factors of safety. The shaft collar dimensions and material properties are designed based on the collar loading.

If the shaft is to be used for mucking and/or transporting materials and supplies and workers, the shaft area will at times be a bottleneck to other tunneling activities. To mitigate this, the shaft area should be organized and maintained. The collar must meet or exceed federal, state, and local safety regulations. The top of the shaft should be reviewed for prevention of falls and dropping things down the shaft. The safety regulations may require only a standard rail of 1.1 m (42 in.), but that may be inadequate for great depths; where possible, a 1.2-m (4-ft) chain link fence on the perimeter of the shaft can eliminate considerable risk. A 10-m- (33-ft-) diameter shaft would require approximately 31 m (105 ft) and appropriate gate(s) would cost less than the project manager's salary for one week but provide a safety environment that will result in cost savings in the long run. Figure 11.8 is an illustration of the equipment located in the collar area.

In the collar area, the muck will be hoisted and moved to a muck pile or bin for hauling from the site. The hoisting equipment may be a mine hoist, crane, or gantry crane/hoist. All services, such as compressed air, water for mining, water removal, and power, should be kept from the collar area piped to the shaft. When possible, services that include piping, such as for compressed air, water, pump water, and electricity, should be buried in the collar area to

Figure 11.8 Gantry Crane above an Internal Shaft

permit better access for handling materials. If the collar area is concreted, it may be necessary to have the conduit run in a trough with access lids or leave a space in the collar slab that is gravel or crushed stone for easy access to the conduit, should maintenance or repair be required. If this appears to be a lot of preparation, it is. One has to “live with” the shaft for the entire project—get started on the right foot.

There are many methods for supporting the ground to excavate through the overburden. Sinking a shaft has an advantage over tunneling when it comes to handling bad ground. With a shaft, one is on top of the ground being excavated rather than having to go under it. That means, when approaching bad ground, one is in a better position to implement ground support measures.

A shaft can be sunk through soil using various types of support depending on depth and material to be excavated. Examples are wood sheeting, sheet pile, soldier piles and lagging, ribs and lagging, liner plate and ribs, slurry wall, and secant piles.

Wood Sheet Piles

Wood sheet piles are boards driven vertically on the shaft perimeter prior to excavation. They are used in very soft ground because the thickness of the boards has resistance to being driven as well as limited strength relative to the

Figure 11.9 Vertical Wood Sheeting Sunk 20 ft to Access Top of Hospital Tunnel at North Shore University Hospital (Courtesy of Macro Enterprises, Ltd.)

force required to drive them. Wood sheet piles are used only for shallow shafts. Although relatively inexpensive and easy to install, wood sheet piles are poor for groundwater control.

Wood sheeting boards are braced from the inside with timber or steel beams. Wood sheeting is relatively inexpensive but has very limited applications. Because of its lack of strength it cannot be driven deep or into the rock interface. Wood sheeting is well suited for shallow shafts such as launching and receiving pits. Figure 11.9 illustrates wood sheeting that is supported by steel beams. Once the sheeting is in place, the excavation of the hole begins. As the shaft is sunk, the horizontal steel beams are installed. It is important that a template be used to maintain the line and plumb of the sheet pile wall.

Sheeting can be driven using standard driving hammers, such as vibratory hammers. The driving forces used must be concentric and must be closely monitored to avoid damaging the boards. Driving has been attempted using the bucket of an excavator. One does not have to do a great deal of study to predict the results.

Steel Sheet Piles

Steel sheet piles consist of steel that is forged to different shapes providing additional flexural strength and an interlocking capability. They are constructed by driving prefabricated sections into the ground. Steel sheet piles are generally limited to depths of 25 m (80 ft). For greater depths, the shaft can be stepped in and another level of sheet pile can be driven. However, the stepped-in area must be wide enough to allow the driving equipment; thus it has limited uses. Steel sheet piles can be driven into tighter, harder ground.

Steel sheeting is desired when depths are so great that wood sheet pile cannot work and where high loads are present. Also, if it is required to leave the sheeting in the ground for a long period of time, or indefinitely, steel sheeting can resist corrosion. More force is required to penetrate the ground, and thus heavier equipment is necessary. Usually driven by a pile hammer or vibrating hammer, the soil conditions may allow sections to be vibrated into the ground instead of being driven by hammer. Usually, steel sheeting is supported laterally using steel beams as waler beams. Also, tiebacks may be drilled into the surrounding ground for lateral support. The full section is used to provide structural resistance. Steel offers many advantages: Its strength permits driving into more resistant soils and it can be fabricated into different shapes giving a greater section modulus. Because of its lock ends, it is very good for water control whereas wood is not (see Figure 11.10).

With steel sheet piles, the full wall is formed by sequentially connecting the joints of adjacent sheet pile sections. Steel sheet piling is most commonly used, because it provides high resistance to driving stresses and is lightweight, and the pile length can be easily changed using welding or bolting. Because of its reuse feature, its cost can be amortized over several projects. During driving, the joints are less likely to deform and, with some protection, can have a long service life below water.

When sheet piles are being installed, they are first laid out in sequence to ensure that they will interlock. Each pile is driven to the planned depth; then the next pile is driven and the two are locked together. This process is continued until the shaft walls are complete.

Problems can develop when trying to drive sheet piles through difficult soils or boulders. One may not be able to drive them to the designed depth. Sheet piles can rarely be used as part of a permanent structure and the shape of the shaft may be controlled by the section and interlocking elements. Because of noise and vibration, there may be complaints from neighbors.

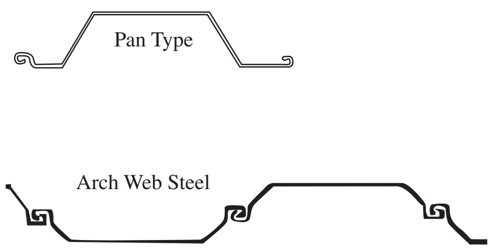


Figure 11.10 Steel Sheet Pile

Soldier Piles and Lagging

Soldier piles and lagging used to retain earth are among the oldest forms of retaining systems in deep excavations. Soldier piles and lagging walls have been used since the late-eighteenth century. The system consists of steel H-piles driven or placed in drilled holes. Once the soldier beams have been installed, the soil is excavated along one side of the beams to partially expose the front faces of the beams. Then, the wood lagging is installed to temporarily hold back the soil. In some cases, shotcrete or reinforced-concrete panels can also be utilized for permanent conditions instead of timber. The soldier pile wall design is based on the depth of the shaft, its shape, and its dimensions; a small circular shaft will not have the strength requirements of a large rectangular shaft. In some cases, because of the geometry of the wall and soil conditions, tiebacks may be installed to provide lateral resistance of the soil and surcharge load. When the shaft is greater than 6 m (20 ft) or the deflection near existing structures, such as buildings and utilities, is very strict, tiebacks are often used. Figure 11.11 is a photograph of a straight soldier-pile-and-lagging wall; note the use of tiebacks.

Soldier piles and lagging consist of steel H- beams being driven, or drilled, vertically into the ground about 1.2–2 m (4–6 ft) apart. The excavating is done in small stages with the lagging being installed as the excavation progresses. The lagging consists of boards 50–75 m (2–3 in.) thick placed and blocked

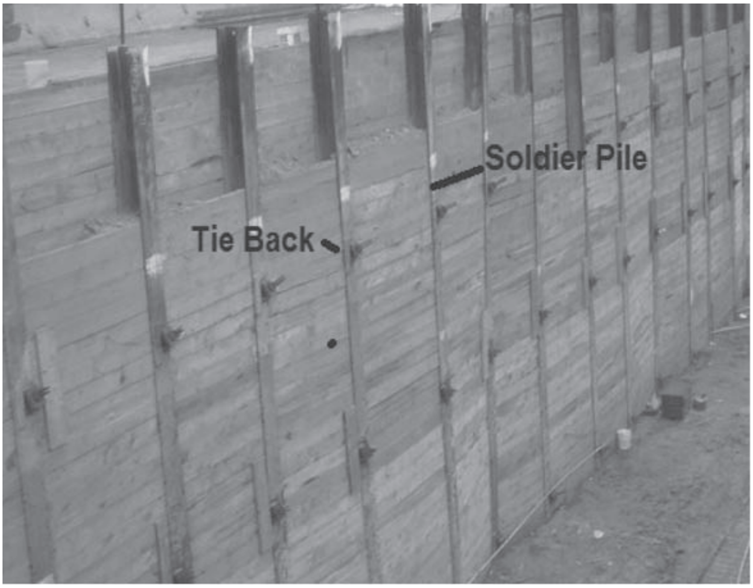


Figure 11.11 Soldier Piles and Lagging. The beams augured without impact noise or vibration, timber lagging, and earth tiebacks to provide a 41-ft-deep excavation in Greenwich Village. New York University Cogeneration Project. (Courtesy of Macro Enterprises, Ltd.)

in the flanges of the piles. Although the installation is relatively easy, the handling of water is poor. It is important to backfill and compact the voids behind the lagging to prevent any soil from moving, causing greater loading on the lagging.

Table 11.1 provides recommended wood lagging thickness values.

With soldier-pile-and lagging walls the moment is resisted by the soldier pile, and the lagging does not provide resistance to the moment. Embedding the soldier piles below the shaft bottom provides passive soil resistance, whereas the lagging between the piles, retaining the soil, transfers the lateral load to the soldier piles.

Soldier-pile-and-lagging walls are easy to construct and, when compared to other support systems, comparatively inexpensive. However, they require extensive dewatering below the water table, which can greatly increase the cost of the system. They are generally limited for use in temporary construction, and if not properly backfilled behind the lagging, there can be a problem with ground losses and surface settlement. With the soldier pile being the only part of the wall embedded beneath the subgrade, it is difficult to control basal movement and the pilet is not as stiff as other shaft support systems.

Ribs and Lagging

A common method for ground support in both shafts and tunnels involves both wood and steel. The method of ribs and lagging uses ring steel with wood lagging placed between the beams. (See Figure 11.12.) The beams are generally placed about 1.2–1.5 m (4–5 ft) apart. The excavation method is determined by the shaft size and material to be excavated. Using 1.2 m (4 ft) of lagging the shaft is excavated in lifts equal to the rib spacing. Once the ring spacing is reached, another ring is placed and the lagging is installed. This relatively simple approach is not good in groundwater situations or in areas that must be excavated the length of the lagging without support. Should unstable ground be encountered, the unsupported spacing can be reduced by changing the ground support to the liner plate.

Using a liner plate, the excavation can be reduced to about 12.7 mm (5 in), thereby improving the stability. Also, the excavation can occur in sections, thus allowing the liner plate to be bolted together without exposing the entire circumference. The spacing of the ribs may be maintained at the previous length or may be increased, because the change to the liner plate may be an increase in strength. In some cases, the beams may not be needed. In addition, water handling is improved.

Liner Plate

Liner plates are thin steel sheets which are corrugated and rectangular in shape. The liner plate can have two or four edges flanged. They are curved longitudinally according to the curvature of the tunnel. The length of the liner plate

Table 11.1 Recommended Wood Lagging Thickness

	Soil Description	Unified Classification	Depth	Recommended Thickness, mm (in.), of Lagging (Rough cut) for Spans, m (ft), of: 1.5 (5)	1.8 (6)	2.1 (7)	2.4 (8)	2.7 (9)	3.0 (10)
Competent Soils	Silts or fine sands or silt above the water table	ML SM-ML							
	Sands and gravels (medium dense to dense	GW, GP, GM GC, SW, SP, SM	0–7.6 (0–25)	50 (2)	75 (3)	75 (3)	75 (3)	100 (4)	100 (4)
	Clays (stiff to very stiff)	CL, CH	7.6- 18 (25–60)	75 (3)	75 (3)	75 (3)	100 (4)	100 (4)	125 (5)
	Clays, medium consistency	CL, CH							
Difficult Soils	Sands and silty sands (loose)	SW, SP, SM							
	Clayey sands (medium dense to dense) below water table	SC	0–7.6 (0–25)	75 (3)	75 (3)	75 (3)	100 (4)	100 (4)	125 (5)
	Clays heavily overconsolidated fissured	CL, CH	7.6- 18 (25–60)	75 (3)	75 (3)	100 (4)	100 (4)	125 (5)	125 (5)
	Cohesionless silt or fine sand and silt below the water table	ML; SM-ML							
Potentially Dangerous Soils	Soft clays	Cl, CH	0- 4.6 (0–15)	75 (3)	75 (3)	100 (4)	125 (5)	—	—
	Slightly plastic silts below the water table	ML	4.6 (7.6)	75 (3)	100 (4)	125 (5)	150 (6)	—	—
	Clayey sands (loose), below the water table	SC	7.6–10.6 (25–35)	100 (4)	125 (5)	150 (6)	—	—	—

Source: Adapted from Lateral Support Systems and Underpinning, FHWA-RD-Report 75–128.
Note: The use of lagging in potentially dangerous soils is questionable.

Figure 11.12 Ribs and Lagging for Shaft Support (Courtesy of Photographer: David Sailors)

is 95.7 cm (37.68 in.). This dimension is pi (π). Thus the number of liner plates used in a set equals the number of feet in the diameter, that is, a tunnel that is 12 ft in diameter will have 12 liner plates for the circumference. Figure 11.13 is a photograph of a shaft supported by a liner plate.

Figure 11.14 is a photograph of four flanged liner plates. Note the bolt holes for fastening them together and the curvature of the plate. Liner plates come in different gages to better match loading.

Grout holes are fabricated in the liner plate to permit the grouting voids behind the liner plate or to stop or reduce the inflow of water. If voids are observed while sinking, material should be stuffed into them to reduce the chances of localized caving. Voids that cannot be filled can continue to expand and eventually cause major ground failure. Hay is often used to fill voids. However, hay is flammable and the risk of fire must be kept in mind.

The liner plate should be small enough and light enough for one person to handle quickly and easily. The exposed ground is 40 cm (16 in.); thus the liner plate can better accommodate ground with little standup time. First the wall is excavated. Once there is enough exposed wall to install a liner plate, the plate is installed. The miner must ensure that there are no voids behind the plate. If there are voids, they should be filled immediately. The fill behind the liner plate does not have to be structural. What the miner is trying to accomplish is to prevent the ground from desiccating or moving; that is, a void can be forced by groundwater or the desiccation of the material to peel off. Preventing the minor raveling or other losses of ground will avoid a bigger collapse later.

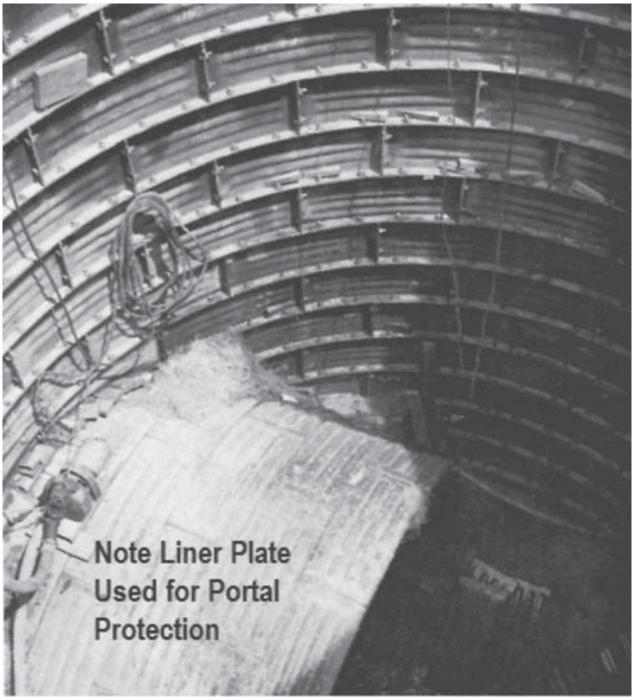


Figure 11.13 Liner Plate Supported Shaft (Proctor and White, 1977)

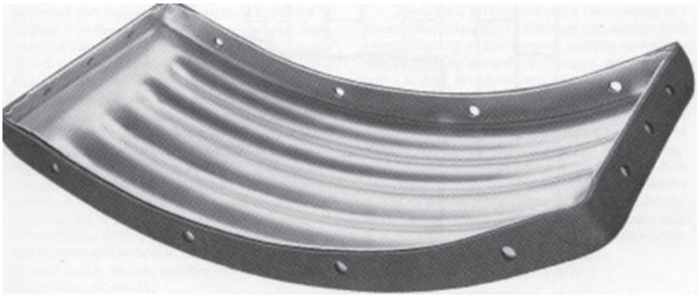


Figure 11.14 Four-Flanged Liner plateK (Proctor and White, 1977)

Slurry Walls

Slurry walls are useful in soil that is unstable or has a high water table. The principal uses for slurry walls in tunneling are walls for cut-and-cover tunnels and shafts. Slurry walls are used as both temporary and permanent earth retention systems to support the sides of deep excavations, such as shafts, as well as to act as cutoff barriers for water control. The slurry wall is typically used to build diaphragm (water-blocking) walls surrounding tunnels and open cuts and to lay foundations.

A trench is dug in which to build a wall using slurry consisting of water and bentonite. Prior to starting the excavation, guide walls are constructed to maintain the wall alignment. The guide walls are constructed on the surface and are approximately 50 cm (18 in.) wide by approximately 1 m (3 ft) deep. The walls act as guides, or templates, to position the slurry wall at the right location.

The trench is excavated and kept full of slurry at all times. The slurry prevents the trench from collapsing by providing outward pressure which balances the inward hydraulic forces and prevents water flow into the trench. During excavation and installation of rebar the viscosity and density are monitored to ensure the density required. Excavation is generally done using a special clamshell-shaped digger. The excavator digs down to design depth, or bedrock, for the first cut. The excavator is then lifted and moved along the trench guide walls to continue the trench with successive cuts as needed. While maintaining the slurry level to prevent cave-in, the excavator is moved to complete the entire length of wall once it reaches rock or the design depth. (See Figure 11.15.)

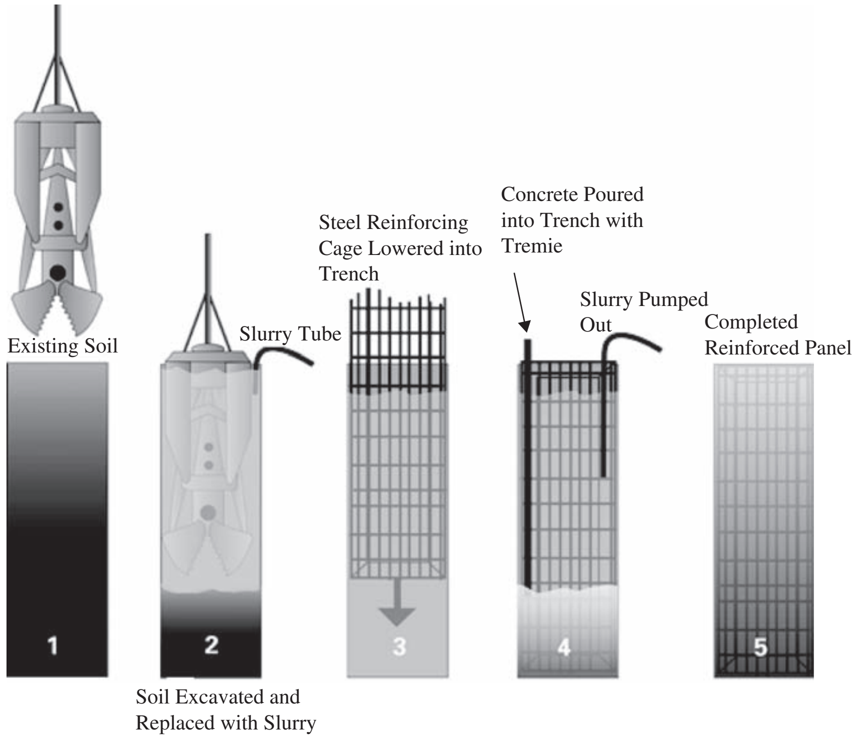


Figure 11.15 Slurry Wall Cycle (Courtesy of Massachusetts General Hospital)

Three-dimensional steel reinforcement cages are fabricated onsite for each panel. The cage is lowered into the slurry-filled trench. Spacers are used to ensure that the cage is accurately located. When the rebar cage is positioned properly, tremie pipes are used to pour concrete into the slurry trench. To have enough fluidity the concrete is placed with a high slump to ensure that all the rebar is encased in the concrete. The concrete is added to the trench from the bottom up using a tremie. The concrete displaces the slurry and the slurry is captured and saved for other excavations.

Sometimes, as with other support systems, ground anchors are used in concert with the slurry wall. This will reduce the bending moment on the wall, reducing dimensions, depth of toe requirement, and reinforcement.

Secant Piles

Secant pile retaining walls are formed by constructing a series of interlocking drilled shafts and can be used where high water tables and/or unstable ground conditions occur. (See Figure 11.16.) Secant pile walls are drilled by high-torque fixed mast rigs. A typical secant wall may consist of 1-m- (36-in.) diameter shafts drilled 75 cm (30 in) on center, depending on loading, wall height, and soil properties. The drilling is typically done through a concrete guide wall that acts as a template that maintains alignment and pile overlap.

Walls that are formed by secant piles are drilled shafts positioned so that the shafts overlap each other. The piles are constructed by either drilling under mud or augering and are reinforced with steel rebar or steel beams. The first piles to be installed are called primary piles. Then secondary piles are constructed between the primary piles. The method involves drilling every other shaft, and after the concrete has set in the initial shafts, that is, in the primary pile, the shafts for the secondary piles are drilled and poured. The piles can be made of low-strength concrete, high-strength concrete, or a combination of low-strength primary and high-strength secondary concrete. When steel beams are used, the secant pile wall design involves the use of weaker than normal concrete.

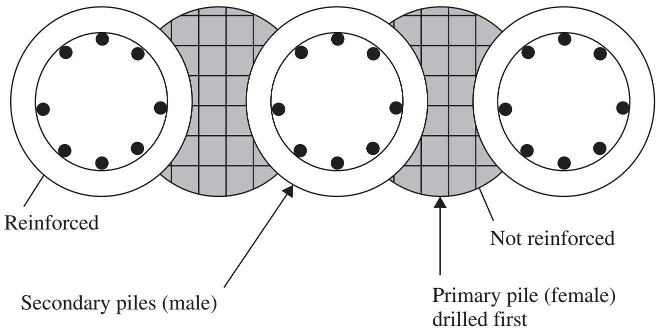


Figure 11.16 Secant Pile Shaft Wall

Using secant pile walls is one of the most economical methods of creating an effective water control barrier. Pile overlap is typically on the order of 8 cm (3 in.). If there is no pile overlap, that is, the piles are constructed flush to each other, it is called a “tangent wall.”

Secant or tangent pile walls allow increased alignment flexibility and thus virtually any shaft shape. The walls are stiffer when compared to sheet piles and can be constructed in difficult ground, like cobbles and boulders, and the construction method is less noisy, reducing potential complaints.

Sometimes it is difficult to maintain vertical tolerances when using deep piles and it can be problematic to waterproof the joints. They are also more costly than sheet pile walls.

Secant pile wall shafts can be backfilled with lean concrete, and in alternate shafts, a steel pile section is installed and the lean concrete provides soil retention between piles. The system can be installed in almost any soil condition and act as a water cutoff as well as provides excavation support. In deeper shafts, lateral bracing may be required, but the small sections in shaft structures generally reduce the necessity for this.

Ground Freezing

Ground freezing is a method for providing ground and water control when sinking shaft. For circular shafts supported by a freeze wall, the wall resembles secant piles. And, because the shaft is circular, the strength characteristics of a circle can be realized.

Drilling and Blasting

Drilling and blasting for shaft sinking are similar to blasting for tunnels. The goal is to move the rock with only one free face. Therefore, the blasting techniques vary from surface or bench blasting in design pattern, detonation timing, and powder factor. The powder factor is the amount of explosives needed per unit of rock. The powder factor for surface blasting can range from 0.4 to 0.7 kg/m^3 (0.75 to 1.25 lb/yd^3), whereas for shafts it can range from 0.91 to 3.6 kg/m^3 to (2 to 6 lb/yd^3).

The primary equipment used for drilling blast holes for shaft sinking are the sinker drill and shaft drill jumbos. Sinker drills are hand held and are commonly seen on the surface or in tunnels for small benches. Generally, the size range of blast holes for production blasting is 3.8–5 cm (1.5–2 in.); the sinker can drill larger holes, such as burn holes, but their torque and the fact that they are hand held restricts their use in special applications. The hand-held drill has been the traditional shaft-sinking tool.

The drill jumbo consists of a column with drill booms attached around its perimeter. The number of drill booms is determined by the shaft and drill sizes. Figure 11.17 is a photograph of a shaft drill jumbo.

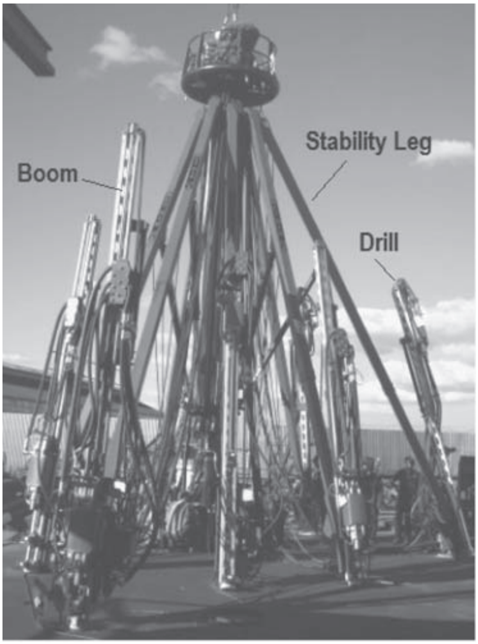


Figure 11.17 Shaft Drill Jumbo (Courtesy of METSHAFT (PTY) Ltd.)

The jumbo is lowered to the shaft bottom and prepared for drilling. Generally the drill steel should be as long as the blast hole and within the length limits of the jumbo boom, and only one drill steel is needed to drill to the required depth. Drilling a shaft bottom can be challenging in that the rock will probably be somewhat fractured from the previous blast and one is generally contending with water inflows.

The use of jumbos has greatly reduced the time spent for shaft sinking, as compared with the hand-held drill. The commonly used technique of benching, or shaft sump cutting, with hand-held drills is no longer as common. The introduction of jumbos has changed shaft bottom drill patterns to full-face drilling rather than benching techniques. The typical pattern now has changed from the V-cut to the full-face cut using the burn cut. This enables the reduction of flyrock and damage to equipment.

As stated earlier, the requirement for additional explosives can be reduced and/or the fragmentation can be improved using the sump cut. In addition, the water can be better managed by having the pump in the sump providing a drier level. The sump cut consists of not maintaining a level shaft bottom when sinking. The shaft bottom is divided into halves. One side is deeper than the other, allowing a second face to which to break the rock and a location for a pump. As the shaft deepens, the sump will alternate sides as the sump is deeper than the rest of the shaft. This method is used with hand-held drills.

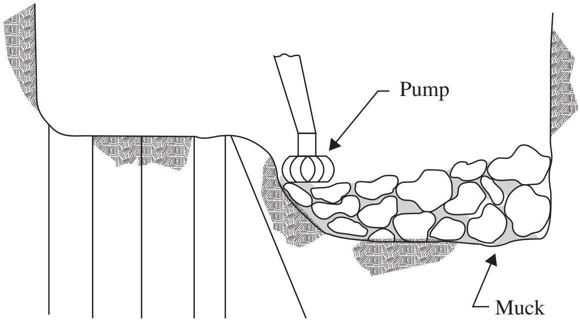


Figure 11.18 Shaft Sump Cut

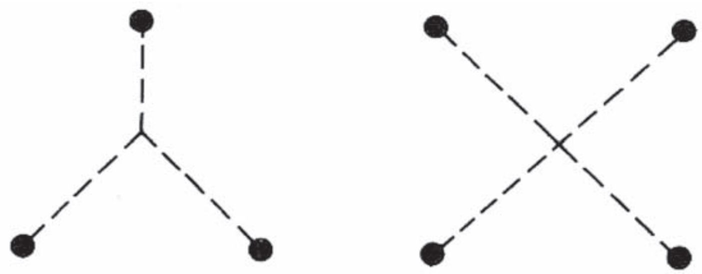
Shaft jumbos negate the need for this method. However, if the situation is such a shallow shaft that a jumbo is not practical, the sump cut can be used. Figure 11.18 is an illustration of the sump cut.

Note that the hole closest to the sump is angled. Using angled holes helps develop the second free face. The rock tensile strength is about 10% of its compressive strength. Therefore less energy is required to break the rock, reducing explosive cost, requiring few holes, improving breakage, reducing collateral damage to shaft utilities, and reducing vibration. The sump cut is the most effective for shaft excavation with hand-held drills.

