Introduction to

TUNNEL CONSTRUCTION

David Chapman, Nicole Metje and Alfred Stärk

APPLIED GEOTECHNICS VOLUME 3

Spon Press

Introduction to Tunnel Construction

Applied Geotechnics

Titles currently in this series:

Geotechnical Modelling
David Muir Wood
Hardback ISBN 978–0–415–34304–6
Paperback ISBN 978–0–419–23730–3

Sprayed Concrete Lined Tunnels
Alun Thomas
Hardback ISBN 978–0–415–36864–3

Forthcoming:

Practical Engineering Geology
Steve Hencher
Hardback ISBN 978–0–415–46908–1
Paperback ISBN 978–0–415–46909–8

Landfill Engineering
Geoff Card
Hardback ISBN 978–0–415–37006–6

Particulate Discrete Element Modelling
Catherine O'Sullivan
Hardback ISBN 978–0–415–49036–8

Advanced Soil Mechanics Laboratory Testing
Richard Jardine et al.
Hardback ISBN 978–0–415–46483–3

Introduction to Tunnel Construction

David Chapman, Nicole Metje
and Alfred Stärk

Spon Press
an imprint of Taylor & Francis

LONDON AND NEW YORK

First published 2010
by Spon Press
2 Park Square, Milton Park, Abingdon, Oxon OX14 4RN

Simultaneously published in the USA and Canada
by Spon Press
270 Madison Avenue, New York, NY 10016, USA

This edition published in the Taylor & Francis e-Library, 2010.

To purchase your own copy of this or any of Taylor & Francis or Routledge's
collection of thousands of eBooks please go to www.eBookstore.tandf.co.uk.

*Spon Press is an imprint of the Taylor & Francis Group,
an informa business*

This publication presents material of a broad scope and applicability. Despite stringent efforts by all concerned in the publishing process, some typographical or editorial errors may occur, and readers are encouraged to bring these to our attention where they represent errors of substance. The publisher and author disclaim any liability, in whole or in part, arising from information contained in this publication. The reader is urged to consult with an appropriate licensed professional prior to taking any action or making any interpretation that is within the realm of a licensed professional practice.

The authors have gone to every effort to seek permission from and acknowledge the sources of images which appear in this publication, that have been previously published elsewhere. Nevertheless, should there be any cases where copyright holders have not been correctly identified and suitably acknowledged, the authors and the publisher welcome advice from such copyright-holders and will endeavour to amend the text accordingly on future prints.

British Library Cataloguing in Publication Data
A catalogue record for this book is available from the British Library

Library of Congress Cataloging-in-Publication Data
Chapman, David N.
Introduction to tunnel construction/David N. Chapman,
Nicole Metje, and Alfred Stärk.
p. cm. – (Applied geotechnics)
Includes bibliographical references and index.
1. Tunneling. 2. Tunnels – Design and construction.
I. Metje, Nicole. II. Stärk, Alfred. III. Title.
TA805.C45 2010
624.1′93 – dc22 2009044487

ISBN 0-203-89515-0 Master e-book ISBN

ISBN10: 0–415–46841–8 (hbk)
ISBN10: 0–415–46842–6 (pbk)
ISBN10: 0–203–89515–0 (ebk)

ISBN13: 978–0–415–46841–1 (hbk)
ISBN13: 978–0–415–46842–8 (pbk)
ISBN13: 978–0–203–89515–3 (ebk)

Dedicated to Professor Reinhard Rokahr who provided the inspiration and first introduced some of us to the eldorado of tunnelling.
Also dedicated to our families.

Contents

Preface

This book seeks to provide an introduction to tunnel construction for people who have little experience of the subject. Tunnelling is an exciting subject and is unlike any other form of construction, as the ground surrounding the tunnel is an integral part of the final structure and plays a pivotal role in its stability. The 'art' of tunnelling cannot be learnt purely from books and a lot of essential decisions are based on engineering judgement, experience and even emotion. There is often no single answer to any question: often the response has to be 'it depends'.

So how can this book help the reader to understand tunnelling? The aim of the book is to provide the reader with background information so that he or she can either make an informed decision and/or consult more specialist references on a specific topic. It will hopefully give the reader the tools needed to critically assess tunnel construction techniques and to realize that not all can be learnt from textbooks. In addition, the book hopes to demonstrate the breadth of the subject and that to become a tunnelling expert, many years of experience are required. At the same time, the book hopes to show the reader the excitement associated with tunnelling and the fact that many unknowns exist which require engineering judgement.

Disclaimer

While every effort has been made to check the integrity and quality of the contents, no liability is accepted by either the publisher or the authors for any damages incurred as the result of the application of information contained in this book. Where values for parameters have been stated, these should be treated as indicative only. Readers should independently verify the properties of materials they are dealing with as they may differ substantially from those referred to in this book.

This publication presents material of a broad scope and applicability. Despite stringent efforts by all concerned in the publishing process, some typographical or editorial errors may occur. Readers are encouraged to bring these to our attention where they represent errors of substance. The publisher and authors disclaim any liability, in whole or in part, arising from information contained in this publication. Readers are urged to consult with an appropriate licensed professional prior to taking any action or making any interpretation that is within the realm of a licensed professional practice.

Acknowledgements and permissions

The authors would like to express their deep gratitude to their colleagues at the Institute of Tunnelling and Underground Construction (IUB), especially Professor Reinhard Rokahr without whose support and encouragement this book would not have materialised. Special thanks also go to Dr Donald Lamont who contributed to the health and safety section of this book, Dr Alexander Royal for his contribution to the sections on pipe jacking and horizontal directional drilling, Graham Chapman for reading through some of the manuscript and Qiang Liu for producing some of the figures. The authors would also like to thank all those people who reviewed the book critically before it went to print and thus making the book better for it, especially Dr Douglas Allenby (BAM Nuttall Ltd), Martin Caudell (Soil Mechanics), Dr Michael Cooper, Colin Eddie (Underground Professional Services Ltd), Robert Essler (RD Geotech Ltd), Dr Dexter Hunt (University of Birmingham), Christian Neumann (ALPINE BeMo Tunnelling GmbH Innsbruck), Dr Barry New (Geotechnical Consulting Group), Casper Paludan-Müller (Cowi A/S), Roy Slocombe (Herrenknecht UK), Dr Alun Thomas (Mott MacDonald) and Dr-Ing. Rudolf Zachow (IUB, Hanover University).

The authors would like to acknowledge the following people and organizations who have assisted and/or kindly granted permission for certain figures, tables and photographs to be reproduced in this book:

Companies and persons who gave permission to use photographs, figures and tables (acknowledged in the text):

- Aker Wirth GmbH
- ALPINE BeMo Tunnelling, GmbH Innsbruck
- Atlas Copco
- Bachy Soletanche Ltd
- BAM Nuttall Ltd. and John Ropkins Ltd
- John Bartlett
- Dr Nick Barton
- Dr John Billam
- British Drilling & Freezing Co. Ltd
- David Caiden
- Professor E.J. Cording
- COWI A/S
- Dosco Overseas Engineering Ltd
- Don Deere
- Dyno Nobel Inc.
- Geopoint Systems BV

- Herrenknecht GmbH
- Peter Jewell
- Dr Ron Jones
- Mike King
- London Underground Ltd
- Lovat
- Professor Robert Mair
- Massachusetts Turnpike Authority
- Mitsubishi Heavy Industries Mechatronics Systems Ltd
- NoDig Media Services
- Prime Drilling HDD-Technology
- The Robbins Company
- Professor Dr-Ing. habil. Reinhard B. Rokahr
- Rowa Tunnelling Logistics
- Dr Alexander Royal
- Alex Sala
- Soil Mechanics
- Wilde FEA Ltd

Those who granted us permission to use figures and tables (in addition to those acknowledged in the main text):

- Figures 2.6, 2.12, 2.13, 2.16, 2.17, 2.18, 2.19, 2.20, 3.1, 3.5, 3.6, 3.7, 4.17, 5.3, 5.12, 5.20, 5.38, 5.39, 5.42, 5.43, 5.44, 5.45, 5.46, 5.48, 7.11, 7.12, 7.13, 7.23 and Table 1.1: Institute of Tunnelling and Underground Construction, Hanover University, Germany
- Figures 2.2a, 2.2b and 2.3: from *Transportation Research Circular E-C130: Geophysical Methods Commonly Employed for Geotechnical Site Characterization*, Transportation Research Board of the National Academies, Washington, DC, 2008, Figures 2a and b (p. 6); Figure 3a (p. 7); and Figure 4a (p. 8). Reproduced with permission from the Transportation Research Board and Dr N.L. Anderson.
- Figure 2.10a: from *NCHRP Synthesis 368: Cone Penetration Testing*, Transportation Research Board, P.W. Mayne, National Research Council, Washington, DC, 2007, Figure 1 (p. 6). Reproduced with permission from the Transportation Research Board.
- Figure 2.11: reproduced with permission of CIRIA from B2 – *Cone Penetration Testing: Methods and Interpretation*, CIRIA, London, 1987, Figure 10 (p. 20).
- Figure 2.15: from DIN 18196 *Earthworks and Foundations: Soil Classification for Civil Engineering Purposes*. Reproduced by permission of DIN Deutsches Institut für Normung e.V. The definitive version for the implementation of this standard is the edition bearing the most recent date of issue, obtainable from Beuth Verlag GmbH, 10772 Berlin, Germany.
- Figures 2.22, A.1 and Tables 2.17, A.1: reproduced with permission from John Wiley & Sons, Inc., from *Engineering Rock Mass Classifications*, Z.T. Bieniawski, 1989, Figure 4.1 (p. 61); Charts A–D (pp. 56–7); Table 4.4 (p. 62); and Table 4.1 (p. 54).
- Figures 2.24, 2.25: reprinted from 'Use and misuse of rock mass classification systems with particular reference to the Q-system', *Tunnelling and Underground Space Technology*, 21(6), A. Palmström and E. Broch, 2006, Figure 7 (p. 584) and Figure 10 (p. 588), with permission from Elsevier; Figure 7 (p. 584), with additional permission from N. Barton.

- Tables 2.2, 2.4 and 2.6: permission to reproduce extracts from BS EN 1997-2:2007, BS EN 1997-2:2007 and BS EN ISO 14688–2: 2004, respectively, is granted by BSI. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop or by contacting BSI Customer Services for hard copies only: Tel: +44 (0)20 8996 9001, Email: cservices@bsigroup.com.
- Table 2.15: reprinted with permission from *Applied Sedimentation*, 1950 by the National Academy of Sciences, courtesy of the National Academies Press, Washington, DC, and also with kind permission from Springer Science & Business Media and James Thomson, *Pipejacking and Microtunnelling*, Table 9.1, original copyright Chapman and Hall, 1993.
- Table 2.16: used with kind permission from the American Institute of Mining, Metallurgical, and Petroleum Engineers, New York from 'Failure and breakage of rock', *Proceedings of the 8th US Symposium on Rock Mechanics (USRMS)*, C. Fairhurst (ed.), 1967, 'Design of surface and near-surface construction in rock', D.U. Deere, A.J. Hendron Jr., F.D. Patton and E.J. Cording, Figure 5b (p. 250).
- Table 2.18: with kind permission from Springer Science & Business Media: 'Engineering classification of rock masses for the design of tunnel support', *Rock Mechanics and Rock Engineering*, 6, N. Barton, R. Lien and J. Lunde, 1974, Table 3, permission also obtained from N. Barton.
- Figures 3.2, 4.11 and 4.12: reprinted from 'Settlements induced by tunneling in soft ground', *Tunnelling and Underground Space Technology*, 22(2), International Tunnelling Association, 2007, Figure 8 (p. 122); Figure 19 (p. 140) and Figure 18 (p. 140), with permission from Elsevier.
- Figure 4.2: reproduced with kind permission from Pearson Education from F.C. Harris, *Exploring Modern Construction & Ground Engineering Equipment & Methods*, 1994.
- Figures 4.4a, 4.5a and b, 4.7, 4.8, 5.56, 5.58a and b: reproduced with kind permission from Taylor & Francis from *An Introduction to Geotechnical Processes*, J. Woodward, 2005, pages 36, 97, 96, 96, 53 and 56, respectively.
- Figure 4.15: reproduced with permission of CIRIA from SP200 – *Building Response to Tunnelling: Case Studies from Construction of the Jubilee Line Extension, London. Volume 1: The Project*, CIRIA, London, 2002, Figure 11.3 (p. 141).
- Figures 4.22, 5.24 and 5.29: reproduced with kind permission from Maney Publishing (www.maney.co.uk) from B.N. Whittaker and R.C. Frith, *Tunnelling: Design, Stability and Construction*, 1990, The Institution of Mining and Metallurgy, London, Figure 4.6 (p. 83); Figure 4.7 (p. 83) and Figure 14.2 (p. 334).
- Figures 5.52, 5.53 and 5.58c: reproduced with kind permission from Springer Publishers from *Tunnel Engineering Handbook*, Second Edition, T.R. Kuesel and E.H. King (eds), 1996, Figure 17.6 (p. 325) and Figures 17.9b and e (p. 330).
- Figures 5.61 and 5.62: reprinted from 'State of the art report in immersed and floating tunnels', *Tunnelling and Underground Space Technology*, 12(2), International Tunnelling Association, 1997, Figure 3.1 (p. 97) and Figure 3.2 (p. 98), with permission from Elsevier.
- Figures 5.73, 5.74, 5.75 and 5.76: used with kind permission from Thomas Telford Ltd and Dr Douglas Allenby, from 'The use of jacked-box tunnelling under a live motorway', *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, D. Allenby and J.W.T. Ropkins, 2004, Figure 3 (p. 232); Figure 5 (p. 234); Figure 9 (p. 237) and Figure 10 (p. 237).

- Figures 7.3 and 7.5: reproduced with kind permission from Taylor and Francis, from 'Theme lecture: bored tunnelling in the urban environment', *Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering,* R.J. Mair and R.N. Taylor, 1997, Figure 22 (p. 2362) and Figure 25 (p. 2363).
- Figure 7.4: reproduced with kind permission from John Wiley and Sons Ltd, from 'The response of buried pipelines to ground movements caused by tunnelling in soil', *Ground Movements and Structures*, J.D. Geddes (ed.), J. Yeates, 1985, Figure 1 (p. 131), original copyright Pentech Press, UK.
- Figure 7.6: reproduced with kind permission from Taylor & Francis from *Soil Movements Induced by Tunneling and their Effects on Pipelines and Structures*, P.B. Attewell, J. Yeates and A.R. Selby, 1986, Figure 4.35 (p. 283), permission also obtained from Dr A.R. Selby.
- Figure 7.7: reproduced with kind permission from Taylor and Francis, from 'The response of full-scale piles to tunnelling', *Proceedings of the 5th International Conference of TC28 of the ISSMGE, Geotechnical Aspects of Underground Construction in Soft Ground*, D. Selemetas, J.R. Standing and R.J. Mair, 2006, Figure 10 (p. 768).
- Figure 7.8: from *Geotechnical Instrumentation for Monitoring Field Performance*, J. Dunnicliff and G.E. Green, copyright 1988, 1993 John Wiley & Sons, Inc., Figure 7.1 (p. 76). This material is reproduced with permission of John Wiley & Sons, Inc.
- Figure 7.26: reproduced with kind permission from Taylor & Francis, from 'The measurement of ground movements due to tunnelling at two control sites along the Jubilee Line Extension', *Proceedings of the International Conference of TC28 of the ISSMGE, Geotechnical Aspects of Underground Construction in Soft Ground*, J.R. Standing, R.J. Nyren, J.B. Burland and T.I. Longworth, 1996, Figures 2 and 3 (p. 752).
- Table 7.3: with kind permission from Springer Science & Business Media, 'Behaviour of foundations and structures', *Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering*, J.B. Burland, B.B. Broms and V.F.B. de Mello 1977, Table 1 (p. 500), permission also obtained from Professor J.B. Burland.
- Figure A.2: reprinted from 'TBM performance estimation using rock mass classification', *International Journal of Rock Mechanics and Mining Sciences*, 39, M. Sapigni, M. Berti, E. Bethaz, A. Busillo and G. Cardone, 2002, pp. 771–88, with permission from Elsevier.
- Tables A.2–A.7: reprinted from 'Some new Q-value correlations to assist in site characterization and tunnel design', *International Journal of Rock Mechanics and Mining Sciences*, 39(2), N. Barton, pp. 185–216, 2002, with permission from Elsevier.
- Quoted material used with permission from Thomas Telford Ltd and Springer Publishers.

Abbreviations

2-D	two-dimensional
3-D	three-dimensional
ADS	anti-drag system
BSI	British Standards Institute
BTS	British Tunnelling Society
CDM	cement deep mixing
CPT	cone penetration test
CTRL	Channel Tunnel Rail Link
EPBM	earth pressure balance machine
EPDM	ethylene-propylene-diene monomer
ESR	excavation support ratio
FSTT	French Society for Trenchless Technology
GBR	Geotechnical Baseline Report
GFR	Geotechnical Factual Report
GIR	Geotechnical Interpretive Report
GSL	ground surface level
GWL	groundwater level (table)
HDD	horizontal directional drilling
HDPE	high density polyethylene
HME	Hypothetical Modulus of Elasticity model
HSE	Health and Safety Executive, UK
ICE	Institution of Civil Engineers, UK
ISRM	International Society for Rock Mechanics
ITA	International Tunnelling Association
ITIG	International Tunnelling Insurance Group
LF	load factor (= N/N_c)
LHS	left-hand side
LVDT	linear variable differential transformer
NATM	New Austrian Tunnelling Method
ÖBV	Österreichischer Beton Verein
PFA	pulverized fuel ash
PiccEx	Piccadilly Line Extension
PJA	Pipe Jacking Association
RHS	right-hand side
RMR	Rock Mass Rating
RQD	Rock Quality Designation

SCL	sprayed concrete lining
SCR	solid core recovery
SGI	spheroidal graphite (cast) iron
SISG	Site Investigation Steering Group, ICE, UK
SPT	standard penetration test
SRF	stress reduction factor
SSP	seismic soft-ground probing
STM	slurry tunnelling machine
SWOT	Storm Water Outfall Tunnel
TAM	tube-a-manchette
TBM	tunnel boring machine
TCR	total core recovery
TSG	tail shield grease
UK	United Kingdom
VSP	vertical seismic profiling

Symbols

γ	(bulk) unit weight of ground (kN/m^3)
γ_d	(bulk) unit weight for ground above the groundwater table (kN/m^3)
γ_{sat}	(bulk) unit weight for ground below the groundwater table (kN/m^3)
γ_w	unit weight of water (kN/m^3)
$\Delta\varepsilon$	change in strain
$\Delta\sigma$	change in stress (MN/m^2)
$\Delta\sigma_m$	average normal stress on the load plates (MN/m^2)
$\Delta S_{Z,R}$	average settlements of the centre and the edge of the load plate (mm)
ΔV	potential difference
$\dot{\varepsilon}$	strain rate
ε_u	ultimate strain at failure
ε_{horiz}	horizontal strain
ε_{pl}	plastic strain
ε_{vert}	vertical strain
η	stress-intensity-index
λ	parameter to describe the proportion of unloading in the convergence–confinement method
λ_d	predetermined value of the parameter λ
μ	Poisson's ratio
σ	total stress (kN/m^2)
σ'	effective stress (kN/m^2)
σ_1, σ_2, (σ_3)	principal stresses (kN/m^2)
σ_3, (σ_2)	confining stress for triaxial test (kN/m^2)
σ_v	total vertical stress (kN/m^2)
σ_v'	effective vertical stress (kN/m^2)
σ_h	total horizontal stress (kN/m^2)
σ_h'	effective horizontal stress (kN/m^2)
σ_s	surcharge acting on the ground surface (kN/m^2)
σ_T	tunnel face support pressure (kN/m^2)
σ_u	ultimate stress at failure (MN/m^2)
$\sigma_{u,adj}$	adjusted σ_u for uniaxial test (MN/m^2)
ϕ	internal friction angle (°)
ϕ'	effective internal friction angle = angle of shearing resistance (°)

ϕ_u	undrained internal friction angle (°)
ω	constant for the type of loading plate
c	apparent cohesion (kN/m^2)
c′	effective apparent cohesion (kN/m^2)
C	overburden to tunnel crown (or cover depth) (m)
c_u, s_u	undrained shear strength (kN/m^2)
c_v	coefficient of consolidation (mm^2/min)
d	sample diameter for uniaxial test and point load index test (mm)
D	diameter of tunnel (m)
D_e	equivalent dimension of the excavation (m)
D_r	relative density of coarse grained soils
E	Young's modulus (kN/m^2)
E′	drained deformation modulus (kN/m^2)
E_d	deformation modulus (kN/m^2)
E_s	stiffness modulus (kN/m^2)
E_v'	vertical drained deformation modulus from oedometer test (kN/m^2)
f_1	factor to allow for the plasticity index
f_s	sleeve friction for CPT (MN/m^2)
G_{max}	shear stiffness/modulus (kN/m^2)
Gs	specific gravity (kN/m^3)
H	depth from the ground surface to tunnel axis (C+D/2) (m)
h	sample height for a uniaxial test (mm)
h	horizontal displacement of footing (mm)
I	current (A)
i	trough width parameter (m)
I_c	consistency index
I_L	liquidity index
I_P	plasticity index
I_S	point load index strength (MN/m^2)
J_a	joint alteration number for Q-method
J_n	joint set number for Q-method
J_r	joint roughness number for Q-method
J_v	sum of the number of joints per unit length for the RQD index
J_w	joint water reduction factor for Q-method
k	permeability (m/s)
K	trough width factor
K_a	active coefficient of lateral earth pressure
K_p	passive coefficient of lateral earth pressure
K_0	coefficient of lateral earth pressure at rest
L	failure load in point load index test (MN)
L_1	interface between two strata
m_v	coefficient of volume compressibility (m^2/MN)
N_{SPT}	standard penetration test blow count
N	stability ratio
N_c	critical stability ratio or stability ratio at collapse
P	length of unsupported tunnel ahead of tunnel shield or lining (m)
P_T	support resistance (kN/m^2)
p_h	horizontal pressure (kN/m^2)

p_v	vertical pressure (kN/m^2)
Q	Q-value for Rock Mass Quality Rating method
q_c	cone tip resistance for CPT (MN/m^2)
Q_c	normalized Q-value
Q_{TBM}	Q-value for TBM tunnelling
r	radius of the load plate (m)
R_f	friction ratio for CPT (%)
S	surface settlement (mm)
S_h	horizontal ground displacement (mm)
S_{max}	maximum surface settlement directly above the tunnel centreline (mm)
S_v	vertical ground displacement (mm)
T_γ	tunnel stability number for the soil load
T_s	tunnel stability number for surface surcharge
u	pore pressure (kN/m^2)
UCS, q_u	unconfined compressive strength (MN/m^2)
V_l	volume loss per metre length of tunnel (m^3/m)
V_o	excavated volume of the tunnel per metre length of tunnel (m^3/m)
V_p	seismic velocity (m/s)
V_S	volume of the surface settlement trough per metre length of tunnel (m^3/m)
V_t	estimated volume loss per metre length of tunnel (m^3/m)
v	vertical displacement of footing (mm)
w	water (moisture) content (%)
w	settlement of the tunnel crown (mm)
w_{crit}	critical settlement of the tunnel crown (mm)
w_L	liquid limit (%)
w_P	plastic limit (%)
x, y, z	co-ordinate axes
y	transverse horizontal distance from tunnel centreline (m)
z	depth from the ground surface (m)
z_w	depth below groundwater table (m)

Note that all logarithmic terms are $\log_{10}$ in this book.