When sump cuts are not practical, angle cuts can be used for developing a second face. As the name implies, angle cuts require the holes be drilled at an angle. They are referred to as wedge cuts and are effective for well-laminated or fissured rock. The angle cut is generally used in larger shafts. Also referred to as the "V" or plow, the holes are drilled symmetrically at an angle of approximately 60°. Because of the increased angle, the powder factor can be lower than for vertical holes.

Wedge cut holes are angled to provide a second free face by detonating the holes to make a V first, thus allowing the subsequent holes to use the second face created by the wedge. A factor of the wedge cut is that, because of the relatively flat angle, the rock fragments are larger than with other cuts. These larger rocks that are catapulted up the shaft can cause considerable damage to ground support and utilities. If this is the type of blasting one chooses to do, the shaft utilities should be kept farther from the blast. The size of the rock fragments can be mitigated by placing a small borehole (a buster hole) at the center of the wedge which will serve to break the fragments into smaller pieces.

A commonly used drill pattern in shafts is the pyramid cut. Although similar to the wedge cut, it has only three or four sides, the pyramid cut holes should be deeper than the other holes. It will create a sump for water handling when drilling the next round and help with breakage of the pyramid. Figure 11.19 illustrates the pyramid cut.



(a) (b)

Figure 11.19 Pyramid Cut

If the size of the round is large, much of the pyramid may break, causing damage to the shaft and utilities and reducing the success of the blast. A "baby pyramid" is drilled within the perimeter of the pyramid drill pattern. This aids the blast geometry and facilitates breakage.

As discussed above, because of the space needed, these drill patterns are generally restricted to hand-held drills.

Rock Shaft Mucking and Sinking

During tunneling or mining, when related to the quantity of material excavated, shaft sinking is a small part of the underground activities. Mechanized shaft sinking has been slower to develop because of its smaller role and the difficulty of vertical excavation relative to horizontal excavation. Most shafts are sunk using blasting and traditional mucking equipment, traditional meaning that this equipment has been around for a long time. Changes in the mucking equipment over the years have increased capacity and durability.

One way to categorize shaft-mucking equipment is by the type of digging appliance. More generally, they can be grouped by their location when excavating. There are those that are located on the shaft bottom and those that are suspended above the shaft bottom. They can be further classified as to their power source. The source of power to operate the appliances can be pneumatic or hydraulic. All shaft muckers dump into buckets or muck skips to be hoisted to the surface.

Shaft bottom muckers can be miniature backhoes and front-end loaders. The loading devices on miniature excavators/backhoes and "bobcat-type" front-end loaders are hydraulic, whereas the Eimco front loader is air operated.

Figure 11.20 is an illustration of the Eimco 630 mucker. The illustrated mucker can also be track mounted for tunnel mucking. It is air operated and digs the muck with lateral force of the machine moving forward and manipulation of the bucket. The mucker is referred to as an overshot loader because the loaded bucket is propelled rearward, causing the contents to be thrown into the nearby muck bucket. Note in the illustration that miner stands on a small

Figure 11.20 Eimco 630 Mucker (Courtesy of the Society for Mining, Metallurgy, and Exploration)

platform, or step, and rides the mucker. Generally, for shaft mucking, a heavy dished bucket is used with a capacity of 0.025 m^3 (9 ft^3).

The Cryderman mucker is a common shaft machine in the low- to medium-capacity range. (See Figure 11.21.) Once pneumatic it is run by using hydraulics for operating the cylinders and the telescopic boom and is equipped with a 0.3-m^3 (10-ft^3) bucket and has an hourly mucking capacity of approximately 11 m^3 (15 yd^3). The boom is suspended from a sectional framework that is usually located at one end of the shaft. It can be suspended from an independent hoisting system located either on the surface or in the shaft. In large rectangular or circular shafts, two Cryderman muckers are generally used.

The cactus grab mucker has muck capacities in excess of 90 m^3 (120 yd^3) per day. (See Figure 11.22.) In circular shafts the operating mechanism is part of the multideck sinking platform, often referred to as the Galloway. The designs can be a central column with a cantilevered boom or a central column with the boom supported on an outboard rail that is attached to the work deck. The boom rotates a complete circle with both types and the grab is suspended from the traveling carriage of the boom. Because the boom rotates and the carriage has radial movement, the grab can be positioned anywhere in the shaft.

Unlike the clam or cactus grab methods, the Alimak mucker has a backhoe action. It is a totally self-contained hydraulic machine that can be completely suspended below the work deck or mounted onto the shaft wall, as depicted in Figure 11.23.

Raise Borer

If the shaft is to intercept an existing tunnel or mine workings the shaft may be excavated from the bottom up. A pilot hole is drilled from the surface

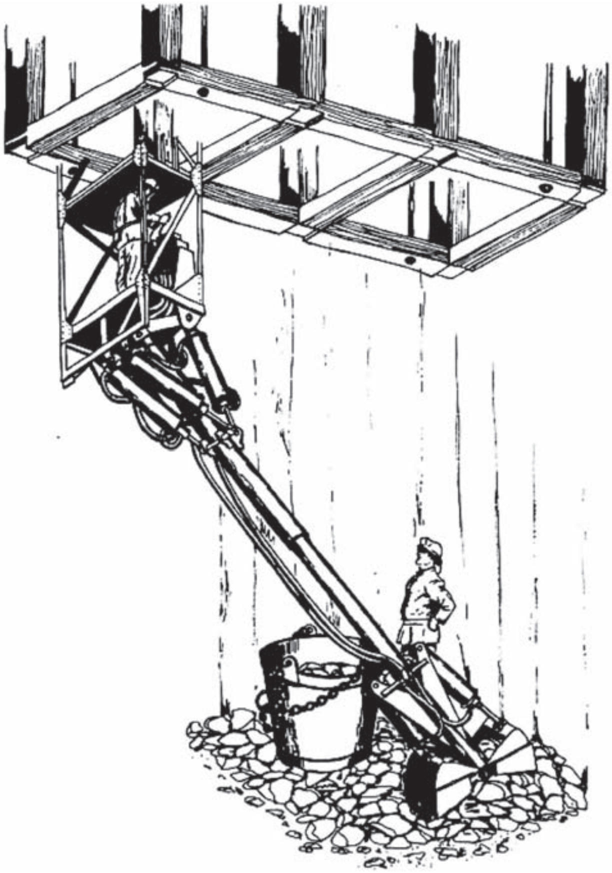


Figure 11.21 Pneumatic Cryderman Mucker (Courtesy of the Society for Mining, Metallurgy, and Exploration)

intercepting the underground opening. A cable and accessory lines are run from the surface to the opening. A reamer is attached to the cables and the reaming tool is pulled to the surface, reaming the hole to the desired diameter. This method has been used for both mining and civil applications for many years. In mining, it is commonly used for ore passes and ventilation shafts. Most common civil applications are ventilation shafts for transportation tunnels and surge shafts in hydroelectric facilities.

A heavy double-walled rod, or drill string, connects the cutting tool to the rotary table located on the shaft-drilling rig. The torque or rotation for the reamer is provided by the rotary table. The shaft is filled with water which is maintained at a constant level throughout the entire shaft development, creating an upflow. The shaft and the hollow drill string are both water filled,

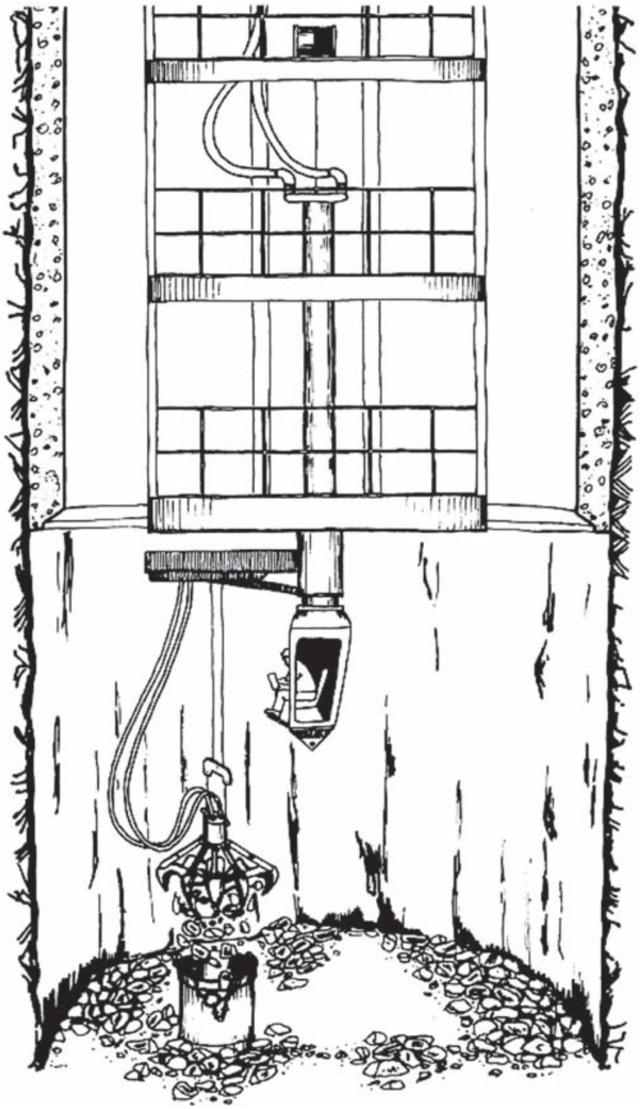


Figure 11.22 Cactus Grab Mucker with Central Column and Cantilevered Boom (Courtesy of the Society for Mining, Metallurgy, and Exploration)

creating two independent columns of water. To displace the fluids and create a much lighter column, compressed air is injected into the water column of the drill string. Because the water column is heavier inside the shaft, it pushes down and across the development face (bottom of the shaft). The water is then forced through a small opening, called a pickup on the reamer body, which has

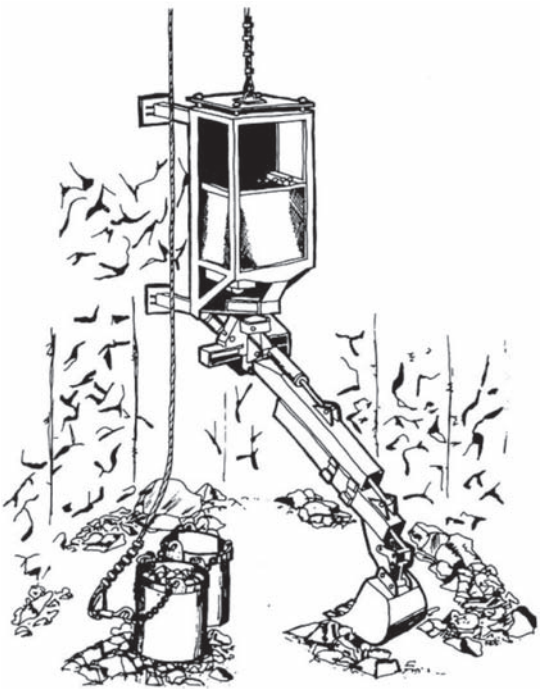


Figure 11.23 Alimak Mucker Backhoe (Courtesy of the Society for Mining, Metallurgy, and Exploration)

water forced through it, displacing the lighter water column in the drill string and resulting in an upward flow or "reverse circulation." A huge vacuum at the pickup point caused by the volume of fluids being replaced, usually about 7570–11,300 lpm (2000–3000 gpm), literally sucks the cuttings from the face. Figure 11.24 illustrates the raise bore method. Note the pilot hole is drilled, followed by the reamer.

When used in a mine, the raise borer setup is made on the upper level connecting to the lower level. Usually it is set up on a concrete slab. For civil tunneling, the raise borer will likely be on the surface, and a pilot hole, which is generally 230–350 mm (9–15 in.) and big enough to accommodate the drill string, is drilled from the surface to the underground target. The target can be a tunnel, an adit, or a cavern. When the drill has holed through into the target opening, the bit is removed and a reamer head with diameter the size of the shaft to be bored is connected to the drill string and raised toward the machine at the starting location. As the reamer head advances, the drill cuttings fall to the invert of the target level. Figure 11.25 illustrates the raise borer reamer. Raise boring is often preferred to conventional drilling/blasting.

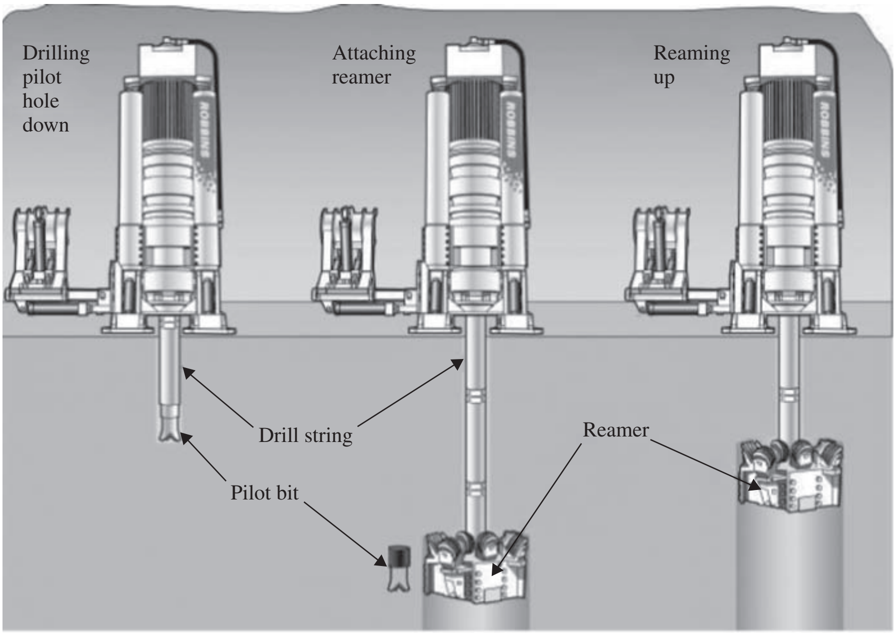


Figure 11.24 Raise Borer Method (Courtesy of Atlas Copco)

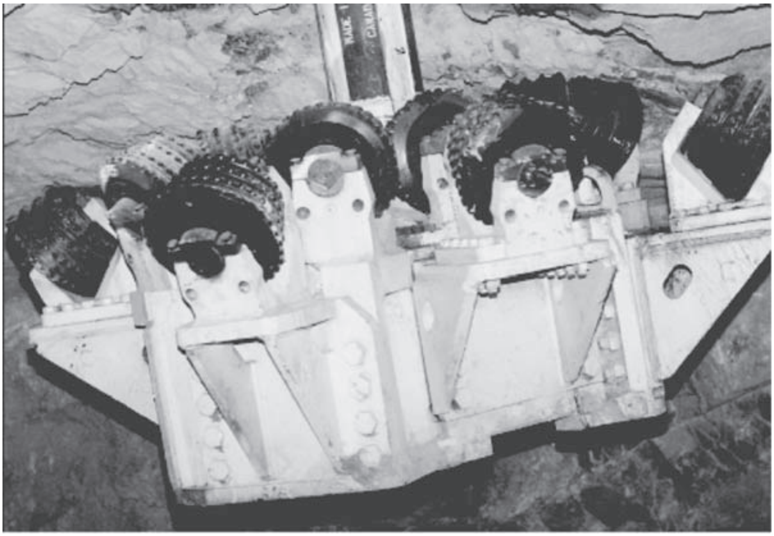


Figure 11.25 Raise Borer Reamer (Courtesy of Atlas Copco)

Paramount is safety; with raise boring, miners are not as exposed to rock falls as in blasting or to fumes and the inherent dangers of explosives. Raises can be drilled in ground that is dangerous and in some cases too hazardous to excavate conventionally. After safety, the reduction of labor and related costs, the speed, and the condition of the shaft when completed are other advantages that raise boring has over blasting.

Raise borer shafts are generally completed much faster than by conventional methods. This not only results in cost savings but also allows follow-up work to start earlier. In addition, it requires a smaller crew, less muck to dispose of, and less ground support requirements. These all add to cost savings.

Boxhole Borer

Although rarely used in civil tunneling, the boxhole borer (or machine roger) is related to the raise borer and, because of its low profile, is used when there is not enough space on the upper however it's also the most difficult raise method. The boxhole borer is mobilized on the lower level, drills a pilot hole to guide the borer, and then advances the reamer bit along the pilot hole from the lower level to the upper level. The cuttings will fall to the lower level, so precautions must be implemented to redirect falling drill cuttings away from the machine and to reinforce the drill string. A common machine that can function as a raise borer and a boxhole borer is the Robbins 34RH model. (See Figure 11.26.)

Figure 11.26 Atlas Copco Robins 34RH Raise Borer Rig (Courtesy of Atlas Copco)

Figure 11.27 Blind Shaft Drill(Courtesy of Shaft Drillers International)

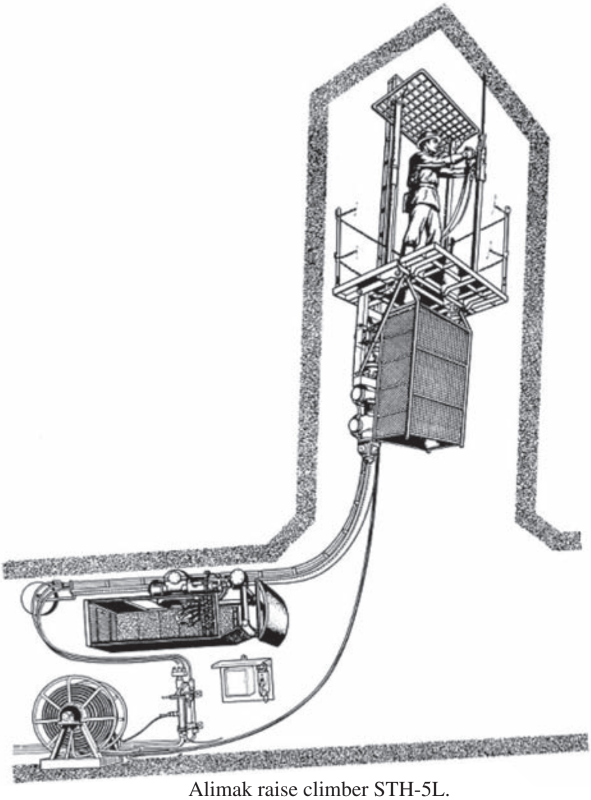
Blind Shaft Drilling The process of mechanically excavating large-diameter vertical shafts into the earth with a full-face drill head is called blind shaft drilling. (See Figure 11.27.) If a pilot hole is not possible, a blind shaft boring machine needs to be used. With blind shaft drilling, the excavated material is transported through the shaft to the surface.

The drill head is part of a submersible assembly that consists of weighted cylinders and stabilizers that provide the motive force for penetrating the rock. A high-torque rotary drive rotates the assembly from the surface. The rock is penetrated by the combination of the weighted assembly rotated over it. A flow of water powered by an airlift removes the cuttings and transports the material to the surface where the rock is separated and the water is recycled into the process. The blind shaft drill in Figure 11.27 is drilling a 5.5-m-(18-ft-)diameter by 100-m-(330-ft-) deep intake air shaft in Kentucky.

The automated process of blind shaft drilling offers certain advantages over conventional shaft-sinking techniques. All of the work is performed on the surface and the crew is generally limited to three or four miners. The blind shaft drilling process, depending on the ground conditions encountered, can normally advance at much quicker rates than a conventional shaft-sinking operation.

Raise Climber

Similar to exterior elevators used on the side of a building during construction, raise climbers climb on a rack pinion welded to a guide rail and carry pipes and hoses for air and water. The guide rail can be extend in one to two increments and, although the standard cross section is about 9 m^2 (97 ft^2), raises as large as 35 m^2 (380 ft^2) have been driven. Figure 11.28 is an illustration of the Alimak raise climber.



Alimak raise climber STH-5L.

Figure 11.28 Alimak Raise Climber (Courtesy of Society for Mining, Metallurgy and Exploration, Inc.)

The cycle for driving a raise begins with the preparation and installation of the raise climber. It begins with excavating the start of the raise a minimum of 3 m (10 ft) to permit the installation of the guide rail. Once the raise climber is installed, the first round is drilled from the work platform and under the heavy steel grizzly-like roof. Once the drilling and loading are completed, the raise climber descends the shaft and travels on the guide rail from under the shaft opening, protecting it from the blast. Once the blast has been ventilated, the raise climber climbs the shaft to the end of the guide rail. The blast area is scaled and bolted where necessary. The guide rail is extended, fastening to the wall with expansion bolts. When the raise climber is in place, the mining cycle begins with drilling.

Note the platform is designed to prevent the miner from falling. Also, the roof protects the miner from rock falling onto the platform, while allowing the miner to drill.

Figure 11.29 illustrates various shaft-sinking equipment and their relationship to one another. The drill jumbo is located at the shaft bottom. Once the

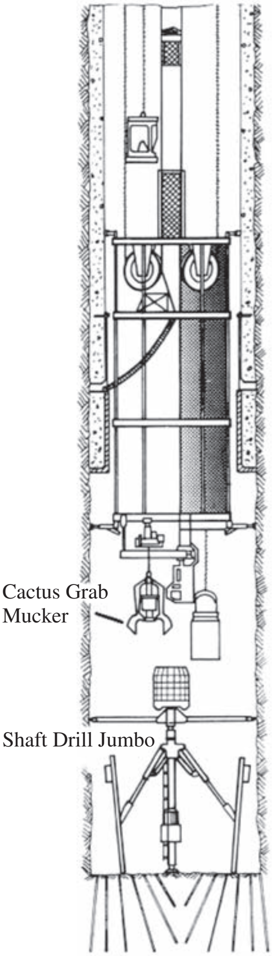


Figure 11.29 Mechanized Shaft-Sinking Equipment (Courtesy of Tamrock)

round is drilled, the drill jumbo will be raised and the round will be loaded with explosives. Once blasted, the shaft bottom will be mucked with the cactus grab mucker. The cactus grab is attached to the shaft wall or the shaft work deck.

Other Mechanical Methods

Methods are being developed to sink shafts by mechanical means. Both Herrenknecht and Robbins have developed a shaft-sinking machine that can sink all types of shafts. They are designed to be used in both stable and unstable soil. They consist mainly of a sinking unit and a shaft-boring machine. Figure 11.30 is a sketch of the Robbins machine.

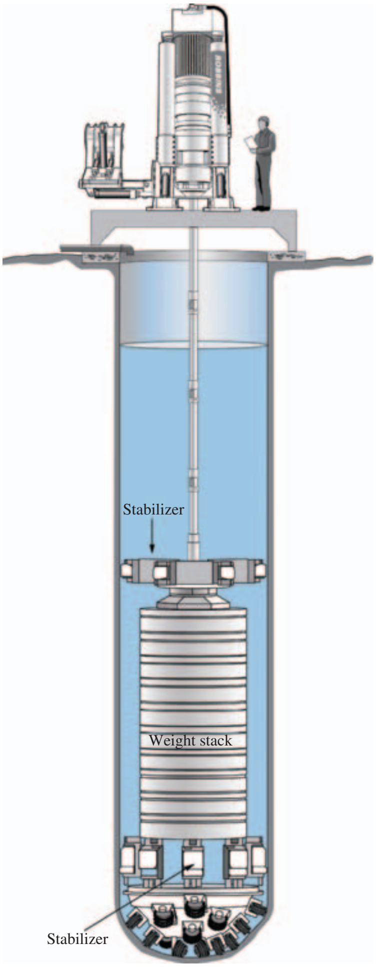


Figure 11.30 Robbins Vertical Shaft Sinking Machine (Photograph courtesy of Atlas Copco)

Functioning like a TBM gripper system, the machine is braced with gripper arms against the solid rock. The machine receives all supplies and is controlled from the surface and, with information being monitored, the operator can be constantly aware of the machine's technical parameters.

REFERENCE

Proctor, R. V., and White, T. L. (1977), "Rock tunnelling with steel supports," Commercial Sheering & Stamping Co., Youngstown, Ohio.

12 Sprayed Concrete (Shotcrete)

Sprayed concrete (shotcrete) is concrete that is pneumatically sprayed rather than formed. The shotcrete is blown onto the applied surface with sufficient velocity to achieve compaction: a velocity of 20–100 m/sec (65–330 ft/sec). The compaction facilitates the bonding and the strength of the shotcrete. Throughout this chapter *sprayed concrete* and *shotcrete* will be used interchangeably. The intent of shotcrete is to get a mix with the lowest rebound that will provide a dense compacted and adhering layer on the substrate. Because it is sprayed on and often overhead, the quality of mix and application are paramount. The spraying method of application and the need for adhesion and early strength require that shotcrete have different additives and mix designs than standard concrete mixes. Accelerators are used to increase adhesion, set time, and early strength. The quick-setting attribute permits a faster production cycle and helps to reduce rebound.

Although there are above-ground applications for shotcrete, that is, for slope stability, most shotcrete utilization is underground, more specifically, tunnels. The principal uses are as ground control and to prevent the rock from moving, thus contributing to the rock's self-support and surface protection; rock bolts may be sufficient to support the crown, but the rock surface may tend to fall off between the bolts, requiring an application of shotcrete. In addition to the obvious hazards from rock spalling from the crown, the sloughing off of the rock can result in greater ground loss by permitting rock that is acting as a binder, or wedge, to be loosened, precipitating greater rock fall. The rock bolts would probably prevent a catastrophic collapse, but the loss of rock in the crown could still be substantial.

Shotcrete is a high-quality concrete that typically has a low water–cement ratio, usually a 3-in. slump with a low water–cement ratio made possible by additives. It is important that shotcrete aggregate meet the American Society for Testing and Materials (ASTM) C 33 requirements and that portland cement meet the C 150 requirements. Fine sand that is not within ACI 506R specifications, although easy to apply, can cause problems with the water–cement ratio. The finer the aggregate is, the greater the surface area, which in turn requires more water. This increase in water can cause more shrinkage-related cracking. A water–cement ratio that is too high can cause a lack of aggregate distribution that does not provide the proper matrix of particle sizes to produce good-quality concrete. Also, the increase in water–cement ratio will have negative effects on the strength and density of the shotcrete (Brennan, 2005). Because of the ridge surface of rock, when first applied on the rock, some of the coarse aggregate, that is, 20 mm (3/4 in.) maximum, tends to rebound, or bounce off, leaving a greater proportion of paste and fine aggregate. As the thickness of the paste increases, the surface being sprayed becomes softer, and the rebound of the coarse aggregate is reduced. Therefore, the in-place aggregate will be finer than the mix. Because of the compacting effect of sprayed concrete, the strength is generally higher than if the same mix was placed in forms. A thin layer of shotcrete on rock will experience more rebound than when sprayed on soil. The quality of the work is directly related to the skill of the nozzleman. The typical mix has 300–425 kg/m^3 (500–700 lb/yd^3). The high end of the shotcrete strength spectrum is approximately 70 MPa (10,000 psi), but typically shotcrete strength is in the range 20–30 MPa (3000–4000 psi). Shotcrete early strengths can reach 7 MPa (1000 psi) at 1 hr and 20 MPa (3000 psi) in 5 hr. Sprayed concrete has greater bonding strength than concrete and, like concrete, is greatly affected by the water–cement ratio. The amount of shrinkage is affected by the quantities of cement and coarse aggregate. The shrinkage can be reduced with the amount of coarse aggregate. When possible, the amount of coarse aggregate should be minimized to reduce rebound, especially with thin layers.

There are two application methods, dry and wet mix.

DRY MIX

Cement and damp aggregate that are thoroughly mixed or premixed and prebagged cement and aggregate are fed through a premoisturizer. The cement–aggregate mixture is then fed to the gun. (See Figure 12.1.) There are two types of dry-mix guns: continuous-feed rotary and single or double chamber.

The continuous-feed guns use the rotating airlock principle to provide the continuous feeding action. The feed bowl and barrel are the two types of rotary

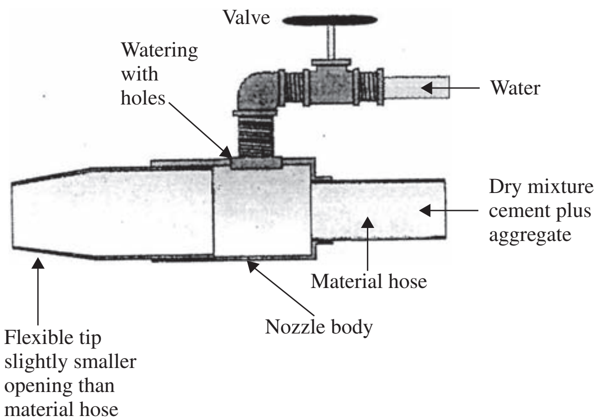


Figure 12.1 Details of Dry-Mix Nozzle (Courtesy of American Concrete Institute)

guns. One sealing segment on the top of the surface of the rotating element is utilized by the feed bowl-type gun. From the hopper, the material is gravity fed into the U-shaped cavities in the rotor and is discharged into the outlet neck. When one of the cavities is full, air is injected into one of the legs of the U and carried into the material hose. The barrel type uses the sealing plates on the bottom and top of the rotating element. The material is gravity fed from the hopper into cylinders in the rotor in one section of its rotational plane. The material is discharged downward by air pressure from these cylinders at the point opposite of its rotation. Proper volume and pressure for material delivery down the hose are provided by the additional air that is introduced into the outlet neck.