1 Introduction

1.1 Philosophy of tunnelling

Tunnels are unlike any other civil engineering structures. In buildings or bridges the building materials have defined and testable properties, whereas this is not the case in tunnelling. Table 1.1 illustrates some of the issues associated with tunnel design when compared to above ground construction projects.

Although a tunnel structure often needs support systems made up of concrete and steel, it is the ground that is the major part of the structure, and this can have both a supporting and a loading role. The key to successful tunnel construction is therefore to understand this material, in particular

Table 1.1 Comparison between tunnels and above ground construction projects

	Above ground construction	*Tunnel construction*
Construction material	The defined properties of the construction materials are guaranteed by the quality control procedures during the production process, including control testing.	The ground, with all its uncertainty, and the general inability to influence its properties (notwithstanding ground improvement techniques) is the construction material.
Loads	The loads for which the structural analysis is carried out are mostly known.	Only by making assumptions is it possible to estimate the loads possible, which means that the magnitude of the load is based on assumption and is thus basically unknown.
Safety	Because the properties of the construction materials and the loads are known, the safety factor relative to failure can be determined.	Because of the number of uncertainties related to the loads and material properties it is not possible to calculate a quantitative factor regarding the safety of the tunnel construction.

its strength and stability characteristics. No matter how much of the ground we test in preliminary site investigations, how many borehole cores we take for testing in the laboratory, we can only ever test a small fraction of the total ground to be affected by the tunnel construction. Therefore, it is up to the engineer to determine the relevant ground conditions and its associated properties. But as only a small fraction of the material can be tested and with limited knowledge of, for example, the effects of layering, fissures and discontinuities, much of this assessment is based on judgement and experience. One might even suggest that emotions are involved. So how can this then be used as the basis for tunnel design? It is up to engineering judgement to interpret the site investigation report and suggest suitable design and construction techniques.

Often, the assumption is that the ground acts as a continuum and allows three-dimensional stress redistribution around the tunnel void, thus taking some of the load, so that not the full overburden acts as the load on the tunnel. But how can anyone determine the percentage of this load-bearing capacity? Again, this comes down to engineering judgement. If a tunnel is lined using sprayed concrete, how can the residual stress-intensity-index be determined for the lining? If the displacement of discrete points is measured, how do we know that the maximum displacement has not been exceeded and the tunnel is not in danger of collapse? When is a crack in the tunnel lining significant and a sign of worse to come? Often it simply comes down to engineering judgement and experience. Many of these questions do not have a single answer, but depend on the individual case. No new tunnel construction is the same as a previous one. During the construction of a tunnel it is important to listen to the miners who have worked in many tunnels and use their experience to respond to different behaviours of the ground when excavating the tunnel. The key is to understand that tunnelling is not a discrete science with definite answers. There are many unknowns and the answer to most of the above questions is 'it depends'.

Experience and engineering judgement help to make a considered and informed decision, but continuous measurements during construction are essential to compare actual behaviour with those predicted. This book does not propose to give the reader all the answers related to any tunnel construction. Rather, its aim is to provide the reader with background information so that he or she can either make an informed decision and/or consult more specialist references on a specific topic. It will hopefully give the reader the tools needed to critically assess tunnel constructions and to realize that not all can be learnt from textbooks but that, to become a tunnelling expert, many years of experience are required. At the same time, this book hopes to demonstrate to the reader the excitement associated with tunnelling and to make it clear that there are many unknowns that require engineering judgement. Solving these issues is the challenge the civil engineer faces. If the reader takes away one message from this book, it should be that the answer to a lot of questions regarding tunnelling design and construction is 'it depends' and sometimes using emotions is essential to overcome the challenges posed by tunnelling.

1.2 Scope of this book

Tunnelling is an extensive topic and so the objective of this book is to provide a general knowledge base and guidance for further reading. It not only concentrates on different tunnel construction techniques but also brings in associated relevant topics such as site investigation, which have a large impact on the final tunnel design and its subsequent construction. It is important to note that tunnels in the context of this book include all types of tunnels not only the larger-scale metro, road and rail tunnels, but also utility tunnels for water, sewerage and cables.

This textbook aims to provide a comprehensive introduction to tunnel construction. It is aimed at undergraduate and postgraduate students with little or no previous experience and knowledge of tunnel construction, as well as recently graduated engineers who find themselves working in this exciting field of civil engineering.

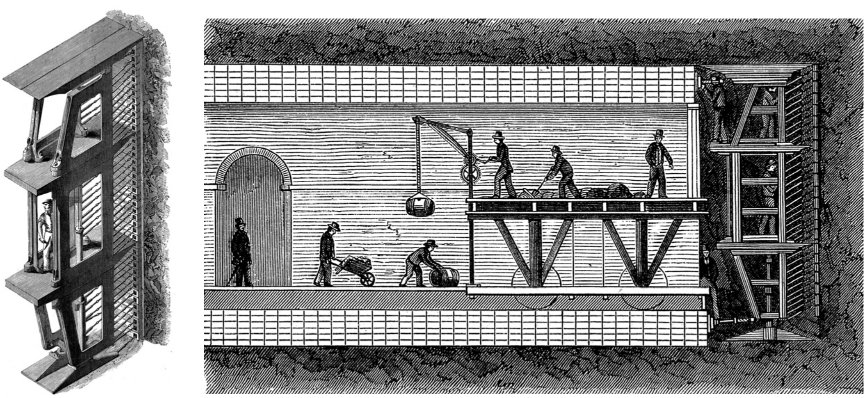
1.3 Historical context

There has been considerable development in tunnel construction techniques in the last 200 years, especially since Marc Brunel's famous first use of a tunnelling shield when constructing the first tunnel under the River Thames in London in 1825. Nevertheless, if Marc or his son Isambard Kingdom Brunel were to look at today's tunnelling methods they would see certain similarities with the techniques used in their day, particularly drill and blast and even tunnel boring machines (TBMs). The primary purpose of a TBM is to provide stability to the face and the surrounding ground, thus improving health and safety for the tunnellers, just as Brunel's own Thames Tunnel shield did. Although they would also notice great advances in technology, it would probably be the extent to which tunnelling has been used around the world and the sheer scale of many of these tunnels in terms of diameter, length and difficult construction conditions that would amaze them the most.

There are a number of detailed histories of the engineering art that is tunnelling, and this history is not reproduced here. The reader is directed to Sandström (1963), Beaver (1973), Megaw and Bartlett (1981), West (1988) and Muir Wood (2000) for further information. However, some tunnel constructions that marked key developments for 'modern' (from 1666) tunnelling are as follows.

- The first recorded use of gunpowder as a construction tool was for a pioneering tunnel of the canal age. This was constructed on the Canal du Midi, a canal built across France in the years 1666–81 connecting the Atlantic Ocean to the Mediterranean Sea. The main tunnel on this route was 157 m long with a rectangular cross section of 6.5 m by 8 m, and was built during the years 1679–81.

- Civil engineering as a profession was largely created in the UK by the development of the canal system, which itself was part of the industrial revolution of the eighteenth century. Two significant tunnels of this era included the 2090 m Harecastle Tunnel, constructed using gunpowder as part of the Grand Trunk canal during the 1770s, and the 5000 m long tunnel at Standedge, constructed through millstone grit. This latter took 17 years to complete and opened in 1811.
- The first tunnel underneath a navigable waterway was a tunnel under the River Thames in London, between Rotherhithe and Wapping. This involved using a tunnelling shield known as 'Brunel's Shield', designed by Marc Brunel. Construction of this brick-lined tunnel started in 1825 and it finally opened in 1842. The key function of this shield was to support the face and provide safety for the miners. The shield was made from cast iron (81.3 tonnes/80 tons), was 11.6 m (38 ft) wide and 6.8 m (22 ft. 6 in) tall and was made up of 12 parallel frames, each 0.9 m (3 ft) wide (Figure 1.1). In addition, there was a movable working platform on which the miners threw the spoil, and which was also used by the masons erecting the brick lining (Sandström 1963).
- Considerable amounts of tunnelling took place in the UK as a result of the coming of the railways, which started with the Liverpool to Manchester Railway opening in 1830. Water was a major problem for many of the tunnel projects. Between 1830 and 1890 over 50 railway tunnels exceeding one mile (1.61 km) in length were completed. I.K. Brunel was appointed Engineer of the Great Western Railway in 1833, at the age of 26, and planned the route from Bristol to London.



a) b)

Figure 1.1 'Brunel's Shield' used for the first Thames Tunnel, a) one of the twelve frames making up the shield, and b) a cross section through this tunnel during construction (after Beamish 1862)

A major tunnel on this route was the Box Tunnel with a length of 2937 m. Water was a major problem on several sections of this tunnel, but it opened successfully in 1841.

- 1857 saw the start of construction on the first major tunnel in the Alpine regions of Europe. The Fréjus Tunnel involved construction between two portals, one at 1344 m above sea level at Bardonnéche and the other at 1202 m at Fourneaux, with the distance between portals being 12,221 m. Rock drills were used extensively on the project and drill carriages mounting four to eight drills were introduced in 1863 and used until the completion of the project in 1870.
- At about the same time as the first Alpine tunnels were being constructed, the Hoosac Tunnel in Massachussetts, USA was started (1855–76). This became known as 'the Great Bore'. It was 7.44 km long (4.62 miles) and was constructed mainly through schist and gneiss. The rate of construction was very slow at 0.32 m per day in 1865, but this improved with the introduction of compressed air rock drills to about 1.65 m per day in 1873.
- 1869 was an important year for subaqueous tunnelling as it marked the successful completion of the Tower subway in London using a shield (designed by J. H. Greathead) and cast iron lining. The shield used is the ancestor of almost all subsequent tunnelling shields (it was circular as compared to Brunel's rectangular shield used on the earlier Thames Tunnel). It even incorporated grouting behind the cast iron lining to fill the void. The system was very efficient and allowed progress of 3 m per day. The tunnel was 2.18 m in diameter and 402 m long.
- Greathead made a number of further developments in shield technology, including a closed face shield with the ground being broken up with jets and the spoil being removed as a slurry, i.e. the forerunner of the slurry shield. (A slurry shield was first used in 1971 at New Cross in London, UK.)
- The first use of hydraulic jacks to propel a shield forward was designed by Beach in 1869 and used under Broadway in New York, USA.
- There were a number of developments in rotary tunnelling machines as part of the various attempts at the Channel Tunnel, UK in the 1880s.
- Compressed air was used as a means of preventing water inflow into the tunnel during the construction of the Hudson river tunnel in New York, completed in 1910. This project also introduced the 'medical lock' for treatment of caisson disease. At about the same time the first (old) Elbtunnel under the Elbe river in Hamburg also used compressed air during construction between 1907 and 1911. It suffered a blowout in 1909 with an 8 m high water fountain being observed. It should be noted that a patent for working in compressed air had been taken out in 1830 by Lord Cochrane in the UK.
- The first use of a combination of a shield and compressed air (together with cast iron segmental lining) was on the City and South London

Railway completed in 1890 (now part of the Northern Line on the London Underground system). The tunnels were twin tubes with a diameter of 3.1–3.2 m and constructed through mainly London Clay, but with occasional water bearing gravel. (Most of the original London Tube lines were constructed by the cut-and-cover technique.)
- Probably the first highway tunnel to use the submerged tube method of construction was the Posey Tunnel in California, USA, opened in 1928. It used 62 m lengths of steel circular shells encased in concrete and lowered into a dredged trench on the river bed.
- The Liverpool to Birkenhead Tunnel under the river Mersey, UK constructed between 1925 and 1933, was at the time the largest underwater tunnel ever built, with a length of 2 miles and 230 yards (3.49 km) and wide enough for four lanes of traffic.

Since these modest beginnings there has been an explosion of tunnelling all over the world and we can now probably claim on a technical level to be able to build tunnels anywhere, through any ground.

Looking to the future, the importance of tunnelling to the sustainability of megacities (defined as metropolitan areas with a total population in excess of 10 million people, or a minimum population density of 2000 persons per km^2) cannot be underestimated as it is vital for the development of the underground space.

1.4 The nature of the ground

There is a tendency for tunnelling projects to be classified as either 'soft ground' or 'hard ground (rock)' tunnels, and in this book the authors have adopted this terminology. However, it must be remembered that there is a transition between these terms and tunnelling projects often have to deal with much more complicated ground conditions, often with mixed ground components. This book uses the broad description of these 'categories' adopted by the British Tunnelling Society and the Institution of Civil Engineers in the UK (BTS/ICE 2004) in their tunnel lining design guide, which suggests that all types of soil and weak rocks would normally fall into the category of 'soft ground' (weak rocks include poorer grade chalk, weak mudstones and weakly cemented and/or highly fractured sandstone). 'Hard ground' would generally comprise all other forms of rock. In this book the word 'ground' is used as a generic term when referring to the material surrounding a tunnel and includes the rock material and, for example, any discontinuities and faults. An alternative term used in the literature is 'rock mass'.

There are many options available these days for the construction of tunnels. The selection of which tunnelling technique to use must be made on the basis of the known and suspected ground conditions, in combination with other aspects such as access, possibly local tunnelling traditions and skills, as well as costs. Adaptability of the technique to variability of the ground could also be an important factor.

One of the key aspects of any civil engineering construction project, particularly relevant for tunnelling, is the observational nature of the process. The ground in particular is not man-made and is infinitely variable. We therefore must treat it with respect. Based on observations, either from previous projects in the area or from the current project as it is progressing, engineering judgement based on performance is essential to inform the design and construction processes. This point is discussed further in section 7.3.3.

1.5 Tunnel cross section terminology

Some useful terminology related to a tunnel cross section is shown in Figure 1.2

Other terminology is explained throughout the book, i.e. when the term first appears in the text. The index can be used to find these explanations.

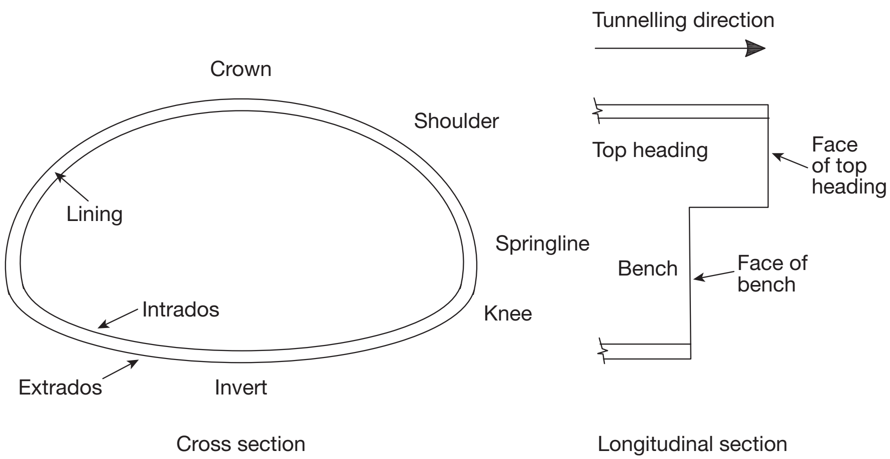


Figure 1.2 Terminology related to a tunnel cross section and longitudinal section

1.6 Content and layout of this book

The book consists of eight chapters (including this chapter) containing the following:

Chapter 2: Introduces the subject of site investigation and the issues of geological properties and classification, including laboratory and field testing.

Chapter 3: Covers preliminary analysis issues, such as calculation of primary stresses, stability in soft ground, preliminary analysis methods and numerical modelling.

Chapter 4: Covers methods of improving the stability of the ground prior to or during tunnel construction to ensure that the tunnel can be constructed safely. This chapter also describes the various methods available for lining a tunnel.

Chapter 5: Covers the main techniques of constructing tunnels.

Chapter 6: Introduces health and safety in tunnelling projects and the concept of risk management.

Chapter 7: Covers additional important issues associated with the construction of tunnels, including aspects related to tunnelling in soft ground such as ground movements and the effect of these ground movements on adjacent structures and services. This chapter also describes the observational method and monitoring and instrumentation related to tunnelling projects.

Chapter 8: Describes three case histories that are used to put into context some of the techniques and issues related to the construction of tunnels as experienced in practice.

There are extensive references within the text in each chapter and a list of these references is given at the end of the book. In addition, a bibliography list suggests further reading material.

2 Site investigation

2.1 Introduction

Tunnel construction is governed by the ground and hence site investigation is vital to obtain ground characteristics and geotechnical parameters. Knowledge of the ground conditions plays a key role in the choice of construction technique, and hence the success of a tunnel project. It is important to realize that the ability to influence the project outcome (in terms of cost and schedule) is easier earlier on in the project programme and much more difficult at a later stage, and the site investigation results can be a key influence on the early decisions. In many respects the site investigation for tunnelling projects is similar to other civil engineering projects and thus general textbooks and standards should be consulted (for example SISG 1993a, b and c, Attewell 1995, Clayton *et al.* 1995, BSI 1999, Simons *et al.* 2002, BSI 2007). However, more specific information related to tunnelling can be found in Dumpleton and West (1976) and BTS/ICE (2005). The new ICE Specification for Site Investigation to be published imminently will have a Tunnelling Addendum (current reference SISG 1993c).

Site investigation is defined in this book as the overall investigation of a site(s) associated with a tunnel construction, including the above and the below ground surface investigations. Ground investigation is defined as a sub-section of the site investigation and is associated specifically with defining the subsurface conditions. The aim of the site investigation is to produce a full three-dimensional model of the site, both above and below ground, and to highlight the associated impact (risks) of the tunnelling works on this environment and also the possible risks to the tunnelling works themselves. These risks can then be assessed and mitigated using appropriate construction techniques (risk management is discussed further in section 6.2). The site investigation comprises a number of key elements as shown in Figure 2.1. This chapter of the book describes these key elements in detail and highlights the investigation necessary for each step. The site investigation culminates in the site investigation report(s) described in section 2.5. It is important to realize that the site investigation information is not fixed at the start of the project and that the ground model develops and evolves with the project.

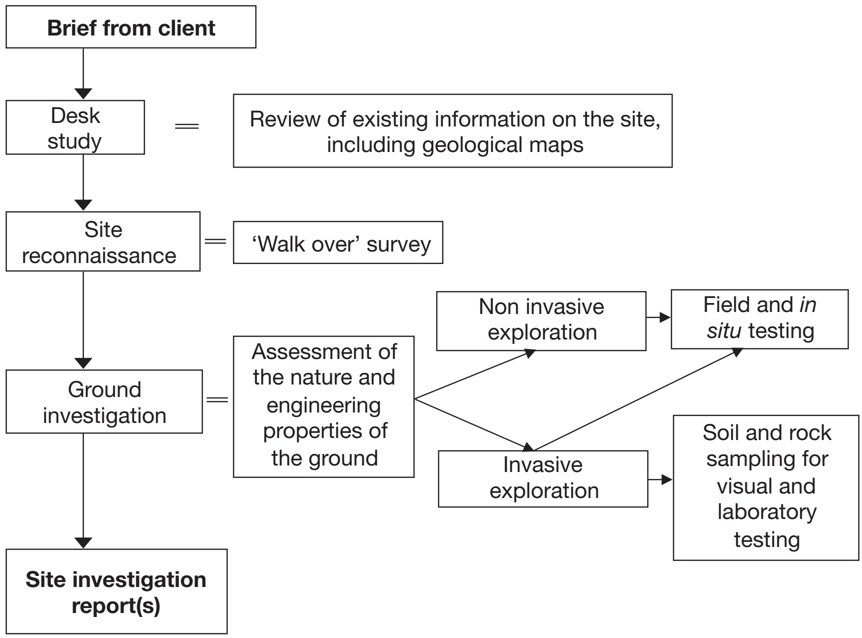


Figure 2.1 Elements of a typical site investigation

The money available to spend on site investigation is usually between 1 to 3% of the total tunnelling project costs. It is therefore important to use this money wisely in order to minimize the subsequent risks during construction. The traditional view is that the more one pays for site investigation the more likely one is to reduce additional costs resulting from unforeseen circumstances. There is some evidence to support this, although it is important to make informed decisions on how this money is spent. However, it is unlikely that more money will be spent on site investigation for tunnelling projects so it is important to use a risk-based approach to maximize the impact of the money available and minimize the risk of overlooking something important.

2.2 Site investigation during a project

2.2.1 Introduction

For any given project there are a number of different types of site investigation, namely: preliminary investigation, design investigation and control investigation (BSI 2007). These may be carried out during different stages of the project and have varying objectives. The main focus of the preliminary investigation is to assess the general suitability of the site and compare different alignments, with due consideration to third parties. The main aim of the design investigations is to provide information required for the design of the tunnel, including the construction method. In addition, control investigations may be required during the construction or execution of the project, and include, for example, checking ground characteristics and groundwater conditions.

A typical site investigation comprises four key elements: the desk study, site reconnaissance, ground investigation and the production of the site investigation report(s) (Figure 2.1). However, when designing a site investigation it is important to be objective and make sure that what is done can be clearly justified and that the desired outcome of the investigation is clear. The desk study and site reconnaissance can help design the subsequent ground investigation. It is essential that the specified sampling and testing are appropriate for the materials and parameters required for the subsequent design.

2.2.2 Desk study

The desk study is a very important stage of any site investigation, which, if done well, can save considerable time, and hence money, later on in the investigation process. The aim of the desk study exercise is to assess the conceptual model developed for the tunnel scheme using all the available records of the area where the proposed scheme is to take place. Desk studies cover all aspects of the site, including current usage, overlying and adjacent structures, historical usage and geology.

It is important that the desk study highlights any issues that could affect the health and safety of personnel during the subsequent site investigation and also the construction of the project. It should also provide as much information as possible to aid the planning of the subsequent stages of the site investigation, which in the case of tunnelling projects is usually the location, depth and type of boreholes.

In most countries there are numerous sources of information available that can aid a desk study, for example, geological maps, geological memoirs, old and new topographic maps (for example Ordnance Survey maps in the UK), aerial photographs, utility company records, site investigation databases (the British Geological Survey in the UK) and local councils.

It is also important to use site investigation companies that are familiar with the local area, as previous experience can be invaluable.

2.2.3 Site reconnaissance

Site reconnaissance (sometimes termed 'walkover survey') is the first site-specific work. With tunnelling projects it is rarely possible to walk along the entire length of the tunnel alignment, but this should be attempted as it can provide excellent detailed site knowledge for future planning. This is particularly important when planning any intrusive ground investigation and for the location of shafts. The objectives of a site reconnaissance include but are not limited to (after Allen 2006):

- location/confirmation of buried services;
- assessment of structures, particularly historic structures likely to be affected by the tunnelling works;
- identification of access restrictions;
- identification of any evidence of existing geology (e.g. exposed cut faces);
- identification of any evidence of existing structural or geotechnical problems, cracks and settlements of structures;
- identification of any new construction works (not shown on current maps);
- identification of any unexpected hazards.

It is important to record site details via photographs, sketches and notes. The information is checked against the desk study findings and further desk studies and/or further site visits undertaken as appropriate. As with the desk study this stage is relatively low cost compared to the later stages of a site investigation and can produce valuable qualitative information (Allen 2006).

2.2.4 Ground investigation (overview)

The ground investigation element of the site investigation should be planned based on the findings of the desk study and the site reconnaissance.

The ground investigation should give information about the stratigraphy of the ground. This is the genesis of the underground strata, i.e. the layering and the types of layers. It is important to conduct description and classification and testing, to determine information on the properties and parameters of the ground. Key general parameters for tunnel design include the strength of the ground to assess stability and the loading on the lining, modulus values such as Young's modulus, E, to assess how much the ground will deform with changes in stress, and the water conditions and permeability of the ground, as water can influence stability and make tunnel construction difficult. Water (the hydraulic regime) is extremely important when conducting underground construction and any investigation should include determining the groundwater level(s), water pressure, confined aquifers and water chemistry (with respect to how aggressive it is towards concrete). Other aspects include determining the swelling properties of clays, cavities (karsts) and abrasiveness characteristics.

This may sound straightforward. However, the ground is generally highly variable and its parameters can change over relatively short distances. It is therefore often a challenge to develop a model of the ground and associated risks along the route of a potential tunnel, and establishing the necessary design parameters is rarely 'straightforward' with considerable engineering judgement being required.

Due to the complexity of this aspect of the site investigation, the ground investigation is covered in detail in a separate section below (2.3).

2.3 Ground investigation

2.3.1 Introduction

A tunnel is commonly a composite structure made up of the tunnel lining and the surrounding material, although there are bare rock tunnels which do not need a lining. The surrounding material not only has a loading function, but is also the medium in which a void is created with the help of the supporting role of the surrounding material. Without this supporting role of the ground an economical tunnel design would not be possible, i.e. the ground is an integral part of the tunnel construction.

To make a judgement about the stability of the tunnel, as with any civil engineering design, the characteristics of the 'building materials' must be known: this includes both the tunnel lining and the surrounding ground material. There is difficulty in determining the ground parameters particularly when there are faults, inhomogeneity and weathering, all of which make it difficult to assign simple statements about the ground behaviour. Laboratory and field experiments can be carried out to give an indication of the soil and rock stability, which can be used to give some, albeit limited, idea of the ground stability.

In this book the ground investigation comprises of field investigations and laboratory experiments to obtain information about the subsurface and its properties. Table 2.1 provides a list of potential parameters required from a site investigation in order to aid the design of a tunnelling project. Many of these parameters are determined during the ground investigation, although some may have been obtained from past investigations as highlighted by the desk study.

The decision as to which techniques should be used during the ground investigation must be considered carefully and in relation to the budget and goals required. It is important to identify the investigation goals in order to avoid wasting time and, consequently, money.

2.3.2 Field investigations

A variety of investigation techniques can be employed as part of the ground investigation. These include intrusive and non-intrusive methods. A combination of various methods is usually the best approach.

2.3.2.1 Non-intrusive methods

Although intrusive methods allow the inspection and testing of the ground itself, they are normally restricted to discrete locations. Non-intrusive

Table 2.1 Ground parameters for the design of tunnel projects (adapted from Morgan 2006, after Jewell 2002, used with permission from Peter Jewell)

Geotechnical design parameter	*Symbol*	*Units*	*Application to tunnel design and construction*
Soil description from light cable percussion boring			Defines type of ground
Soil and/or rock description from rotary coring			Defines type of ground
Grade of rock			Extent of ground support
Percentage core recovery and core condition (total core recovery, solid core recovery and rock quality designation respectively)	TCR, SCR, RQD		State of weak or hard rock
Unit weight	γ	kN/m^3	Overburden pressure
Relative density of coarse grained soils	D_r		State of natural compaction of cohesionless soft ground
Moisture content	w	percent	Profiling of property changes with depth
Specific gravity	Gs	kN/m^3	Type of ground
Plasticity and Liquidity Indices (liquid limit, plastic limit, plasticity index, liquidity index, consistency index respectively)	w_L, w_P, I_P, I_L, I_c	percent	Type and strength of cohesive soft ground
Particle size distribution			Composition of soft ground
Unconfined (or Uniaxial) compressive strength, UCS	q_u	MPa or MN/m^2	Intact strength of hard rock

Table 2.1 (*continued*)

Geotechnical design parameter	*Symbol*	*Units*	*Application to tunnel design and construction*
Point load index strength	I_S	MPa or MN/m^2	Intact strength of hard rock
Undrained shear strength	c_u, s_u	kPa or kN/m^2	Shear strength of soft ground (short-term strength of fine grained 'cohesive' soils)
Effective shear strength	c'	kPa or kN/m^2	Long-term apparent 'cohesion' of soft ground (fine grained soils)
Angle of internal shearing resistance	ϕ'	degrees	Long-term shear strength of 'cohesive' soft ground (fine grained soils) Short and long-term shear strength of 'cohesionless' soft ground (coarse grained soils)
Ultimate stress at failure	σ_u	MPa or MN/m^2	Characterizing rock
Ultimate strain at failure	ε_u		Characterizing rock
Modulus of elasticity (Young's modulus)	E	MPa or MN/m^2	Stress increment per strain increment, i.e. directly related to strength
Drained deformation modulus	E'	MPa or MN/m^2	Long-term stiffness
Poisson's ratio	μ		Influences stiffness values
Coefficient of lateral earth pressure (at rest, active and passive values respectively)	K_0, K_a, K_p		Ratio between horizontal and vertical effective stresses
Permeability	k	m/s	Characteristic ground permeabilities and variations, waterproofing
pH, sulphate and chloride content	pH, SO_3, Cl		Concrete and steel durability
Chemical contamination			Extent of ground contamination
Abrasion			Rate of cutter tool wear

methods can be used for determining additional information about the ground and include geophysical methods. Geophysical methods can be used to obtain information over a relatively large area of the subsurface ground, and hence can be used to help locate boreholes, provide information about the nature and variability of the subsurface between existing boreholes, or can be used where access for intrusive methods is not possible. It should be noted that interpretation of the output from these methods is not easy and usually requires a borehole(s) to correlate results. Some of the more appropriate geophysical methods for tunnel projects are briefly described below.

SEISMIC METHODS

Seismic techniques are based on the generation of seismic waves on the ground surface at a source, S, and the measurement of the time taken by the waves to travel from the source, through the ground to a series of receivers, R. They utilize the fact that elastic waves travel with different velocities in different rocks. The seismic wave can be generated using a drop hammer or a 3 kg sledgehammer to give a penetration depth of up to 20 m. For deeper penetration depths falling weight devices or even explosives can be used (Waltham 2002). Geophones are commonly used as receivers. Two main travel paths for the seismic wave are possible. The wave can travel along the interface between two rock types (L1), i.e. it is refracted (Figure 2.2a), or it can be reflected off this interface (Figure 2.2b).

Knowing the distance between the source and the receiver and the travel time it is possible to determine the shear velocity, and hence the depth to the refracting/reflecting interface. If the density is known (calibrated from borehole information) the shear stiffness, G_{max}, of the material can be inferred from the surface waves resulting from seismic surveys. Seismic reflection and refraction can be useful for determining depth to bedrock and depth to groundwater table, but reflection can give better resolution and can also identify multiple layers and faults.

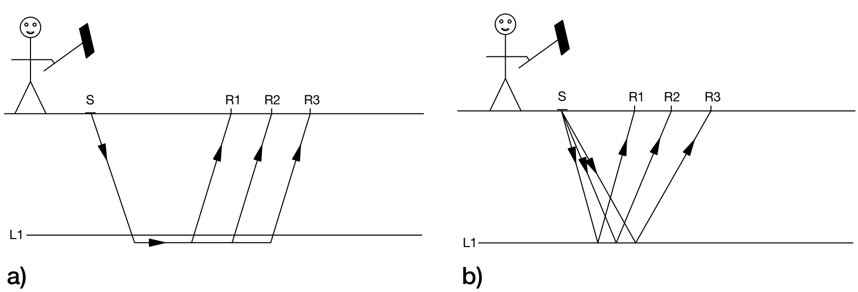


Figure 2.2 a) Seismic refraction and b) seismic reflection (after Anderson *et al.* 2008)

RESISTIVITY/CONDUCTIVITY

The results from these methods are particularly useful when combined with seismic refraction. They are especially useful for determining the soil/water interface, soil profiles and also for characterizing contaminated groundwater plumes.

Figure 2.3 shows the principle of these techniques. In the electrical resistivity technique a current (I) is induced between paired electrodes (C1, C2). The potential difference (ΔV) between paired voltmeter electrodes P1 and P2 is measured. Apparent resistivity is then calculated (based on I, ΔV, and the electrode spacings). If the electrode spacing is expanded about a central location, a resistivity–depth sounding can be generated. If the array is expanded and moved along the surface, 2-D or 3-D resistivity–depth models can be created (after Anderson *et al.* 2008).

BOREHOLE GEOPHYSICAL LOGGING

A wide variety of in-hole methods are available adapted from the petroleum industry. Borehole geophysical logging can be useful for special circumstances and includes sonic and electrical resistivity methods. It is good for determining the properties of the ground at depth such as density, but must be used selectively in order to be cost-effective.

CROSS-HOLE SEISMIC TECHNIQUES

This technique provides improved definition of geology at depth when compared to surface seismics. It can be good for the characterization of underground caverns. This method, however, can be expensive due to the need for closely spaced boreholes. It typically involves high-frequency acoustic pulses generated at predetermined source locations at different levels in the source borehole. The amplitude and arrival time of direct arrivals

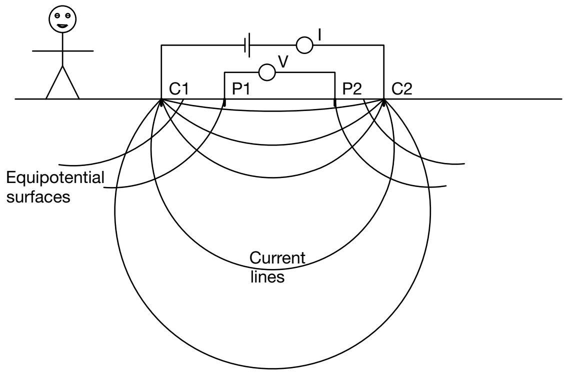


Figure 2.3 Resistivity/conductivity (after Anderson *et al.* 2008)

(and others) is recorded at predetermined receiver locations at different levels in the receiver borehole. The recorded travel time–amplitude data are statistically analysed and used to generate a velocity–attenuation cross sectional model of the area between the source and receiver boreholes. Attenuation is defined as the reduction in signal strength as a result of it passing through a medium, in this case the ground (after Anderson *et al.* 2008).

Other geophysical techniques include magnetic methods (good for locating buried foundations, mineshafts and ferrous utilities and obstructions), gravitational methods (good for cavity detection) and electromagnetic methods (good for locating utilities, ground and pollution mapping). For further information on geophysical methods commonly employed for site characterization, the reader is referred to McDowell (2002) and the Transportation Research Circular E-C130 (Anderson *et al.* 2008).

2.3.2.2 Intrusive exploration

Intrusive exploration is used for obtaining samples/cores of the ground for visual examinations and laboratory testing, and also for conducting *in situ* testing to determine the ground characteristics and primary stress conditions.

IN SITU SAMPLING

The principle methods for obtaining samples/cores include trial pit excavations, percussive drilling, rotary drilling techniques and even trial tunnels. Trial pit excavations are used for relatively shallow investigations to a few metres, but depending on available space can open up a relatively large area of the ground. Percussive boring (known as either cable percussion or shell and auger boring) is the most common technique in the UK for soft ground (soil/weak or weathered rock) as it is relatively cheap, simple, flexible and robust. Through suitable ground it can be used down to 60 m. Figures 2.4a and b show a typical cable percussion rig. As the name implies, the boring is conducted by continuously raising and dropping weighted hollow drilling tools which gradually penetrate the ground. Rotary drilling is used in rocks and can drill down to hundreds of metres, although smaller rigs are available for shallower investigations. Figure 2.4c shows an example of a small rotary drilling rig. The standard approach in the UK is to use cable percussion boring to rockhead, and if required the borehole is extended by rotary coring. However, some strata, for example weathered rock, overconsolidated clay and most chalk, may be sampled by either cable percussion or rotary drilling methods. Samples can be obtained during these intrusive operations and the quality of these samples is described in section 2.3.3. Cable percussion boring allows discrete samples to be obtained, for example using a U100 driven sample tube (100 mm

Figure 2.4 a) and b) Cable percussion rig and c) rotary drilling rig (courtesy of Soil Mechanics)

Figure 2.5 Example of continuous rock cores obtained from rotary drilling

Figure 2.6 Triple core barrel sampler (double core barrel with a plastic liner) used with rotary drilling

diameter, 450 mm long sample). BSI (1999) suggests that in soft ground, samples should be obtained at the top of each new stratum and thereafter every 1.0 to 1.5 m, and standard penetration tests (see *in situ* testing) conducted immediately afterwards. Figure 2.5 shows an example set of rock cores obtained from a rotary drilling operation, which allows continuous samples to be obtained.

Figure 2.6 shows a triple core barrel sampler used with rotary drilling, which incorporates an outer drill tube that rotates and has the cutter attached to the end, an inner steel tube (core barrel) that does not rotate (the gap between these two tubes is used to pass drilling fluid to the drill bit), and a plastic liner to help preserve the sample during retrieval from the core barrel and transportation to the laboratory for logging and sampling.

In soft ground it is also usual to obtain disturbed samples from cable percussion techniques. These are samples where there is no attempt made to preserve the shape or the fabric of the soil, but if sealed correctly (bagged) can give useful information on particle size and water content, and if the soil is clay, information on plasticity indices (see section 2.3.2).

A recent development in intrusive investigation is the use of horizontal directional drilling that allows core drilling in practically any direction, for example along the alignment of the tunnel. Horizontal directional drilling is discussed in detail in section 5.12.

The transportation, storage and labelling of samples needs to done carefully and a satisfactory procedure adopted to ensure they can be readily identified. If not done adequately it can lead to sample deterioration and hence influence subsequent laboratory test results.

It is important to consider the position of the boreholes carefully. Although it is essential to get a representative sample of the ground, accessibility to the location for the drilling rig might be a limiting factor. The possibility of lateral realignment of the tunnel should be considered in the drilling plan. Even though boreholes should be filled in properly, it is not always done satisfactorily, and this can create problems during the later construction stages, for example water ingress and pressure losses. Therefore, these should not be drilled directly on the alignment of the tunnel. However, when creating large openings (caverns) it may be necessary to drill into the later void. It should be noted that creating a hole is expensive and it is therefore often worth utilizing the borehole to incorporate instrumentation used during the construction of the tunnel (monitoring and instrumentation is described in section 7.3).

For tunnels and shafts it is important to take the exploration to a generous depth below the proposed invert level of the tunnel because changes in design may result in a lowering of the level of the tunnel, and because the zone of influence of the tunnel may be extended by the nature of the ground at a greater depth (BSI 1999).

The number of boreholes associated with a particular tunnelling project depends on the ground conditions and extent of tunnelling works, and there are no fixed rules on the spacing or number. As a rule of thumb, however, for relatively long tunnels 300 m spacing would be sensible for the main tunnel and 30 m spacing at the portals.

From the borehole results and associated testing (see section 2.3.3) it is possible to obtain a geological section along the tunnel and in a plan showing the layering, i.e. a geological model along the route of the tunnel (Figure 2.7). It may also be useful to conduct some shorter angled drillings to help develop the stratigraphic section and produce a more complete

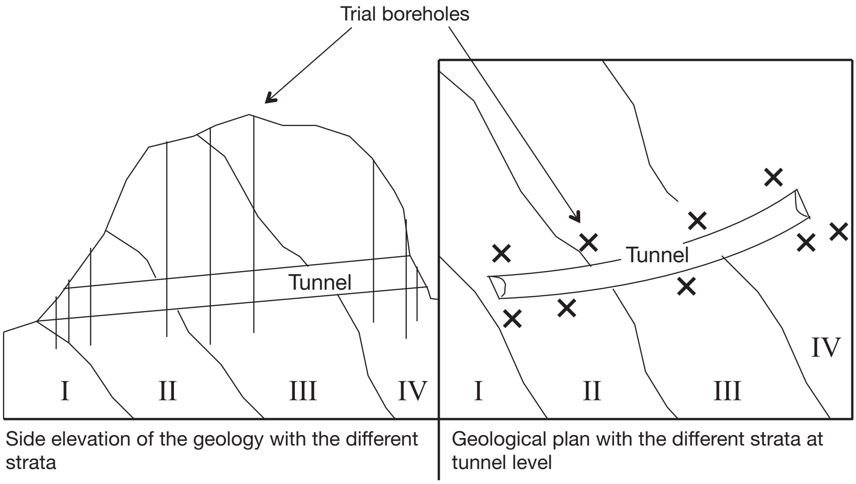


Figure 2.7 Possible borehole locations for a mountain tunnel (Note: exaggerated vertical scale)

picture. This is important if the strata are highly dipping and relatively thin, as the vertical boreholes could miss some of these strata.

The following points should also be noted related to the borehole investigation:

- The borehole positions should be shown accurately on the proposed plans for the tunnel and the ground level at each borehole position must be recorded.
- The majority of information from the investigatory boreholes is derived from cores from the whole depth. This allows undisturbed samples of the rock to be obtained and tested.
- If the boreholes are very deep, it is sometimes preferred to get cores only at specific sections, not for the whole depth.
- It is desirable to create a 3-D model of the geology associated with the project from the boreholes. However, it is also important, if possible, to get a personal observation of the site during the borehole drilling and hence obtain a good 'feel' for the ground conditions.
- Simply looking at photographs of the cores and reading the associated reports can give a false impression. Colours on photographs, for example, can be misleading and information could be missing from the report such as, for example, whether it was wet? What was the Rock Quality Designation (defined in section 2.4.4.1)? It is important to spend time studying the cores directly and noting any irregularities.
- Boreholes should also obtain information on the hydrology, ground-water level and layers holding water.

If it is not possible to determine aspects of the subsurface details using boreholes from the ground surface, trial excavations/tunnels can be used. The Gotthard Base Tunnel in Switzerland, for example, used a trial tunnel of a couple of kilometres to determine further details of the 'Piora fault'.

Once samples have been obtained, it is then possible to conduct laboratory tests on the material, including uniaxial, triaxial testing and basic index tests. Some of these tests are discussed in more detail in section 2.3.3.