The mixture is introduced into the delivery hose via a metering device, such as a feed wheel. Compressed air is added to the gun and the mixture is carried through the delivery hose to the nozzle. The nozzle is fitted inside with a perforated water ring, through which water and admixtures are introduced under pressure and intimately mixed with other ingredients as they go through the nozzle. The concrete is propelled from the nozzle at high velocity onto the receiving surface.

These guns can produce 0.8–1.5 m^3/hr (1.0–2.0 yd^3/hr) of mixture and are used primarily for dry mixes, but they can be adapted for wet mixes.

With the dry mix, the water is added at the nozzle. (See Figure 12.2.) Basically, a mixture of aggregate and cement is put into the machine and conveyed with compressed air through hoses to the nozzle where the water is added. As stated earlier, the water–cement ratio is a major factor in determining the quality of the shotcrete. Since some coarse aggregate is lost due to rebound, the relative cement content increases. Adding water at the nozzle, the water–cement ratio is controlled by the nozzleman. Although there is no way to accurately determine the water–cement ratio, it is generally below 0.5. The water–cement ratio is visually judged by the dryness of the shotcrete, which is reflected by the adherence of the shotcrete and the amount of moisture seen.

The natural moisture content affects the quality of the shotcrete. If there is too little moisture, the mix is too dry, and there will be excessive dust. If

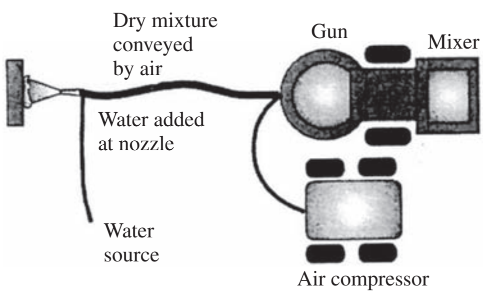


Figure 12.2 Dry-Mix Process (Courtesy of American Concrete Institute)

the dry mix has too much natural moisture, the amount of shotcrete passing through the nozzle decreases substantially because of buildup. Also, the system, pumps, hoses, valves, and so on, can become encrusted and blocked. Because of the natural moisture content of the aggregate, generally between 3 and 6%, no water should be added until the mix has reached the nozzle. The moisture content can be increased by adding water prior to entering the spraying machine by using equipment feeding devices or a special prewetting nozzle. Typically, an accelerator is the only admixture used in dry mixes. The nozzleman adjusts the amount water added until the shotcrete has a slight sheen with occasional dry spots.

In the dry-mix process, cement and damp aggregate are thoroughly mixed or premixed and prebagged cement and aggregate are fed through a premoisturizer. The cement–aggregate mixture is then fed to the gun. The mixture is introduced into delivery hose via a metering devise, such as a feed wheel. Compressed air is added at the gun, and the mixture is carried through the delivery hose to the nozzle. The ID of the material hose should be three times the size of the largest aggregate, and the air hoses should be able to withstand at least twice the operating pressure; typical air pressure is 700 kPa (100 psi) for dry mix (Krause, 2002). The nozzle is fitted inside with a perforated water ring, through which water and admixtures are introduced under pressure and intimately mixed with other ingredients as they go through the nozzle. The concrete is propelled from the nozzle at high velocity onto the receiving surface.

The operator should introduce compressed air into the hose, and then mixed material is added slowly as ordered by the nozzleman. When preparing the gun, the gun should be aimed away from the surface to be shotcreted to avoid contaminating the surface before the proper consistency is reached. To achieve quality shotcreting and reductions and variations in the shotcrete consistency, water pressure sufficiently greater than the air pressure should be available at the nozzle. The nozzleman will not be able to adjust the water fast enough, and the operation should be stopped if the delivery of the material is pulsating. Failure to do so will result in shotcrete that is either over- or underwet. At shutdown, delivery of material should be terminated first, followed by delivery of air, being sure to not aim the gun at the shotcreting surface.

Using a dry mix generates a great deal of dust, creating a health hazard; it can be mitigated by prewetting. Being dry, it is more abrasive and causes wear and tear on the equipment, especially rubber gaskets. Dry mix has a high degree of rebound, depending on whether the rebound is vertical or overhead. The rebound will range between 10 and 35%, averaging about 20–25%, whereas in wet mix the rebound averages about 15%.

WET MIX

With a wet mix, as the name indicates, the mix arrives at the gun with the water in the mix. (See Figure 12.3.) It is sprayed using either a continuous-feed gun

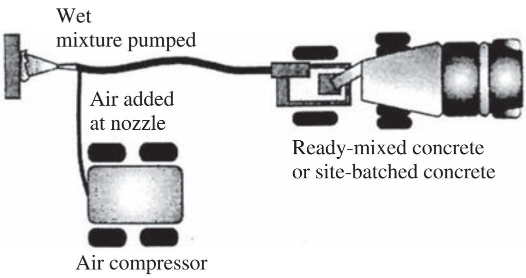


Figure 12.3 Wet-Mix Process (Courtesy of American Concrete Institute)

or a concrete pump. The concrete pump pushes the concrete through the hose for delivery to the nozzle. Concrete pumps are generally positive displacement using piston action to push the concrete. At the hopper, there is a valve that allows the mix to flow into the empty piston while pushing material out with the second piston.

Also used for wet-mix shotcrete placement is the squeeze pump. The squeeze pump, which uses mechanical rollers, squeezes the hose, much like a toothpaste tube, thereby pushing the concrete out of the hose.

Like the dry mix, the typical wet mix has 300–425 kg/m^3 (500–700 lb/yd^3) of cement and 20 mm ($^3/_4$ in.) or less of aggregate. Wet shotcrete has a greater spraying capacity while generating less dust. Because the water is in the mix, spraying under severe ground conditions with applying with a robot and using fiber reinforcement is possible. The typical batched water–cement ratio is 0.3–0.4. Quality control with wet-mix spraying is reasonably stable with low spread and low speed. It is easier to produce a uniform quality throughout the spaying process with a wet mix. The optimum air required is about 7–15 m^3/sec (10–20 yd^3/sec) at 7 bars (100 psi) for a hand-held nozzle, whereas for robot spraying, it will be about 15 m^3/sec (20 yd^3/sec). Figure 12.4 illustrates a wet nozzle. The optimum air flow is important to maximize compaction, adhesion, and compressive strength and to minimize rebound. The maximum slump is 75 mm (3 in.). If the slump is greater, the shotcrete will lose strength and not adhere, sloughing off.

At about 10–15% rebound, wet mixes generate less dust than dry mix. The reduction in dust provides a better environment. Because the addition of admixtures increases adhesion and reduces set time, thicker layers can be achieved. With the water being controlled during mixing, the management of the water cement–ratio is improved. There is also a consistently high compressive strength. Because of the aforementioned ability to increase layer thickness, production is increased, thereby reducing costs.

Because it is not prepared at the heading, wet mix has a limited conveying distance, increasing the demands on aggregate quality. Because the water is already in the mix and the setup process has begun, lengthy delays cannot be tolerated. Since the mix is wet prior to the nozzle, the cleaning is more difficult.

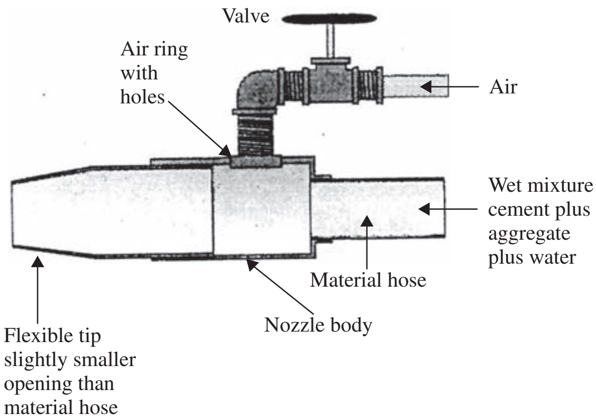


Figure 12.4 Details of Wet-Mix Nozzle (Courtesy of American Concrete Institute)

Using a robot to spray large surfaces, 60–100 m^3 (80–130 yd^3) may be placed with potentially less than 10% rebound.

Rebound is the sprayed concrete material that bounces off of the substrate, the surface being sprayed. Efforts in the form of mix design, such as increasing fine-grained materials, cement, which can reduce shrinkage, or silica, and spraying techniques are being made to minimize the rebound. The rebound is principally coarse aggregate, and because of the hard surface being sprayed, the amount of rebound is greatest with the first layer of shotcrete applied. As the shotcrete builds up, the shotcrete surface becomes softer, thus reducing rebound. Because of the greater loss of coarse aggregate with the first pass of shotcrete, there is a greater percentage of fine material in the first layer, against the rock surface. Rebound precipitates a difference in the designed mix and the mix that is adhering to the substrate. Rebound costs money because of the loss of materials and the additional time required to develop the required shotcrete thickness. The principal conditions that effect rebound are the nature of the aggregates, substrate surface quality, mix design, nozzle pressure/velocity, layer thickness, and skill of the nozzleman.

The shotcrete is compacted by the force of subsequent layers of shotcrete being applied. The sprayed shotcrete adheres to the rock with a bonding strength of 0–3 MPa (0–0.5 psi). The shotcrete completely encases and insulates the rock surface with fines being pushed into cracks and joints. The shotcrete mortar and fines that are pushed into the rock joints and cracks provide a wedging effect that may help to prevent any movement of small wedges or blocks, thus assisting the rock to be self-supporting. The tensile strength is assumed to be zero and thus, without reinforcement, the shotcrete cannot carry a tensile load. Unreinforced sprayed concrete will not be able to carry tensile loads because of cracks. The design mix should minimize the shrinkage. Low shrinkage facilitates adhesion, reduces cracking, and improves durability. The principal factor in the mix design that affects shrinkage is the amount of cement. The concrete should have the minimum amount of cement possible. A low water–cement ratio, that is, less than 0.5, preferably near 0.4, is important. For hydration, a water–cement ratio of 0.25 is sufficient. However, additional water is needed for the concrete to be workable. The water required above the hydration requirement of 0.25 water–cement ratio for workability can be reduced by using admixtures. There are various admixtures on the market that can be used to increase workability/pumpabilty with lower water content. To maintain a low water–cement ratio yet provide workability, a polymer is added. The polymer acts as a partial water reducer, thus allowing a higher slump without an increase in the water–cement ratio.

Cement, aggregates, and admixtures (except accelerators) are thoroughly mixed. The mixture is fed into the gun and propelled through the delivery hose to the nozzle by compressed air or pneumatic or mechanical pumping. Air is injected at the nozzle to disperse the stream of concrete and generate enough velocity for shotcrete placement.

In the pneumatic-feed equipment, the premixed mortar or concrete is conveyed from the gun through the delivery hose to the nozzle by slugs of compressed air. At the nozzle, additional air may be added if needed to increase the velocity and improve the gunning pattern. This equipment can handle mixtures of a consistency that is suitable for general shotcrete construction, using mixtures containing up to 20 mm (3/4 in.) aggregate. Guns with a dual mixing chamber and a two-way valve allow mixing of materials and continuous-flow operation.

In positive-displacement equipment, the concrete is pumped or otherwise forced through the delivery hose without the use of compressed air. Air is injected at the nozzle to disperse the stream of concrete and impart the velocity necessary for concrete placement. Positive-displacement delivery equipment requires a wetter mixture than pneumatic-feed equipment, and the velocity of the shotcrete being applied is lower. It is difficult to apply shotcrete to vertical and overhead surfaces by this method unless a suitable accelerator is used. This equipment can also satisfactorily shoot material containing 20 mm (3/4 in.) aggregate.

Most shotcrete is batched and mixed in the field using portable mixing equipment or delivered in mixer trucks from local plants. Mixing equipment for shotcrete is of the batch or continuous type. In the continuous type, individual ingredients are fed to a mixer screw by means of variable-speed augers, belt-feed systems, or a combination of both. A hopper is sometimes used in high-production units of both types to collect and feed the mixture as required. Water-metering systems are also available to redampen the mixture. Batching and mixing equipment must be capable of maintaining an adequate and continuous flow of homogeneous material. Batching by mass is preferred and will normally be required. Water may be batched by mass or volume. Many small shotcrete jobs have a low production rate and are in isolated locations, and mixing is often done by a small drum mixer at the job site. Prior to pumping, the pump should be inspected in accordance with the manufacturer's recommended procedure. The delivery hose must be checked and lubricated with either a water and cement mixture or prepackaged lubricant. The lubricant will be wasted, because it has only lubricating value, that is, no structural value. Controlled by the nozzleman, the compressor operator charges the hose, monitors and controls the pump pressure and feed hopper, and watches the nozzleman for signals. Should the pump or hose in a hydraulic pump become plugged, the pump operator must relieve the pressure in the line by reversing the pump so that it is emptying the line at the material source instead of at the nozzle. Since it is the pump operator who monitors the pump pressure, no one should try to open the hose until the operator informs them that the pressure has been relieved. A joint or a coupling may burst, spilling material and potentially injuring someone. The nozzleman should leave a little air flowing in the nozzle when the pump is down to avoid plugging the nozzle with paste.

ADDITIVES

As stated earlier, the accelerator is the most important additive and generally the only additive in a dry mix. The proportion added is 2–6% of the cement by weight depending on the application. The accelerator's importance is it reduces setting time. With quick setting and early strength, the nozzleman can apply subsequent layers of thicker shotcrete and do it sooner. This in turn increases productivity, not only reducing cost but also providing ground support sooner. The accelerator will cause a reduction in the 28-day strength. The strength reduction can be more than 35% in 28 days, as opposed to what it would be for nonaccelerated concrete. However, when designing the mix, this can be compensated for. The accelerator can be in liquid or powder form. The powder can be added while feeding the spraying machine, but adding powder by hand makes it unlikely that one will get exact proportions; therefore, there is no guaranty that the shotcrete is receiving the optimum dosage. Generally, the mix is overdosed, causing the shotcrete to be very unevenly applied. Therefore, manual dosing is not recommended and generally not done.

When liquid accelerator is used, it is added to the mixing water and the nozzle. The liquid accelerator also avoids the caustic components of dust caused by a dry accelerator. Flash set is prevented when dosing is done at the nozzle.

Microsilica (silica fume), which is commonly referred to as "fly ash," increases the density and compressive strength of concrete. It also improves bonding, which in turn allows thicker layers. Fly ash can reduce rebound by 50%. Fly ash is a fine, glasslike powder recovered from gases created by coal-fired electric power generation. Fly ash is an inexpensive replacement for the portland cement used in shotcrete. It improves strength, reduces shrinkage, and facilitates pumping. Improved workability means less water is needed, thus lowering the water–cement ratio and resulting in less segregation of the mixture. Although fly ash cement is less dense than portland cement, the produced concrete is denser.

REINFORCEMENT

The principal reason for the reinforcement of shotcrete is to provide ductile support. Shotcrete covers the uneven surfaces that can have loads and deformations, and the shotcrete must distribute the load. The highest possible fracture energy is the best margin of safety in the sprayed concrete layer.

The typical options for shotcrete reinforcement are fiber reinforcement, wire mesh, or steel reinforcing bars. The FRS contains discrete discontinuous fibers approximately 12–75 mm (0.5–3 in.) long that are added to the shotcrete mix and pneumatically projected at high velocity onto the surface. Fiber reinforcement can be steel, polypropylene, or fiber glass.

The relationship of postcrack capacity to strength at first crack is toughness. Toughness depends on the quantity of fibers and their effect on strength.

Steel Fibers

Steel fiber-reinforced shotcrete (SFRS) is with steel fibers added. It has higher tensile strength and impact resistance than unreinforced shotcrete and can be used in wet- or dry-mix shotcrete. In addition, steel fibers improve the crack resistance (or ductility) capacity and toughness of the shotcrete. Whereas rebar is used to increase the tensile strength of concrete in a specific direction, fibers are multidirectional. Although fibers provide some toughness in multiple directions, shotcrete generally fails due to the lack of tensile strength, possibly started by loss of adhesion. Fiber-reinforced shotcrete is less expensive than installing welded wire fabric or hand-tied rebar, while still increasing the tensile strength many times. The shape, dimension, and length of fiber are important. A thin and short fiber, for example, short hair-shaped glass fiber, is only effective the first hours after pouring the concrete by reducing cracking while the shotcrete is developing strength, but it will not increase the concrete tensile strength. Longer fibers, whether steel or fiberglass, that are generally sized 1 mm (0.04 in.) in diameter and 45 mm (1.75 in.) long will increase the concrete tensile strength; steel will provide the most strength but is subject to corrosion.

Steel fibers provide small increases in flexure and improved load-carrying ability postcracks. They add little to the compressive strength of the shotcrete but can increase the ductility by 50–200 times overt nonreinforced shotcrete. The fibers are added to the mix at about 1-2% by volume, and the fiber is typically 20–45 mm (0.75–1.75 in.) long. There is a hazard of injury from steel fibers that are in the rebound. The maximum-size aggregate when using fibers is about 8–10 mm (5/16–3/8 in.).

Synthetic Fibers

Collated fibrillated polypropylene (CFP) fibers are used in shotcrete for the same purpose as steel fibers. They lack the strength of steel but are resistant to corrosion and provide control of thermal and drying shrinkage cracking. There is less hazard of injury from fibers that are in the rebound than steel fibers.

Although not as ductile as steel fibers, glass fiber is inexpensive and corrosion proof. Spun basalt is stronger and less expensive than glass but historically has not resisted the alkaline environment of portland cement well enough to be used as direct reinforcement.

Welded Wire Fabric

Wire fabric helps to prevent or mitigate delamination or debonding of shotcrete. To resist movement of the fabric during shotcreting and reduce the number of required fabric layers, small rebar may be added to the fabric. If made of quality steel with shotcrete thicknesses of 100–150 mm (4–6 in.), wire mesh can provide an improvement in ductility as with fibers. However, cold-drawn wire is too brittle to achieve this. Wire spacing should be a minimum of 100 mm (4 in.) on center, preferably 150–200 mm (6–8 in.). Fabric should be tied tightly, similar to bar reinforcement, to minimize vibration. If the wire moves or vibrates during shotcreting, then the shotcrete may not bond to the wire fabric. Fabric sheets should be lapped one and a half spaces in all directions. When more than one layer of wire fabric is required, the first layer is covered with shotcrete prior to placing the next layer, with ties extending from the first layer to the next. A minimum of one layer of fabric is used for each 75 mm (3 in.) of shotcrete.

Reinforcing Bar

Rebar must be sized and positioned to reduce the amount of labor required and to minimize interference with shotcreting. Generally, the maximum-size bar should be no more than No. 5. Larger rebar may be used, but it may require more time and labor and, more importantly, may cause greater difficulty when encasing them with shotcrete. When possible, lapped bars should be separated by at least three bar diameters. For a shotcrete section 200 mm (8 in.) or less, one layer of reinforcement is generally sufficient. Thicker sections require several layers of bars, with the outer layer spaced to allow easy access to the inner layers. Because of the difficulties of adequately shotcreting behind rebar, the rebar spacing should not be less than 150 mm (6 in.) and whenever possible the first layer of rebar should be shotcreted before the second mat is installed. To prevent vibration caused by the force of the sprayed concrete and prevent sagging of the bars, bars should be rigidly tied with 16-gage wire. Tie wires should be free of large knots that can cause voids and should be bent flat in the plane of the mesh.

APPLICATION

The most important ingredient in quality shotcreting is the skill of the nozzleman. One must be meticulous when selecting the nozzleman; he or she must be well trained and have excellent experience applying shotcrete.

Techniques for shooting shotcrete will vary, depending on whether it is a dry or wet system, the location (i.e., overhead or vertical), the thickness of the shotcrete required, the type of surface to be sprayed, and the reinforcement.

Since the nozzleman is so important, it is apparent that good nozzling techniques are required. The mix fans out as it leaves the nozzle; this means that the center of the spray has the shortest distance to travel, therefore having the greatest velocity when it hits the surface. As mentioned earlier, the larger aggregate bounces off, until the paste builds up enough to provide a soft substrate, reducing the rebound. The impact of the shotcrete spray compresses the shotcrete, increasing strength. The part of the indirect spray that hits the surface at an angle and with less force has more ricochet than the direct spray, or it can overspray. Overspray occurs when the material is carried by the spray airstream but is not placed on the point of application. This overspray has less cement content and is not well compacted by the impact of the high-velocity stream, resulting in an area of low-strength material. If the overspray is encased in the shotcrete, a sand pocket develops. Overspray is the result of inappropriate nozzle technique. Overspray consists of both coarse and fine aggregate and a little cement paste, and can land on adjacent surfaces or reinforcement, contaminating them. Although there is enough paste in the cementitious material to cause the rebound to be sticky, there is not enough to fill the voids between the aggregate. This results in porous and weak shotcrete.

Overspray and rebound can be controlled by shooting perpendicular to the surface and cleaning the overspray as it occurs. Also, areas that collect overspray, such as corners, should be sprayed first. The nozzle should be at right angles (90°) to the surface being shotcreted to ensure optimum compaction and fiber orientation. Shooting at an incorrect angle will increase rebound and overspray and reduce the compaction. The only time when this can be varied is when shotcreting over welded wire fabric or rebar. To ensure the concrete fills the area behind the reinforcement, the nozzle angle has to be adjusted to achieve this. The optimum distance from the substrate surface to be shotcreted and the nozzle will be about 1 m (3 ft). If the nozzle is closer than 1 m, the intense pressure of the shotcrete stream will cause an increase in rebound. The reduced pressure due to being farther than 1 m will cause an increase in rebound, and the shotcrete will be less compacted and thus low in strength.

Because the water is controlled, dry-mix shotcreting can be a little more demanding than wet mix. The nozzleman observes the surface for indications of the correct amount of moisture and adjusts the water accordingly. He or she looks for a slight sheen with few or no dry spots. If the shotcrete is dark and has a sandy appearance, it is too dry. The mix is too wet when the shotcrete sags or has a running, wet appearance. The nozzleman must be prepared to turn the gun from the surface should anything happen to a consistent flow of mix until the flow can be restored.

When shotcreting, the nozzle should be moved in a steady or circular overlapping pattern. This will permit the material to mix, providing a smooth surface. If the spray is directed to one spot for too long, rebound can occur and there will be difficulty controlling the thickness. Anyone who has spray painted knows that if the spray is too close or too far away or remains on one spot too long instead of moving steadily, the paint will be too thick at the concentrated spot and too thin adjacent to it. Also, where the paint is too thick it will run—it will not adhere. When sprayed concrete is applied too long at one location, the area will be well compacted with adjacent areas not well compacted.

Prior to shotcreting the surface should be cleaned using compressed air and water to provide a bondable surface that is free of materials that will prevent or reduce bonding. As with pumping concrete, cement slurry should be run through the hose to lubricate it for the initial shotcrete and to prevent the paste from the first shotcrete pump from being used to lubricate the hose. It is recommended that, prior to applying the shotcrete layers, a thin layer be applied to the planned spray air to minimize any defects that can be caused by overspray. Generally, when shotcreting drill blast rounds, timing is not a problem, but in larger areas it is important that the surface to be sprayed is not so large that the first layers sprayed do not have sufficient initial (*initial set* is a term in concrete referring to when the concrete first hardens) set prior to being covered with the next layer. Also, the batch of shotcrete should be checked to ensure that it has been properly mixed. The use of bonding agents is not recommended, because, if the bonding agent cures prior to shotcreting, it can act as a bond breaker. A good bond in shotcrete is the result of high-velocity impact.

When spraying a vertical surface, such as the rib, the sprayed concrete should be applied from the bottom up. This prevents spraying on top of overspray or locking in rebound. The nozzleman generally works horizontally along the wall, shooting widths about 1–2 m until initial set has taken place. If the shotcrete is stacked too high for its level of setting up, sags and voids will occur. When the dry-mix process is used to shoot a vertical surface, typically the vertical layering method is used. When using vertical layering, multiple thin, vertical layers of shotcrete are placed on the previous layer. It is supported by adhesion to the previous layer. The nozzle should be at 90° to the surface and directed slightly upward, especially if steel is being encased. The thickness of a layer applied vertically depends on the roughness of the substrate and the stiffness of the mix; the smoother the substrate is, the thinner the layer that can be applied.

When applying shotcrete overhead, the layers must be very thin to prevent sagging or shotcrete from dropping out. Generally, accelerators are added to permit thicker layers, and higher air pressures can help. As with vertical shotcreting, the nozzle should be pointed perpendicular to the surface.

When shotcreting reinforcing bar, the nozzle should be held close to the reinforcement at a slight angle from perpendicular to avoid shadows and voids behind the rebar. (See Figure 12.5.) The mix should be slightly wetter than

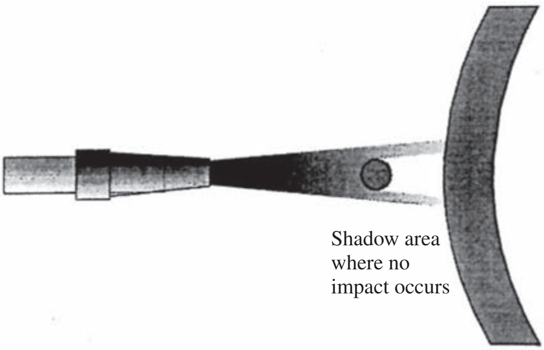


Figure 12.5 Resulting Shadow Behind Reinforcing Bars Where No Impact Occurs (Courtesy of American Concrete Institute)

normal, without being so wet as the cause unwanted sloughing. A slump of 75–100 mm (3–4 in.) will require less impact to force material to flow to the backside of an obstacle. The nozzleman can tell whether there is sufficient impact velocity and plasticity when the face of the steel will glisten and remain clean. Rebar deformations will be visible and a ridge up at the back, rather than a valley, will develop behind the bar.

When encasing bars that are larger than No. 5, a higher slump is required. The shadow can be reduced or eliminated by directing the shotcrete spray at a slight angle from both sides to force the material behind the bar. Sometimes the volume and nozzle stream should be directed to permit the material to hit directly behind the reinforcement; this will reduce the shadow area. It may be necessary to reduce the air volume so the nozzle can be held closer to the target.

If reinforcing steel is not properly encased, there will be adverse effects on the structure and the potential for corrosion of the rebar that is not properly encased. (See Figure 12.6.) When shotcrete is sprayed against rebar, a shadow occurs behind the steel caused by the spray that the bar blocks. The size of the shadow increases if there is an increase in the size of the rebar or if the gun is held closer to the steel. The shotcrete in the shadow area behind the steel is not compacted because the stream of material does not impact the shadow area. Thus, voids can develop. The shotcrete must have good plasticity and sufficient impact velocity to force the shotcrete to flow around the bar and spread into the shadow area, with flow pressure causing some compaction. If a mix is too stiff or does not have sufficient impact velocity, the material will build up on the face of the rebar, creating large obstacles, thus forming a void behind the rebar. The stiff mix, to some degree, can be overcome by a high-impact velocity or holding the nozzle closer to cause the material to flow around the bar. Reducing the distance to 0.6–1.2 m (2–4 ft) will increase the velocity. Also, adding more at the gun can increase impact velocity. A wet shotcrete mix should have a slump of 50–75 mm (2–3 in.) to properly encase rebar. The

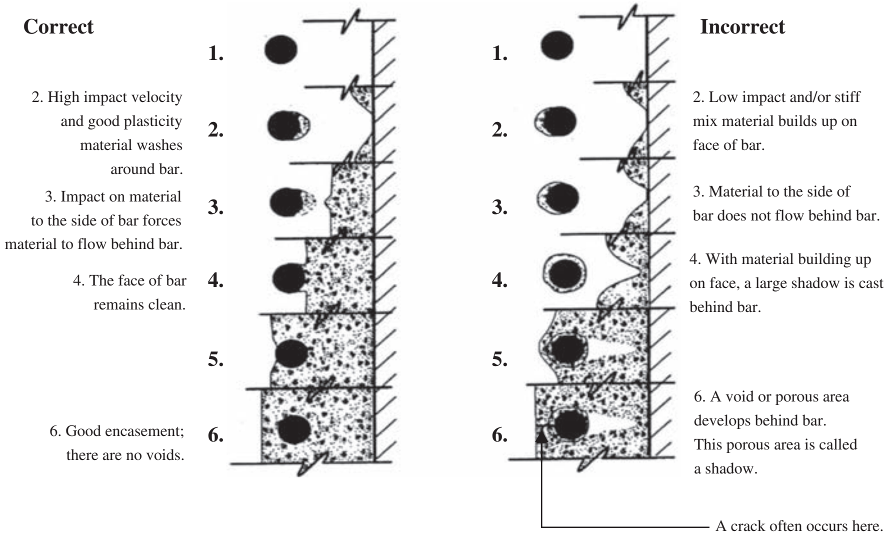


Figure 12.6 Correct and Incorrect Way to Encase Reinforcing Steel (Courtesy of American Concrete Institute)

key to good encasement is proper plasticity. It requires less impact velocity to force flow of material around to the backside of a rebar if a higher plasticity mix with a 75–100-mm- (3–4-in.-) slump is used.