IN SITU TESTING

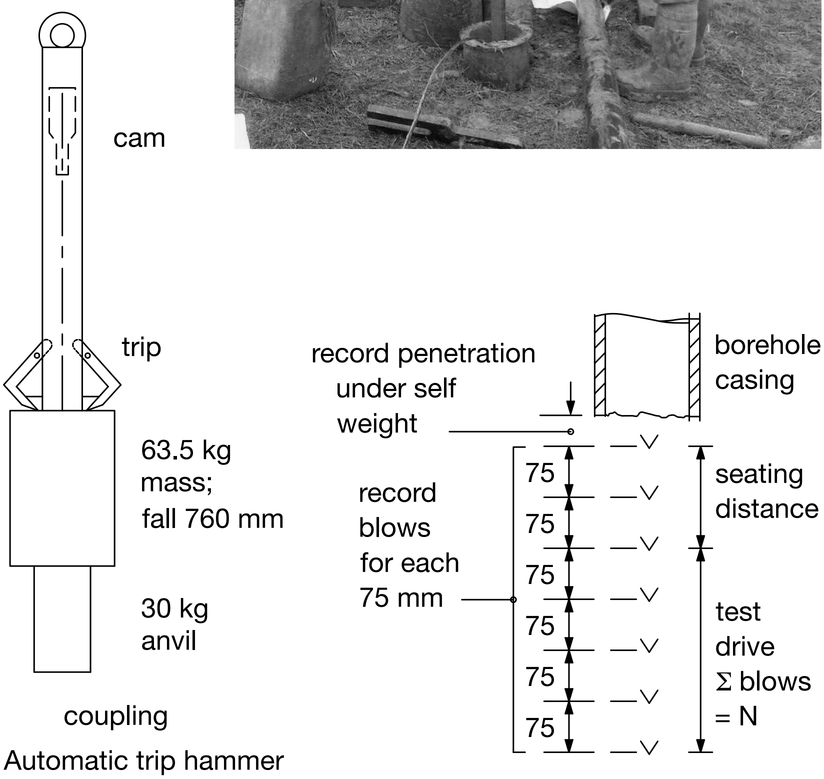
It is also possible to conduct *in situ* testing (with or without associated sampling) and these can include standard penetration testing, cone penetration tests and pressuremeter or dilatometer testing. The reader is directed to the appropriate standards in their own country for further details, for example BSI (2007). It should be noted that most field tests only provide indirect measures of the ground properties and very often empirically derived relationships need to be applied to obtain design parameters. Several of the more common tests are briefly described below:

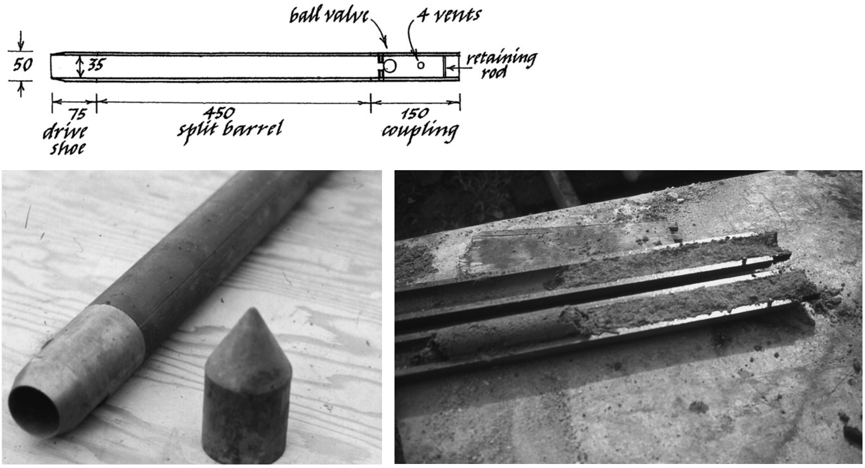
Standard penetration test In soft ground, cable percussion (or shell and auger) boring techniques are often employed during field investigations. As part of this boring process, *in situ* standard penetration tests (SPTs) can be conducted as the borehole is created. The SPT test is performed at the bottom of the borehole (the boring level reached at that time). It is carried out by driving a standard sampling tool (for example a split barrel sampler) through a prescribed distance (450 mm), with a known weight (63.5 kg). The weight is dropped from a fixed height of 760 mm onto an anvil placed on top of the drill rods and the number of blows required for it to penetrate 450 mm is recorded (generally in 75 mm intervals). Normally the number of blows recorded to penetrate the first 150 mm is discarded as the ground in this region is normally disturbed by the boring process. The number of blows recorded to drive the sampler the next 300 mm is taken as the blow count, N_{SPT}. Figure 2.8a shows a standard penetration test being conducted and Figure 2.8b shows a schematic of the test procedure. The split barrel sampler is shown in Figure 2.9a and although this is a poor quality sampler from the point of view of soil sampling (the dimensions are such that the soil is disturbed too much as it enters the sampler), it does provide a sample for visual inspection at the level at which the test was conducted (Figure 2.9b). A significant effect on the results may begin to occur when the borehole diameter is greater than 150 mm. The water level in the borehole casing (boreholes are often cased to prevent collapse) should be kept above the natural groundwater level to avoid instability at the base of the borehole due to water flowing towards the borehole.

Figure 2.8
a) Standard penetration test carried out using a cable percussion rig (shell and auger) (courtesy of Soil Mechanics) and
b) schematic of the SPT automatic trip hammer arrangement and procedure (courtesy of J. Billam)

a)

b)





a) b)

Figure 2.9 Split barrel sampler used for the standard penetration test: a) the open tube is used for fine grained soils and the cone for coarse grained soils and b) in fine grained soils a sample can be obtained for visual inspection (courtesy of Dr Ron Jones)

Table 2.2 Approximate relationship between blow count and density (BSI 2007)

Density	*Very loose*	*Loose*	*Medium*	*Dense*	*Very dense*
Normalized blow count (N_{SPT})[a]	0–3	3–8	8–25	25–42	42–58

Note: (a) Normalized blow count is adjusted for the energy transmitted down the rod and the effective overburden pressure. These descriptors should not be used for very coarse soils.

The SPT is popular because it is relatively simple and cheap to carry out. However, interpretation can be difficult as although the word 'standard' is used in the title of the test, there is a large variation in the equipment used. It was originally developed for coarse grained soils because of the difficulty in obtaining samples in these soils, and it is useful for giving an indication of density (Table 2.2).

Empirical relationships have been developed for other design parameters, for example the undrained shear strength, c_u, of overconsolidated clays can be approximated using equation 2.1.

$$c_u = f_1 N_{SPT} \tag{2.1}$$

where f_1 is a factor to allow for plasticity index, I_P, as shown in Table 2.3. (Plasticity index is defined in section 2.3.3.)

Table 2.3 Relationship between f_1 and plasticity index (after Stroud 1989)

Plasticity index (I_P)	*15*	*25*	*35 and over*
f_1	7.0	4.8	4.2

There are also correlations with the angle of shearing resistance (effective internal friction angle), ϕ', however, as this value depends on the stress level, care must be taken when determining this value from charts. Further information on SPT testing can be found in Clayton (1995).

Cone penetration test The cone penetration test (CPT) is used in soft ground and consists of pushing a cone (penetrometer tip) attached to the base of a series of rods into the ground. As the rods are pushed into the ground (at a constant rate of 20 mm/s), the cone tip resistance, q_c, and the

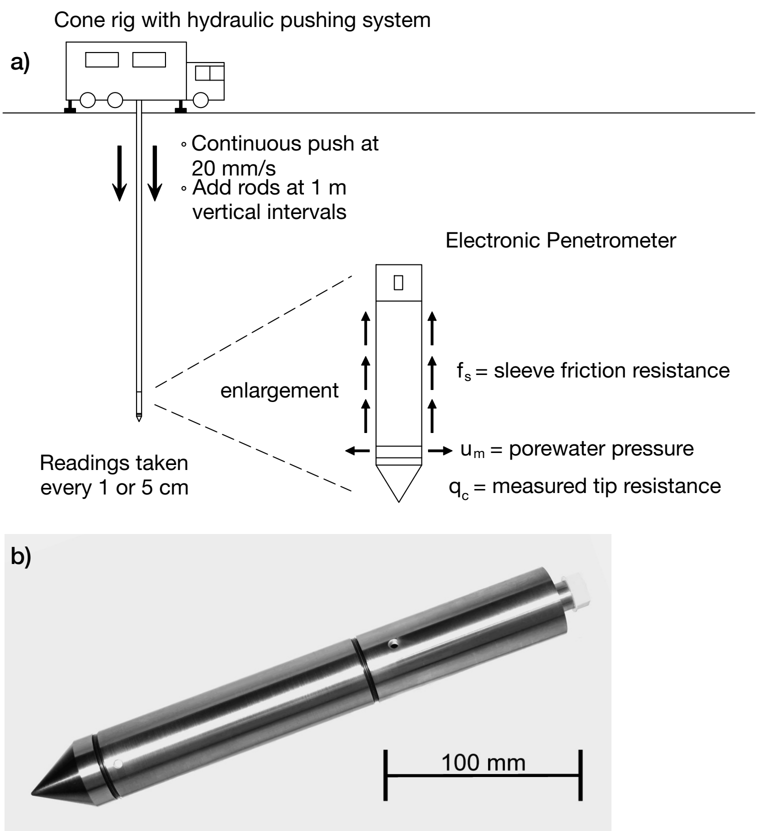


Figure 2.10 Cone penetration testing: a) test arrangement (after Mayne 2007) b) example of an electric penetrometer cone (courtesy of Geopoint Systems BV)

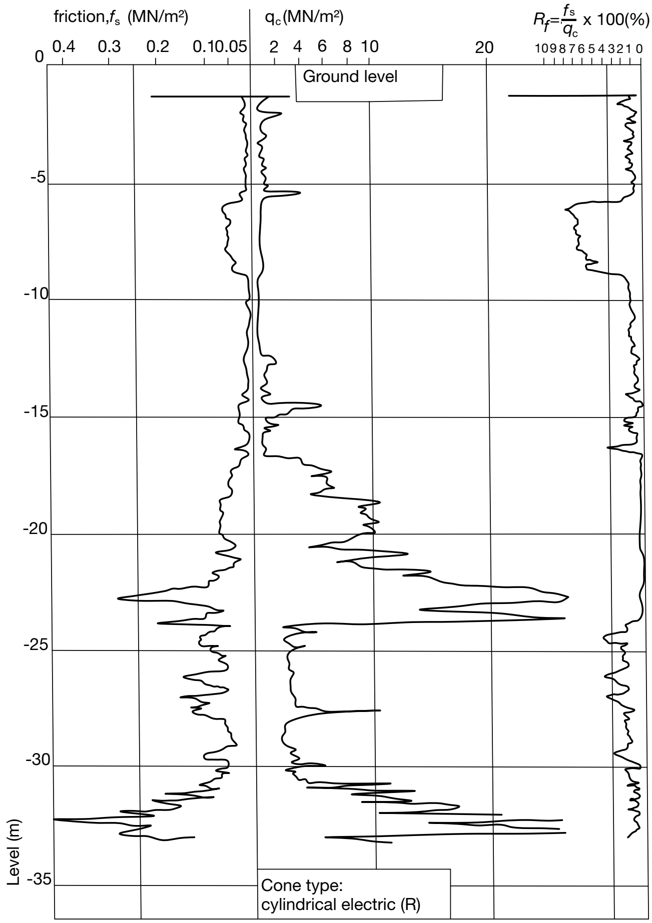


Figure 2.11 Typical plot from a cone penetration test (after Meigh 1987)

sleeve friction, f_s, are recorded by the penetrometer tip (Figures 2.10a&b). The cone angle on the penetrometer tip is 60° and the cross sectional area is 1000 mm^2. There are different types of equipment available on the market to conduct this test, but the most common involves a 20 tonne truck, to push the penetrometer tip into the ground.

A typical plot from a CPT is shown in Figure 2.11. The friction ratio, R_f is the ratio of the sleeve friction divided by the cone tip resistance, i.e. $R_f = f_s/q_c$. The zone in Figure 2.11 where the f_s and q_c values are higher indicate stiffer ground.

It has been found that by plotting the values of q_c against R_f an approximate description of the soil type can be obtained (Figure 2.12). Other relationships have been developed over the years between q_c and a number of other parameters, for example undrained shear strength, and angle of

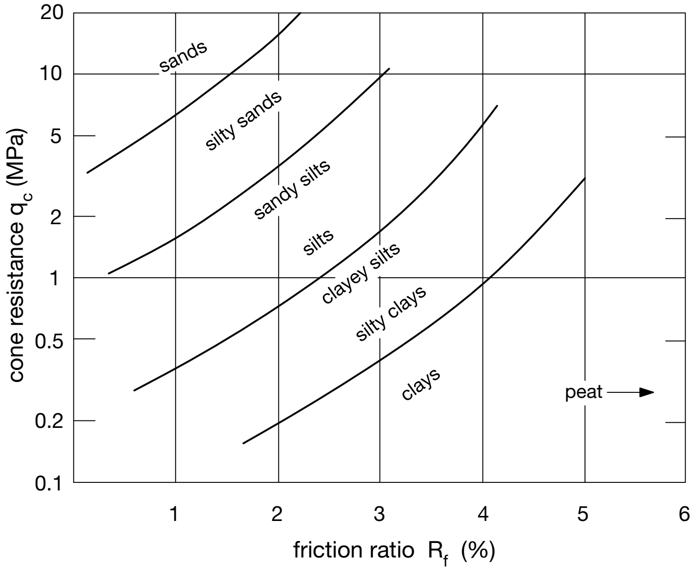


Figure 2.12 Simplified version of a friction ratio cone resistance plot used to obtain an approximate description of the ground

shearing resistance, but local experience and correlation with laboratory tests is essential. There are also penetrometer tips with pore water pressure measurement (piezocone) and geophysical testing (seismic cone and resistivity cone). Further details on cone penetration testing can be found in Mayne (2007).

Dilatometer/pressuremeter A dilatometer (or a pressuremeter, the terms seem interchangeable for rock testing) is a borehole deformation device. It is used as a rock/soil loading test in boreholes with a defined diameter. The aim of the dilatometer test is to determine the deformation modulus of the ground (see Figure 2.17, 'Definitions of different modulus values') and horizontal stress. The dilatometer consists of a cylindrical pressure cell containing strain arms within a cylindrical rubber membrane, which is pressed hydraulically against the borehole wall (Figure 2.13a). The borehole walls are loaded and then unloaded (cyclically) causing the borehole walls to deform (measured by the strain arms), thus allowing an estimate to be made of the deformation modulus of the material for an associated change in radii. By conducting the test in different directions within the ground, it is possible to determine the deformation modulus in different directions and hence obtain information on anisotropy within the ground. As with all *in situ* experiments the validity of the results are potentially limited because only small areas/sections of the ground are tested. Problems in validation can occur if the ground has many fractures, the borehole wall is not even or when the borehole is not stable and rock is collapsing into the bore.

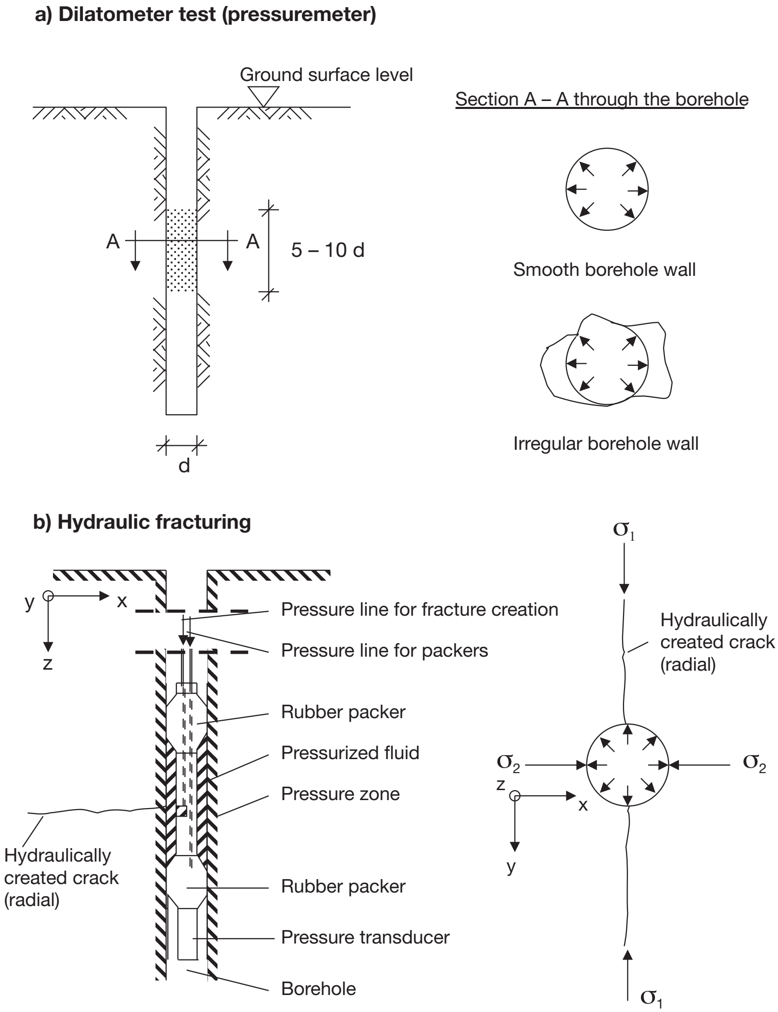


Figure 2.13 Schematic of the a) dilatometer and b) hydraulic fracturing tests

In these cases, it is necessary to secure the sides of the borehole or smooth out the contours to allow the experiment to be conducted. This is usually done with concrete or cement slurry resulting in an improvement of the ground at the borehole but can lead to possible false results for the test.

In soft ground (soils and weak rock), there are three types of pressuremeters available; a pre-bored pressuremeter (dilatometer type or Ménard type – Ménard invented the original pressuremeter), a self-boring pressuremeter and push-in pressuremeters. The pre-bored pressuremeters, typically 1 m long and 74 mm diameter, are lowered into a slightly oversized pre-bored hole. As the name implies, the self-boring pressuremeter bores itself into the ground, replacing the soil, and can operate up to pressures of approximately 4.5 MN/m^2. This pressuremeter is particularly useful for measuring horizontal stress. Push-in pressuremeters are usually 50 mm in diameter and displace the soil, but have a pressure capacity of only half that of the self-boring pressuremeters.

Determination of the principal* in situ *stresses using hydraulic fracturing prior to construction For any tunnel construction it is important to determine the primary stresses, i.e. the stresses in the ground prior to construction of the void (i.e. the tunnel). This will help the tunnel designer to estimate the likely stress redistribution when the tunnel is constructed and hence the loading on the tunnel lining. Of primary importance are the principal stresses, i.e. the largest and smallest possible stress where the shear stress is equal to zero. Further information on calculating stresses is given in section 3.2.

The vertical principal stress can easily be calculated because the unit weight, γ, and the height of material above the proposed tunnel axis depth, H, are easily determined, i.e. $\sigma_v = \gamma H$. This is not so with respect to the horizontal principal stress, $\sigma_h = K_0 \gamma H$. The value of K_0, the coefficient of lateral earth pressure at rest, is a difficult parameter to determine, especially as it can vary in magnitude in different directions (see section 3.4). It is assumed that the principal stresses are initially acting vertically and horizontally.

The following procedure can be used to determine the smallest lateral pressure and its direction in a borehole. It is important to ensure that there is a reasonable length of borehole above and below the location of the measurement and that this is crack free. This section of the borehole is then sealed with packers and pressurized with air or liquid (generally water) until there is a sudden drop in measured pressure. After noting the maximum pressure the system is closed and the smallest principal stress can be deduced from the adjusted pressure. The pressure drop develops when the ground fractures and the liquid flows into the ground. Two main fractures occur in the direction of the largest principal stress, σ_1. Figure 2.13b shows a diagram of the hydraulic fracturing test. In this figure, x and y are the principal stress directions and σ_1 and σ_2 are the principal stresses. In this example, the direction of the largest deformation gives the smallest principal stress direction and thus the smallest value of K_0. The largest value of K_0 in this example is found in the y-direction, but cannot be determined in this experiment. The value of K_0 determined from this experiment is still only an estimation and it is therefore advisable to do design calculations for a range of K_0 values.

Double load plate test With the double load plate test, load plates are pressed against the rock using a hydraulic jack (Figure 2.14). At the load plates the deformation of the ground is measured and is used to determine the deformation characteristics, and hence elasticity (Young's modulus, E).

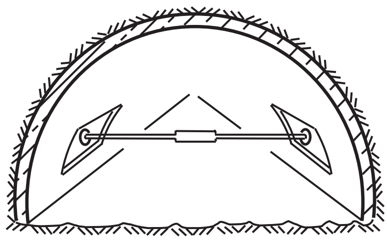


Figure 2.14
Diagram showing one method of conducting the double load plate test

The double load plate experiment is normally performed in test or trial tunnels where it is possible to use the opposite wall, or the crown or invert, as a reaction. Equation 2.2, based on Hooke's law, can be used to determine deformation modulus E_d:

$$E_d = \omega\left(1-\mu^2\right) r \frac{\Delta\sigma_m}{\Delta s_{Z,R}} \tag{2.2}$$

where μ is Poisson's ratio, ω is a constant for the type of load plate, flexible or rigid, r is the radius of the load plate, $\Delta\sigma_m$ is the difference between two load stages of the average normal stress on the plates, $\Delta s_{Z,R}$ is the difference between two load stages of the average settlements of the centre and edge of the plate.

2.3.3 Laboratory tests

After conducting *in situ* sampling it is possible to visually inspect these samples, describe the material in accordance with appropriate standards and carry out testing in the laboratory. It should be noted that the results from the samples tested only provide information on the sample itself and engineering judgement is essential to translate this information into ground characteristics (section 2.4).

It is important to visually inspect the samples collected in order to gain a preliminary profile with depth before conducting any laboratory experiments. For rock cores, total core recovery (percentage of core recovered, solid and pieces, relative to the overall length of the core interval), solid core recovery (total length of pieces of core recovered which have a full diameter, expressed as a percentage of the overall length of the core interval) and rock quality designation (RQD, see section 2.4.4.1) values should be obtained as these give an indication of the fracturing and fragmentation.