In summary, the way to ensure good-quality shotcrete is by controlling the shotcreting crew. When selecting materials, the crew must confirm the aggregate gradations, the moisture content of the aggregate, and the source of cement and ensure the materials are properly mixed in the correct proportions. The crew must maintain the equipment. Shooting with damaged hoses can cause inconsistent flow, resulting in poor quality. The substrate should be properly prepared; grease oil, dirt, overspray, and rebound should be removed. As stated earlier, the ambient temperature and the temperature of the mix must be, at placement and for 24 hr after placement, above 5°C (40°F). In underground applications, this is generally not a problem. Prior to placement, the materials have to be pretreated for temperature and moisture content, and the cement and aggregate must be thoroughly premixed.

The reinforcing steel must be properly placed and securely tied to prevent the steel from vibrating excessively. Vibrating steel will reduce the bond between the steel and the shotcrete. The quality of the shotcrete crew and the effectiveness of the nozzleman can make the difference between a good and a bad product. The nozzleman must control the impact velocity to attain optimum compaction of the material by maintaining the air volume and the nozzle distance. He or she must control the nozzle angle and make plans on how to control rebound and overspray. Sometimes it may be appropriate to use an air lance to remove rebound and, between layers, use a broom or scrape the applied shotcrete to create a better bond.

To conclude, shotcrete provides not only surface protection but also structural support when cured and is a valuable tool in tunneling. Unlike poured concrete, it is not forgiving. That is, with poured concrete, a change in the variables may cause some changes in the product, and it may not achieve the specifications, but it may be "good enough" or may have to be removed and replaced. However, because it is contained in a form and above ground and is supported by some type of false work or bracing, it probably does not result in any danger to the workers. This is generally not the case with shotcrete. If shotcrete is not properly applied, there is a danger to personnel and equipment.

There are four principal variables that will affect the quality of shotcrete: surface condition, layer thickness, shotcrete mix (especially water content and additives), and operator competency. If any of the variables are not what they should be, the shotcrete may be adversely affected. The most dangerous effect is loss of adhesion; that is, the shotcrete does not adhere to the surface, causing considerable danger. The lack of adherence may not be obvious, and the shotcrete may remain in place for a period of time before it separates from the substrate and falls. There have been miners killed by shotcrete failing and falling out.

To avoid the fall of shotcrete, the mix should be consistent and the surface should be clean. The surface is prepared by scaling any loose rock and using high-pressure water to clean the substrate. Not only does this apply to the rock surface, but also between each application the substrate must be pressure washed prior to application of subsequent shotcrete layers. After washing, aggregate should be exposed at the interface. This is an indirect indication that all laitance and other surface defects affecting bond have been removed. If there is water flowing in the area to be shotcreted, the water has to be panned away. Generally, a small tube is placed at the fissure or crack. Then the remainder of the crack and around the tube is packed with cement or covered with some type of polymer drainage fabric, and the water is panned away from the shotcrete. The water has to be diverted, not stopped, because of the potential head buildup.

The importance of the operator's competence cannot be overstated, and nozzle operators should be certified to perform the function. In particular, the qualification should include shooting panels with a thickness equivalent to the application thickness proposed for the application. This is especially critical for overhead applications, as noted below. In addition to controlling the water content when using a dry mix, the operator controls the layer thickness of either mixes, the distance from the nozzle to the surface, and the movement of the nozzle. In addition, some recommend having the operator shotcrete panels as close to the conditions that will be encountered underground to be cored and tested. The crown of the tunnel is the most important; overhead panels are good for mimicking the crown of the tunnel.

It should be realized that the thicker the layer is, the greater the required adhesion. For this reason, many recommend that layer thickness not exceed 75–100 mm (3–4 in.). In cases in which the operator is exceptionally qualified or there is support, such as welded wire fabric or rebar, the thickness can be increased. Also, the geometric shape is a factor. If the crown arch has a small radius or is narrow, affecting the load being transferred to the walls to any extent, the thickness is not as critical. However, for flat surfaces or locations where the inclination of bonding surfaces in overexcavation areas is 30° or less, support of shotcrete by bolts/dowels and welded wire fabric (WWF) or straps/channels may be necessary to prevent fallout. Finally, vertical or invert shotcrete allows relaxing the above recommendations. No one should be allowed under the shotcrete for a minimum of 4 hr (longer is better), and many recommend 8 hr between applications. Therefore, work must be planned and sequenced so as to minimize delays to production while waiting for the shotcrete to set.

After the shotcrete has set, it is a good idea to sound the shotcrete with a hammer to locate any voids. Also, when the thickness reaches 500 mm (18 in.) and is not reinforced, one should consider installing bolts and straps to provide additional stability to the shotcrete. If the rock surface is rough and jagged, with holes where wedges have fallen out, the additional surface area may negate the need for the additional support.

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13 New Austrian Tunneling Method and Norwegian Method of Tunneling

This chapter discusses two methods by which selection is based on the type of ground being mined: the new Austrian tunneling method (NATM) and the Norwegian method of tunneling (NMT). Both are considered design-as-you-monitor, whether by instrument or observation, in that the ground support is flexible and allows changing to the optimum ground support, based on the conditions encountered as the tunnel advances.

The NATM can be defined as a tunneling method that is observational and adaptable, optimizing the ground–structure interaction that is fully integrated with operational, contractual, and technical considerations. The rock and soil formation surrounding a tunnel are integrated into an overall support structure. That is, the geologic stresses of the surrounding rock mass or soil are used to provide stabilization and act as the main load-bearing components helping to support the tunnel. However, some argue over whether this is a tunneling method or a philosophical approach.

The NATM starts with exploration and data collection, from which several design alternatives are developed for each of the anticipated soil conditions. During construction, the actual conditions are compared to the anticipated conditions. The ground is monitored to determine if the amount of ground movement is acceptable for the actual conditions. If indicated by observation and monitoring that the ground support should be changed, another support system that had been designed for the particular ground anticipated is implemented. The supports should be optimized according to the permissible deformations. Better suited and more often used in soil, the design philosophy that includes the strength of the ground through which the tunnel is being mined and is used as much as possible to assist with the support. This is attained by permitting controlled movement or deformation of the ground but not to the extent that the ground is weakened. The excavation should maintain a circular shape.

The initial support used matches the load–deformation, or flexibility, to the ground conditions, and the installation occurs at an appropriate time with regard to ground movement. That is, the support should not be too stiff. To monitor the deformation of the installed support and determine any changes in the approach to the design and excavation, instrumentation is installed. A thin sprayed concrete lining is used to increase the stability of support girders. If steel reinforcement and arches should be used, the liner should not be thickened. Closing of the invert at the appropriate time is required.

The tunnel should be sequentially supported and excavated using varied sequences, and a full face should be used when conditions permit it. The initial support consists of shotcrete, with or without fiber reinforcement, welded wire mesh, arches or lattice girders, and spilling or soil reinforcement, such as soil nails; the final liner is usually cast-in-place concrete.

The principal advantage with this method is flexibility; that is, the tunnel can be any shape or diameter. The lining thickness can vary, and one can respond rapidly to changes in ground conditions. Because it is not dependent on major equipment, a project will have a short lead time, requiring little mobilization and low capital costs.

The geomechanical behavior of the material must be taken into consideration during planning. Deformation and adverse states of stress must be avoided by installation of the appropriate support at the appropriate time.

An important factor is the relationship between the ground and the liner. Figure 13.1 illustrates the interaction between the ground loading and the liner. As the crown is loaded, the sides of the liner push outward, thereby causing the ground and liner to interact to support the ground. Note how the liner bulges out, pushing against surrounding earth, thus preventing the crown from deflecting. Completion of the circle by closing the tunnel invert gives the liner a ringlike structure, with the static properties of a tube or circle. The net effect is the entire lining is in compression.

The main design principle is to take advantage of the capacity of the ground, whether soil or rock, that is being excavated by permitting the rock/soil mass to deform in a controlled way. The ground characteristic curve in Figure 13.2 illustrates the relationship between time, ground loading, and lining installation. As the ground converges, represented by the ground line curve, it reaches a point of equilibrium where the lining can be installed having to support a

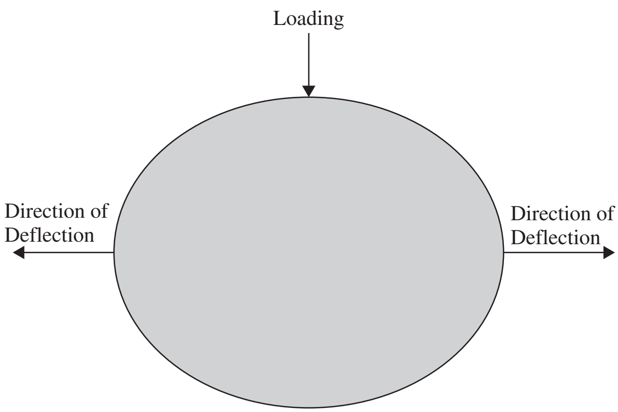

Figure 13.1 Ground–Lining Interaction

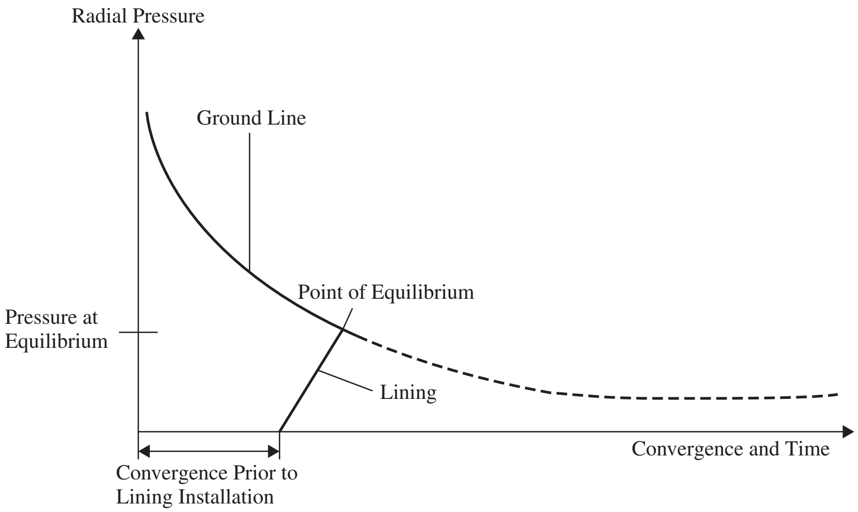

Figure 13.2 Ground Characteristic Curve(Adapted from Safety of the New Austrian Tunnelling Method (NATM) Tunnels, Health and Safety Executive, 1996)

smaller load. After the lining is installed, the ground continues to converge until the lining is loaded. At this point, the convergence stops.

This requires knowledge of the relationship between ground deformation and load, whereby control measures and constant checks on the optimization of the supports must be performed.

As the word *sequential* implies, the excavation and support are done in sequence; that is, the heading is divided into manageable pieces. Each piece is excavated and supported before beginning the next piece. The options are to divide the heading into multiple drifting, but as few as possible. For example, in Figure 13.3, there are six headings, labeled A through F. This represents

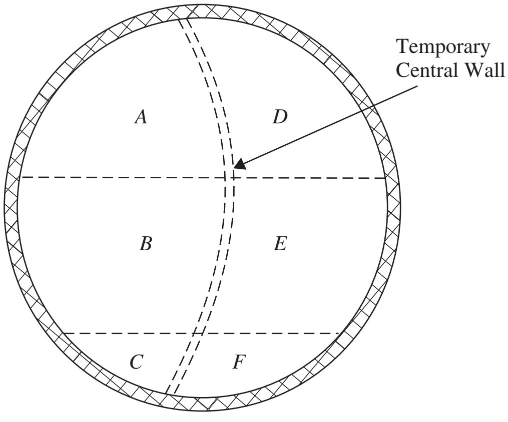

Figure 13.3 Multiple Drifting (Adapted from Safety of the New Austrian Tunnelling Method (NATM) Tunnels, Health and Safety Executive, 1996)

the cutting of the face into six different drifts. In the example, the A section is excavated and supported first. Part of the support is to install a temporary central wall, as depicted in the illustration.

The drifts are removed alphabetically. The sections can be referred to as "side gallery" and "enlargement," with A being the crown section side gallery, the bench section, B, side gallery, and C being the invert section side gallery. As the side gallery is excavated, the temporary wall is extended so that it is installed from the crown to the invert. The remaining sections are the enlargement sections: D is the crown section enlargement, E is the bench section enlargement, and F is the invert section enlargement. Generally, the temporary wall will be removed when the full ring has been completed.

Figure 13.4 is an illustration of the relationship between the ground and lining. Points *A*-*C* indicate that, as the excavation proceeds, the ground moves into the tunnel and the radial pressure that is required to achieve equilibrium reduces as the ground strength is mobilized.

Point *B* represents the completion of the lining whereby the ground load causes the inward movement of the lining. This leads to the point of equilibrium when the radial pressure required for equilibrium is provided by the lining at point *C*. Point *D* represents the end of the convergence of the lining. Without the support of the lining, the convergence increases and the ground may ultimately collapse into the tunnel.

The lack of stability of the ground is key in determining the amount of face subdivision and the length of the round. One indicator of the round length is the speed with which the support can be installed. In more stable ground, the top heading may be driven, with the bench following at a distance determined by the

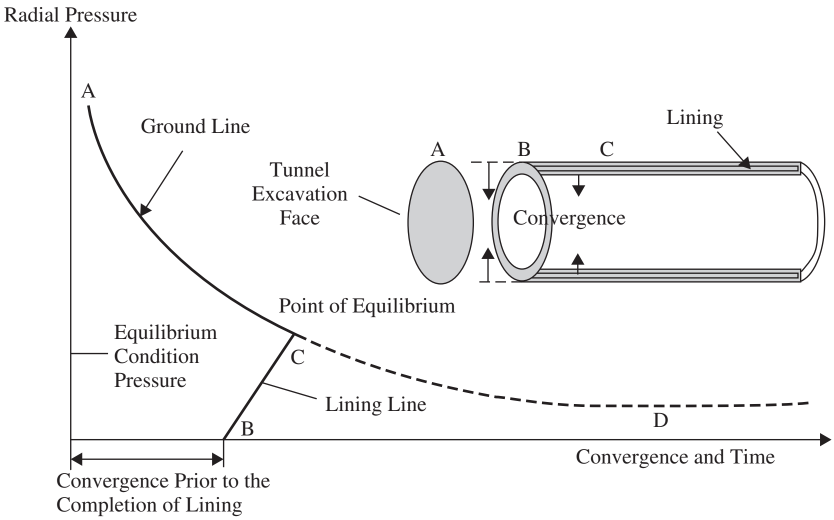

Figure 13.4 Ground–Lining Relationship (Adapted from Safety of the New Austrian Tunnelling Method (NATM) Tunnels, Health and Safety Executive, 1996)

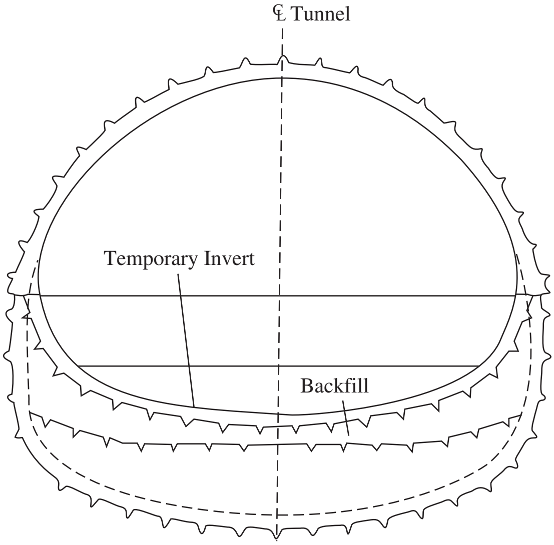

Figure 13.5 Typical Top Heading and Bench Tunnel Cross Section with Temporary Invert

ground stability. Figure 13.5 is a sketch of a potential excavation sequence. In this case, the face is subdivided into a top heading and a bench. The top heading is excavated with shotcrete, and lattice girders are installed immediately behind the excavation. The timing for support installation is generally established in the specification, subject to change once experience with the material has been gained. It is important to note that a temporary invert is installed to maintain the circle. If the invert is not closed at the appropriate time, the benefit of the structural shape of a circle being in compression is not realized. Instead, the lining is exposed to moments and other stresses, which will cause a collapse.

Note the thickness of the temporary invert is 300 mm (1 ft), on top of which backfill is deposited. The backfill serves to protect the invert from damage by the equipment working on it, provide working platform, and help to provide stabilization to the invert by compressing the invert shotcrete between the unexcavated ground and the backfill.

Once the prescribed length that the top heading leads the bench is reached, the final invert is closed. This requires that the temporary invert be excavated with the material below it to the final excavation line. The bench is excavated in relatively short lengths to minimize the length of the shotcrete lining that does not have the invert closed.

Figure 13.6 is a photograph of the final invert being closed in very weak rock. Note the short length, approximately 2 m (6 ft), being installed.

The type of support is determined by the rock class. Table 13.1 provides the definitions of rock classes and the general excavation support requirements. Although not indicated in the table, the sequential excavation method (SEM)

Figure 13.6 Installing Rebar for Final Invert

would not be the best choice for any rock class below Vb. For worst ground conditions, a TBM designed for tunneling through ground with some fluidity or tendency to flow, for example a SPB or and EPB, or other ground improvement techniques, such as compressed air, would have to be considered.

In Figure 13.5, the ground is probably rock class II. It would require rock bolts, ribs or lattice girders, and shotcrete. To continue the lattice girders to the final invert, there has to be a connection to attach the rib for the bench excavation. The footing for the lattice girder is widened to allow for the location of the interface of the temporary invert and the lattice girder extension once the bench is excavated.

Figures 13.7, 13.8, and 13.9 illustrate the stages of outer lining reinforcement, that is, the top heading crown excavation, the top heading invert excavation, and the bench excavation, respectively.

Figure 13.7 is a sketch of the position of the lattice girder at the conclusion of the top heading excavation. The lattice girder is installed with connection bars for the remaining excavation. In Figure 13.8, wire mesh attached to the connecting bar is used to reinforce the shotcrete for the top heading invert.

In Figure 13.9, the lattice girder is attached to the end of the top heading lattice girder and lapped with the connection bars.

Center walls for multisectioned headings can follow this process and then be removed when the heading is complete.

Table 13.1 Type of Support by Rock Class

Rock Class	Behavior	General Requirements for Excavation and Support
I	**Very Stable**. Massive with tight joints with large joint spacing.	Occasional spot bolting.
II	**Stable**. In general is elastic, ground resists secondary stresses without failure. Due to unfavorable orientation of discontinuities local overbreaks by gravity; insignificant deformations.	Subdivision of excavation face. Light tunnel support with shotcrete and steel ribs, rock bolts. Typical length of round at top heading 1.5–2.0 m (5.0–6.5 ft).
III	**Short Time Stable**. Development of local shear failure zones with limited disintegration of rock mass; small deformations.	Subdivision of excavation face. Immediate shotcrete sealing. Shotcrete lining with steel ribs as main support; rock bolts. Typical length of round at top heading 1.2–1.5 m (4–5 ft).
IV	**Unstable**. Development of shear failure with pronounced disintegration of rock mass; moderate deformation.	Additional subdivision of excavation face with temporary support elements. Advancing support elements; shotcrete lining with rock bolts and steel ribs. Typical length of top heading round 1.0–1.2 m (3–4 ft).
Va	**Squeezing.** Delayed, deep reaching development of shear failures and plastic zones; slow shear failure propagation; slight swelling potential; large deformations.	Additional subdivision of excavation face into small headings with systematic face support and advanced supporting elements. Heavy support by shotcrete lining with long bolts and steel ribs. Typical length of round at top heading 0.8–1.0 m (2.5–3 ft).
Vb	**Heavily Squeezing**. Rapid development of extensive shear failures and plastic zones immediately after excavation; swelling potential; very large deformation.	Additional subdivision of excavation face into small headings with systematic face support and advancing support elements. Heavy support by shotcrete lining with long rock bolts and steel ribs. Typical length of round at top heading 0.8–1.0 m (2.5–3 ft).
VIa	**Running**. Cohesionless, unraveling ground. Immediate instability of exposed ground.	Subdivision of excavation face. Due to specific ground conditions measures will comprise advancing support elements and possibly ground treatment ahead of face; face support; Shotcrete lining with steel ribs similar to rock class IV. Length of round according to ground stabilization methods.
VIb	**Flowing**. Mudflow-like flowing water, saturated ground immediately after excavation.	Additional subdivision of excavation face with temporary shotcrete support elements; face support; advancing support elements. Shotcrete lining with long rock bolts and steel ribs, similar to rock class Va. Ground treatment prior to excavation. Length of round according to pretreatment.

(*continued overleaf*)

Table 13.1 (*Continued*)

Rock Class	Behavior	General Requirements for Excavation and Support
P/Sa	**Portal/Shallow Cover Class A**. Gravel or loose sand with overburden less than 15 m (50 ft).	Subdivision of excavation face. Face support. Shotcrete lining with steel ribs as main support; rock bolts at bench. Typical length of round at top heading 0.80–1.0 m (2.5–3 ft). Jet grouting umbrella.
P/Sb	**Portal/Shallow Cover Class B**. Silt or cemented sand with overburden less than 15 m (50 ft)	Subdivision of excavation face. Face support. Shotcrete lining with steel ribs as main support; rock bolts at bench. Typical length of round at top heading 0.80–1.0 m (2.5–3 ft). Pipe roof umbrella.

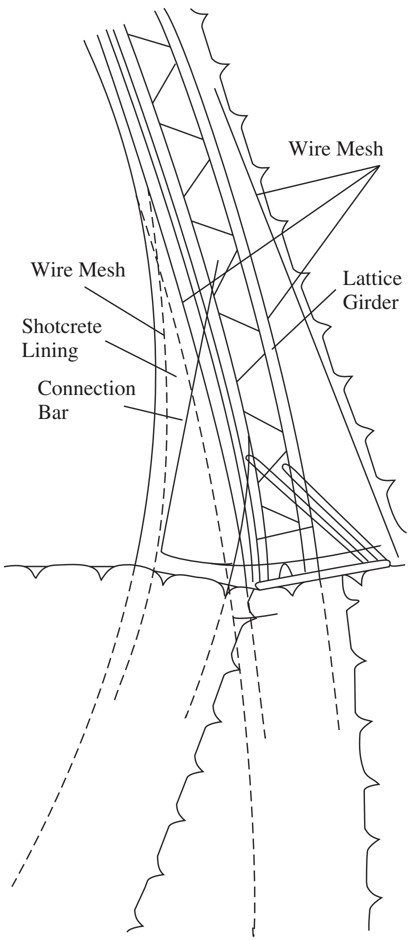

Figure 13.7 Top Heading Crown Excavation

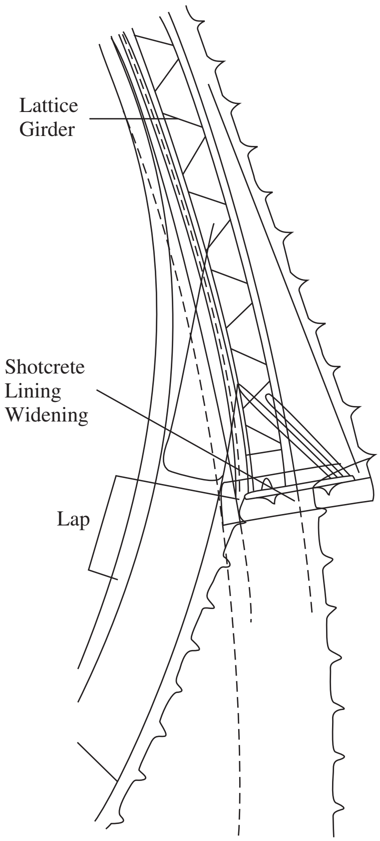

Figure 13.8 Top Heading Invert Excavation

If the ground does not have adequate stand-up time to allow the installation of the lattice girders and shotcrete, additional means may be needed to provide that time. The methods that may be employed include forepoling, spiling, freezing, and jet grouting.

Forepoling refers to the installation of dowels or plates outside the tunnel crown perimeter excavation in the direction of the drive to provide support of the crown. When gravel or boulders are encountered, steel members such as channels may be used. Forepoling may consist of plates or boards that are driven ahead of the face to provide crown support. The plates are effective in running ground in that they may be interlocked along the sides, similar to steel sheet pile. Forepoling boards are generally wood boards whose length is twice the distance between the ribs. They are positioned above the rib closest to the face and under the second rib. This provides a cantilever effect (see Figure 13.10).

Spilling involves drilling or driving pipes or rods closely together outside of the excavation line and grouting, as in Figure 13.9. The pipes are installed

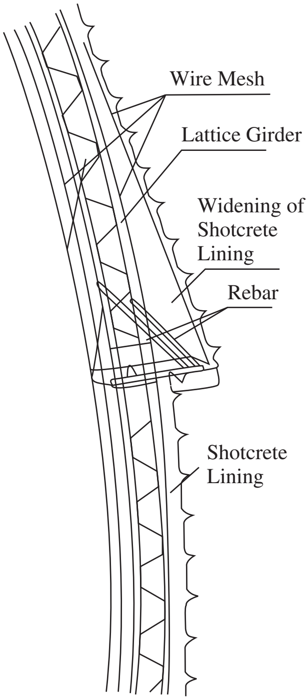

Figure 13.9 Bench Excavation Lattice Girder Installation

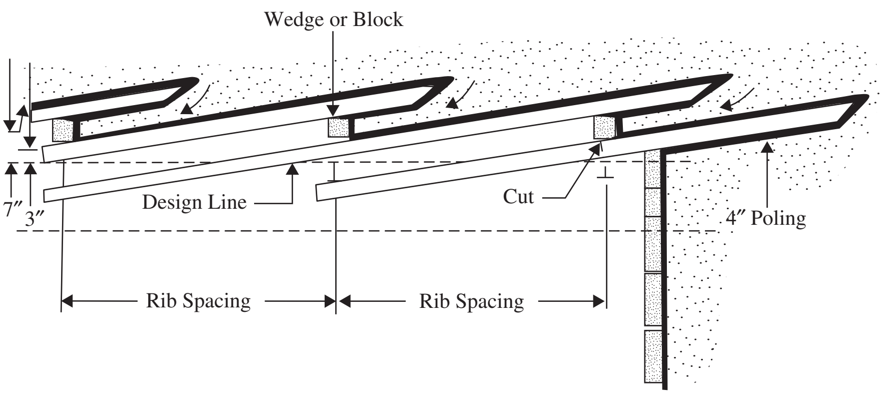

Figure 13.10 Forepoling Method (Proctor and White, 1977)

ahead of the face, and the length should be greater than a round length and as long as can be effectively installed. The pipes, being close together, form an umbrella above the tunnel supporting the crown. Because of interference with ribs or other ground support, the length of the spile is limited. The angle required to use the girders or ribs requires that the longer the spile is, the

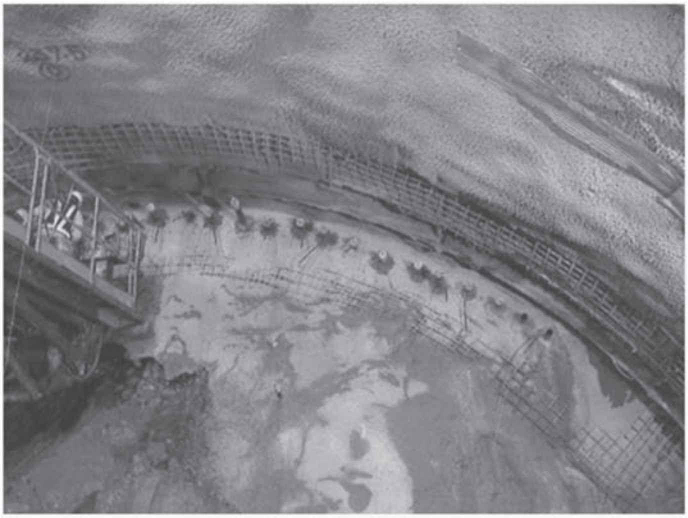

Figure 13.11 Grouting Pipe Roof

farther the spile will be from the excavation line getting a flat angle. (See Figure 13.11.)