Depending on how the sampling has been carried out on site and the care taken in their subsequent handling, soil samples are classified into five quality classes with respect to the soil characteristics that remain unchanged (BSI 2007). These classes are described in Table 2.4, with quality class 1 being the best. The table shows the possible information, as indicated by the dots, which can be obtained for the five different quality samples. Also indicated on Table 2.4 are sample categories A, B and C (A being the best

Table 2.4 Quality classes for soil samples for laboratory testing (after BSI 2007)

Soil properties/quality class	*1*	*2*	*3*	*4*	*5*
Unchanged soil properties					
particle size	•	•	•	•	
water content	•	•	•		
density, density index, permeability	•	•			
compressibility, shear strength	•				
Properties that can be determined					
sequence of layers	•	•	•	•	•
boundaries of strata – broad	•	•	•	•	
boundaries of strata – fine	•	•			
Atterberg limits, particle density, organic content	•	•	•	•	
water content	•	•	•		
density, density index, porosity, permeability	•	•			
compressibility, shear strength	•				
Sampling category to be used	A	A	A	A	A
			B	B	B
					C

quality and C being the worst). These relate to the techniques used in the field for obtaining the samples. For example, drive sampling, in which a tube or a split-tube sampler having a sharp edge at its lower end is forced into the ground either by a static thrust (by pushing), by dynamic impact or by percussion are mostly category A or B sampling methods. Rotary core sampling methods, in which a tube with a cutter at its lower end is rotated into the ground, are usually category B. Auger sampling with hand or mechanical augers are usually category C sampling methods.

Although samples are often described as disturbed or 'undisturbed', there is no such thing as a truly undisturbed sample as the very act of retrieving the sample from the ground disturbs it, the stress conditions are changed for example. Hence the term 'undisturbed' is often written in parentheses to indicate this fact. The quality of the sampling technique also dictates how disturbed the sample is. For example a 'bagged' sample as described above is highly disturbed.

Table 2.5 Particle size ranges

Component	*Size range (mm)*
Clay	< 0.002
Silt (fine, medium and coarse)	0.002–0.006, 0.006–0.02, 0.02–0.06
Sand (fine, medium and coarse)	0.06–0.2, 0.2–0.6, 0.6–2.0
Gravel (fine, medium and coarse)	2.0–6.0, 6.0–20.0, 20.0–60.0
Cobbles	60.0–200.0
Boulders	> 200.0

For investigations involving soil samples in the laboratory, it is important to determine the water content, the particle size distribution, the mineralogy of clay soils, and the percentage of air voids associated with the material. Table 2.5 gives an indication of the particle size ranges associated with various soil components.

For soils containing clay-sized particles, plasticity information is particularly useful to gauge its behaviour, for example Atterberg limits (liquid limit (w_L) and plastic limit (w_P)), and plasticity indices (plasticity index ($I_P = w_L - w_P$) and liquidity index ($I_L = (w - I_P)/(w_L - w_P)$ where w is the water content). Figure 2.15 shows a plasticity chart showing the A-line, i.e. the distinction between soils with predominantly clay-sized particles (above the A-line) and those with predominantly silt sized particles (below the A-line). For example, a high plasticity clay, i.e. one with high I_P and w_L values, can be susceptible to large swelling and shrinkage behaviour when subjected to small changes in water content.

From the Atterburg limits and the water content, w, of the soil, the consistency index, I_c, can be determined as shown in equation 2.3.

$$I_c = \frac{w_L - w}{I_P} \tag{2.3}$$

Consistency index is a useful way of estimating the state or condition of silts and clays (Table 2.6).

Furthermore, it is important to determine the time dependent behaviour of the materials in the long-term. For example, for fine grained soils it is useful to do oedometer (one dimensional consolidation) tests as this can

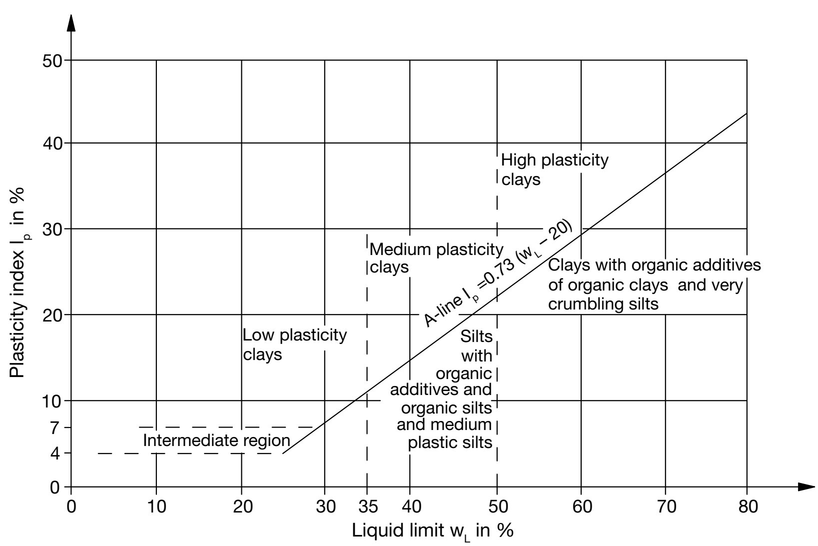


Figure 2.15 Plasticity (Casa grande) chart (after DIN 2006)

Table 2.6 Consistency index (BSI 2004c and Stein 2005)

Consistency of silts and clays	*Consistency index (I_c)*	*Description of soil*
Very soft	< 0.25	When pressing it in the fist, the soil squirts between the fingers.
Soft	0.25 to 0.50	Easily kneaded
Firm	0.50 to 0.75	Hard to knead, but can be rolled in the hand to threads of about 3 mm in diameter without tearing or crumbling.
Stiff	0.75 to 1.00	When trying to roll it into threads of about 3 mm in diameter, it crumbles or tears, but it remains moist enough to be able to reform it into a lump.
Very stiff	> 1.00	When dried its appearance (colour) is very light. It cannot be kneaded, but only broken. Reforming it into a lump is no longer possible.

give information on the characteristics of how it will deform with time and also its permeability can be estimated. The parameters obtained from this test are coefficient of volume compressibility, m_v (note, this parameter is stress dependent and so the value has to be calculated for the appropriate stress range), coefficient of consolidation, c_v, and vertical drained deformation modulus, E_v'.

In addition, for clayey and unsaturated soils, the swelling characteristics should be established as any increases in volume could induce large forces onto the tunnel lining that need to be included in any structural analysis of tunnel linings.

For rock, a point load index test can be conducted to obtain the point load index strength, I_S, for the material (ISRM 1985). This is conducted by applying a point load diametrically across the rock core. The I_S value is determined from equation 2.4.

$$I_S = \frac{L}{d^2} \tag{2.4}$$

where L is the load required to break the specimen and d is the core diameter. I_S varies as a function of size and so a size corrected value corresponding to a d = 50 mm, i.e. $I_{S(50)}$, is used. This test can also be conducted on blocks and lumps of material. If d is in millimetres, an approximate relationship between I_S and unconfined compressive strength, UCS, is given by equation 2.5 (Hoek and Brown 1997).

$$UCS = (14 + 0.175d)I_S \tag{2.5}$$

There are also laboratory tests for determining the abrasiveness of rocks in order to gauge the wear on cutting tools. One such test is the CERHAR Abrasiveness Test (CAI Test) developed at the Centre d'Étude et de Recherche du Charbon (Büchi *et al.* 1995). This test provides an index value that can be used as a gauge for the abrasiveness of different rock types. The index value ranges from 0.3 for very low abrasiveness to 6.0 for extremely abrasive. Using this test, basalt has an abrasiveness index of 2.7, gneiss 4.4 and granite 4.9.

Further details on laboratory tests can be obtained from standards, such as Eurocode 7: Geotechnical Design – Part 2: Ground investigation and testing (BSI 2007). This document includes guidance on both soil and rock testing. In addition, details on identification and description, and classification of soft ground can be obtained from BSI (2002a) and BSI (2004a) respectively. For the identification and classification of rocks the reader is referred to BSI (2003). Details of shear strength tests can be obtained from BSI (1990). In addition, Head (1997 and 2008) and Head and Keeton (2010) provide extensive descriptions of all the main soil laboratory tests.

In order to determine the strength parameters for rock and soil, and the modulus, E, uniaxial and triaxial tests can be conducted. These tests are briefly described below.

UNIAXIAL TEST

This is a standard experiment for rock cores in order to obtain failure parameters (unconfined compressive strength UCS, σ_u, ε_u), E and μ. For the uniaxial test the core sample is loaded in one direction. Laboratory samples are made up of cylindrical shaped cores with diameters of at least 90–100 mm. During the test, load is applied to the end of the sample (Figure 2.16a). Some considerations for sample preparation include:

- the end surfaces must be flat and even;
- the ends must be parallel and at right angles to the sample axis in order to avoid bending stresses being induced into the sample (which will give a reduced value of strength);
- during the test, friction is generated between the end surfaces of the sample and the end loading platens. This has the potential for increasing the failure load of the sample as it restricts the sample expansion. This is negligible if the height (h) to diameter (d) ratio is greater than or equal to 2 (h/d $\geq$ 2). If h/d is less than 2 (h/d $<$ 2), for example if there is not enough intact material in the core sample, then equation 2.6 can be used to adjust the stress, $\sigma_{u,adj}$.

$$\sigma_{u,adj} = \frac{8 \cdot \sigma_u}{7 + 2 \cdot \frac{d}{h}} \tag{2.6}$$

Care should be taken as the failure load can be reduced by up to 11% between stumpy and slender samples.

The rate of loading is also important. However, as the load increment is dependent on the deformation of the material, there are no set values. The strain calculated from the displacement measuring transducer attached to the test rig should increase at a rate of, $\dot{\varepsilon}$ = 0.05%/min, with the expectation that the test should last at least 5 minutes. This is the general guideline for a material that does not deform much. If a more ductile material is being tested and large strains are expected, the rate can be a lot higher, for example rock salt $\varepsilon_u \approx 10\%$, therefore the rate is increased to 0.25%/min. In comparison, for concrete $\varepsilon_u \approx$ 6 to 8‰, i.e. approximately 10 times smaller than for soft rock, and for granite $\varepsilon_u \approx$ 3 to 4‰.

Experiments in which either the strain or stress are regulated are called strain and stress controlled tests respectively. For strain controlled tests it is unavoidable that the sample is completely destroyed at ε_u and so there is no information on the after failure strain response. However, the demands put on the testing machine, i.e. the control techniques, are less than those for a stress controlled test.

One of the most important material parameters is obtained from Hooke's law, which is the ratio of stress over strain (where E is the elastic modulus in equation 2.7).

$$E = \frac{\Delta\sigma}{\Delta\varepsilon} \tag{2.7}$$

The required values are taken from the stress/strain graph (Figures 2.16b, 2.18c). Depending on the section of the curve investigated, several different moduli can be identified as shown in Figure 2.17.

As a rule, to determine the value of E from the results of laboratory experiments, look at the $\sigma - \varepsilon$ graph where the sample behaves elastically and also where it is linear, i.e. where E is constant (equation 2.7 assumes linearity). When doing an analysis on hard rock, the most reliable results are obtained using a reloading modulus. As a rule the middle third of the reloading modulus should be used and additionally the initial load should not exceed 60% of the failure load as this avoids local overstressing because of inhomogeneities and microcracks within the sample. This means that E can be determined from the intact sample. The value of E is often dependent on the stress situation; generally the higher the isotropic stress, the higher E. It is therefore worth noting that in the case of the actual ground, there is a possibility that E will change with depth and as a consequence the value of E at the crown and invert of the tunnel could be different.

The value of E in rock can well exceed 100,000 MN/m^2 (this is an order of magnitude greater than the value of concrete). This means that E of the ground cannot be higher than that of the rock/soil itself and in reality, on

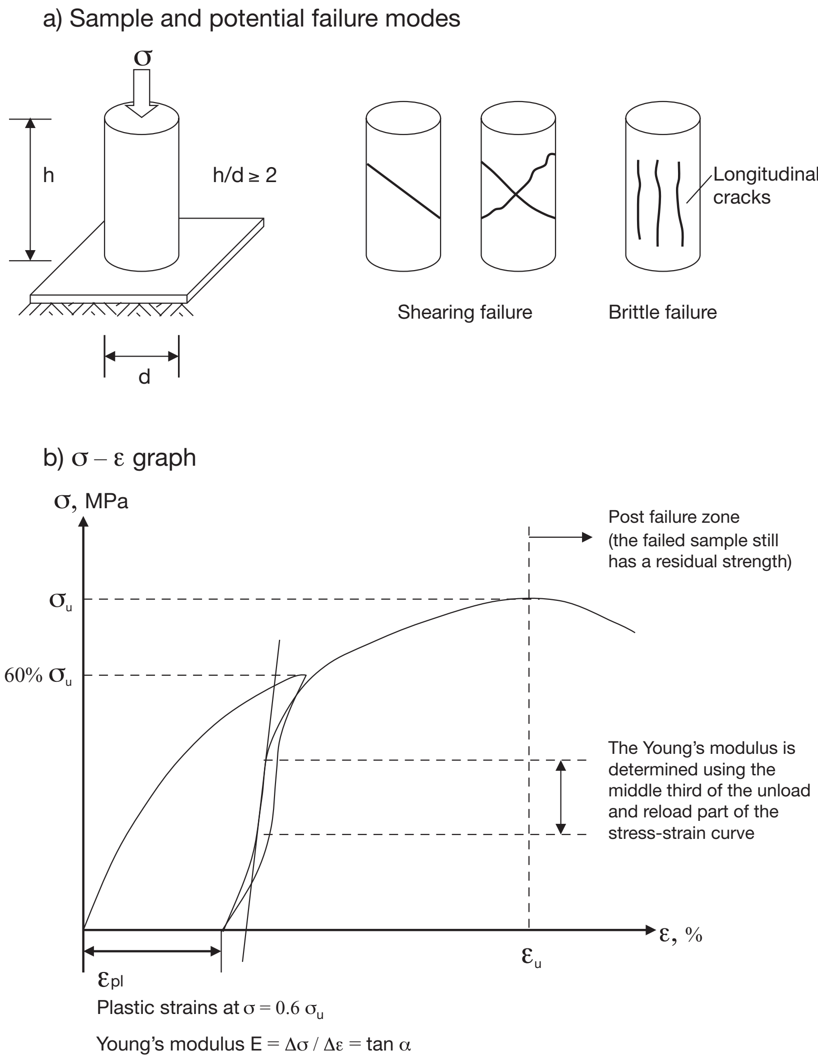


Figure 2.16 Uniaxial unconfined compression test

average, the value of the ground is 10 to 20 times less. The value of E for the ground is therefore an estimate and so it is always advisable to plan and do calculations based on a range of values.

The value of Poisson's ratio, μ, is important for structural analysis as this provides the ratio of horizontal strain to vertical strain (equation 2.8).

$$\mu = \frac{\Delta\varepsilon_{horiz}}{\Delta\varepsilon_{vert}} \tag{2.8}$$

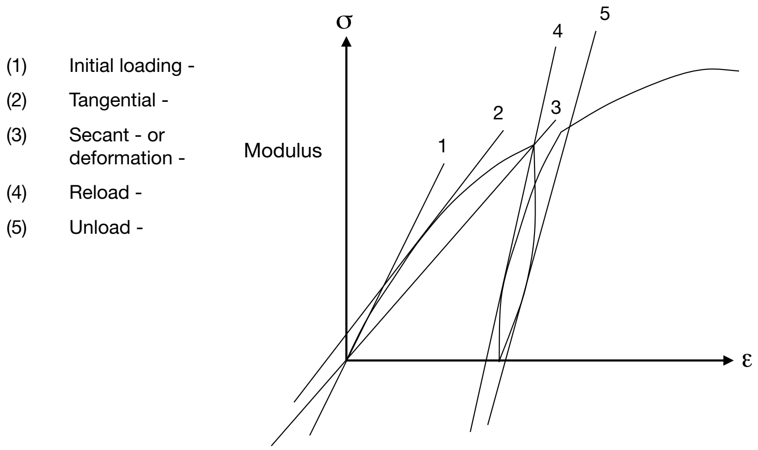


Figure 2.17 Definitions of different modulus values

It is determined at the same location on the stress/strain graph as E. The Poisson's ratio, μ has values of 0 to 0.5. A material with $\mu = 0.5$ maintains volume under load. It should be noted that in German literature, for example, Poisson's ratio has the inverse definition, i.e. it is between 2 and infinity.

The Modulus Ratio can be a useful parameter and is defined as E/UCS and is approximately 300 for most rocks; > 500 for some strong rocks and stiff limestones; < 100 for deformable rock, clays and some shales (Waltham 2002).

TRIAXIAL TEST

The difference between the triaxial and uniaxial test is the application of pressure to the circumference of the sample as well as vertically along the sample axis (Figure 2.18a). It can be considered that the uniaxial test is one in which $\sigma_2 = \sigma_3 = 0$, i.e. a special case of the triaxial test. Therefore, the experimental procedures are similar and so is the analysis. The sample size requirements, i.e. the h/d ratio, and the need for parallel end platens to be at right angles to the sample axis are exactly the same. In addition, these tests can be conducted under both stress and strain control. The only difference is that in a strain controlled triaxial test the axial and radial strains are increased equally until the axial strain has reached the desired value. In the stress controlled test the radial stress is kept constant while the axial stress is increased (it should be noted that in an indirect tension test, the radial stress is increased and the axial pressure is kept constant).

Triaxial tests are usually done when there is an interest in the shear strength parameters from which one can estimate the stand-up time for the ground, which is particularly important for weaker materials such as soils.

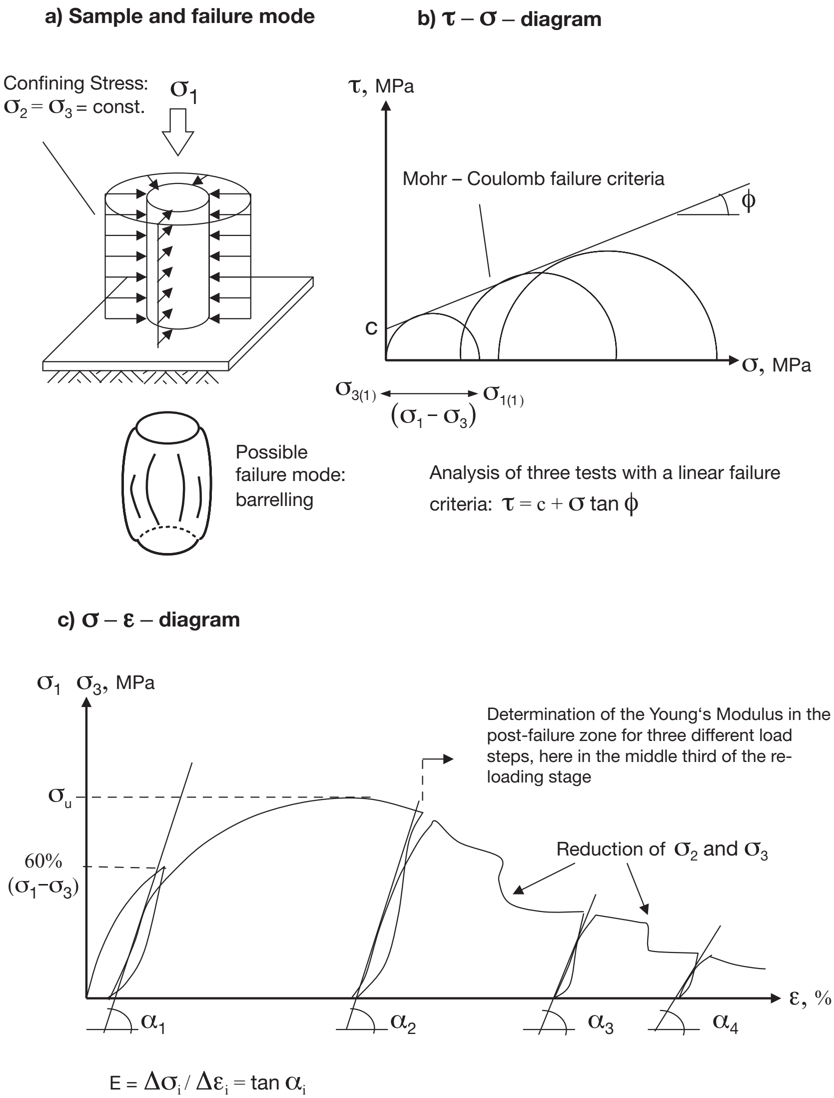


Figure 2.18 Triaxial test

The stand-up time allows an understanding to be obtained of the time that an open void can stand on its own without any support. Another reason to obtain the shear strength parameters of the material is to gain an idea of the deformation characteristics of the sample, i.e. deformations in the tunnel that are independent of E. Deformations that are dependent on E are elastic deformations, however, in addition to these deformations, there are plastic deformations, which are dependent on the apparent cohesion and internal friction angle of the material. These can get much larger than the elastic deformations. Plastic deformations in rock can develop because of crevasses and softening zones. The apparent cohesion, c, and internal friction angle, ϕ, give indications of how strong the matrix structure of the whole ground is when disturbed.

The triaxial compression test can determine E and the compression stability of the soil and rock, and they can be described in the same way as the uniaxial test. It should be noted that in the $\sigma - \varepsilon$ plot, the stress difference, $\sigma_1 - \sigma_3$, should be plotted on the y-axis (Figure 2.18c).

Figure 2.18b shows the results of three triaxial tests on the same material at different confining stress, σ_3 ($\sigma_1 = \sigma_3$ + the additional vertically applied stress during the test, and $\sigma_2 = \sigma_3$) on a $\tau - \sigma$ diagram (shear stress versus direct stress). For each pair of principal stresses (principal stresses are stresses acting on a plane where there are no associated shear stresses) Mohr circles can be drawn. In order to determine the shear parameters the failure condition needs to be defined, which in this case uses the linear Mohr-Coulomb failure condition as a basis for this estimation. The Mohr-Coulomb strength line forms an envelope for all the Mohr's circles by touching each circle at one point (i.e. a tangent to each Mohr's circle). In theory only two circles would be sufficient to construct this tangent, but, as we know, experimental results are subject to variability and so it is advisable to do at least three tests. The intercept with the shear stress axis, τ, is called apparent cohesion, c, and the gradient of the line is the internal friction angle, ϕ. Depending on the type of triaxial test conducted, different strength parameters can be obtained. For example, a quick undrained test will give undrained shear strength parameters (c_u and ϕ_u), which are useful for short-term design calculations in fine grained soils. A consolidated undrained test with pore water pressure measurement or a drained test will allow effective shear strength parameters (c' and ϕ') to be determined. These are used in long-term designs in fine grained soils or short-term (and long-term) designs for coarse grained soils. Effective shear strength parameters are indicated by the ′ next to the parameter.

It should be noted that triaxial extension tests can also be conducted to obtain information on the tensile behaviour of the rock or soil.

There is also more of an emphasis these days on obtaining information related to the small strain behaviour of soils. This is because the stiffness of soils is highly non-linear with increasing strain and is considerably larger at lower strains. Therefore, to get accurate indications of this relationship in triaxial tests, various techniques can be used. These include on-sample strain measurements, rather than using external measurements, and bender elements (where a vibrating element induces a shear wave into the sample that is picked up by another element on the opposite side of the sample; the travel time being indicative of the sample stiffness, in a similar way to seismic tests for the ground). This knowledge is important as input parameters for constitutive soil models used in numerical analyses (section 3.6).

The main questions that must be asked are: do these laboratory values determined from small samples of material relate to the ground *in situ*? What effect is there on these values from, for example, layering, fissures and water? It is ultimately the ground characteristics that are critical when designing a tunnel.

2.4 Ground characteristics/parameters

After determining the soil/rock material parameters from laboratory testing, for example, it is important to determine the ground parameters as these can be significantly different. It is only possible to do this in conjunction with the engineering geology report. The engineering behaviour of the ground is not solely controlled by the strength and stiffness of the rock/soil material. Discontinuities, such as joints, faults and bedding, act to reduce the *in situ* ground material properties compared to those of the rock/soil material. Only in the very rare case of a homogeneous and isotropic ground are the characteristics of the ground equal to those obtained for the rock/soil material. However, there is no 'recipe' as to how to determine ground parameters from field and laboratory experiments and geological descriptions.

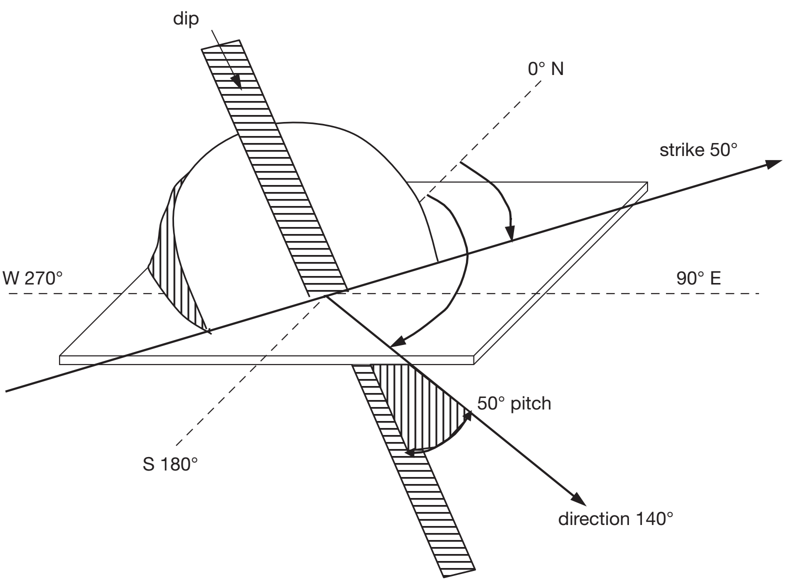
It is necessary to take a holistic look at the ground, treating it as a matrix of several soils/rocks with, for example, dips and faults (defined below), and including any water. From the information obtained from a site investigation, the objective is to develop a geological model, and a hazard model, for the site, i.e. to highlight important information relevant for the design and construction (and potentially longer-term issues in the case of environmental aspects).

Material (soil or rock) tested in the laboratory is only a sample of the whole ground mass and these tests can only to some extent simulate the real conditions. A laboratory test of a rock sample that gives 40000 MPa for the Young's modulus, for example, may only yield 500 MPa on site (based on experience). This large reduction in Young's modulus can depend on large faults or lots of small faults, or faults filled with clayey material (the clay acts as a lubricant and the ground can move without exceeding the failure stress of the rock). Faults are fractures in the ground where relative displacements have occurred. Section 2.4.1 gives an example of how layering can affect the modulus of the ground. Reference should be made to Hoek and Brown (1997) for further information on how to practically estimate ground strength. Hoek and Diederichs (2006) also give information on empirically estimating the ground modulus. For a comprehensive list of geological terms, the reader is directed to a dictionary of geology, for example Whitten and Brooks (1972).

The descriptive engineering geology report resulting from the site investigation has a very important function as it contains information, for example, on the size and frequency of faults, about their characteristics, i.e. healed, closed, open, filled and the type of filling material and also the angle and direction of the faults relative to the tunnel axis in both the vertical and horizontal direction. The report also gives information on layering, folding of the strata (i.e. undulations), fabric, fragmentation, separation layers related to the ground, and also two angles of dip (azimuth and inclination relative to the tunnel location). The definitions for 'dip' and 'strike' are shown in Figure 2.19.

The positions of any discontinuities, any inhomogeneities and any anisotropy should be noted. Discontinuities are where the ground properties change abruptly. Inhomogeneities are where the properties of the ground change, either due to a change in material or to more subtle changes within the same material. Anisotropy is where the properties of the ground are different in various directions, for example different in vertical and horizontal directions. Unconformities are also important as these are planes or breaks between two sequences of rock with different dips.

Tables 2.7 and 2.8 provide information on some important geological descriptions about the ground used when designing tunnelling projects. These geological descriptions can affect the stress redistributions around the tunnel, influence the support requirements and also affect the loads



Definitions of dip and strike:

Strike is the direction in which a horizontal line can be drawn on a plane in relation to geographic north (Whitten and Brooks 1972).

Dip is the angle of maximum slope of the beds of rock measured from the horizontal at any point (Scott 1984).

Figure 2.19 Definitions of dip and strike

Table 2.7 a) Descriptions used for layer thickness and joint spacing (after BSI 1999, Prinz and Strauß 2006, and Anon 1977), b) Bedding inclination (after Forschungsgesellschaft für das Straßenwesen 1980)

a)		*Layer thickness*	*Spacing between joints*	*b)*	*Ranges of inclination and description*
	> 2 m	Very thickly bedded	Extremely wide		0° to 10° horizontal
0.6	to 2 m	Thickly bedded	Very wide		10° to 30° level
0.2	to 0.6 m	Medium bedded	Wide		30° to 60° inclined
0.06	to 0.2 m	Thinly bedded	Moderately wide		60° to 90° steep
0.02	to 0.06 m	Very thinly bedded	Moderately narrow		
0.006	to 0.02 m	Laminated	Narrow		
0.002	to 0.006 m	Thinly laminated	Very narrow		

Table 2.8 State of weathering (after BSI 1999, and Forschungsgesellschaft für das Straßenwesen 1980)

State	*Rock sample characteristics*	*Ground characteristics*
Unweathered	No effect of weathering visible	No loosening of the interfaces due to weathering
Partially weathered	Some noticeable weathering of individual mineral particles on freshly fractured surfaces	Partial loosening at discontinuities
Distinctly weathered (softened)	Softening due to weathering, but minerals still bonded together	Complete loss of strength at discontinuities

on the tunnel lining. Table 2.7 provides some descriptions related to the thickness of the layers, spacing and angle of dip for strata within the ground. Table 2.8 shows some weathering descriptions. Weathering weakens the ground through the action of water, wind and temperature.

Groundwater levels should be noted, together with the chemical composition of the natural groundwater with respect to concrete attack (sulphates and chlorides). Groundwater levels must be carefully assessed as there can be multiple levels depending on the relative permeability of the strata making up the ground. For example, in London, UK there is an upper and a lower aquifer and hence two groundwater levels.

Information on the geological history, age and mineralogy composition of the rock or soil (stratigraphy and petrography, i.e. the description and systematic classification of the rocks) is important. In clay soils, for example, it is the microstructure of the material that controls its properties such as strength and compressibility.

It is of primary importance for the tunnel builder to understand how all these factors influence the mechanical behaviour of the rock within the overall ground. Due to the variety of the influences there is no simple or definitive answer with respect to rock behaviour.

In addition, an extensive and well performed site investigation is only equivalent to a ‘pin prick’ as far as the problem is concerned and therefore there is always uncertainty and this can be found in the choice of the expert’s words when writing the report. While reading the engineering geological report, be aware of the phraseology used: ‘it can’t be excluded that . . .’, ‘the layer boundaries could be uneven . . .’, or ‘expect fault thickness of, for example, 3 cm and more . . .’. Unfortunately, there is no definite mathematical formula, so when judging such non-committal language your own impressions of the cores and site can be very helpful. Due to the unavoidable uncertainties of site investigations, it is extremely important to check the predicted situation during the construction phase.

2.4.1 Influence of layering on Young’s modulus

It often occurs that the ground is layered and there is a potentially large difference in material behaviour in these various layers. In the laboratory, layers can only be investigated separately, and in many cases soft layers are not present in the sampling core as they are washed out with the drilling fluid. In such cases it can be impossible to estimate the influence of layering on the overall behaviour of the ground. However, if it is assumed that the layering is uniform, it is possible to do a simple estimation. An example is shown in Figure 2.20 and the calculation is given below.

$$\varepsilon = \alpha \cdot \frac{\sigma}{E_S} + \beta \cdot \frac{\sigma}{E_C} \quad \Rightarrow \quad E = \frac{1}{\dfrac{\alpha}{E_S} + \dfrac{\beta}{E_C}} = \text{overall E} - \text{modulus}$$

where α and β are percentages of the soil material in the whole.

$$\alpha = \frac{d}{t+d} \quad \text{with} \quad d = \sum_i d_i \quad \text{and} \quad \beta = \frac{t}{t+d} \quad \text{with} \quad t = \sum_i t_i$$

$$\alpha = \frac{1.50}{1.53} \cong 0.98 \qquad \beta = \frac{0.03}{1.53} \cong 0.02$$

$$E = \frac{1}{\dfrac{0.98}{7000} + \dfrac{0.02}{70}} \cong 2350\text{MPa}$$

This illustrates that even though the clay is only 2% of the overall soil mass, it has a significant effect and reduces the overall E of the system by about two-thirds of the original sandstone value.

In this very simple example, no account has been taken of any crevasses, faults etc. which would reduce this value still further.

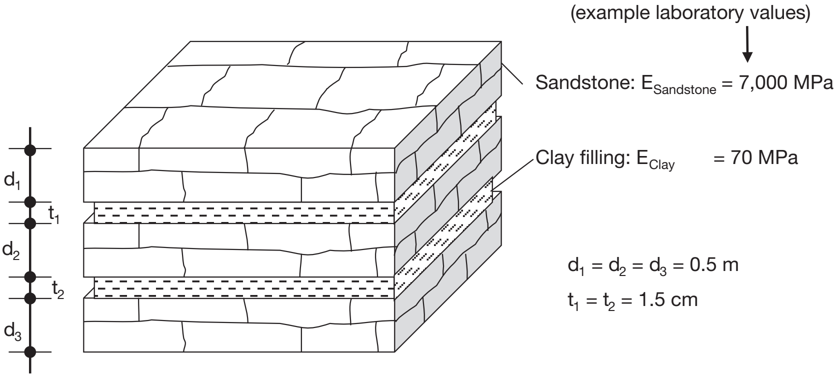


Figure 2.20 Example of how layering affects E-modulus

2.4.2 Squeezing and swelling ground

Squeezing and swelling of a ground can create high pressures on tunnel supports and such situations must be managed either within the construction process or by providing flexible or 'ductile' supports.

SQUEEZING

Squeezing rock is a plastic material that moves into an underground opening primarily because of pressures exerted by loads of overlying rocks. Although a clear distinction between swelling and squeezing ground is not always possible, squeezing ground differs from swelling in that it undergoes no appreciable volume increase owing to the penetration of water. However, ingress of water, even in small amounts, promotes plastic behaviour and contributes to the easy movement of squeezing ground (Wahlstrom 1973).

Unlike swelling ground, movements in squeezing ground involve materials outside the immediate area of the tunnel opening, and the volumes of material that have the potential of moving into the tunnel opening may be very large.

When the uniaxial compressive strength of the rock is less than 30% of the *in situ* stress, severe or extreme squeezing can occur (Thomas 2009a).

SWELLING

Swelling in mineral aggregates is caused by one or a combination of processes including (Wahlstrom 1973):

- adsorption of films of water attracted and held by surface forces of very small mineral particles;
- adsorption of free water by clay minerals such as montmorillonites;
- hydration;
- expansion of pore water as a consequence of the release of confining pressure.

With the exception of anhydrites, which swell because of chemical incorporation of water to form gypsum, swelling is most pronounced in rocks that contain abundant clay minerals (montmorillonite having the greatest volume change, with other clay minerals such as illite and kaolinite being much less susceptible to volume change), or clay-sized particles of other minerals. Some common materials that swell are shales, claystones and mudstones, where the swelling is directly proportional to the amount of clay minerals, especially montmorillonite, that are present.

Swelling is commonly a slow process, primarily because of the fine grain of the minerals that are prone to swelling. The permeability of such minerals is low, and penetration of water of any origin takes considerable time. Swelling is likely to be accelerated if the construction process brings water in contact with a soil, which is capable of swelling.

2.4.3 Typical ground parameters for tunnel design

Due to the many influences that determine the ground behaviour and the disparity that often exists between this and the properties of the individual soils/rocks it is not possible to determine binding parameters for a rock or soil type, i.e. it cannot be put into any 'Standard'. However, some typical values are provided in the following tables. These provide the reader with a 'feel' for the magnitudes of the values involved with certain parameters.

TYPICAL ROCK AND SOIL PARAMETERS

Tables 2.9 to 2.12 provide some typical values for the strength and deformation characteristics of rocks, generic hardness classes, and some strength and permeability values for soils respectively.

Table 2.9 Strength and deformation characteristics of some typical rocks (after Reuter 1992 and, Klengel and Wagenbreth 1987)

Ground	*Compressive strength (MPa)*	*E (MPa)*
Basalt	160–400	48000–105000
Dolomite	50–180	32000–100000
Gabbro	80–345	75000–120000
Gypsum	9–40	10000–29000
Granite	100–300	37000–72000
Chalk	20–240	16000–90000
Sandstone	10–290	6000–71000
Rocksalt	20–30	16000–24000
Slate	20–210	23000–85000
Concrete C20/25	25 (cube crushing strength)[a]	29000

Note: (a) For the influence of the shape of the specimens on the compressive strength see uniaxial compression test

Table 2.10 Generic hardness classes (after Reuter 1992 and Fecker and Reik 1987, simplified)

Ground	*Compressive strength (MPa)*		*Apparent cohesion (kN/m²)*	*Internal friction angle (degrees)*
Hard rock	high	> 100	> 2000	> 40
	middle	20–100	200–2000	30–40
	low	5–20	20–200	20–30
Transitional rock	very low	1–5	< 20	< 20
Soft ground	very soft	< 1	0– 10	0–10

Note: That the numbers in this table should only be treated as 'ball park' values.

Table 2.11 Example shear strength parameters for soils (after Waltham 2002, BSI 1986)

Soil type	*Apparent cohesion*[a] *(kN/m²)*	*Internal friction angle (degrees)*
Till and tertiary clays	Hard > 300	
	Very stiff 150–300	
	Stiff 75–150	28
Alluvial clay	Firm 40–75	
	Soft 20–40	19
Medium dense sand	–	32–36
Dense sand	–	36–40
Sandy gravel	–	35–50
Silty sand	–	27–34
Clayey silt	20–75	25
Consistency of clays	*(after BSI 1986)*	
Very soft	< 20	–
Soft	20–40	–
Firm	40–75	–
Stiff	75–150	–
Very stiff	150–300	–

Note: (a) Short-term or undrained shear strength (c_u), the long-term or drained (effective) shear strength (c') for clays is difficult to quantify, but is usually relatively small (< 10–15 kN/m²) *and decreases rapidly on disturbance and weathering.*

TYPICAL GROUND PARAMETERS

Table 2.13 provides some typical values for the shear strength of various rocks and Table 2.14 provides an example of a rock classification system. Further information on rock mass classification systems is given in section 2.4.4.

Table 2.12 Typical permeability values for soils

Type of soil	*Permeability, k (m/s) – limiting values*
Coarse gravel	10^{-1} to 5
Fine gravel	10^{-4} to 10^{-2}
Coarse sand	10^{-5} to 10^{-2}
Medium sand	10^{-6} to 10^{-3}
Fine sand	10^{-6} to 10^{-3}
Silt	10^{-9} to 10^{-5}
Clay	10^{-12} to 10^{-8}

Note: Below 10^{-8} is very low permeability, 10^{-6} to 10^{-4} is permeable and above 10^{-2} is very high permeability

Table 2.13 Shear parameters for several rocks (after Reuter 1992)

Rock	*Condition*	*Apparent cohesion (kN/m²)*	*Internal friction angle (degrees)*
Granite	–	200–3000	30–50
Sandstone	Parallel to joints	100	60
Limestone	Joints without fill	700	40
Limestone	Joints with loose material	100–300	22–27
Limestone	Joints with clayey fill	0–100	11–17

Table 2.14 Rock mass classification (after Bieniawski 1984 and Fecker and Reik 1987)

Description	*Very good rock*	*Good rock*	*Medium rock*	*Weaker rock*	*Very weak rock*
Average stand-up time	10 years with a 5 m span width[a]	6 months with a 4 m span width	1 week with a 3 m span width	5 hours with a 1.5 m span width	10 minutes with a 0.5 m span width
Apparent cohesion of the rock mass (kN/m²)	> 300	200–300	150–200	100–150	< 100
Internal friction angle of the rock mass (degrees)	> 45	40–45	35–40	30–35	< 30

Note: (a) The span width indicates the unsupported length in the direction of the tunnel advance.

2.4.4 Ground (rock mass) classification

In his book 'Engineering Rock Mass Classifications', Bieniawski (1989) states 'Rock mass classifications are not meant to be taken as a substitute for engineering design. They should be applied intelligently and used in conjunction with observational methods and analytical studies to formulate an overall design rationale with the design objectives and site geology.'

The objectives of rock mass classifications are therefore to (after Bieniawski, 1989):

- identify the most significant parameters influencing the behaviour of a rock mass;
- divide a particular rock mass formation into groups of behaviour, that is, rock mass classes of varying quality;
- provide a basis for understanding the characteristics of each rock mass class;
- relate the experience of rock conditions at one site to the conditions and experience encountered at others;
- derive quantitative data and guidelines for engineering design;
- provide a common basis for communication between engineers and geologists.

An early classification system for soft ground is the Tunnelman's ground classification as shown in Table 2.15 and provides information on the likely tunnel working conditions and some idea of the types of soils in which these conditions might occur.

For harder ground, a number of classification systems have been developed. Three of these classification systems are briefly described in this book: Rock Quality Designation (RQD), which is one of the simpler classification methods and is described in section 2.4.4.1, the Rock Mass Rating (RMR) system and the Rock Mass Quality Rating (Q-method) (sections 2.4.4.2 and 2.4.4.3 respectively). Further details are given in Appendix A. The reader is encouraged, however, to read the original, and subsequent, publications by the relevant authors of these systems in order to fully appreciate their usefulness and limitations.

2.4.4.1 Rock Quality Designation

The Rock Quality Designation index was developed by Deere in 1967 (Deere *et al.* 1967, Deere 1989) to provide a quantitative assessment of ground quality from drill cores. RQD was developed for assessing rock and can be misleading in soft ground. RQD is defined as the total length of 'solid' core pieces each greater than 100 mm between natural (not drill-induced) discontinuities expressed as a percentage of the total length of each core run, measured along the core axis. A solid core is defined as a core with at least one full diameter (but not necessarily a full circumference)

Table 2.15 Tunnelman's ground classification (after Thomson 1995, from Terzaghi 1950)

Classification	*Tunnel working conditions*	*Representative soil types*
Hard	Tunnel heading may be advanced without roof support	Very hard calcareous clay; cemented sand and gravel.
Firm	Tunnel heading can be advanced without roof support and the permanent support can be constructed before the ground will start to move.	Loess (e.g. wind blown soil deposits) above the water table; various calcareous clays with low plasticity such as the marls of South Carolina.
Slow ravelling	Chunks or flakes of material begin to drop out of the roof or the sides some time after the ground has been exposed.	Fast ravelling[a] occurs in residual soils or in sands with clay binder below the water table. Above the water table the same soils may be slow ravelling or even firm.
Fast ravelling	In fast ravelling ground, the process starts within a few minutes; otherwise it is referred to as slow ravelling	
Squeezing	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in the ground surrounding the tunnel. (May not be noticed in the tunnel, but will cause surface subsidence.)	Soft or medium-soft clay.
Swelling	Like squeezing ground, moves slowly into the tunnel, but the movement is associated with a very considerable volume increase in the ground surrounding the tunnel.	Heavily precompressed clays with a plasticity index in excess of about 30; sedimentary formations containing layers of anhydrite.