Jet grouting operates under the same method as vertical jet grouting on the surface. For crown support, the holes are drilled above the excavation line. The jet grouting action removes mixes and replaces some of the material with cement to bind the material together. These cylinders are installed along the perimeter of the tunnel, generally to the spring line, and act as an umbrella, preventing material from falling from the crown.

When the face is hard to control, it may be stabilized with shotcrete, rock bolts/soil nails, or other methods to hold the face. In some tunnels breast boards may be used. Another method for holding the face is the use of a buttress, or an unexcavated wedge. The center of the face follows the heading some distance behind the face. Because of its mass, the buttress holds the face. As the face is advanced, the buttress is excavated, maintaining a distance behind the face. The photograph in Figure 13.12 illustrates the use of a buttress for face support.

Note that the crown area of the face has been advanced and shotcreted. In some cases it may be necessary to shotcrete the buttress also.

Both the NATM and the NMT are ideally suited for caverns, such as underground subway stations. Substantial instrumentation must be used to monitor the ground movement and lining stress. More instrumentation is needed in caverns, because the spans are larger. It is important to have an engineer monitor the instrumentation and to be prepared to make modifications to the ground support.

Because of extensive overbreak, it may be impractical from a cost and/or safety perspective to install steel sets. Therefore, use the Norwegian Method.

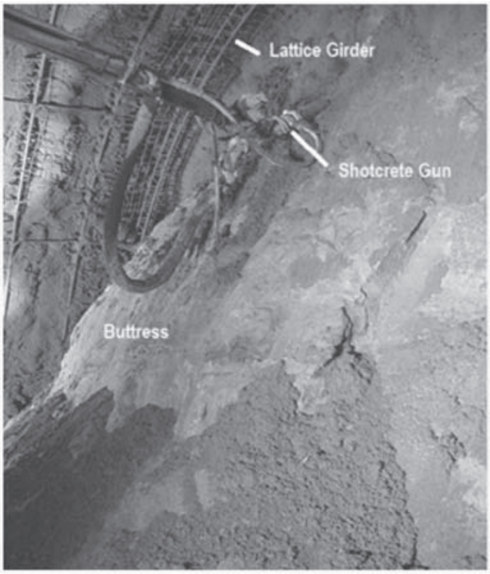

Figure 13.12 Buttress Support

It differs from NATM in that, when the tunnel is to be mined through highly jointed rock, the tunnel is advanced by drilling and blasting. Between rounds, rock bolts and SFRS are remotely installed to ensure the safety of the miners.

Whereas the NATM is better suited for soft ground and machine-excavated tunnels, the NMT is the choice for highly jointed rock that requires blasting. Generally not instrumented, the NMT uses SFRS and bolts, whereas the NATM uses lattice girders in addition to passive support. Also, the NATM frequently requires cast-in-place concrete for a final liner and a closed invert, whereas the bolts and SFRS initial ground support of NMT often remains as the final lining.

The selection of NMT is based on using the Q-system for rock mass classification; the types and rock support techniques appropriate for any given type geologic conditions, as well as the type of support, can be determined.

As discussed in earlier chapters, the methods of mining and ground support are a function of various geologic variables, including whether it is rock or soil. The NATM is probably the best design-as-you-monitor for soil, and NMT is the best choice for jointed rock.

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14 Water Handling

In this chapter, we will discuss methods for handling water during tunneling operations. Generally, the main benefit of preventing water inflow is to keep the ground from deteriorating by washing out of fines. The methods include panning, pumping, wellpoints, and freezing. Grouting, which is typically used, is discussed in a separate chapter.

The best way to handle water is to keep it out of the tunnel. This is done by forming a water barrier by grouting or freezing or pumping the water before it reaches the tunnel. Nevertheless, one must be prepared for handling water in the heading.

PANNING

Because water presents the biggest difficulty when tunneling, it is important to have knowledge of ways to control or minimize the flow. With small amounts of water, the flow can sometimes be controlled by panning. Panning involves directing the water flow to a location to be pumped or otherwise removed from the heading. Corrugated metal or plastic sheets are often used to direct the flow of water. The sheets are of a size to be handled easily by one miner; often, dimensions are approximately 750×1200 mm (30×48 in). As an example, assume there is 40 l/min (10 gpm) of water flowing down the rib in a tunnel that is being driven downhill. The flow separates, causing the water to flow in more than one direction. Although the water make is not large, if not controlled, it will become a problem in the heading. Panning sheets are mounted upstream from where the flow separates. The panning directs the water toward a sump or other location for removal from the tunnel.

PUMPING

The amount of pumping needed to prevent water from causing too much difficulty while tunneling depends on the amount of water flow to be removed and the distance it has to be pumped for disposal. Some TBMs are designed to permit them to work for high water inflows. This will reduce the amount of grouting required, but the water will still have to be removed from the heading.

In the heading, the option is pumping. This is fairly straightforward. A sump is established in which a pump is located; as the water flows into the sump, it is pumped from the heading to the surface, if the heading is close enough, or to an interim sump or tank. That is, the water may be pumped in stages if the distance to the portal and disposal requires handling it several times. Provided the water flow into the tunnel (make) is not too great, handling water by pumping is not difficult. One should check the color and content of the water for fines. Presence of a significant amount of fines introduces other problems, discussed in Chapters 2 and 3.

Generally speaking, a sump is located as close to the face a practicable. A submersible electric or pneumatic pump is placed in the sump and removes the water from the heading. The sizing of the pump will be based on the experience of the miners. In general, a pump that has the capacity of the water make should be used.

DEWATERING

Dewatering is the temporary removal of ground or surface water from a construction site to allow construction to be conducted under dry conditions. Dewatering of cofferdams, trenches, shafts, and tunnels is a common practice during construction. For shallow excavations, water is usually removed using wellpoints, drilled wells, and power-driven pumps.

In granular soils, lowering the water table can be an effective way to keep water from entering the tunnel by the use of deep-well pumping or wellpoints.

Deep Wells

Deep wells typically consist of holes drilled in the ground from the surface in which are fitted a slotted liner and an electric submersible pump. (See Figure 14.1.) By pumping water from these holes, a cone of depression, or pump drawdown, lowers the water table at the tunnel location enough to reduce the head of the water, making it easier to handle the water in the tunnel. Using deep-well pumping allows water to be extracted at reasonable depths; cost is generally the determining factor. Deep wells work best in soils with specific permeability profiles (k equals $1 \times 10^{-3} - 1 \times 10^{-5}$ m/s), and generally the amount of drawdown that a well can achieve is limited only by the size of the pump. A hydraulic gradient is formed, and water flows into the well, forming a cone of depression around the well in which there is little or no water remaining in the pore spaces of the surrounding soil. Deep wells can be installed around the perimeter or along the alignment of an underground excavation to lower the water level and maintain a safe, dry site.

Typical deep-well equipment is comprised of a casing used to prevent collapse of the hole, material falling into the well, undesirable water from entering, and good water from escaping. A screen is required at the bottom of the hole to prevent the walls from collapsing or caving into the hole, prohibit fine sand from entering and plugging the pump, and permit the inflow of water. Since

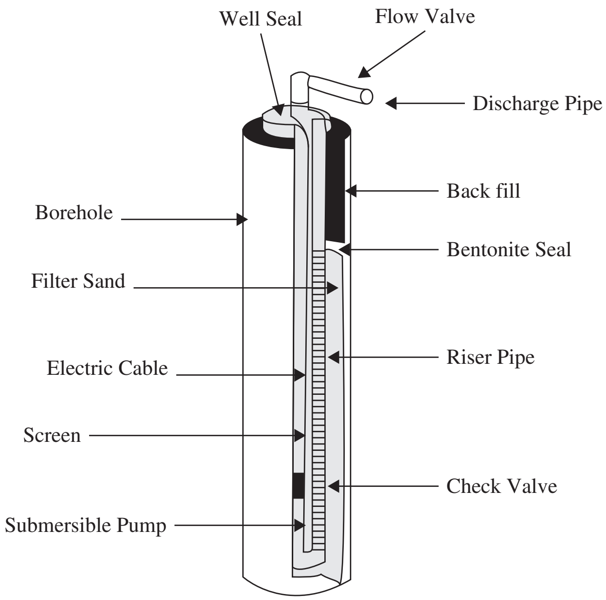

Figure 14.1 Deep-Well Details (Adapted from Griffin Dewatering)

the useful life of the well is greatly affected by the durability of the screen, the screen material is corrosion resistant. (See Figure 14.2.)

Deep wells can be more expensive to install and maintain; however, in many applications, they can be the most cost-effective choice.

Irrespective of their installation cost, in some cases deep wells are indicated because of their efficiency and their low operational cost compared to multistage wellpoints. Deep wells can be installed from the surface around the shaft or tunnel area. They will lower the water table to 30 m (100 ft) or more in one single stage, with spacing from 6 to 30 m (20 to 100 ft), depending on the soil condition. One well consists of the well itself and its casing, the well screen, the sand filter, when needed, the pump, and the discharging piping. The diameter of the well and the spacing will be determined from soil study and excavation depth according to the volume of water to be removed to achieve a dry and stable excavation bottom.

Wellpoint

Wellpoint dewatering is the most versatile and common form of dewatering for coping with the low flows from silty sands and the larger flows from coarse sands and gravels. Wellpoints are tubes that are sunk in the ground to draw water using suction pumps. The tubes are perforated and covered with screen to keep them from being fouled by sand. (See Figure 14.3.)

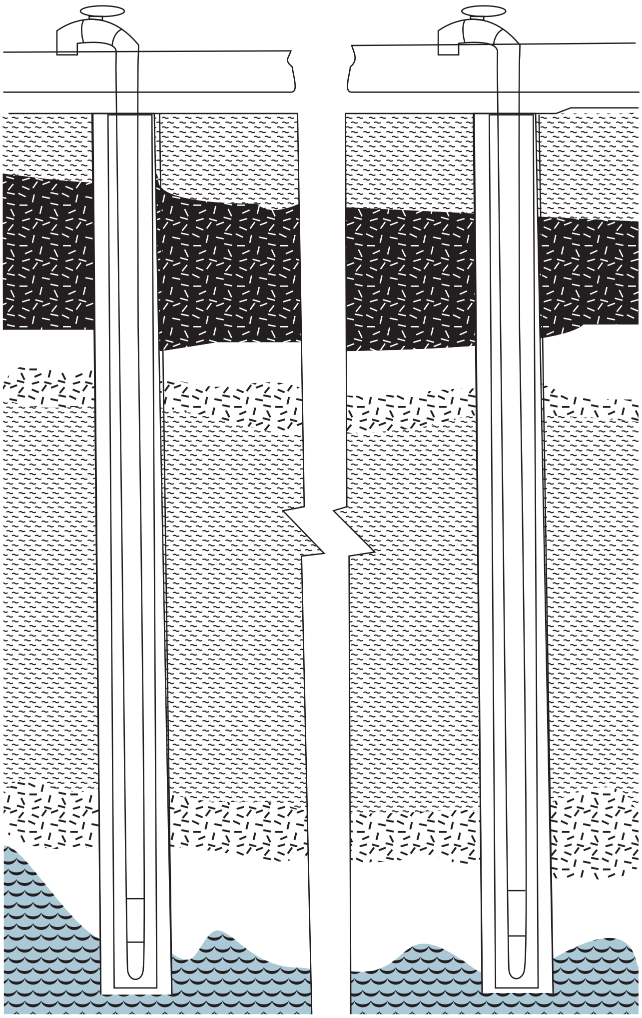

Figure 14.2 Deep-Well Section

A wellpoint system consists of a series of small wells installed around the perimeter of, in the case of a shallow tunnel, the area in which the water table is to be lowered and is connected to a high-vacuum pump by means of a set of header pipes with couplings and fittings. The system is based on the induced flow of water in the water-bearing strata toward filters (wellpoints) kept under vacuum by a high-vacuum pump. The pressure difference created between the pressure of the groundwater (atmospheric pressure) and the filters (wellpoints) directs the groundwater flow toward the wellpoints at a velocity that depends on the permeability of the various types of ground through which it has to pass.

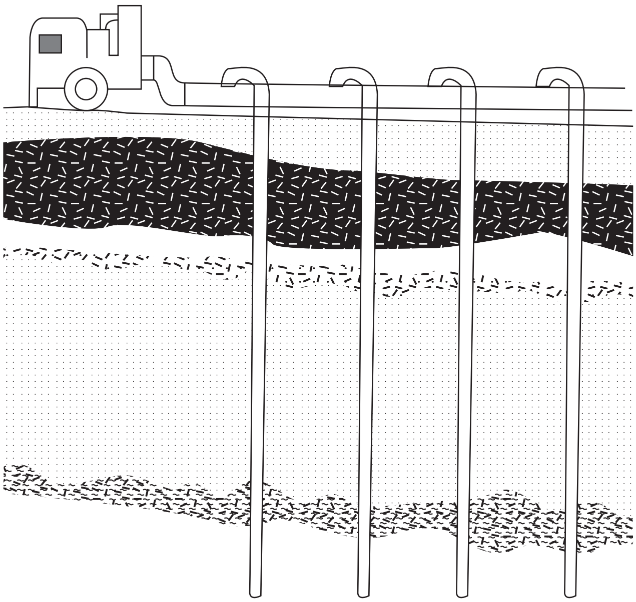

Figure 14.3 Wellpoints

A typical dewatering set is 50 wellpoints connected to one 150-mm (6-in.) dewatering pump. Additional wellpoints can be added or more than one pump can be used to increase the water drawdown. A wellpoint system consists of a number of wellpoints spaced along a trench or around an excavation site, all of which are connected to a common header attached to one or more wellpoint pumps. Wellpoint assemblies are made up of a wellpoint, screen, riser pipe, and swing joint which are installed by drilling or jetting. Entry of the water into the system is affected by creating a vacuum to the wellpoint. The water is then pumped off through the header pipe. A wellpoint system is a combination of two pumps, one which pumps water from the header and the other, a vacuum pump to remove air that enters the system. Control of air is important, as excessive air causes cavitation, reducing pump efficiency.

Although limited to shallow tunnels and shafts, wellpoint systems are frequently the logical and economic choice for dewatering areas where the required lowering of groundwater level is on the order of 6 m (20 ft) or less. These could be typically shallow shafts, launching pits, and cut-and-cover tunnels or starter tunnels. Greater lifts are possible by lowering the water in two or more stages. The 6-m lift restriction results from the fact that the water is lifted by vacuum.

In a typical system, wellpoints are spaced at intervals of 1–3 m (3–10 ft). Depending on their diameter and other physical characteristics, each wellpoint can draw from 0.5 to 100 l/m (0.1 to 25 gpm); total systems can have capacities exceeding 75,000 l/m (20,000 gpm).

In addition to being used on the surface, wellpoints can be used in the tunnel for dewatering. They can be installed in areas away from the face and out of the way to lower the water level in the immediate area. In small tunnels (hand mined), miners have attached PVC pipes to the suction line, pushing them into the bottom of the face at an angle to pull water from the face as they mine. For larger tunnels, wellpoints may be driven at an angle at the invert so that the bottom is below the face to draw out water.

COMPRESSED AIR

The compressed air method, also called the "plenum method," is used to prevent the flow of water into the tunnel, making tunneling under water bodies relatively safe by creating pressure greater than the water head when a tunnel is constructed within the groundwater. Unlike deep wells and wellpoints, compressed air is used in the tunnel to provide a pressure bulb. Developed in cofferdams, air pressure can control water flow, can provide stability below the groundwater table, and is one of the oldest pressurized face support methods used in tunneling. The principle is that the pressure under which the entire perimeter of the tunnel is subjected to can be counteracted by making the pressure inside the tunnel greater than the pressure of the water and soils. Gravity gives this air weight. This weight creates a pressure of about 1013 bars (14.7 psi). This is the pressure reading on a barometer. Pressure is defined in many ways:

Atmosphere absolute (ata); atmosphere (atm): 1 atm = 14.7 psi

1 bar = 14.5 psi

Pounds per square inch gauge (psig): air pressure measurement does not include atmospheric pressure; thus 1 psi = 15.7 psi

Feet of seawater (fsw): pressure that exists at any given depth of seawater in feet; 10 ft of seawater has a pressure of 4.45 psig; 33 ft of seawater = 1 atm (Note seawater is heavier than other water.)

Meters of seawater (msw): pressure that exists at any given depth of seawater in meters

The rule of thumb is about $^1/_2$ lb (3000 Pa, 0.43 psi) of air pressure for each foot of water head. Often the air requirement is less than theoretical due to the restriction to water flow of the soil and the stability of the soil.

Best suited in sands, gravels, and water-bearing silts, compressed air resists the pore water pressure. The pressure head of water is dependent on the relative elevations of the water table and tunnel and the porosity and permeability of the ground. In rock, the aperture of the discontinuities and the filling of them affect the water head along with relative elevation.

Water will not flow through tight clay quickly, because it is nearly impervious, meaning it is effectively supported by the air and it seals the ground, preventing the air from escaping. Sometimes it can be effective enough that the primary lining is not needed prior to the installation of the final lining. Silt acts somewhat like clay when it is first exposed to the compressed air. But then it will start to dry out and flake from the crown and top of the face.

The behavior of the top of the tunnel compared to the bottom of the tunnel will be different, especially in a large tunnel. The invert of the tunnel is lower than the crown. Therefore, the pressure required to prevent the incursion of water at the invert is greater than at the crown. This means that a 20-ft tunnel may require 8.6 psi more pressure at the invert than at the crown. However, there is a danger of blowout if the crown receives that much pressure. To mitigate this problem, one may be able to install crown support, such as a liner plate. The liner plate with backfill will permit the pressure to be increased without increasing the risk of a blowout. Thus, the air pressure can be increased to permit the invert to be managed. However, it is essential that the liner plates be made airtight by spreading some type of sealer (wet clay) along the joints. To make the lagging impermeable, one can use building paper, which is water tight and durable. In sand, the air will infiltrate several meters at the crown and leave the invert wet enough that boards will have to be used to stop sand from running in; something like excelsior or oakum may have to be stuffed between the boards. Also, as stated earlier, wellpoints can be driven at the interface of the invert and face.

A blowout is rapid loss of the air pressure from the tunnel to the surface. As the air under pressure passes through the soil, it can suddenly escape, causing a sudden pressure drop in the tunnel. A rapid pressure drop in the tunnel pressure is accompanied by a loud banging sound with the formation of a mist. This can lead to the tunnel caving in and generally will cause an increase in the loss of air from the tunnel. A rule of thumb to avoid this is to make sure that the primary stress at the crown exceeds the air pressure by 10%. Therefore, if the primary stress at the crown is equal to 1 bar (14.5 psi), the air pressure should not exceed 0.9 bar (13.0 psi).

Tunneling through soft underwater mud has a high risk of this occurring. Should any outward leak occur, it is imperative to immediately plug it with any material that is available. Sometimes clay or other material is used to construct an embankment on the surface above that area of the tunnel, with its width being about six times the diameter of the tunnel to increase the impermeability of the soil around the tunnel.

Driving a tunnel under compressed air is no longer a common method. However, manned interventions into the cutting head may be required for inspection, maintenance, and repair. The same principles and gas laws apply. When work has to be done in the head, the mechanisms for maintaining the face with EPB, treated soil in the head, and slurry machines, slurry mix, have to be removed to permit entry into the head. If the face cannot stand without these mechanisms, another method must be used. This other method is compressed air. By the injection of compressed air into the head, the face can be supported and the water inflow restrained. The machine will have two locks to enable passage in and out of the head. Once the compressed air is injected into the face area, the miner climbs into the first chamber. At this time, the chamber is at the ambient air pressure of the open tunnel. Once in the chamber and the chamber is sealed, the air pressure in the chamber is gradually increased until it matches the air pressure in the head. Once this happens, the chamber hatch into the face area can be opened. When the miner is ready to leave the head, he or she enters the chamber. At this time, the air pressure in the chamber is the same as at the face. The chamber is sealed and the air pressure is reduced, decompressed, until it equals the tunnel ambient air pressure. One can relate the procedure to that of locks for passing ships. The lock has a gate at each end to contain the water. The ship enters the lock at the same water surface elevation. The lock is sealed from the water that the ship had just left and the water level in the lock rises or falls depending on the elevation of the water on the opposite side of the locks.

Because of the health hazard, the decompression of the chamber must be conducted slowly, in conformance with established procedures, or the miner could get several related illness. The most common is the “bends.” A further discussion of compressed-air-related illnesses is beyond the scope of this book. It must be appreciated that it is dangerous, and all of the pertinent regulations and procedures must be adhered to. Also, there are companies that specialize in managing entry into compressed-air environments and provide all of the necessary safety and medical equipment.

GROUND FREEZING

Ground freezing is a process that is used for groundwater cutoff to provide structural underpinning, temporary supports for excavations, and the stabilization of ground for tunneling in wet, weak soils. It does this by making water-bearing strata temporarily impermeable and increasing shear and compressive strength by changing joint water into ice. The objective of freezing is to simultaneously seal against water and substantial strengthening of incoherent ground.

Ground freezing is not new. For over 100 years, it has been used extensively for groundwater control and excavation support in underground construction. In 1863, the German scientist F. H. Poetsch patented his artificial ground-freezing method. His technique is believed to have been first used around 1862 in the coal mines of South Wales. By 1888, it was used in the United States at the Chapin Mine in Iron Mountain, Missouri. The first successful use for sinking shaft occurred in England in 1901. Two shafts were sunk through 24–21 m (80–90 ft) of wet sand and boulder clay at the Washington Glebe Colliery, near Sunderland.

All ground-freezing systems are quite similar in principle. A refrigerated coolant is circulated through a series of subsurface pipes to convert soil water to ice. The watertight material created is so strong that it is routinely used as the primary method of groundwater control and soil support for the construction of shafts hundreds of feet into water-bearing soils. The single most important component of a ground-freezing system is the subsurface refrigeration system. It consists of a series of refrigeration pipes installed by various drilling techniques. It should be remembered that, when frozen, the ground will expand; the amount of expansion depends on the temperature and amount of water in the soil. This will be taken into consideration during the design. Ground freezing can be achieved by using either a large portable refrigeration plant or liquid nitrogen. Like other refrigeration, the required refrigeration capacity is significantly reduced to maintain the frozen barrier once the initial freezing has been completed and the frozen barrier is in place.

Poetsch's basic concept is still the basis for twenty-first century ground freezing. The in situ pore water, when frozen, fuses together particles of soil or rock like a bonding agent that creates a frozen soil mass with markedly greater compressive strength and impermeability. While advanced refrigeration technology has refined modern efforts, the core method of achieving the freeze still dates back to Poetsch.

Closed-end freeze pipes that are small diameter are inserted into vertical drilled holes. They are in a pattern consistent with the shape of the area to be improved and the required thickness of the wall or mass. Heat is extracted from the soil, causing the ground to freeze around the pipes as the cooling agent, typically chilled "brine," is circulated through the pipes. When the brine is returned to the refrigeration plant, it is again cooled. The frozen earth forms, in the shape of vertical, elliptical cylinders, around the freeze pipes. The cylinders gradually enlarge, eventually intersecting one another to form a continuous wall. The thickness of the frozen wall will expand with time as long as heat extraction continues at a rate greater than the heal replenishment. Once the frozen wall has achieved its design thickness, the freeze plant may be operated at a reduced rate to maintain the condition during shaft excavation and liner placement.

Temperature sensors installed at various levels in monitor pipes are located strategically along the frozen wall to monitor conditions during formation and maintenance. Refrigeration is discontinued, allowing the ground to return to its normal state once the excavation and shaft lining are completed. Successful freezing operations require specialists who must be skilled in refrigeration and able to analyze thermal problems. The specialist must be knowledgeable and experienced in groundwater and geotechnical engineering. It is vital that the specialist have a good understanding of the strength and behavior of frozen earth.

Once in place, proper instrumentation can provide complete assurance of the integrity of the freezing to depth prior to reaching it with the excavation. Because the frozen ground is the ground support and the temperature is monitored, construction can be scheduled without needing time to be scheduled for probing ahead, adding ground support, or dealing with groundwater. The soil and rock interface, which is generally the most difficult geology in which to create a groundwater cutoff by other methods, is perfectly implemented through the soil–rock interface. Unlike other methods, at increasing depths, any discontinuity in a temporary support system can be difficult to rectify unless the hydrostatic head is externally relieved; this is a task that cannot practically be achieved at depth. However, a frozen wall, by design, is continuous into the underlying cutoff and resists the loads imposed by full groundwater and soil pressures.

The main disadvantages to using ground freezing as support and water control are the cost and equipment required. Ground freezing is an expensive method, but it is used frequently because it is the only method that will work under certain circumstances. In addition, when excavating in frozen ground, especially at depth, the entire shaft cylinder freezes, requiring methods that are not needed for soil. Generally, excavation is done with sensitive, or careful, blasting or a roadheader. In addition, for the construction of shafts specialized equipment is required for the excavation of the frozen ground. With continued operation of the freezing system, the frozen ground will encroach further within the shaft excavation at greater depths. In deep shafts, it is common for the entire cylinder to freeze solid. In frozen shafts excavation is either by roadheader or by careful drilling and blasting.

The thermal properties of the underlying soils and the related response to the freezing system govern the design of the frozen soil barrier. The rates of development of the formation of the frozen soil barrier depend on the thermal and hydraulic properties of each stratum. Rock and coarse-grained soils typically freeze faster than clays and silts.

This methodology does not require that extraneous materials be injected, and other than contingency for frost heave, the ground generally returns to its normal condition. Regardless of the structure, grain size, or permeability, ground freezing can be used. Although it can be used in a wide range of soils, it does take a considerable time to develop a significant ice wall and it must be maintained by uninterrupted refrigeration for as long as needed. Despite being useful in rock, it is more effective in soils.

Usually, a row of freeze pipes are installed vertically in the soil. The pipes are used to remove heat energy, similar to pumping groundwater from wells. Ground freezing may be done in any shape, size, or depth. The high cost is somewhat mitigated in that the cooling plant, like other construction equipment, can be reused on subsequent projects. The use of ground freezing, as the freezing progresses, reduces the need for other water-handling equipment, such as compressed air, and dewatering and reduces the need for ground support.

Freezing will increase the strength of the ground to that of medium to strong rock by cementing the particles with ice created by the presence of water. The greater the water content, the more impermeable the ground will become.

Freezing of water in the pores occurs when the ground temperature reaches 0°C (32°F) and then cooling increases. With granular soils, the groundwater will easily freeze, and in saturated sand, the results are excellent at only a few degrees below freezing. In this situation, the temperature will decrease only marginally. In clays, the groundwater is at least in part molecularly bonded to the soil. When cooled to the freezing point, some portion of its pore water in soft clay begins to freeze, causing the clay to begin to stiffen. Further reduction of the temperature results in an increase of the pore water freezing, thereby increasing the strength of the clay considerably. Because of the density of the clay, it may be necessary to supply a significantly lower temperature to realize the required strength. Depending on the material, the temperature may vary considerably; in sands, -6°C ($+20^{\circ}$F) may be adequate, whereas it may require a temperature as low as -28°C (-20°F) to achieve the required strength in soft clay.

As the earth freezes, vertical cylinders encircle the freeze pipes and the cylinders enlarge gradually until they intersect, forming a continuous wall. As the heat extraction is continued at a rapid rate, the frozen wall expands. The freeze plant operates at a reduced rate to remove heat flowing toward the frozen wall to maintain the condition once the designed thickness is reached.

Circulating brine, which is a strong saline solution of calcium chloride, is the most common freezing method. The freeze pipe is inside a drop tube. The chilled brine is pumped down the drop tube to the bottom of the freeze pipe. It then flows up the freeze pipe, drawing heat from the surrounding soil. (See Figure 14.4.)