Table 2.15 (*continued*)

Classification	*Tunnel working conditions*	*Representative soil types*
Cohesive running	The removal of the lateral support on any surface rising at an angle of 34° to the horizontal is followed by a ‘run’, whereby the material flows like granulated sugar until the slope angle becomes equal to about 34°. If the ‘run’ is preceded by a brief period of ravelling, the ground is called cohesive running.	
Running		Running occurs in clean, coarse or medium sand above the water table.
Very soft squeezing	Ground advances rapidly into the tunnel in a plastic flow.	Clay and silts with high plasticity index.
Flowing	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and sides, but also through the bottom. If the flow is not stopped, it continues until the tunnel is completely filled	Any ground below the water table that has an effective grain size in excess of about 0.005 mm.
Bouldery	Problems incurred in advancing shield or poling; blasting or hand-mining ahead of the machine may be necessary.	Boulder glacial till; rip-rap fill; some landslide deposits; some residual soils. The matrix between the boulders may be gravel, sand, silt, clay or combinations of these.

Note: (a) The term ‘ravelling’ is used to describe a situation when the ground collapses at the crown of the tunnel.

measured along the core axis between two natural discontinuities (Davis 2006). The procedure is shown on an example core in Figure 2.21.

The RQD provides a general assessment of rock quality and can be used as a basis for descriptive rock quality terms as shown in Table 2.16. However, it is limited to the mechanical structure of the rock and provides no information on discontinuity properties or strength (Davis 2006).

There are some potential issues with assessing RQD in the field or when reviewing borehole logs as it is frequently mis-logged. The key issues are (Davis 2006):

- natural discontinuities and drilling features are often not differentiated;
- drillers often only include cores of full circumference greater than 100 mm instead of full diameter cores.

Both of these issues lead to reduced RQD values.

Palmström (1982) suggested that if no core is available, but continuity traces are visible in surface exposures or exploration adits, the RQD may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free ground is given in equation 2.9.

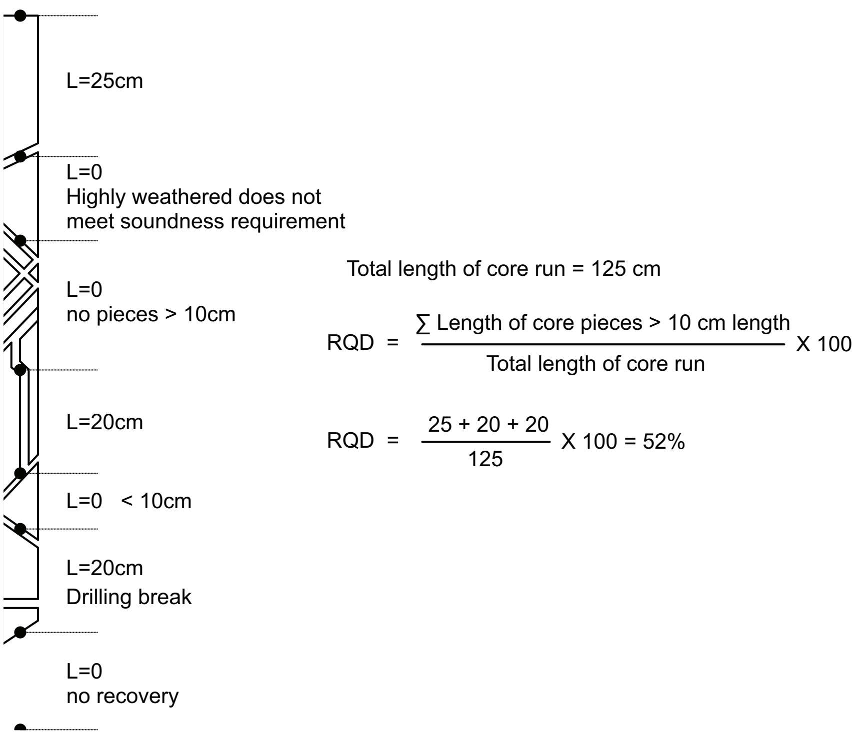


Figure 2.21 Procedure for measurement and calculation of RQD (after Deere and Deere 1989, used with permission from Don Deere)

Table 2.16 RQD values related to rock quality descriptions (after Deere and Deere 1989, from Deere *et al.* 1967)

RQD (%)	*Rock quality*
<25	Very poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Excellent

$$RQD = 115 - 3.3J_v \tag{2.9}$$

where J_v is the sum of the number of joints per unit length for all discontinuity sets.

2.4.4.2 Rock Mass Rating

The Rock Mass Rating system was developed by Bieniawski in 1972 and has been modified over the years as more data have become available (Bieniawski 1989). It should be noted that the RMR system was developed for hard rock conditions and it is of only limited use in soft ground.

The following six parameters are used to classify the ground using the RMR system:

- uniaxial compressive strength (UCS) of the rock material;
- RQD;
- spacing of discontinuities;
- condition of discontinuities;
- groundwater conditions;
- orientation of discontinuities.

The way to apply this system is to divide the rock into a number of structural regions in such a way that certain features are more or less uniform within each region. Appendix A (section A.1) provides details of the classification system used (Table A.1) and how to use this table to determine the RMR value for each region of the rock mass.

For tunnels, information can be obtained on stand-up time and maximum stable rock span for a given RMR (Figure 2.22).

In terms of application of the RMR system to tunnelling, Table 2.17 (after Bieniawski 1989) provides guidelines for the selection of rock reinforcement for tunnels in accordance with the RMR system (although it should be noted that this only applies to a 10 m span tunnel constructed using drill and blast, and no indication is provided as to how to extend this to other sizes of tunnel). These guidelines depend on such factors as depth to tunnel axis (*in situ* stress), tunnel size and shape, and the method of excavation.

Figure 2.23 shows how the rock mass classes in Figure 2.22 are modified for tunnel boring machines (TBM). It can be seen that the rock mass rating has to be higher to achieve similar stand-up times and roof span values. The letters indicate different TBM classes.

2.4.4.3 Rock Mass Quality Rating (Q-method)

The Rock Mass Quality Rating (Q-method) was proposed by Barton *et al.* (1974) for the determination of rock mass characteristics and tunnel support requirements. It is based on empirical data obtained from 200 tunnel construction projects in Scandinavia. It is probably the most widely used rock mass classification system today. It should be noted that the Q-method was developed for hard rock conditions and it is of only limited use in soft ground.

The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1000 and is defined by equation 2.10.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{2.10}$$

where

RQD = Rock Quality Designation ($0 \le RQD \le 100$)
J_n = joint set number ($0.5 \le J_n \le 20$)
J_r = joint roughness number ($1 \le J_r \le 4$)
J_a = joint alteration number ($0.75 \le J_a \le 20$)
J_w = joint water reduction factor ($0.05 \le J_w \le 1$)
SRF = stress reduction factor ($0.5 \le SRF \le 400$)

Appendix A (section A.2) provides Tables A.2–A.7 that give the classification of individual parameters used to obtain the Q-value for the rock mass. A detailed explanation of these parameters can be found in Barton *et al.* (1974)

The Q-value can be related to the stability of the excavation and support requirements. In order to do this, Barton *et al.* (1974) defined an additional parameter, which they called the equivalent dimension, D_e, of the excavation. This dimension is obtained from equation 2.11.

$$D_e = \frac{\text{excavation span, diameter or height in (m)}}{\text{excavation support ratio, ESR}} \tag{2.11}$$

The value of ESR is related to the intended use of the excavation and to the degree of safety, which has an influence on the support system to be installed in order to maintain the stability of the excavation. Typical ESR values are given in Table 2.18.

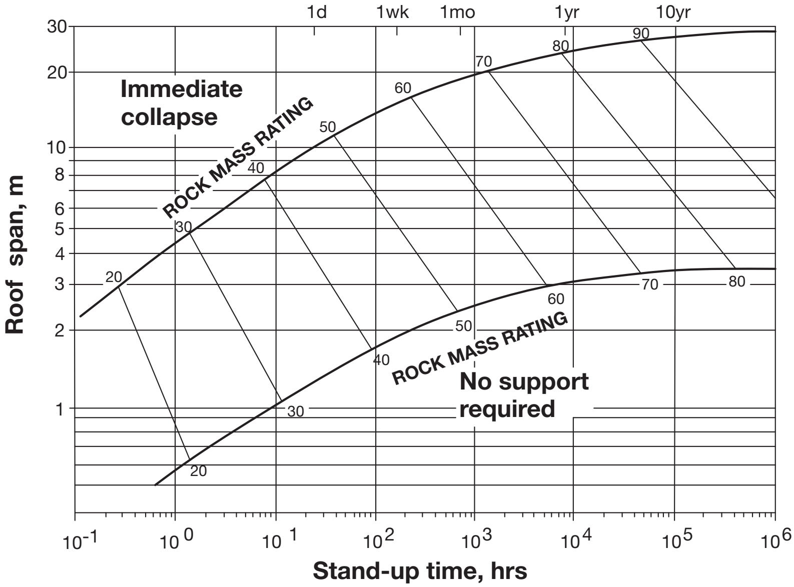


Figure 2.22 Relationship between the stand-up time and roof span for various rock mass classes (after Bieniawski 1989)

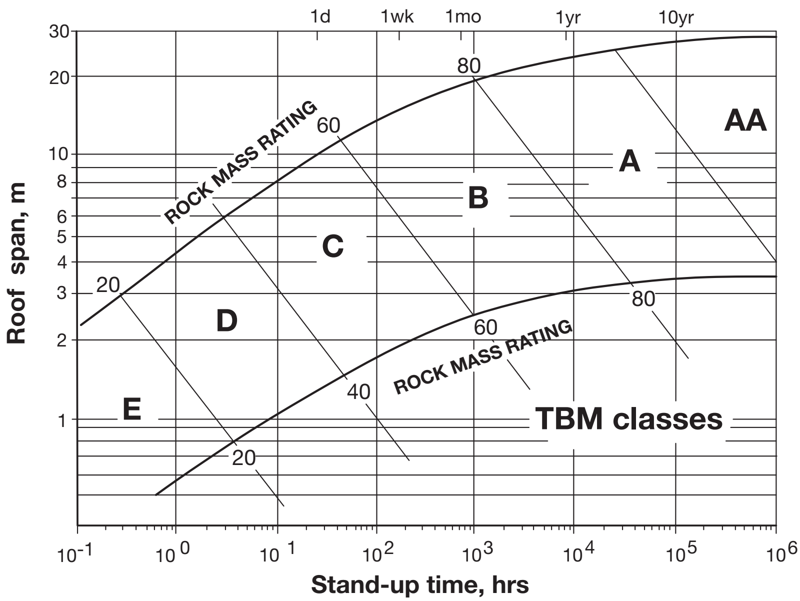


Figure 2.23 Boundaries of rock mass classes for TBM applications (after Bieniawski 1989, modified from a plot by Lauffer 1988, used with permission from VGT Verlag GmbH and taken from *Felsbau*)

Table 2.17 Guidelines for excavation and support of rock tunnels in accordance with the RMR system[a] (after Bieniawski 1989)

		Support		
Rock mass class	*Excavation*	*Rock bolts (20 mm dia. fully grouted)*	*Sprayed concrete*	*Steel sets*
Very good rock I RMR: 81–100	Full face. 3 m advance.	Generally, no support required except for occasional spot bolting		
Good rock II 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
Fair rock III 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
Poor rock IV 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.
Very poor rock V RMR: < 20	Multiple drifts 0.5–1.5 m advance in top heading. Install support concurrently with excavation. Sprayed concrete 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.

Note: (a) Shape: horseshoe; width: 10 m; vertical stress: < 25 MPa; construction: drilling and blasting.

Table 2.18 Suggested excavation support ratios (ESR) (after Barton and Grimstad 1994, from Barton *et al.* 1974)

	Type of excavation	*ESR*
A	Temporary mine openings	2.0–5.0
B	Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers	1.6–2.0
C	Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels	1.2–1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections	0.9–1.1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels	0.5–0.8

The equivalent dimension, D_e, plotted against the value of Q, is used to define a number of support categories in a chart published in the original paper by Barton *et al.* (1974). This chart has been updated a number of times to directly give the support requirements. Grimstad and Barton (1993), for example, modified it to reflect the increasing use of steel fibre reinforced sprayed concrete in underground excavation support. Figure 2.24 is reproduced from this updated chart.

In a further development, Barton (1999) proposed a method for predicting the penetration rate and advance rate for TBM tunnelling. This approach is based on an expanded Q-method of rock mass classification and average cutter force in relation to the appropriate rock mass strength. The parameter Q_{TBM} can be estimated during feasibility studies, and can also be back calculated from TBM performance during tunnelling. This method is briefly described in Appendix A (section A.2.1).

Barton (2002) provides some other useful correlations for the Q-value to assist site investigation and tunnel design. For example, a relationship between Q-value and seismic velocity (V_p) as used for some geophysical site investigation techniques (see section 2.3.2.1) is given in equation 2.12.

$$V_p \sim 3.5 + \log Q \tag{2.12}$$

where V_p is in units of km/s. This relationship was developed from tests in hard rock, but this has been developed further for application to weaker and harder ground conditions. This has been achieved by normalizing the Q-value using 100 MPa as the hard rock norm. The relationship for the normalized Q-value, Q_c is shown in equation 2.13.

$$Q_c = Q \times q_u/100 \tag{2.13}$$

where q_u is the unconfined compressive strength of the rock mass.

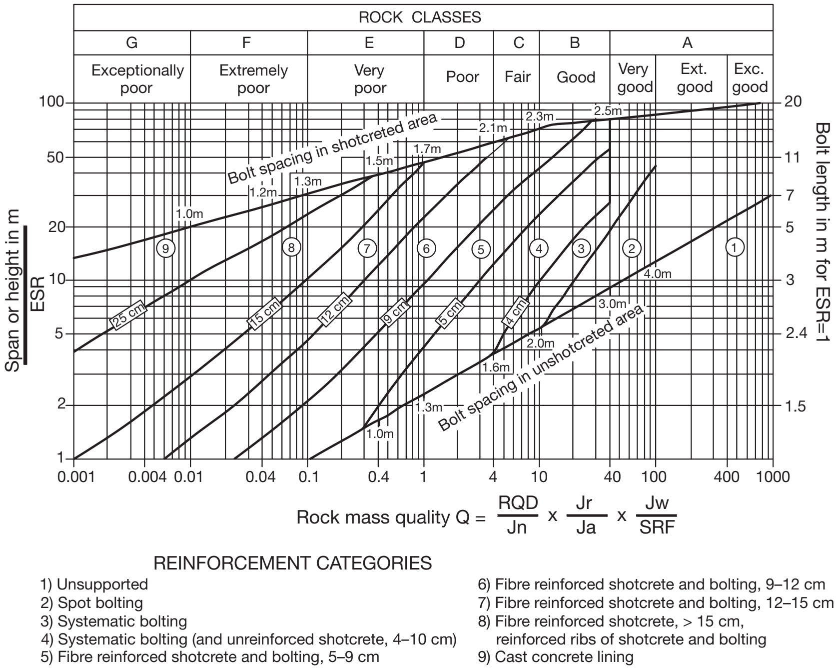


Figure 2.24 Estimated support categories based on the Q-value (after Grimstad and Barton 1993, reproduced from Palmström and Broch 2006)

Substituting equation 2.13 into equation 2.12, yields equation 2.14.

$$V_p \sim 3.5 + \log (Q_c{*}100/q_u) \qquad (2.14)$$

Barton (2002) also proposed a relationship between the Q_c and the modulus, E, of the ground as shown in equation 2.15.

$$E = 10\ Q_c^{1/3} \qquad (2.15)$$

Further details of these and other relationships can be found in Barton (2002).

2.4.4.4 *A few comments on the rock mass classification systems*

In this book there is only space to provide a brief overview of some of the rock mass classification systems currently in use. It is important to understand the basis of the systems and hence their applicability and limitations. It is therefore recommended that if the reader intends to use these systems, further, he/she conducts more detailed, background reading using the references provided.

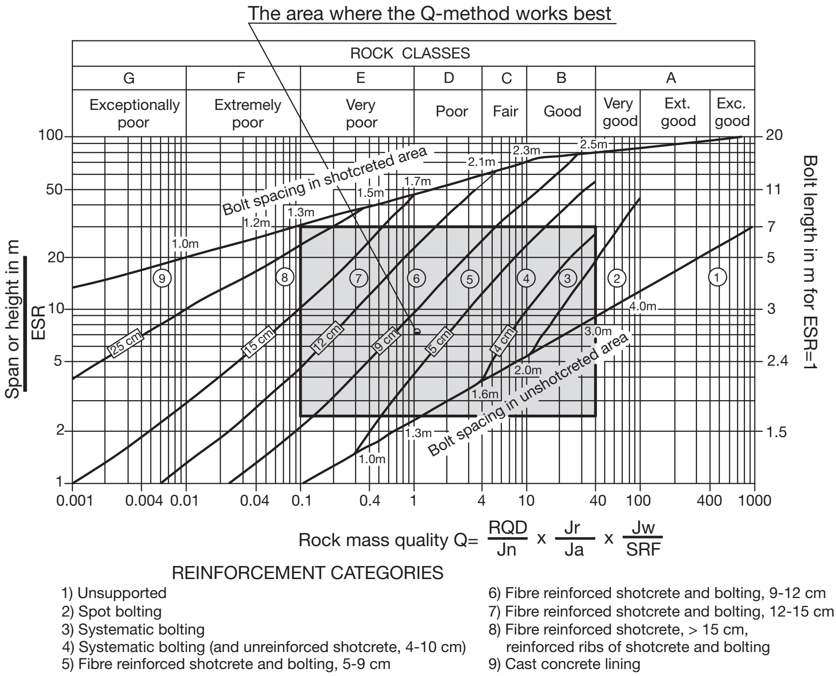


Figure 2.25 Limitations of the Q-method for rock support. Outside the shaded area supplementary methods/evaluations/calculations should be applied (reproduced from Palmström and Broch 2006)

One of the benefits of the RMR system is that it is relatively easy to use. The result produced by the RMR classification, however, is rather conservative. This can lead to an overestimation of the support measures (Maidl *et al.* 2008). As the RMR system and the Q-method are empirically devised they inevitably have their own deficiencies (as well as good points). As there are some reasonably consistent relationships between these systems, it is advantageous to apply both systems to the field data as a mutual check. There is an empirical relationship between the RMR and the Q-value as shown in equation 2.16 (Barton 2002).

$$RMR \sim 15 \log Q + 50 \tag{2.16}$$

Palmström and Broch (2006) investigated rock mass classification systems and particularly the Q-method, including Figure 2.24, and showed that actually the Q-method is most applicable within a certain range of parameters as shown by the shaded area in Figure 2.25. Outside this area, supplementary calculations and methods of evaluation are recommended.

For poorer quality ground these systems are less effective as shown in Figure 2.25. In these lower quality grounds the modulus values (or support criteria) are sensitive to small changes in the rating values.

For most tunnels for civil engineering projects, the ground can be considered as a continuum and tunnels are designed on this basis, i.e. the movement of the ground towards the excavation will load the lining. Rock mass classification systems such as RMR and Q-method are best used where the ground strength adequately exceeds the ground stresses and a support system, which increases the strength and stiffness of the discontinuities is appropriate. Where the ground requires a continuous structural lining for support, such is the case for weaker rocks, continuum analysis methods are more appropriate (BTS/ICE 2004). Continuum methods are discussed further in section 3.5.

2.5 Site investigation reports

The main outcome of any site investigation is the written report(s) that presents the findings and recommendations in a clear and concise manner so as to aid the tunnel designer. With respect to tunnelling projects, the site investigation reports will be used in the subsequent choice of the tunnelling method adopted, the design of the tunnel and the pricing and timescale of construction, and is therefore vital to the success of a tunnelling project.

2.5.1 Types of site investigation report

There are several types of report that can be produced from a site investigation and depending on the country will include a separate geology report, for example in Germany (after Hansmire 2007).

GEOTECHNICAL FACTUAL (OR DESCRIPTIVE) REPORT

This report should contain only factual information from the site investigation consisting of analysed data and objective consideration in accordance with existing standards, codes or specification. It does not have engineering interpretations. BS 5930 (BSI 1999) sets out, in general terms, the content of the Geotechnical Factual Report (GFR).

GEOTECHNICAL INTERPRETIVE REPORT

This is a report containing subjective considerations, interpretations, and comments from the engineer in charge; all in accordance with his knowledge and experience. The Geotechnical Interpretive Report (GIR) can be a project-specific report that presents the geological and engineering interpretation of the data. In its most simple form, it is a single report on a well-defined project. A geological profile is a geotechnical interpretation. In practice, many reports are written, revised, and in some cases superseded by later work. An interpretive report will address the project issues, and will often have design analysis, such as where rock mass classification is used to characterize the tunnel ground conditions upon which ground support and final lining requirements are established. The GIR is prepared primarily for use by the designers.

The GFR and GIR reports are often combined into one report in which the GIR makes up the main part of the report and the GFR the appendix. This report then forms the basis of the tendering process.

GEOTECHNICAL BASELINE REPORT (ESSEX 1997)

Within tunnelling contracts, a significant cause of cost overrun has historically been associated with contractors' claims for ground conditions significantly different from those expected at the time of tender. It has been difficult to assess these claims without well-defined benchmark conditions agreed at the outset between all the parties. The Geotechnical Baseline Report (GBR) has been designed as a tool to address this problem. The idea of a GBR is not new and has been the usual practice in the United States for many years. It is being increasingly more widely used in the UK and is a useful addition to tunnelling contracts irrespective of their type.

The GFR and GIR will form the basis for the GBR as appropriate. However, the GBR serves a different purpose and should be an entirely separate document. The GBR is intended to be contractual and to establish baseline conditions upon which a tender would be prepared. The GBR identifies the specific geotechnical data information from prior investigations or tests to be carried out in accordance with the contract that is in turn to be used to establish means and methods, and cost. It sorts out what data and past reports are relevant. It can also indicate specific previous work that is relevant, such as data obtained for different alignments or early interpretive engineering reports. An example of a 'baseline' is setting a maximum unconfined rock strength and rock hardness as the basis for the design of a TBM. During construction, the baselines in the GBR would be used to establish whether a change in geologic conditions has been encountered, resulting in financial consequences, for example the merit of additional payment to the tunnel contractor or benefit to the client.