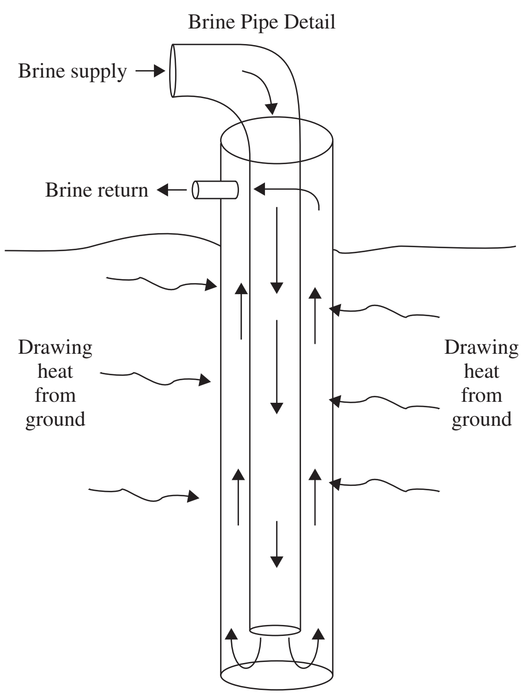

Figure 14.4 Brine Pipe Detail

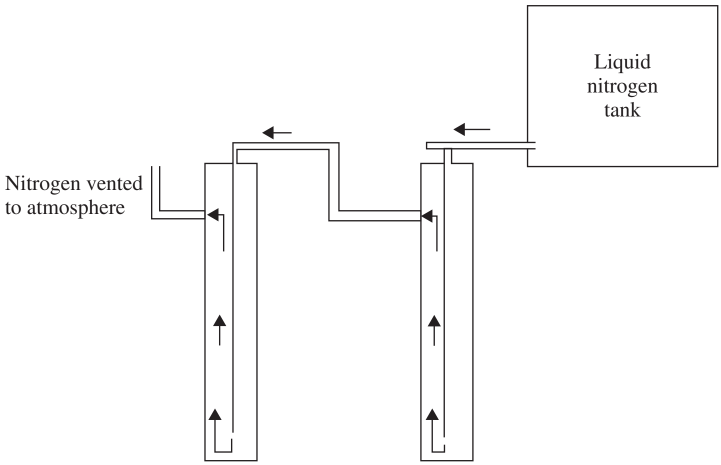

Figure 14.5 Schematic of Liquid Nitrogen Plant

When extremely low temperatures are needed quickly, liquid nitrogen (LN_2) has been used with success. It is considerably more expensive per unit of heat than liquid brine, but for small short-term projects, especially emergencies that need fast, extremely low temperatures, it can be cost effective. Because low temperatures can result in rapid freezing, one can attain frozen clay with high strengths. (See Figure 14.5.)

Since the penetration of freeze is not greatly affected by permeability, which allows it to be quite versatile, it can be adapted to various situations. It is considerably more effective than cutoff grout, and in stratified soils, freezing has fewer problems than dewatering. Because of the strengths achieved, freezing can function as both water cutoff and ground support, thus eliminating the sheeting and bracing or other active earth support equipment. In tunneling, freezing can be used as a water cutoff and/or support for shafts and shallow tunnels or as retaining walls for cut-and-cover tunnels.

Ground freezing has the versatility that it can be applied in all types of soil and groundwater conditions, such as running sand, clay, and gravel, allowing one to work in all of these soils. When groundwater is a concern, it can be the most effective ground support system when either pumping or cutting off the water flow cannot be readily achieved with other methods. When properly executed, ground freezing can provide complete water cutoff and can be used in difficult ground conditions. Demobilization of ground freezing can be done easily and completely simply by allowing the ground to thaw naturally, and it has no long-term effect on the subsurface environment. When freezing, the work can be easily instrumented using temperature and pressure sensors.

The type of water control/handling system used depends on the situation, schedule, and cost. The method selected will be the fastest, least expensive, and most effective available. Sometimes the approach to use is very obvious, whereas with more complex situations, careful engineering and cost analyses may be required.

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15 Trenchless Excavation

Trenchless excavation consists of a number of nondisruptive techniques for installing, replacing, or rehabilitating underground pipes and cables without open-cut excavation; therefore, there is no surface disturbance. By not disturbing the surface, disruption to local traffic, whether pedestrian or vehicular, is eliminated or mitigated.

Conventional trench excavation techniques generate costs to the community beyond those of the obvious constructions costs. Many of these costs are the result of conflicts with existing utilities and surface obstacles that trenchless methods avoid or mitigate. Opening up a sidewalk or street for traditional trenching can result in having to detour traffic, which can cause negative effects on commercial establishments. If a business entrance is blocked or in any way restricted, a financial hardship to the business in the form of loss of sales and revenue can result. Not only will this cause the business to have a negative attitude toward the construction and the entity for which the project is being performed, but also the entity will likely be responsible for the negative commercial effects as well as physical repairs such as damage to the sidewalk, parking lot, and so on. Often the financial losses are difficult to quantify. The cost to repair a parking lot can be determined, but what about the loss of revenue? Do you take to business owner's word for it, or do you match the previous year's revenue during the same time period? Is the business climate the same as the previous year, or is it better or worse? As is evident, it can become a very complex undertaking to fairly compensate the business owner (as well as the lawyers!). What if there are several businesses affected? Thus, although trenchless excavation may appear to be more expensive at first glance, it can easily be less expensive when all of the construction, commercial, and social costs are considered. Actually, it can be considerably less expensive than deep open-cut excavation.

In contrast to these often complex social problems caused by trenching, there are generally less environmental impacts. Other than the jacking and receiving pits, there is no opening of the ground surface, and erosion and water runoff are mitigated.

Beyond the financial considerations related to trenchless excavation, microtunneling and pipejacking make technical sense. With jacked pipe, generally the greatest loads that the pipe is exposed to are the construction loads and stresses; that is, the jacking of the pipe puts greater stresses on it than its service use. Because of the ability of the pipe to withstand jacking forces, once a pipe is in the ground, the stresses to which the pipe is exposed are less than jacking; the pipe is durable.

Since there is generally little displacement of the soil with pipejacking or microtunneling, there is minimal surface disturbance, resulting in less chance of ground loss and settlement, thus minimizing or eliminating the need for surface restoration. Generally, the pipe is installed with the excavation, causing the ground to be constantly supported as the pipe is advanced.

There are three ways to create a hole for installing a pipe: compression/displacement, percussion, and cutting. Compression/displacement is the act of driving a pipe through the soil by compressing and displacing the soil along the pipe's path. That is, the soil is not excavated; it is pushed aside, compressed into the surrounding soil, and displaced by the pipe. Percussion is similar to compression/displacement in that the pipe is driven into the soil and the pipe displaces the soil, but in this case, the soil is not pushed, or compressed, into the surrounding soil. Rather, the soil that is being replaced by the pipe is removed through the pipe. Cutting is the method whereby the soil along the pipe alignment is cut, or dug, from the pipe alignment. The best example of this is an auger drill. As the auger rotates, the cutting edge of the cutter head slices the soil, excavating it from the hole for the pipe.

RAMMING

Probably one of the earliest forms of trenchless technology, pipe ramming is a simple trenchless method of installing steel pipes or steel casings for distances of around 50 m (150 ft) with diameters up to 1500 mm (60 in.). Impact hammers are very simple devices that consist of an outer, normally steel shell within which there is a heavy reciprocating piston that is driven by either compressed air or hydraulic fluid. Like a pile driver, the forward stroke of the piston impacts the shell, driving the unit forward. Once the length of pipe is buried, the reverse stroke positions it for placement of the next length of pipe and the next forward stroke.

This method is most commonly used for crossing under railway lines due to stringent settlement requirements. Typically, a casing is used that acts as a conduit for wires or other pipes. The ramming tool is attached to the rear of a steel pipe and drives the pipe into the ground with repeated percussive blows. It can be carried out in a short series of smaller rams, or lights, or for the entire pipe length. The ground conditions and the available space for the driving pit setup are the principal factors in the decision of whether to do the entire pipe length or short rams. Even though a short pipe segment is being rammed, the pipe is driven the entire length and then returns back to the tool's starting position, where a new segment is welded or mechanically attached to the previous segment. During the installation of the pipe, the soil generally enters the pipe because the end is open. The soil inside the pipe can be removed as the pipe is installed or after the installation, by auger, compressed air, or water jetting. Although the pipe may have a closed end, this option is usually selected only for certain ground conditions or for small-diameter pipes. Once installed, other pipes or cables can be inserted through it.

Although both pneumatic and hydraulic hammers are used to provide the impact, when compared to a pneumatic impact hammer, a hydraulic system provides higher energy utilization efficiency, longer service life, and smaller dimensions and weight requirements and is more environmentally friendly.

The correct rammer size is based on the casing diameter, length of ram, and ground conditions. Typically, installation begins with the first section of steel pipe being placed in the starter pit on the correct line and grade. Then the hammer is attached, by using a series of size adaption collets, to the rear of this pipe. When the hammer is activated, the reciprocating piston drives the pipe section forward into the ground. There is no necessity to push against a reactive force; therefore, unlike pipejacking, thrust blocks or plates are not required in the launch pit.

Mechanically, cutting edges are on the head of the pipe to facilitate soil penetration and friction is reduced by lubricating it with the addition of bentonite. The risk of road surface heave or settlement is minimized by the small amount of displacement volume in the area that the cutting shoe eliminates. Therefore, impact driving can be conducted even from shallow pit depths, providing the advantages of minimal settlement above the pipe; thus restoration is not required and set-up and pipe laying times are short. A thrust block is not needed, meaning that the pits can be simple. With a special ram cone, this simple technique can be adapted for all pipe diameters.

Although pipe ramming can be used in a many different soil types, some soils are better suited for ramming than others. Soft to very soft clays, silts and organic deposits, and all sands above the water table ranging from very loose to dense, are among the most suitable soils for pipe ramming. In addition, soils with cobbles, boulders, and other obstacles of significant size but smaller than the pipe diameter and soils with cobbles in extremely wet conditions, even with running water, can be penetrated. In denser materials, such as medium to dense sands below the water table, medium to very stiff clays, hard clays, highly weathered shale, soft or highly fractured rocks, marls, chalks, and firmly cemented soils, pipe ramming is more difficult. Pipe ramming is unsuitable for use in solid rock. In rocky ground conditions, the ramming may be begun using a pneumatic tool to punch the pilot hole first and then ram the pipe afterward. The disadvantage to this approach is that the pneumatic tools can be easily deflected by rocks or other obstacles in the ground. To reduce the potential for course change during operation, manufacturers have developed a variety of head designs that they claim can hammer through harder ground and to break up smaller boulders.

AUGER BORING

The technique for the bored installation of a casing in the ground is called "auger boring." Casings are generally steel pipes that are welded together. It is an excavation technique in which a cutterhead followed by an auger penetrates the earth. As the cutterhead advances, the rotating auger transports the excavated material, muck or spoils, to the pit for excavation to the surface. It is not used for jobs requiring a high degree of accuracy in line and grade, because it is usually unguided.

There are two types of auger boring: with a casing and without a casing, or "uncased." Boring without a casing is called "free boring" and is generally used for small-diameter holes, that is, less than 150 mm (6 in.). Because of the inherent risk of the hole collapsing and subsequent ground subsidence, many jurisdictions do not allow this method under roads and other public areas. Therefore, the majority of auger bores are simultaneously cased. The final conduit can then be installed in the casing when desired.

Auger boring is normally cheaper than microtunneling or pipejacking, but it is limited to the type of ground in which it can be used. It has limitations in wet ground. With a specially adapted head, it can be used in some rocks. Figures 15.1 and 15.2 are photographs of typical auger boring machines.

Figure 15.1 Auger Bore (Courtesy of American Auger)

Figure 15.2 Auger Boring Machine with Casing and Cutterhead (Courtesy of American Auger)

Horizontal Directional Drilling

Developing rapidly over the past several years, the horizontal directional drilling (HDD) technique is generally used for installing pipes up to 500 m (1650 ft). Much of the development of HHD came from the water well and oil well drilling industries. HDD is a steerable trenchless method of installing underground pipes, conduits, and cables in a shallow arc along a predetermined bore path using a surface launched drilling rig. It is especially useful for drilling under water, such as rivers and other surface obstructions that cannot be done open cut. Directional boring is used when trenching or excavating is not practical. It is suitable for a variety of soil conditions and projects, including road, landscape, and river crossings. Applications include gas lines, pressure lines for wastewater, and cable protection conduits, basically any application that requires pipe to be installed for significant distances where the greatest depth is greater than that which can practically be done by other trenchless methods (i.e., river crossings).

The method is environmentally friendly in part because of the small footprint, or work area, required, and because of its small footprint, it is well suited for an urban environment, and there is little or no disturbance to wetlands or nature preserves. In addition, HDD can provide for lower construction costs in that less area is required, the time required for setup is relatively shorter, there is less spoil to remove, and surface restoration and interference with traffic or surface activities are reduced or eliminated. In addition to directional capabilities offered by HDD, deeper and longer installations are possible.

There are three main system components to the drilling unit: the drill rig and drilling tools, the bentonite mixing and recycling system, and the hydraulic power unit to mix the bentonite.

Generally, the selection of the drilling unit is based on the length of the bore, the size of the pipe, and the geologic material in which the pipe is to be installed. The most difficult aspect is maintaining the line and grade of the pilot bore, especially when the fluid-assisted technique fails because of soil conditions. High thrust and pulling forces are needed to deal with this problem. Bentonite inclusion may help, but in difficult soil conditions with coarse-grained material and rock or rubble, the addition of a pneumatic hammer is needed. This allows the drill to break through the difficult ground. Combined with fluid-assisted drilling, this allows the drill to advance while maintaining the line and grade in difficult soils.

To meet the demands for precision of the alignment of the bore, the location of the head of the drill is monitored using a method referred to the transmitter–receiver principle. Basically, there are two types of equipment for locating the bore head: the “wireline” and the “walk-over” locating systems. In both systems, a transmitter, or sonde, is located behind the bore head, which registers direction, rotation, angle, and temperature data. Encoded into an electromagnetic signal, this information is transmitted through the ground to the surface in a walk-over system. The signal decoded and steering directions are relayed to the bore machine operator through a surface receiver (usually a hand-held “locator”) that is manually positioned over the sonde. With a wire-line system, there is a cable fitted within the drill string through which this information is transmitted.

Once the pilot hole breaks through, the borehead is replaced with a larger head (backreamer). One or more intermediate sizing prereamings are conducted prior to pipe installation. Once the hole is enlarged to approximately 25% larger than the pipe diameter, the conduit can be pulled through it. The pipeline is prefabricated at the opposite end of the hole from the drill rig. A reamer is attached to the drill string and is then connected to the pipeline by a pulling head and swivel. The swivel permits the reamer to rotate without the pipe turning. Beginning the pullback operation, the rig rotates and pulls on the drill string while circulating drilling fluids. The pullback continues until the reamer exits the bore hole by the drill.

Pipes are made of plastic, steel, or cast iron. Pipes 1.2 m (48 in.) in diameter have been installed on shorter runs and installation lengths up to approximately 6500 m (21,300 ft) have been completed. They can be installed/pulled singly or bundled. Caution must be used that the permissible pulling forces are not exceeded when pulling plastic pipes.

Although there are a few methods to change the borehole direction, they all share one common element: they produce a side-deflecting thrust to force the drill bit to the desired direction. The most common is the side-deflecting whipstock. The whipstock is a long inverted steel wedge that is placed at the end of the hole. It has a concave face in the direction of the axis of the hole. When the drill advances down the hole, it is forced to the opposite side of the hole.

PIPEJACKING AND MICROTUNNELING

Originally developed for installing casings for crossings, pipejacking has developed into a major trenchless technique. The trenchless technology method for installing a prefabricated pipe through the ground is known as “pipejacking” (PJ). “Pipejacking” is the name given to the technique of installing pipe of man-entry size by pushing sections from a jacking pit. As well as for jacking, the principle of using hydraulic rams to push pipe sections to line the hole cut by a cutting head, or shield, is also applied to auger boring and microtunneling. Whether installing a 150-m (500-ft) auger bore casing or driving a concrete underpass, the principle is the same. The narrowest definition of *PJ* is it is a method for installing a lining.

The first recorded use of PJ in the United States was for installation projects in the Northern Pacific Railroad Company between 1896 and 1900. The pipe is driven/jacked/propelled from the launching, or drive, shaft to a reception shaft or receiving pit. The jacking force is conveyed to the face of the excavation through the pipe. The muck is transported to the jacking pit through the pipe either manually or mechanically, where it is removed and disposed of out of the pit. Both excavation and mucking require labor inside the pipe during the jacking process. The presence of workers in the pipe necessitates that the minimum recommended diameter of pipes jacked is 108 cm (42 in). However, it is feasible to jack reinforced-concrete pipe (RCP) with an inside diameter as small as 900 mm (36 in.).

The pipe is jacked into a hole that is being cut in the soil by a cutting edge, with the pipe providing continuous ground support. Pipejacking is a cyclical operation in which the process involves thrusting hydraulic jacks against the pipe, advancing it against the face as the muck is removed through the pipe. After the pipe length has been installed, the rams of the hydraulic jacks are retracted, another pipe segment can be placed in position in the jacking frame, and the jacking cycle begins again. The large jacking forces against the pit wall require that the design of the jacking pit include a thrust block or some other structure to act as a reaction to the force. The resistance required increases with the size of the pipe. Without a properly designed reaction structure, the jacking process can fail. If the back wall is vertical and the jacking forces are not too great, the simplest reaction is to use a steel plate, that is, a road plate, for the jacks to push against. The plate is stable and can withstand great shear and punching forces. The load is distributed to the earth wall by the plate.

The PJ process begins with the excavation and preparation of the launch pit and the receiving pits are the first work done to begin the jacking process. The size of the pits varies based on the size of the pipe to be jacked. The jacking frame and hydraulic jacks are set up in the jacking pit to the desired line and grade. The jacking frame holds the pipe in place while being jacked. Figure 15.3 is a typical jacking frame.

Figure 15.3 Jacking Frame

The jacking shield is then launched from inside the launching pit. To provide a stable structure against which to jack pipes, the thrust wall is constructed behind the jacking shield. A thrust ring (push plate) is also used to transfer loads to ensure the jacking force is distributed equally through the entire diameter of the jacked pipe. To ensure that the thrust from each pipe is consistent, jacks are interconnected hydraulically. The number of jacks and the force of the jacks needed to install the pipes are based on the size and length of the pipe being installed, the strength of the jacking pipes, and the frictional resistance.

During the jacking process, the correct alignment of the pipeline is routinely checked. With the use of a steerable shield and by placing guide rails inside the thrust pit to set the pipe, accurate pipe alignment can be checked and carried out. Traditional surveying equipment is used for checks for shorter pipelines. Once jacking is completed, the shield is removed from the reception pit and the pipeline is installed.

In detail, the jacking process begins with the excavation and preparation of the jacking pit. Once the pit is set up, the jack frame is installed and the frame and jacks are adjusted to the proposed line and grade. If laser is being used for guidance, it is installed in the shaft. When switched on, the laser points are set to the designated position. The operator, who is located next to the machine continuously, monitors the mark on the steering head and the laser point. If the operator notes a deviation, he or she will articulate the head to bring the bore back on line and grade.

The machine is lowered into the driving shaft and set up. The thrust ring and the shield or TBM are mated. The frame against which the main cylinders push to advance the boring head and pipe is called the “thrust ring.” The ring has a 360° circle that provides surface against the pipe to avoid a point load, reducing the potential for breakage of the pipe. At this time, the shield or TBM is advanced through the prepared opening in the forward shaft support structure. The excavation and muck removal process is begun and advanced until the shield or TBM is installed. The control panel outside the boring machine controls the movement of the jacking machine, whereas the drilling operation is controlled by the control levers inside the boring machine.

Now that the machine is ready for operation, the jacks and jacking ring are retracted enough to allow space for the placement of the pipe length and place the first pipe length on to the jacking tracks. The jacking plate and the pipe are mated and forward advancement is initiated. This process is repeated until the pipe line is completely installed. The shield is removed from the reception pit and the jacking equipment is removed from the jacking pit. Then the site is restored to the required condition.

Drive Lengths

The drive length is the length that can be successfully driven. It depends on the jacking equipment capacity, the jacking stress capacity of the pipe, and the material through which the pipe is being jacked.

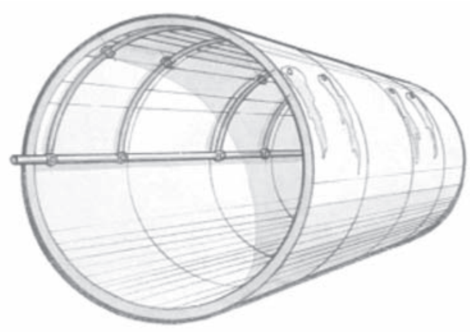

Figure 15.4 Pipe-Lubricating System (Courtesy of Pipe Jacking Association)

Although the jacking pipe is designed for jacking loads, there are limits. The factor that affects the amount of force required is the resistance to jacking. The resistance is principally the result of friction and the friction is a function of the friction factor, the weight of the pipe, the face pressure, and the material through which the pipe is being jacked. To reduce the force required to move the line, a lubricant containing bentonite and perhaps other polymers is used to reduce the friction and thus the required thrust load. Figure 15.4 illustrates a pipe-lubricating system.

Eventually, the length of the line will advance past the point where lubricating the line is insufficient. To mitigate this, intermediate jacking stations are used. They are used on pipes generally 1.2 m (36 in.) and larger. They are placed along the line between the drive shaft jacking plate and the jacking shield or TBM. They redistribute the total required jacking force on the pipe, reducing the amount of force needed from the jacking pit, and the stress to which the pipe is subjected is reduced because the short length of pipe requires less thrust. They consist of a steel cylinder that is installed between two lengths of the pipe being jacked. The cylinder has hydraulic jacks spaced around the internal perimeter. The pipe is pushed forward by the hydraulic rams. Once the jacks have completed their stroke, the pistons are retracted. The main jacks push the pipe to the retracted cylinders of the intermediate jacks. The intermediate jacks again expand, advancing the pipe. This cycle is continued until the pipe reaches the reception pit. A drive may require more than an intermediate jacking station.

Figure 15.5 is a photograph of an intermediate jacking station. Note the placement and size of the cylinders. The cylinders are generally smaller than those of the main jacking station.

Some of the technical benefits to PJ include:

The lining is inherently strong.

Being precast, the pipe provides good flow characteristics.

It is a one-pass method.

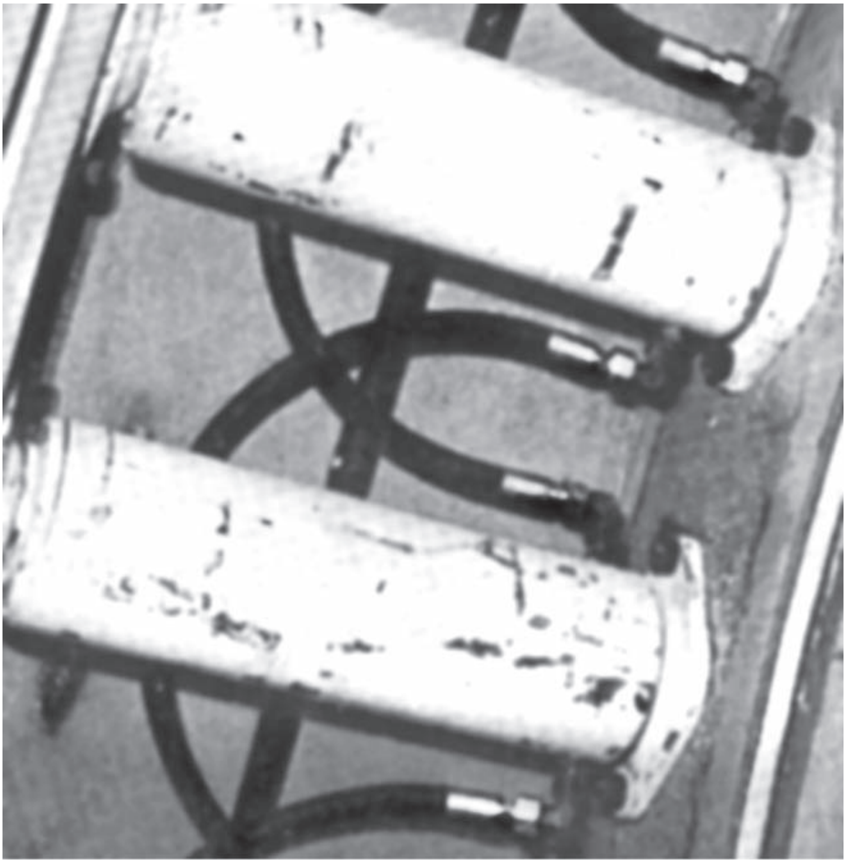

Figure 15.5 Photograph of Intermediate Jacking Station

There are fewer joints than in a segmental tunnel.

Pipes sealed with flexible joints will help to prevent groundwater ingress.

The risk of ground settlement is reduced.

There is less conflict with existing utilities.

Jacking pipes usually undergo greater stress when being jacked than during their end use. Therefore, the strength of the liner will be greater than required for its functional use. Since the pipe is precast, the surface of the pipe's interior is generally quite smooth, resulting in a lower friction factor and improving flow characteristics. With the pipe advancing with the excavation, there is only one pass through the ground. Since there are no longitudinal joints, there are few joins to leak. If flexible seals are used, they will help to prevent or mitigate water ingress. Figure 15.6 is an illustration of typical joints for concrete pipe.

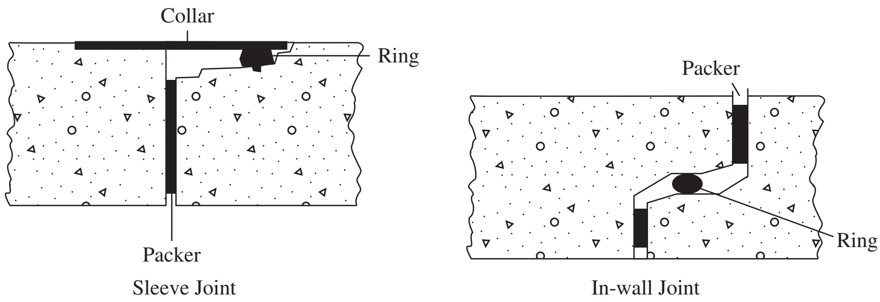

Figure 15.6 Concrete Pipe Joints (Courtesy of Pipe Jacking Association)

Pipejacking tends to be low-maintenance construction. As with other trenchless methods, there is a significant reduction in social costs when compared to open-cut trenching in urban areas and there is reduced environmental conflict.

Pipejacking consists of four elements: the face, the line, the jacking pit, and the top side. The face is where the excavation, ground control, mucking, and monitoring and adjusting line and grade occur.

There is a shield fitted to the leading end of the pipe being jacked that not only acts as protection but also is where the excavation activity takes place. It can be anything from a simple steel cutting ring fixed to the leading pipe to a sophisticated fully automated TBM. In addition, it can provide a hard cutting edge fabricated to the required excavation size. Being covered workers have a safe place to work. In addition to providing a location for mounting mechanical excavation equipment, it provides a safe working environment. Also, the shield provides a mount for face stabilization devices, a place for monitoring line and grade, and a means of adjusting directional attitude to provide line and grade correction.

The shield is effective for short drives above the water table and may allow man-entry working and control. For longer drives, mechanical cutting tools are needed. When working below the water table or in unstable soils, remote-controlled pressure balance techniques are used.

Microtunneling shields are miniaturized, remotely controlled versions of mechanized PJ equipment with monitoring and control systems that, in some cases, have computer-controlled automatic adjustment.

Types of Excavators

Excavation can be done manually where shovels, air spades, and other handheld tools are used. It can also be conducted using an auger bore or a digger arm. Roadheaders, TBMs, or EPB machines can be used, depending on the material being excavated.

Although best used in cohesive soil, the mechanical excavation methods used in PJ are very similar to other tunneling methods with a wide range of shields, excavation methods, and face supports available to make PJ possible in a variety of ground conditions. For example, one can effectively deal with PJ in either cohesive or noncohesive soils in dry or watery conditions. Also, the ability to jack through very hard rock and ground conditions comprised of mixed composites such as cobbles and big boulders is made possible by the use of mechanized boring equipment such as TBMs.

Mucking is the removal of the material from the shield to the pit or surface. One can use hand skips, carts on rail, wheelbarrows, and so on, anything used by hand that will fit in the pipe. Auger or screw conveyors are generally part of the excavator, as is slurry pumping. Conveyors can be used as part of the machine mucking system or a miner can hand muck, dropping the muck on the conveyor to be transported to the shaft.

The Line The line is principally comprised of the pipe and has several functions during construction. It provides a lining for the tunnel, provides protection for the workers, and serves to transfer the jacking load to the shield, or cutterhead. It provides a path for muck transportation and allows facilities to be run to the shield. The most important elements of the line, the pipe and casings, are installed in three ways: single pass, double pass, and within the casing.

The single-pass system is when the pipe becomes the final lining. This is most common with drainage, sewer, and water lines, that is, a conduit for fluids. With the double pass, a temporary casing is first installed and then jacked out by the permanent pipe. The principal use for the double-pass system is when the pipe installed by a single pass will not meet specifications. In the within-the-casing system, the permanent pipe is laid within an outer conduit that has been jacked in place. Once the inner pipe is in place, the annular space is filled. A classic example of this approach is when a pipe is filled with electrical or other conduit. The primary pipe acts as the basic conduit.

The jacking pipe has to be capable of transmitting the required jacking forces from the thrust plate in the jacking shaft to the jacking shield or the TBM. The most common types of jacking pipe are steel pipe, RCP, and glass-fiber-reinforced plastic pipe (GFRP). In Europe, polymer concrete pipe (PCP) is commonly used for PJ and microtunneling.

To assist in transmitting the jacking loads between pipes evenly across the cross section of the pipes, a cushioning material should be used. The cushioning prevents point loading on the joint causing damage to the pipe. The most common type of cushioning is plywood and particle board.

There are certain standards that jacking pipe must have to be effective. The joint has to be formed within the wall thickness to eliminate any external or internal projections; that is, both the interior and exterior of the joints are even from one pipe to the other. This prevents a disruption of flow characteristics within the pipe and the outer wall smoothness is necessary to jack the pipe through the earth. Pipe joints also have to be watertight and be able withstand the design pressure.

The most common form of joint for gravity or nonpressure applications is butt faced with an external collar set within the exterior of the pipe diameter to provide a flush surface.

Pipe generally is available in lengths ranging from 1 to 6 m (3 to 20 ft). When selecting the length of the pipe section, it should be realized that the fewer the joints, the less there will be the potential for leaking. Also, although longer pipes require fewer pipe changes, longer pipe lengths are harder to handle and a larger drive pit is required.

The size of the pit will vary based on the jacking needs. The surface site must provide space for storage and handling of pipe and muck and adequate space for the shaft. This can include hoisting equipment to remove the muck from the shaft. The size of the jacking shaft is determined by the space needed for the pipe diameter, pipe joint length, jacking shield, muck handling, jacking system, thrust wall, pressure rings, and guide rail system. As an example, the jacking pit size needed for a PJ project using pipe 1.5 m (5 ft) in diameter with lengths of 3.3 m (10 ft) could require a 3.6-m-(12-ft-)wide and 7.5–9.6-m-(25–32-ft-) long shaft, depending on selection of jacking and excavation equipment.

Iseley and Gokhale (1997) suggest that a reasonable productivity range for PJ projects is 10–18 m (33–60 ft) per shift with a four- or five-person crew. The productivity can be affected by factors that include the presence of groundwater, unanticipated obstructions such as boulders or other utilities, and changed conditions such as encountering wet silty sand after selecting equipment for stable sandy clay.

Microtunneling, as the name implies, is a digging technique used to construct small tunnels. The trenchless method involves installing pipe that is generally less than 900 mm (36 in.) in diameter to a predetermined line and grade by jacking behind a remotely controlled and steerable, guided, articulating microtunnel-boring machine (MTBM). The method is totally performed by remote control. Being remotely controlled, the machine’s distinctive feature is the use of sophisticated advanced technology equipment consisting of TV monitors and a laser guidance system which ensures safe, efficient, and highly accurate operation through a wide range of soils with a counterbalance cutting face. The systems provide graphical display of the cutterhead position at all times and the required data can be printed out. The quantity of excavation is completely controlled by the advancement rate of the machine at all times. This reduces the potential for surface subsidence by overexcavation.

In high-density areas, where it is necessary to drive through areas congested with underground utilities, accuracy in the installation of a new pipe is very important. Microtunneling is the most accurate line and grade pipeline installation method available to date.

Simply put, microtunneling is remote-controlled PJ. An operator located above ground conducts the boring and PJ entirely by remote control. It is not required to have personnel located inside the pipe and the muck is removed from the cutting head positioned at the leading end of the new pipeline that is advanced by PJ. With microtunneling, excavation is always in a closed trench, that is, the trenchless boring and PJ excavation, and muck and material transportation occur simultaneously. Because microtunneling does not need long lengths of open trench for pipe laying, it is deemed to provide a much safer work environment.

Other than being solely electrically powered and attached to the head of the pipe that follows the path of the tunnel as it is being bored, MTBMs are basically miniature replicas of full-scale, heavy construction TBMs.

Box Jacking

Box jacking is a method whereby square or rectangular four-sided boxes are, like PJ, jacked through earth to provide a conduit without open excavating. It is used for large conduits, generally greater than 2 m (6 ft) in both the walls and invert and roof, with the most common size often found when the conduit size is larger than 3 m (10 ft) in diameter. The equipment is the same except for shapes that the equipment can manage. As would be assumed, the jacking pit setup is the same and the frame is the size of the box. (See Figure 15.7.)

Generally, the method starts with the construction of the backstop for the jacks to push against. The launch slab is constructed to act as the guideway for the box. Large boxes are often fabricated onsite using typical reinforced-concrete methods for concrete box culverts. Sometimes conduits for posttensioning tendons are cast into the structure. When the concrete is cured, jacking begins by locating the box on the launch slab. It is important that the grade of the launch slab be as close to the design line and grade of the completed tunnel

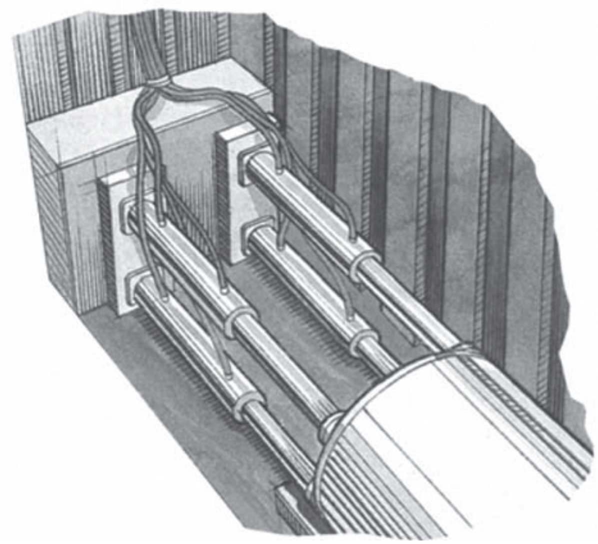

Figure 15.7 Box Jacking Frame (Courtesy of Pipe Jacking Association)

Figure 15.8 Typical Box Shield (Courtesy of Tunnelcorp)

as possible. A difference between the box jacking alignment and PJ alignment is the tilt of the box. Tilt is not an issue with pipe because it is a circle whereas with a box it is very important for a structure to have a flat invert.

Next, the launch slab is constructed, which will form a guideway for the box. The box is then cast onsite using similar techniques to those used for reinforced-concrete box culverts. Once the box is cured, jacking begins. Figure 15.8 shows a box shield. Like PJ, it is a cyclical method that includes pushing the box, excavating the face and hauling spoils, and extending the thrust members. This process is repeated until the box reaches its final position.

As with PJ, the size of the box, the overburden, the muck carried in the shield, and the ground type are used to determine the forces needed to jack the box. Boxes can be jacked through embankments that have a wide range of ground types.

Although mostly used for box culvert types of structures, box jacking can be effectively used for constructing road underpasses. This is done without affecting the traffic on the road above and the lining is in place when jacking is completed.

REFERENCE

Iseley and Gokhale (1997), "Trenchless installation of conduits beneath roadways." National Cooperative Highway Research Program (NCHRP) Synthesis 242. Transportation Research Board/National Research Council, Washington, D.C., 36.

16 Tunnel Ventilation

INTRODUCTION

Since humans began going underground to mine, providing breathable air to miners has been one of the primary problems that had to be overcome. Cultures that used slaves as miners had no concern about air quality, but enlightened societies made attempts to solve the problem of underground ventilation. The Greeks would build small fires at the bottoms of shafts for the warm air passing the tunnel to draw air out, causing a circulation. As the centuries passed, humans developed semimechanical means for furnishing air to the miners. Figure 16.1 is a sketch from *De Re Metallica*, written in the sixteenth century, of a man using a bellows to push breathable air into a tunnel. Note that the bellows has a heavy rock on top of it. The miner pushes down on the small log on the frame structure, which acts as a fulcrum that expands the bellows. When the miner releases the log, the weight of the rock depresses the bellows, moving the air through the pipe and into the tunnel.

Tunnels need to be ventilated to remove dust, toxic gases, explosive gases, and heat and to provide fresh breathable air. Ventilation can be natural, traffic induced, mechanical, or a combination of the three. Natural ventilation can be caused by a tunnel having at least two openings to the surface through which a natural flow of air occurs. Also, because of differences in gradient, temperature, tunnel cross section, or pressure, the movement of air can occur. Although it is not enough to adequately ventilate the heading, this movement can assist with ventilation. Traffic moving in the tunnel can cause aerodynamic drag or a piston effect. Like natural ventilation, traffic-induced ventilation is not enough to ventilate the face adequately. Mechanical ventilation refers to the use of fans to create air flow.

Ventilation continued to develop as the knowledge of hazards increased. One cannot tunnel or mine without ventilation, and yet in some cases it does not remove all of the hazards. For example, the removal of coal dust and methane in a coal mine is an obvious need and an easy one about which to realize the hazards, as they cause not only explosions but also black lung disease. Mines are ventilated and, although rare, explosions still occur. Another hazard is silicosis, a disease caused by crystalline silica dust. Like black lung, it is a killer. It usually results from long-term exposure (10 years or more) to relatively low concentrations of silica dust and does not appear for 10–30 years after

Figure 16.1 Early Mechanical Ventilation Using Bellows (Agricola, 1556)

first exposure. It can also be acute enough to develop in a few weeks to five years after exposure to high concentrations of silica dust. This is reduced with adequate ventilation, filtering, and using water as the medium for removing drill fines. Often it is still required to wear a filter mask to prevent the dust from entering the lungs. Another hazard that has been recognized during our modern history is radon. Most common in uranium ore, it can be found in some granites; there are very low levels of radon all over the earth's surface. Although hazardous levels are not common, the U.S. Environmental Protection Agency has designated radon as the second leading cause of lung cancer. Figure 16.2 shows a tunnel air filter.

There are three types of mechanical ventilation systems: longitudinal, transverse, and semitransverse. In the longitudinal system the air flows along the tunnel toward the heading. The air is fed at the portal or shaft and the air return is from the heading to the portal or shaft. This is the most common in tunnels. The amount of air flow is determined by the amount of air pushed by the fan minus the friction. In longer tunnels, it is often necessary to install booster fans along the ventilation conduit. With only one source of outside air,

Figure 16.2 Tunnel Air Filter

caution must be used to prevent the contamination of the air by dust, fumes, or other hazardous particulates. Therefore, care must be taken that there are no internal combustion engines operated at the intake location and no smoke- or fume-generating materials located near it.

Reversible systems blow fresh air to the face, with the air exhausting along the tunnel to the portal and sucking air from the face to vent the area, particularly after blasting.

Not generally found in tunnels because of the lack of laterals, transverse system fresh air is introduced into the tunnel along its length and is discharged through grills in the duct. The air is generally fed from the invert area and exhausted through ducts in the vent pipe near the crown. This approach is more common in mines.

The semitransverse system is a combination of the longitudinal and transverse systems. With this system, the fresh air is distributed along the entire tunnel length, and the contaminated air travels the length of the tunnel and is exhausted at the opening to the surface. Like the transverse system, the semitransverse system is used in the mining industry.

What factors must be considered when determining how much air has to be blown into the underground area? The factors are principally determined by the operational activities, that is, how many people will be working underground, the type of haulage (a diesel scoop tram generates the need for more ventilation than an electric train locomotive), other equipment, and heat-, dust-, and fume-producing activities. If blasting is being conducted, the amount and type of explosives used will determine the fumes generated. The tunneling method is a factor in determining air flow needs. For example, a TBM not only generates a lot of dust but also generates a lot of heat. TBMs are equipped with mechanisms to reduce dust, such as filters and spraying the face with water, but, although reduced, heat is still present. Also, drilling and blasting will produce a rougher wall than TBM tunneling, thereby increasing the friction to resist the air flow. The geologic conditions can have an effect with regard to dust as well as how the material generates dust when drilled and the gas and heat present. In some geologic areas, the heat of the rock and water requires additional ventilation for cooling. In some deep mines, such as in the Coeur d'Alene mining district in northern Idaho, air conditioning is required, and the mines still have hot environments. The length of the tunnel affects the length that the air must be pumped, increasing friction and requiring more energy to provide the face with the required air flow.

The U.S. Occupational Safety and Health Administration and Cal/OSHA (California) agree that there should be a minimum of 5.7 m^3/min (200 cfm) of air flow per person to the face and, because of comfort and reduced head losses, the air velocities should not exceed 450 m/min (1500 ft/min). The flow to the face should not be less than 18 m/min (60 ft/min) where drilling, blasting, or other dust-producing activities are conducted. However, an exception to this is for tunnels that are greater than 9 m (30 ft) in diameter, where the flow may be reduced to 9 m/min (30 ft/min).

Because of the fumes generated, diesel equipment has air flow requirements based on bhp (brake horsepower). The air flow requirements for new equipment are determined and published by the manufacturer. For older equipment, the regulatory agencies have directed that the minimum required air flow be 2.83 m/bhp (100 cfm/bhp). New equipment with more efficient air scrubbers and filters can greatly reduce the air flow requirements.

Once the air flow to the face is decided, the fan size is determined. This is the determination of the horsepower needed to push the required air from the free air source down the conduit to the face. The density of the air, the friction in the pipe, and the fan efficiency are used to make this determination.

FUNDAMENTALS OF AIR FLOW

The quantity of air flow is dependent on the difference in pressure at the start of the system and the end of the system and the size of the opening. Other factors are the roughness of the airway wall and the severity and number of times the air flow has to change direction. Loss in energy in the duct due to the roughness of the duct wall is called "friction pressure loss." Air flow is the pressure differential between two locations. The higher pressure will flow to the lower pressure. As the air moves through the duct, there are three laws of physics that affect it. The first is the conservation of mass, which states that matter cannot be created or destroyed. Only its form can change. The amount of air mass that arrives at a constriction, say a half closed valve, is equal to the air mass leaving the valve.

Another law of physics affecting ventilation is the law of conservation of energy. It states that energy can neither be created nor destroyed: It can only be transformed from one state to another. The total amount of energy in a system remains constant over time, that is, conserved over time. Therefore, the only thing that can happen to energy in a system is that it can change form: mechanical, electrical, magnetic, thermal, chemical, nuclear, and so on. As time progresses, it tends to become less and less available. As long as energy has not entered or left the system in the form of work or heat, there is no change in the total amount of energy. Finally, the conservation of momentum states that a body will remain in a constant state of rest unless it is caused to move by another force; so air will not move without something moving it.

The types of flow of air are laminar flow, parallel to the boundary layer, that is, the pipe, and turbulent flow, which is perpendicular and near the center of the duct. At low fluid velocities the stream lines of flow are almost parallel to each other. Shear resistance between the airstream lines is caused by the friction between the layers moving at different velocities. Frictionless or ideal flow, that is, flow with no resistance to movement, does not exist. Real flow is when shear movement of the fluid is resisted. However, the resistance can be so small that it is insignificant.

It is convenient to calculate pressure in airways using as a base an atmospheric pressure of zero. Supply ducts have positive pressure and return or exhaust ducts have negative pressures. Air flow through a ventilation duct system creates three types of pressures: static, dynamic (velocity), and total pressure. Each of these pressures can be measured. Air flowing through a duct causes both static and dynamic (velocity) pressure on the duct. Static pressure is the cause of most of the force on the duct walls. However, dynamic pressure introduces a rapidly pulsating load. Static and dynamic pressures are increased by the fan energy. Although fan ratings are based on static pressure only, they are still commonly used.

Pressure losses in the vent pipe have three components: frictional losses along the duct wall, dynamic losses in the fittings, and to a small degree, component pressure losses. Pressure loss is the loss of total pressure in a duct or a fitting. There are principally three observations that describe the benefit to using calculations and testing to determine total pressure loss rather than using just static head. Total pressure in the vent line always drops in the direction of flow, whereas static and dynamic pressures do not follow this rule. Air is a fluid that will constantly deform when subjected to shear forces. The energy level measurement in an airstream is represented by total pressure only. Pressure losses in a duct are the total pressure loss.

Ventilation constantly changes from potential energy (static pressure) to kinetic energy (velocity pressure), and the energy source must be maintained to ensure that the air continues to flow. That is, if the energy source is removed, the air flow will stop. Static pressure is the measurement of the potential energy of a unit of air in a particular cross section of an airway. The air pressure on the wall of the duct is considered static. To appreciate the concept, if one pictures a fan blowing into a closed duct, only static pressure is present because there is no air flow through the duct. Another example is of an inflated balloon. The balloon is expanded by static pressure only.

Static pressure is a measure of the potential energy of a unit of air at any cross section of the pipe. The air pressure that loads the wall of the pipe is considered static. To differentiate it from dynamic pressure, if a system is completely sealed off and the fan blows into the pipe and there is no flow of air through it, there is only static pressure.

As already stated, total pressure is the sum of velocity pressure and static pressure. It is convenient to calculate the pressure in the vent pipe, with atmospheric pressure being considered zero. Air flow through the vent pipe causes three types of pressure: static, dynamic (velocity), and total pressure. The static pressure is responsible for most of the force on the walls of the vent pipe. Dynamic pressure causes a rapidly pulsating load.

Dynamic (velocity) pressure is the kinetic energy of a unit of air flow in the airstream. The components of dynamic pressure are velocity and density. The denser the air or the greater the speed with which it flows, the higher the dynamic pressure is. Dynamic pressure can be determined by the equation

$$\text{Dynamic pressure} = P_d = (\text{density})(\text{velocity})^2/2$$

The flow velocity is determined by the local pipe cross section. Therefore static and dynamic pressures are mutually convertible.

The total pressure is the pressure the air applies in the direction of flow (velocity pressure) and the pressure the air exerts perpendicular to the pipe through which it is moving. Therefore,

$$P_t = P_v + P_s = \text{total pressure}$$

The most common influence on air density is the temperature and barometric pressure at sea level. The temperature used is 20°C (70°F), and the barometric pressure is 29.92 in. Hg (14.696 psi). The ratings found in fan performance tables and curves are based on standard air. To be defined as “standard,” the air must be clean and dry, with a density of 1.2 kg/m^3 (0.075 lb/ft^3) at sea level and 20°C (70°F). For other than these conditions, to operate a fan requires selecting a fan with adjustments to both static pressure and brake horsepower. The volume of nonstandard air is not affected, because the fan will move the same amount of air, regardless of the density. That is, if at 20°C the fan moves 150 m^3/min (5300 cfm), at 120°C (250°F) it will also move 150 m^3/min. However, since the density of the air is only 34% of the density at 20°C, it will require a fan with less brake horsepower, but the pressure generated will be less than required.

When a fan is specified for a certain flow rate and the static pressure and other conditions are not standard, correction factors must be applied to select the proper size and brake horsepower to meet the new conditions. As the air

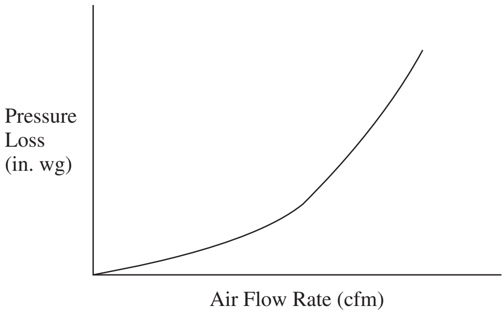

Figure 16.3 Relationship between Pressure Loss and Air Flow Rate

flow increases, the pressure loss increases exponentially. Figure 16.3 illustrates the relationship between pressure loss and flow rate.

Air cannot travel along the vent pipe or drift as fast as the fan can move that quantity of air because of back pressure; that is, the pipe or tunnel walls resist the movement. The static head is a direct result of this back pressure. The length of the pipe, reduction in cross-sectional area, and changes in direction increase friction. Because of the increase in friction resulting from bends and reduction of cross-sectional area, it is important that when using a vent bag, an effort should be made to prevent sags and twists in the bag.

A common fan is an axial type fan. Axial fans use vanes or propellers to pull the air into the fan and discharge it in the same axial direction. Figure 16.4 is a photo of an axial fan.

The friction, or resistance to flow, is also affected by an increase in the perimeter, or circumference, of the vent line, increasing it. Circular duct is preferred, because it has a smaller perimeter per unit of area; it is more efficient at moving air than a square or rectangular duct. Like any conduit, as the

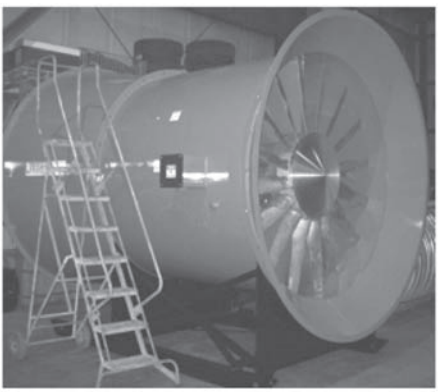

Figure 16.4 A 72-in. Axial Fan (Courtesy of Mining Equipment Ltd.)

Table 16.1 *K*-Factors for Various Types of Mine and Tunnel Airways

		Values of $K \times 10^{10}$											
					Sinuous or Curved								
		Straight			Slightly			Moderately			High Degree		
Type of Airway	Irregularities of Surfaces, Areas, and Alignment	Clean (basic values)	Slightly Obstructed	Moderately Obstructed	Clean	Slightly Obstructed	Moderately Obstructed	Clean	Slightly Obstructed	Moderately Obstructed	Clean	Slightly Obstructed	Moderately Obstructed
Smooth lined: (bored holes, lined shift)	Minimum	10	15	25	20	25	35	25	30	40	35	40	50
	Average	15	20	30	25	30	40	30	35	45	40	45	55
	Maximum	20	25	35	30	35	45	35	40	50	45	50	60
Sedimentary rock (or coal) (drifts)	Minimum	30	35	45	40	45	55	45	50	60	55	60	70
	Average	55	60	70	65	70	80	70	75	85	80	85	95
	Maximum	70	75	85	80	85	95	85	95	100	95	100	110

Source: Society for Mining, Metallurgy and Exploration, Inc.

roughness of the vent increases, so does the friction. Table 16.1 consists of various *K*-factors, which are the variables used to factor the roughness. Also, the friction of an airway increases as the length increases. As the volume increases, the effects of the friction increase; therefore, since velocity is a direct result of volume, an increase in velocity will increase resistance to flow.

The static head in inches of water gage using all contributing factors can be calculated by the following:

$$H_s = (KPLQ^2)/(5.2A^3)$$

where H_s = static head in inches of water gage
K = roughness factor (K-factor)
P = perimeter (circumference) in feet
L = airway/duct length in feet
Q = air quantity in ft^3/min
A = area of airway/duct in ft^2

where 1 in. of water = 249.089 Pa (0.25 kPa)

Velocity head is the pressure created by the movement of the air. If you approach the face and walk in front of the vent pipe and suddenly your hardhat is thrown toward the face, then that is velocity head. It is also described as the pressure one can feel on the face.

Table 16.2 Fan C K-Values for Steel and Flexible Duct Fixed-Pitch Fan

Type of Duct	New-Duct K-factor	Used-Duct K-factor
Steel vent tubing	15×10^{-10}	20×10^{-10}
Flexible ducting	20×10^{-10}	25×10^{-10}– 75×10^{-10}, depending upon quality of installation
Ventilation borehole	15×10^{-10}	25×10^{-10}

Source: Society for Mining, Metallurgy and Exploration, Inc.

The velocity head in inches of water gage is calculated by the following:

$$H_v = (V/4005)^2$$

where H_v = velocity head in inches of water
V = air velocity in feet per minute

The K-factor is the value given to roughness for different kinds of airways. Table 16.2 provides various K-factors reflective of the conduit roughness.

The velocity in the duct may be determined by dividing the air flow by the cross-sectional area of the duct, or airway, or it may be obtained by using instruments such as an anemometer, a device used for determining wind speed, or a Pitot tube, which measures air flow. The Pitot tube is commonly used on aircraft to measure airspeed.

To calculate the combined (total) head, the following equation can be used:

$$\begin{aligned} H_c &= H_s + H_v \\ &= [(KPLQ^2)/(5.2A^3)] + [(Q/A^2)/(4005)] \end{aligned}$$

Pressure in vent lines is usually expressed as water gage (w_g) and water column (w_c). A water gage reading of 27.7 in. is equal to 6.89 kPa (1 psi). One inch of water gage equals 0.249 kPa (5.2 psf) where 4500 is a constant for round duct.

It is better to use total pressure for duct calculations rather than a static pressure measurement only. The flow of any fluid has pressure losses due to resistance to the flow. Pressure loss is the loss of pressure in a duct or a fitting. Static pressure is the pressure on the discharge side of the fan system; that is, it is the sum of all resistances that the fan must work. Dynamic pressure is the result of changes in the flow's direction and velocity. The energy need is determined using total pressure, because the total pressure in ductwork always drops in the direction of flow. This principle is not followed by static or dynamic pressures alone. Also, the energy level measurement of the air flow is represented by the total pressure only, and the pressure losses in the duct are represented by potential and kinetic energy loss combined, that is, the total pressure loss.

When the airway changes shape or direction, the airstream loses energy as a result of "shock loss." In vent pipes or tubing, acute bends and pinches are the most common cause of shock loss, especially in flexible duct pipe. Shock losses are the result of changes in the direction and velocity of the air flow. Shock losses occur whenever the airstream makes any turn or diversion. In tunnel ventilation, bends and lack of straightness of the vent line are the most common examples. If flexible pipe or tubing is used, the friction is generally greater. The velocity streams are reorganized at these places by the development of vortexes that cause the conversion of mechanical energy into heat. The disruption of the flow stream begins before it reaches the diversion and ends some distance once it passes, that is, more than six diameters past. Shock loss calculations generally equate the friction losses to additional pipe length in the original equation. Table 16.3 gives the lengths for various sources of shock loss.

Fan Characteristic Curves

Fan characteristic curves illustrate the relationship between the loads that are imposed on the fan and a fan's output in terms of the amount, that is, volume,

Table 16.3 Equivalent Lengths for Various Sources of Shock Losses

	Lengths	
Source of Shock Loss	m	ft
Bend, acute, round	0.9	3
Bend, acute, sharp	45.7	150
Bend, right, round	0.3	1
Bend, right, sharp	21.3	70
Bend, obtuse, round	0.2	0.5
Bend, obtuse, sharp	4.6	15
Doorway	21.3	70
Overcast	19.8	65
Entrance	0.9	3
Discharge	19.8	65
Contraction, gradual	0.3	1
Contraction, abrupt	3.0	10
Expansion, gradual	0.3	1
Expansion, abrupt	6.1	20
Splitting, straight branch	9.1	30
Splitting, deflected branch, 1.57 rad (90°)	61.0	200
Junction, straight branch	18.3	60
Junction, deflected branch, 1.57 rad (90°)	9.1	30
Mine car or skip (20% of airway area)	30.5	100
Mine car or skip (40% of airway area)	152.4	500

Source: Society for Mining, Metallurgy and Exploration, Inc.

of air delivered per unit of time. As the load increases, the output decreases. The load is primarily due to friction between the airstream and the walls of the airway. Load is the power necessary to overcome the resistance to flow, that is, the "back pressure" of the vent line or drift. The three main components of a fan characteristic curve are the flow volume, the static pressure, and the brake horsepower.

Pitch is the blade setting and is one of the factors in determining fan performance. A fixed pitch allows only one blade setting.

The relationship that links primary variables associated with its operation is the most important characteristic of a fan. The relationship between the pressure rise and volume flow rate for a constant impeller speed (rpm) is the most commonly used fan characteristic. The most commonly used system characteristic is the relationship between pressure loss and volume flow rate.

Fan pressure rise characteristics are expressed as total pressure or, mostly in the United States, static pressure. The fan volume output is measured in cubic feet per minute (cfm). Typically, system pressure losses and volume flow rate are expressed as static pressure at some cfm. See Figure 16.5.

Figure 16.5 illustrates a fan pressure–volume curve from a laboratory test of a vane axial fan varying the flow from a no-flow "block-off" condition to full flow. Comparing the *x* (horizontal) with the *y* (vertical) axis, it becomes apparent that the greater the pressure, the lower the flow. Like our earlier example of a balloon, the static pressure is greatest when the air is not flowing. As the air flow increases, as represented by the *x* axis moving to the right, the lower the static pressure is. Therefore, if the flow is 5.5 cfm, the static pressure is 1. Point *A* is the maximum static pressure, point *D* is zero static pressure

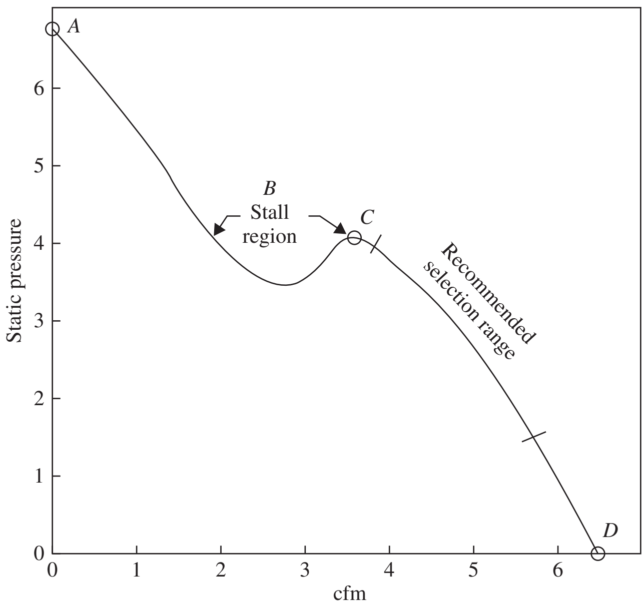

Figure 16.5 Typical Static or Vane Axial Fan (Adapted from AMCA)

and air flow, and point *C* is the maximum air flow. A line drawn parallel to the *x* axis from a point on the *y* axis to where it intercepts the curve and then a vertical line drawn to the *x* axis gives the relationship between the static pressure and the air flow. For example, a horizontal line is drawn from 2 on the *y* axis. A vertical line is drawn from where the horizontal line intersects the curve. Following the line to the *x* axis yields the value of 5. Thus, a static pressure of 2 yields a flow of 5 cfm.

Once the static pressure has been determined and the air flow is given in cfm, the operating brake horsepower can be determined. The fan performance curve can be completed by adding the bhp curve to the static pressure curve in Figure 16.6. By extending vertically the cfm point established in Figure 16.6 until it intersects the bhp curve, a horizontal line is drawn from this point to the right to the bhp scale to establish the bhp needed. The value of 32.8 kW (44 hp) corresponds to the previously determined performance of 640 m^3/min (22,600 cfm) at 2.48 kPa (10 in water gauge) static pressure.

The fan and system are in a stable equilibrium when the fan pressure rise (SP)/volumetric flow rate is cfm at the operating point; that is, the SP/CFM is on the curve that represents the point of operation or design point. This curve will have all the possible combinations of SP/CFM. This corresponds to the state where the fan SP/cfm characteristic intersects the system pressure loss/flow rate characteristics.

On occasion, the fan system does not operate according to the design. For example, the fan system may be delivering too much air or not enough. This is resolved by slowing down or increasing the fan speed depending on the situation.

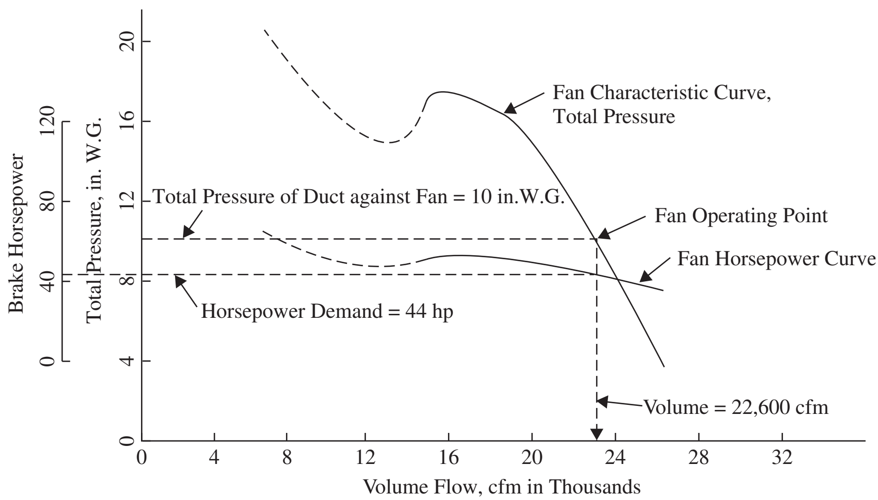

Figure 16.6 Fan-c Characteristic curve Curve for fixedFixed-pitch Pitch fanFan (Courtesy of Society for Mining, Metallurgy and Exploration, Inc.)

The following fan laws can be applied because the fan must operate somewhere along the system curve and therefore the fan performance at other speeds can be predicted: the volumetric flow (cfm) varies with the rpm, the static pressure (SP) varies as the rpm^2, and the bhp varies as the rpm^3.

Figure 16.6 shows the characteristic curve for an axivane mine fan with a fixed pitch allowing only one blade setting. Therefore, the figure has only one fan curve (the upper curve). The fan produces a certain quantity of air at any pressure. Using the graph as an example, assume there is a 305-ft (100-m) ventilation duct in a tunnel with a 37.3-kW (50-hp) fan at the free air end of the duct. From measurements taken with a manometer, it is determined that the total pressure against the fan is 2.48 kPa (10 in. wg). Using Figure 16.6, the amount of air being produced by the fan is determined by first finding the value of 10 in. wg on the left side of the graph. Then, follow a line horizontally across the graph until it intersects the fan's characteristic curve. The point where the lines intersect the fan curve is the operating point of the system and is the point where the capabilities of the fan match exactly the requirements of the particular airway and pressure. That is, the power of the fan exactly matches the load on the fan by the friction in the duct; the output is stable.

Now that the operating point has been located, the volume of the flow of air can be determined by moving straight down the graph to the *x* axis. The value at this intersection is the quantity of air flow. For the example, the value is 10.7 m^3/sec (22,600 cfm). This value is the volume of air that will be produced given the fan and duct.

Part of the fan curve is shown as a solid line and part as a broken line. The broken line represents the stall region of the fan; it is an area of very inefficient operation. Air flow insufficient to satisfy the "bite" requirements of the fan blades is the cause of the stall. The acute dip in the curve is referred to as the "stall trough," and no fan should be operated in that region. Not only does the stall trough waste substantial electric power, but it also can damage the fan. Obviously, systems should not be designed with the fan operating in the stall region.

The second curve is the "brakepower" curve. It shows the power consumed at different output rates. The power needed to pass a certain volume of air can be determined by returning to the operating point for the fan characteristic curve and drawing a vertical line from that point to the power curve. In our example, the power required is determined to be approximately 32.8 kW (44 hp) to affect the movement of 10.7 m^3/sec (22,600 cfm) of air.

Corresponding to the stall region of the fan characteristic curve, the power curve has a broken line. This represents the region of power demand when the fan is operating in the stall region. The demand of power increases considerably as the fan stalls. It rises to 67.1 kW (90 hp) in the totally stalled condition that is known as the "shutoff." The maximum pressure that a fan can develop when blowing into a blocked duct is the shutoff pressure.

The behavior of fans varies, because they have many blade settings, with each setting having its own fan characteristic and power curves. For example,

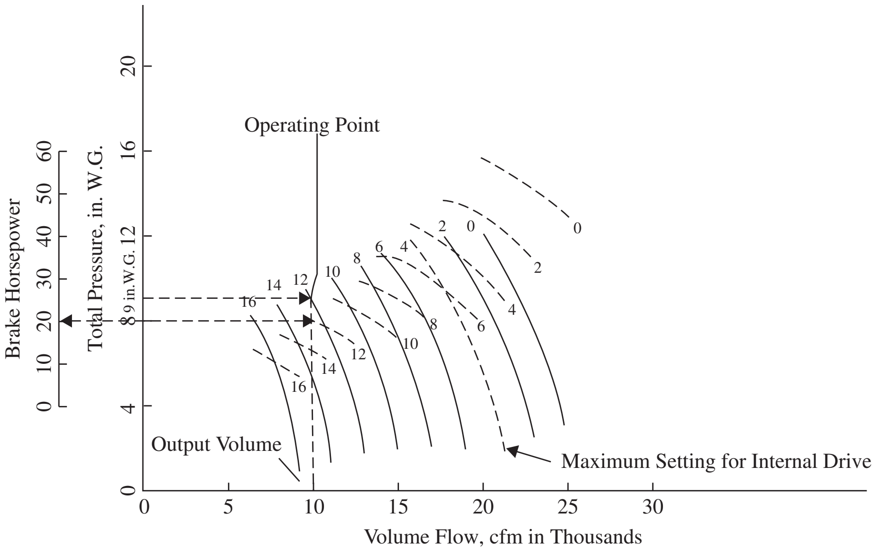

Figure 16.7 Fan Characteristic Chart for Multipitch Fan (Courtesy of Society for Mining, Metallurgy and Exploration, Inc.)

the graph in Figure 16.7 illustrates a chart for a 14.9-kW (920-hp) fan. The blade diameter for this fan has 641 mm (25.25 in.), a hub diameter of 445 mm (17.5 in.), and 361 rads/sec (3450 rpm) rotational speed. There are separate characteristic and power curves for each blade setting. In Figure 16.7, the characteristic curves are solid lines, whereas the power curves are broken lines. A single fan can be adapted to a wide range of applications by using different blade settings. The power and physical size of the motor and fan casing limit the blade settings. To illustrate, a 149-kW (20-hp) motor is limited in the amount of air without exceeding the power limits of the motor. The fan can operate within that region with certain blade settings, yet others cause the fan to operate outside that range.

Figure 16.7 is an example. If a 14.9-kW (20-hp) fan is installed onto a blower bag and from the Pitot and manometer measurements it is learned that the total pressure in the bagging is 2.24 kPa (9.0 in. wg), this is the load against which the fan has to push. The blades are at a setting of No. 12, and on the scale, the total pressure of 2.24 kPa (9.0 in. wg) is located and a line is drawn horizontally along this level until the solid curve marked “12” is intersected. That point, the operating point of the fan and bagging system, is the point at which the demands of the system are met exactly by the capabilities of the fan. A line drawn straight down from the operating point intersects the output volume at 4.58 m^3/sec (9700 cfm). The line also passes through a broken line numbered “12.” The broken line is the power curve for a blade setting of No. 12 at 2.24 kPa (9.0 in. wg), and 4.58 m^3/sec (9700 cfm) requires the power developed of 14.7 kW (19.7 hp).

To reduce clutter and confusion by having too much information, some parts are not shown on a graph. This is especially true when various blade settings are on the graph. Fans should not be operated in a system that requires higher pressure than indicated for a specific blade setting. No. 12 is the setting in the fan characteristic curve in Figure 16.6 and should not be used if a system requires 2.49 kPa (10 in. wg), because the fan curve goes only to 2.30 kPa (9.25 in. wg). The allowable operating ranges at each setting that should not be exceeded are represented by the curves.

An example of what happens if the blade setting used is too high can be demonstrated if one assumes that the fan in Figure 16.7, instead of having a setting of No. 12, has a setting of No. 8. If the fan were operating against a pressure head of 1.99 kPa (8.0 in. wg), as illustrated in Figure 16.7, the operating point would be 6.89 m^3/sec (14,600 cfm), but the increased air flow would require an increase in power to 194 kW (26 hp), which is a 30% increase in power required over 14.9 kW (26 hp). The fan operating with a fan blade setting of No. 8 would require 22.4 or 29.8 kW (30–40 hp). This additional power requirement would require a bigger fan motor that may be installed in the current casing if it fits. If it does fit, the area between the casing and the fan would decrease, requiring too much power to push the air through the fan.

The importance of properly sizing the fan is obvious. This includes the need to build air flow systems with maximum efficiency, which, in addition to the fan, is dependent on the conduits through which the air is moved. The line should be as straight and as clear of fittings and turns as is practical. The better the efficiency is, the greater the life of the fan and the more reduced the energy costs. The importance of being energy efficient is increasing yearly. Between environmental restrictions on energy development and political interference, the cost of energy is going to increase greatly, increasing the need for cost-saving energy conservation on a project.

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17 Tunnel Lining

There are various reasons for lining a tunnel: to maintain the tunnel shape, that is, to support the ground; to keep fluids in or keep fluids out of the tunnel; or to provide a certain friction coefficient for the flow of fluids.

TUNNEL LINING There are two classes of linings: primary (initial) and secondary (final). The primary lining is the ground support that is installed immediately after excavation and provides a temporary structure for safety and to maintain tunneling operations. Normally, the types of support used with hard rock tunneling include steel sets, rock dowels, and shotcrete.

Although the linings vary in materials and approaches used, they share certain characteristics (Bickel et al., 1996):

- The processes of ground pretreatment, excavation, and ground stabilization alter the preexisting state of stress in the ground, before the lining comes into contact with the ground.
- A tunnel lining is not an independent structure acted upon by well-defined loads, and its deformation is not governed by its own internal elastic resistance. The loads acting on a tunnel are ill defined, and its behavior is governed by the properties of the surrounding ground. Design of a tunnel lining is not a structural problem, but a ground-structure interaction problem, with the emphasis on the ground.
- Tunnel lining behavior is a four-dimensional problem. During construction, ground conditions at the tunnel heading involve both transverse arching and longitudinal arching or cantilevering from the unexcavated face. All ground properties are time-dependant [sic], particularly in the short term, which leads to the commonly observed phenomenon of stand-up time, without which most practical tunnel construction methods would be impossible. The timing of lining installation is an important variable.
- The most serious structural problems encountered with actual lining behavior are related to absence of support-inadvertent voids left behind the lining—rather than to intensity and distribution of load.
- In virtually all cases, the bending strength and stiffness of structural linings are small compared with those of the surrounding ground. The properties of the ground control the deformation of the lining, and changing the properties of the lining will not significantly change this deformation. The proper criterion for judging lining behavior is therefore not adequate strength to resist bending stresses, but adequate ductility to conform to imposed deformation. In short, the lining is a confined flexible ring.

Normally for cast-in-place concrete, the final, or secondary, lining follows the primary support; that is, it is independent of the excavation.

The materials that have been used for tunnel linings are wood, masonry and stone, cast iron, steel, and concrete. These materials may form linings that are put in one piece at a time, such as wooden boards, bricks, or pipes, or segmental, sprayed, or cast-in-place concrete.

Although rarely used today as a permanent lining, wood was sometimes used as the primary support in a tunnel and then left as the final lining. This would have occurred more in mining than in civil tunnels. It would have been in the form of boards and timber blocks and logs. Although wood can be installed fairly easily, it is not durable and will eventually decay.

Although liner plate is a primary lining, it is sometimes left in place as the only liner. The most common occurrence is in short tunnels (less than 30 m) that carry pipes and other conduits. One must be aware of corrosion if the liner plate is not treated, but it is a relatively inexpensive way to support this type of tunnel.

At one time, bricks and stones were extensively used for lining in water, sewer, and railroad tunnels, and can still be found in older railroad tunnels. They were laid in multiple courses or layers, generally with an arch shape. This type of lining was very labor intensive, and with labor costs today, it would be prohibitive to use, plus it is less effective than modern methods.

With the advent of the shield in soft ground tunneling, it became necessary that the lining have the strength to withstand the pressure of the thrust jacks pushing against the shield to advance the machine. This requirement for a tunnel lining to be strong enough for jacking pressures and the need for faster lining installation made way for the use of cast iron segments. These segments varied in size and were placed with the use of an erector arm. Figure 17.1 is a rendering of a cast iron segment erector on the St. Clair Tunnel.

Ductile iron, also known as “ductile cast iron,” is a type of cast iron that was invented because most varieties of cast iron are brittle. Ductile iron is much more flexible and elastic, due to its nodular graphite inclusions. However, cast iron segments are still in use. On the Chunnel project, cast iron segments were used in addition to high-strength concrete segments. Figure 17.2 is a photograph of two sandhogs tightening bolts of segments in the Midtown Tunnel in New York in the early 1930s. Note the push cylinders in the background and segment bolts. (Author’s note: This photo has some extra value to me, as my father was a sandhog on this project.)

Ductile iron segments provide a greater strength-to-weight ratio than cast iron segments, have been used for the past 30 years, and now are used for lining vast segments of the London Underground network. With its inherent strength and impact resistance over concrete segments, ductile iron is commonly used in the opening sets and an assortment of shaft functions, such as for lifts, escalators, ventilation, and sumps. Consequently, the tunnelling industry has seen continued development of segmental lining systems from brick to cast iron and steel to the precast concrete systems in common use today.

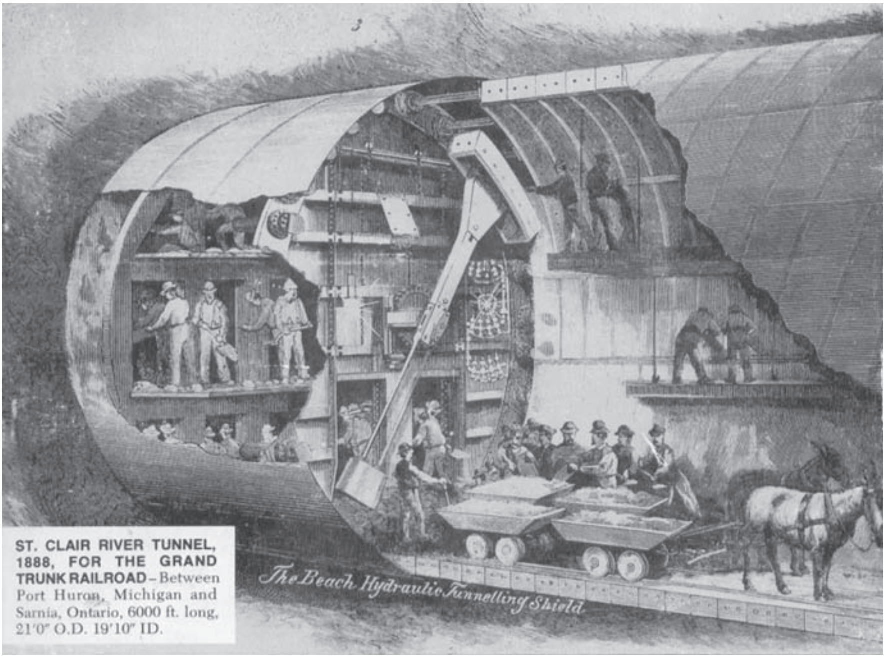

Figure 17.1 Cast iron Segment Erector on the St. Clair Tunnel

Figure 17.2 Tightening Segment Bolts in the Midtown Tunnel (Photographer Michael Bobco for Somach Photo Service, Feb. 26, 1939. Courtesy of MTA Bridges and Tunnels Special Archive)

Pipe can be either precast concrete or steel. With fewer joints, there will be less propensity for leaks than with segmental linings, but it can be more difficult to handle. Generally, the tunnel cross section is larger to facilitate the installation of the pipe. The larger cross section will result in the need of more grout in the annulus between the pipe and the tunnel. However, it can have excellent flow characteristics and may be necessary due to internal pressure or outside water pressure. The internal pressure can be the result of pumping fluid, such as water, or gravity head as with a penstock. Great external pressures can be the result of very deep passage through a mountain with a much higher water table.

SHOTCRETE

Shotcrete is often used because of its ease of installation. The shotcrete may cover lattice girders for additional strength. With a good nozzle operator, shotcrete can be applied, filling overbreak, leaving a fairly smooth surface. Shotcrete is much less expensive that cast-in-place concrete and can be completed much faster. However, a concrete invert is usually used with a shotcrete lining. For more in-depth discussion of shotcrete, see Chapter 12.

ONE- AND TWO-PASS LINING

Generally comprised of precast concrete segmental lining, one-pass lining is used as both initial and final lining. Two-pass lining is installed after the temporary ground support. With two-pass segmental lining, the primary lining is precast concrete segments and the final lining is cast-in-place concrete.

Concrete Segments

Like cast iron and ductile iron segments, precast tunnel liner segments are used where the cylinders push off from to drive the shield. The segments are specially engineered and can be fabricated to virtually any diameter and service application. There are generally six or more per ring, and they are assembled and placed in the tail of the TBM as it excavates the tunnel. With a minimum practical diameter of 3.5 m (12 ft), liner segments are generally practical only when the tunnel diameter is too large for precast jacking pipe.

A ring is made up of segments that perform as interlocking structural elements that support the ground. The ring is a continuous structure that is self-supporting and generally has a circular shape.

The principal reasons for using precast concrete segments are soft ground tunnel, difficult ground conditions in a hard rock tunnel, that is, laminated shale embedded with clay, and conditions below the water table. In addition, the use of precast concrete segments can be driven by schedule. Although generally more expensive than a two-pass system, using precast concrete segments for a one-pass lining can save considerable time. Also, with the threat of high groundwater infiltration, the segments can have gaskets to withstand high water heads.

The segments are designed based on temporary and permanent loads. Temporary loads are the loads imposed prior to them being completely installed. They include storage/stacking, demolding, transportation, handling, erection, and grouting pressures. Permanent loads include traffic, adjacent foundation or pile loads, internal pressure such as water under the head, nearby construction such as adjacent tunnel construction, and floatation, or buoyancy, forces. Often the construction loads are greater than the permanent loads. Figure 17.3 illustrates water leaking through (repairable) segment damage during installation.

Segments are installed by an erector that can be a mechanical connection or vacuum. Figure 17.4 is a photograph of a vacuum erector. Tunnel segments are installed under the tail shield, which provides protection from the ground. The segment is fabricated with tapered rings that permit it to adapt itself to a specific curvature. Other than some expanded linings that have no structural connections, segmental linings may be connected by bolts or dowels.

Segments have included ethylene propylene diem monomer (EPDM) gaskets, which are a synthetic rubber gaskets installed in a groove at all mating faces near the extrados of the segment. The gasket is critical when waterproofing the lining. (See Figure 17.5.)

Figure 17.6 is a sketch representing the joining of two segments. Things of note are the EPDM gasket, adjacent to which is a hydrophilic gasket. The segments are held together using joint bolts.

Figure 17.3 Leakage through Cracks in Segments

Figure 17.4 Photograph of a Vacuum Erector

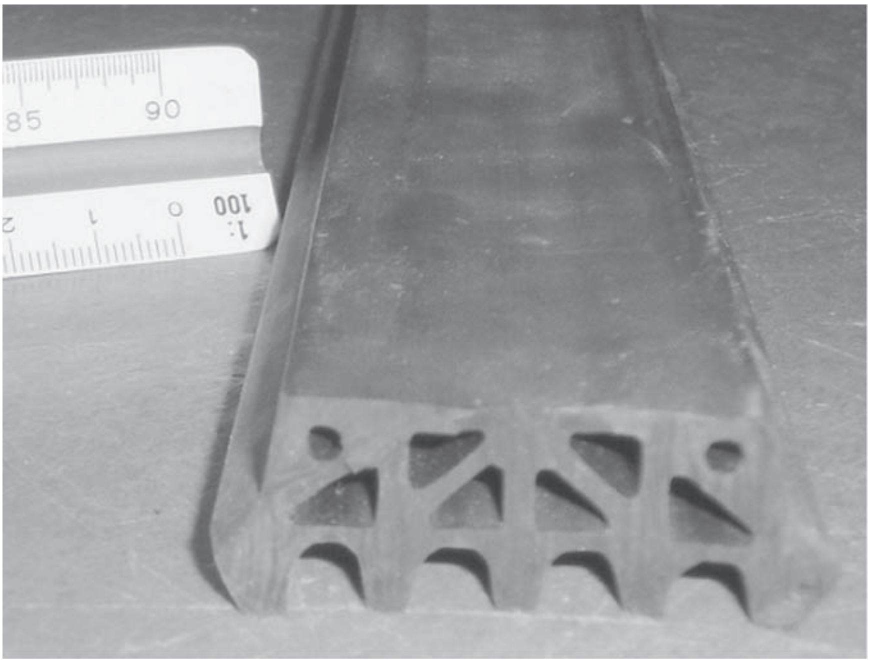

Figure 17.5 Photograph of a Gasket

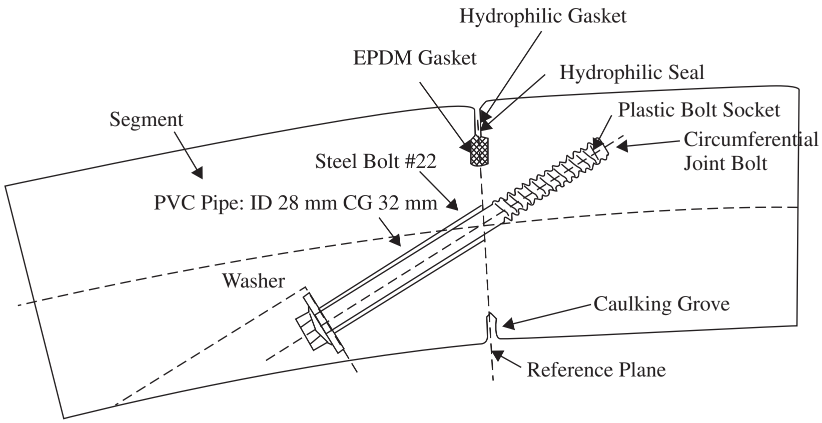

Figure 17.6 Bolted and Gasketed Segments (O'Carroll, 2005)

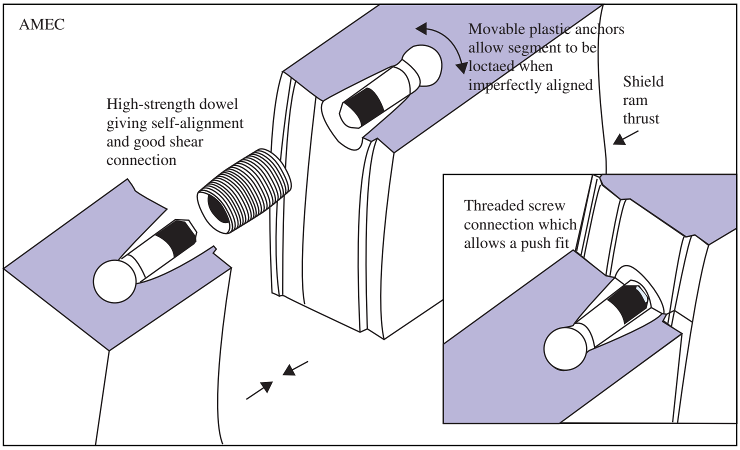

Figure 17.7 Circumferential Joint Dowel Connectors (O'Carroll, 2005)

To maintain lateral positioning and shear strength, a high-strength dowel is placed at the joint. See Figure 17.7.

The photograph in Figure 17.8 illustrates segments with holes cast for installation of segment bolts.

Cast-in-Place Concrete

Cast-in-place concrete is as the name implies: The forms are placed at lining locations, and the forms are filled with concrete. There are various types of forms based on their utility. Generally speaking, the cast-in-place method requires pouring the invert first. The form is placed on the invert and secured.

Figure 17.8 Bolt holes in segment

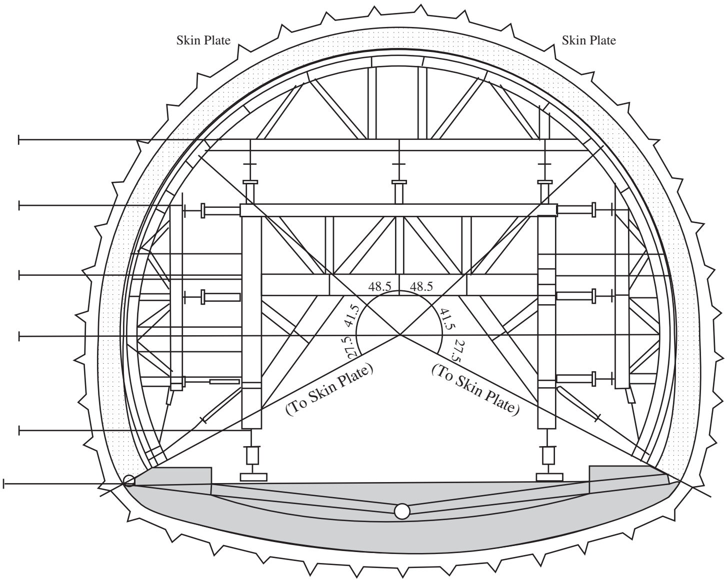

Figure 17.9 Horseshoe-Shaped Form

Because the invert slab serves as the foundation for the tunnel form, it is imperative that the invert be poured to the design elevation. Figure 17.9 illustrates a tunnel form that is placed on a poured invert. As one can see in the photograph, with a horseshoe-shaped form, the form must be well braced to prevent the sides from moving inward. If the invert is poured correctly, the elevation should be relatively easy to set. However, one still has to ensure that the line is correct. This is accomplished by careful surveying of the form installation. Although a properly poured invert is a good start, the remainder of setting the forms should be surveyed in, both for its initial set and then when bracing the form prior to the pour.

If the perimeter of the lining is poured including the invert pour it's part of the pour, as when using a cylindrical form, it will require not only placing the form on supports or some type of brace, as one normally would, but also, it is necessary to stabilize the form against the crown, because the form will float if not properly secured. The pour should be slow as practical, without allowing the concrete to set up, resulting in the causing of a cold joint. Also, both sizes should be poured simultaneously, alternating between sides, to maintain the same level of concrete to equalize the lateral forces against the form. Also, the crown has to be supported well, to reduce the stress on the form caused by the propensity of the form to float and to prevent lateral movement of the forms.

The concrete can be steel fiber reinforced or rebar. Steel-fiber-reinforced concrete is cast or pumped directly into the forms. It is well suited as a final lining when there are flexural and tensile stresses in the lining requiring it and it permits the traditional steel bar reinforcement.

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