2.5.2 Key information for tunnel design

Although certain information that is common to all tunnelling projects is required from a site investigation, there is some information that is particularly important depending on the type of tunnelling technique to be adopted. Some of these requirements are described below (after Kuesel and King 1996).

DRILL AND BLAST

Data are needed to predict the stand-up time for the size and orientation of the tunnel and the conditions for blasting during construction, i.e.

strength, stratigraphy, description and classification of the ground, water, gas, quartz content and abrasivity (this is obviously essential for all tunnelling techniques, except for possibly immersed tube tunnels).

HARD GROUND TUNNEL BORING MACHINES

Data are required to determine cutter costs and penetration rate. In addition, data to predict stand-up time are necessary to determine the type of machine which is to be used. Water inflow information is also important.

OPEN FACE SOFT GROUND TUNNEL BORING MACHINES

Face stability is important, i.e. stand-up time, and whether there is a need for mechanical devices to support the face built into the machine (face-breasting plates). Information is necessary to determine the requirements for filling the tail void. There is a need to characterize all potential mixed-face conditions.

CLOSED FACE SOFT GROUND TUNNEL BORING MACHINES

There is a need for data to make reliable estimates of the groundwater pressures, strength and permeability of the ground to be tunnelled. It is essential to predict the size, distribution and quantity of boulders. Mixed-face characteristics must be fully characterized.

PARTIAL FACE TUNNELLING MACHINES (FOR EXAMPLE ROADHEADERS)

Data are required on jointing to evaluate if the roadheader will be dislodging small joint blocks, or will grind away at the rock. Data on the hardness of the rock are essential to predict cutter/pick wear and hence costs. Quartz content and abrasivity are also important parameters.

IMMERSED TUBE TUNNELS

There is a need for ground data in order to reliably design the dredged slopes, to predict any rebound of the unloaded material and settlement of the completed immersed tube structure. Testing should emphasize rebound modulus (elastic and consolidation) and unloading strength parameters. There is also a need to ensure that all potential obstructions and/or rock ledges are identified, characterized and located. Any contaminated ground should also be fully characterized (*also important for all tunnelling techniques*).

CUT-AND-COVER TUNNELS

Exploration should be conducted over a sufficient plan area in order to define the conditions closely enough so as to reliably assess the best and most cost effective location to change from cut-and-cover to mine tunnels. The investigation should also evaluate the ground and groundwater conditions in order to aid design of the construction techniques and the excavation support systems to be adopted.

CONSTRUCTION OF PERMANENT SHAFTS

There should be at least one additional borehole for each shaft location. Data are required to design the construction method to be adopted and how to deal with groundwater conditions, both temporarily and permanently.

These tunnelling techniques will be described in detail in Chapter 5.

3 Preliminary analyses for the tunnel

3.1 Introduction

After obtaining ground characteristics from laboratory and field experiments, it is necessary to calculate the primary and secondary stresses in the ground, in order to assess the stability of the ground and likely loading on the tunnel lining. This will aid the selection of a suitable tunnelling method, assess whether ground improvement methods are necessary as well as provide the input parameters for preliminary analysis and modelling of the tunnel. This chapter focuses on obtaining the additional information, especially in soft ground, as well as the preliminary analysis techniques that may be employed.

The stability of a tunnel depends on certain key information:

- the tunnel depth and geometry;
- a detailed geological profile;
- the thickness and strength of the ground layers;
- the permeability of the ground and water pressures;
- the support provided during tunnelling.

3.2 Primary stress pattern in the ground

Primary stresses are the stresses in the ground prior to the construction of the void (tunnel). These stresses depend on the bulk unit weight and the depth at which they are determined as well as the coefficient of lateral earth pressure. The estimation of these initial or primary stresses is extremely important as it forms the basis of the loads that act on the combined tunnel support system, i.e. the ground and the tunnel lining. Commonly, the primary stresses are determined in the vertical, σ_v and horizontal, σ_h direction (Figure 3.1), which can be determined using equations 3.1 and 3.2.

Initial vertical stress: $$\sigma_v = \gamma z \tag{3.1}$$

Initial horizontal stress: $$\sigma_h = K_0 \gamma z \tag{3.2}$$

where γ is the bulk unit weight, z is the depth from the ground surface and K_0 is the coefficient of lateral earth pressure. An explanation of how to estimate K_0 is given in section 3.4.

In the structural analysis, either the full initial stress or a proportion of it is taken as the load on the tunnel.

If the tunnel is constructed in soft ground within the groundwater, two stress components have to be considered when determining the primary stress condition. First the effective ground stress, σ', and second the pore

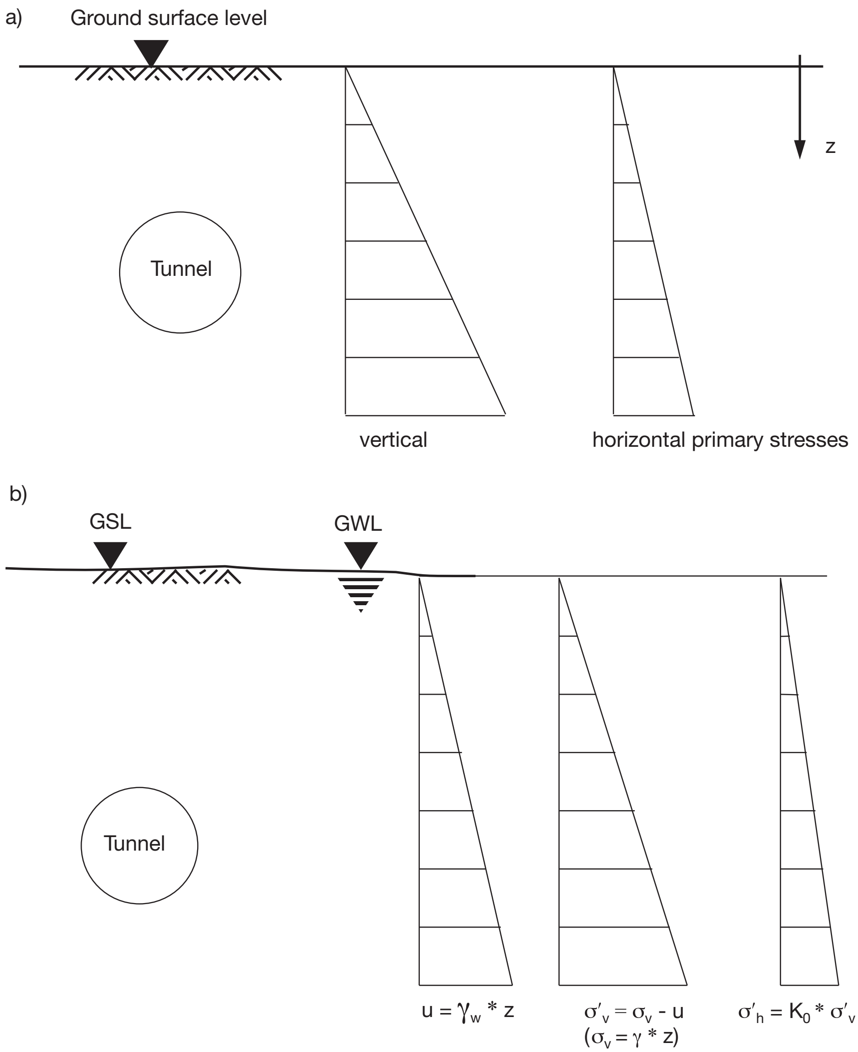


Figure 3.1 Primary stress distribution a) above the groundwater table and b) below the groundwater table

water pressure within the ground, u (below the groundwater level, GWL, this is equal to the water pressure in the ground). The total primary stress is then the summation of σ' and u (equation 3.3).

$$\sigma = \sigma' + u \tag{3.3}$$

It is often the effective stresses which dictate the behaviour of the ground in terms of shear strength as the pore water is assumed to have no shear strength. However, it should not be forgotten that the water pressure must be included when determining the loads acting on the tunnel lining, i.e. the effective stress generates bending and the total stress generates normal forces in the tunnel lining. The procedure to calculate the vertical and horizontal primary stresses (both total and effective) is as follows:

1 Calculate the vertical total stress using equation 3.1 (if the ground is layered then this is the summation of all the layers above the tunnel depth, i.e. $\gamma_1 z_1 + \gamma_2 z_2$ etc., where the subscripts 1 and 2 refer to different strata above the tunnel).
2 Calculate the pore water pressure at the tunnel depth. For example, if the tunnel is below the groundwater level and the water pressure can be assumed hydrostatic, then $u = \gamma_w z_w$, where γ_w is the unit weight of water and z_w is the depth below the level of the groundwater level (if the groundwater is flowing then u will be different).
3 The effective vertical stress, σ_v', is then calculated from $\sigma_v' = \sigma_v - u$, i.e. the effective vertical stress is the average stress acting between the particle to particle contacts within the ground material.
4 Multiplying the effective vertical stress by K_0 gives the effective horizontal stress, $\sigma_h' = K_0\sigma_v'$.
5 To determine the total horizontal stress, σ_h, the pore pressure, u, determined previously (water pressure acts equally in all directions, i.e. K_0 is 1.0 for water) is added to σ_h', i.e. $\sigma_h = \sigma_h' + u$.

When a tunnel is excavated, it disturbs the primary stress conditions. Assuming that the tunnel construction is stable, this requires a redistribution of the stresses around the void. This is known as arching. The stresses form a new equilibrium and this is called the secondary stress condition. It can also happen temporarily for partial or separate construction phases, for example if there is a partial heading far in advance of the remaining heading construction.

3.3 Stability of soft ground

One of the key parameters that influences the choice of tunnelling technique is the stability of the ground as the tunnel is constructed. This is particularly critical around the tunnel heading. Depending on the stability of the ground itself, i.e. the stand-up time, a decision has to be made on the face support required, for example open face or close face (see Chapter 5). Furthermore, decisions on the ground improvement measures are made depending on the stability of the ground (see Chapter 4). This section provides guidance on how to estimate the stability of the face and the face support pressure required. There is a significant difference in how to estimate the stability between fine and coarse grained soils, and this is mainly due to the difference in the permeability of the soil (and with respect to the construction of the tunnel, the advance rate and geometry). In coarse grained soil (where the permeability is greater than approximately 10^{-7} to 10^{-6} m/s and construction advance rates 0.1 to 1 m/hour or less) any excess water pressures generated during construction will dissipate quickly and 'drained' conditions should be used in assessing stability. In fine grained soil with low permeability, 'undrained' conditions are more important, i.e. where the excess pore water pressures do not dissipate quickly, although if there is a stoppage in construction drained conditions may become more relevant (Mair and Taylor 1997).

3.3.1 Stability of fine grained soils

In saturated fine grained soils the short-term stability is dominated by the undrained shear strength of soil, c_u. Broms and Bennermark (1967), drawing on earlier work related to bearing capacities below foundations and field measurements, performed extrusion tests on a clay soil supported by a vertical retaining wall. They postulated the idea of a stability ratio N, which compared the overburden stress to the undrained shear strength of the soil in the form of a ratio (equation 3.4).

$$N = \gamma H / c_u \tag{3.4}$$

where H is the depth to tunnel axis (C+D/2), γ is the bulk unit weight of the soil and c_u is the undrained shear strength of the ground prior to excavation. The higher the value of N, the lower the stability. In the more general case where there is a surcharge at the ground surface and a support pressure is used at the face, for example as applied via an earth pressure balance machine (EPBM), the stability ratio, N, can be expressed as shown in equation 3.5.

$$N = (\sigma_s + \gamma H - \sigma_T) / c_u \tag{3.5}$$

where σ_s is the surcharge acting on the ground surface and σ_T is the support pressure applied at the face (note this is zero for a sprayed concrete lining). Figure 3.2 shows the parameters used, including P, which is the unsupported advance length of the tunnel (note this is zero for a shield TBM).

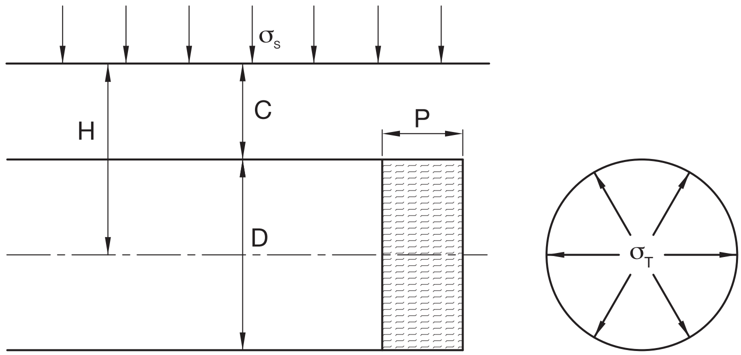


Figure 3.2 Stability parameters (after ITA/AITES 2007)

Various authors have published observations on the value of N, for example Peck (1969b) suggested N values ranging from 5 to 7. ITA/AITES (2007) suggests the following typical values:

- when $N \leq 3$, the overall stability of the tunnel face is usually ensured;
- when $3 < N \leq 6$, special consideration must be taken of the settlement risk, with large amount of ground losses being expected to occur at the face when $N \geq 5$;
- when $N > 6$, on average the face is unstable.

There are also other parameters that should be considered with respect to the stability of the face. These are:

- C/D, which controls the effect of depth on the stability condition, for a $C/D < 2$ a detailed face stability analysis is required;
- $\gamma D/c_{u,}$ which accounts for the possibility of localized failures occurring at the face, a value of $\gamma D/c_u > 4$ would indicate localized failure at the face is likely;

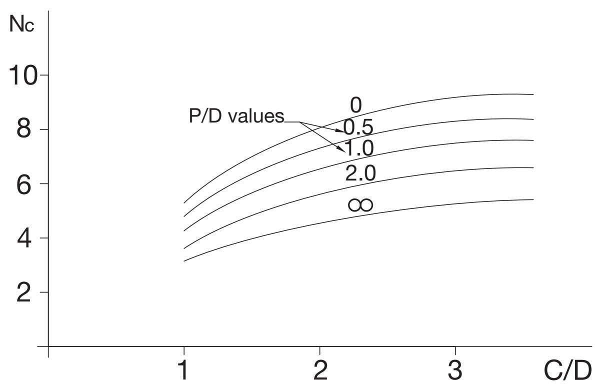


Figure 3.3 Critical stability ratio (N_c) (after Dimmock and Mair 2007b, used with permission from Thomas Telford Ltd and Professor R.J. Mair)

- P/D, which accounts for the distance behind the face until the lining is installed: an indication of the effect of this ratio on the critical stability ratio (N_c) is shown in Figure 3.3, using data from centrifuge tests.

Centrifuge modelling has been used to investigate many aspects of soft ground tunnelling including stability, ground movements and the effects of tunnelling on adjacent piles and structures. Centrifuge modelling involves constructing a small-scale physical model of the problem to be investigated. This is constructed in a strong box, which is then attached to the end of a beam and rotated at high speed. The rotation increases the gravitational forces on the model. This means that everything in the model weighs more and thus, for example a small depth of soil in the model simulates a much larger prototype depth in the field. Thus the dimensions, and many of the physical processes of the prototype, can be scaled correctly if an 'Nth' scale model is accelerated by N times the acceleration due to gravity. For example, lengths are scaled by 1/N and stresses are scaled 1:1 in the centrifuge, meaning that a dimension of 0.25 m in the model when spun at 100 g is equivalent to 25 m. An example of a centrifuge test apparatus would consist of a 1.7 m radius beam centrifuge, capable of spinning a 500 kg payload at 100 g, the equivalent to 230 rpm. Further information on centrifuge testing can be found in Taylor (1995a).

3.3.2 Stability of coarse grained soils

Atkinson and Mair (1981) describe a method for calculating the required tunnel face support pressure (σ_T) for coarse grained soil above the groundwater table and the general equation is shown in equation 3.6.

$$\sigma_T = \sigma_s T_s + \gamma D_s T_\gamma \tag{3.6}$$

where T_s is the tunnel stability number for the surface surcharge (σ_s), D_s is the diameter of the shield and T_γ is the tunnel stability number for the soil load. T_s and T_γ are equivalent parameters to the stability factor, N for fine grained soils. T_γ depends on the effective internal friction angle of the soil (ϕ') and can be determined from the graph shown in Figure 3.4a. T_s depends greatly on the depth of cover (C) as well as ϕ' and can be determined from the graph shown in Figure 3.4b.

If the tunnel is below the groundwater table, equation 3.6 should theoretically be modified to equation 3.7 (after Thomson 1995).

$$\sigma_T = \sigma_s T_s + [\gamma_d (C - h_w) + \gamma_{sat} h_w] T_\gamma + \gamma_w h_w \tag{3.7}$$

where γ_d is the bulk unit weight for the soil above the groundwater table, γ_{sat} is the bulk unit weight for the soil below the groundwater table, γ_w is the unit weight of water, h_w is the depth of the tunnel crown from the

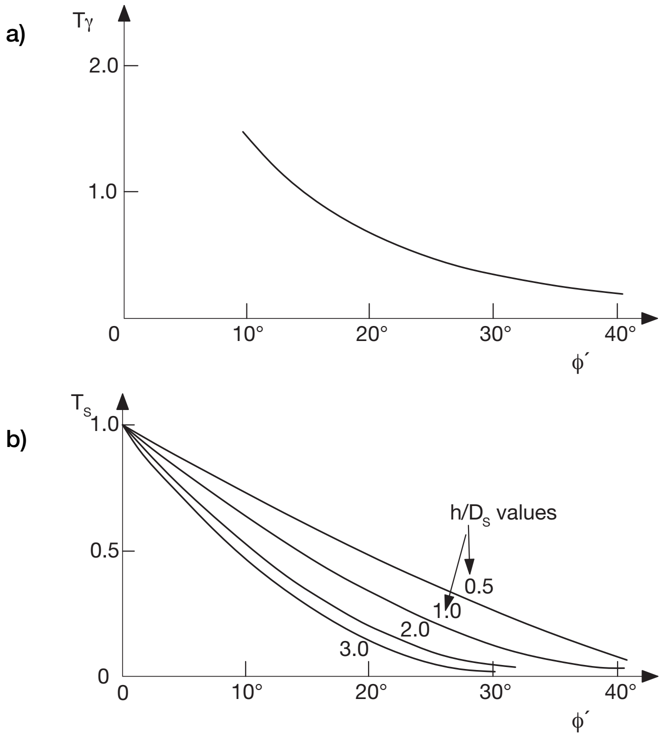


Figure 3.4 a) Determination of the tunnel stability number in coarse grained soils for the soil load, b) determination of the tunnel stability number in coarse grained soil for the surface surcharge (after Thomson 1995, from Atkinson and Mair 1981, reproduced with permission from Emap Ltd)

groundwater table and the other parameters have their usual meanings. It should be noted, however, that other factors influence the stability in this case, such as seepage forces towards the tunnel face and hence these must also be considered. This will result in considerably greater support pressures being required in order to prevent water inflows and provide drained stability. This can be achieved using compressed air support or slurry tunnelling machines. For coarse grained soils above the water table, there will probably be sufficient water content to enable some suction effects to develop (apparent cohesion), helping to stabilize the face. Stability solutions for face stability of slurry tunnel boring machines, based on limit equilibrium methods, have been developed by Anagnostou and Kovari (1996) and Jancsecz and Steiner (1994).

3.4 The coefficient of lateral earth pressure (K_0)

The coefficient of lateral earth pressure at rest can have a range of values ($0.1 < K_0 < 3$). In practice this parameter is difficult to obtain, but several aspects can be considered when estimating K_0. It should be emphasized again that the estimation is based on engineering judgement and any assumptions have to be checked by measurements as described in section 7.3.

LATERAL PRESSURE IN A SILO

Due to the difficulties of determining K_0 and because of the issues of determining the ground mechanics, there have always been experiments to estimate K_0. A common mistake, which even today leads to misunderstandings, is the determination of K_0 from the silo pressure. In a silo (Figure 3.5a) K_0 can be calculated from Poisson's ratio, μ, as shown in equation 3.8.

$$K_0 = \frac{\mu}{1-\mu} \tag{3.8}$$

However, because μ is generally in the range 0 to 0.5, using equation 3.8 would lead to K_0 values in the range of 0 to 1.0. The realistic range of μ for the ground is between 0.2 to 0.35, which leads to K_0 values of between 0.25 and 0.54. This example calculation shows that K_0 values of greater than 1.0 are not possible with this equation and values of K_0 of greater than 0.54 are only fully covered if one uses a μ value, which is not necessarily realistic for the ground. This equation represents a simplified case and is based on the assumption of elasticity in the ground and is only valid in rare circumstances in underground construction.

The value of K_0 is always going to be an estimation. To determine its value one needs to take into account the historical development of the earth, and hence the rock. The determination (or better estimation) of K_0 is part of the engineering geological survey. Five possible reasons are listed below for the variation in K_0, i.e. where K_0 has no relationship to μ (which is true for the majority of cases).

- **Ice age preloading.** It is possible that the lateral horizontal pressure of earlier times is impregnated into the ground and is still present today. The pressure of huge glaciers from past ice ages is an example of this. In this case K_0 can be higher than 1.0 (Figure 3.5b).
- **Layering, synclines, anticlines (saddles and troughs).** Figure 3.5c shows the influence of layering and layering with saddle structures depending on the position of the tunnel relative to the geological formation and hence the need to use different K_0 values. In this case a potential rotation of the principal stress conditions would be expected, i.e. the assumption that the vertical and horizontal stresses are principal stresses is no longer valid, or limited. This also applies if the layers are dipping.
- **Crevasses.** In open crevasses K_0 is very small (Figure 3.5d). This is the same as for the cases where the crevass contains soft soil or water. If K_0 were high in this situation, the crevass would be most likely closed.

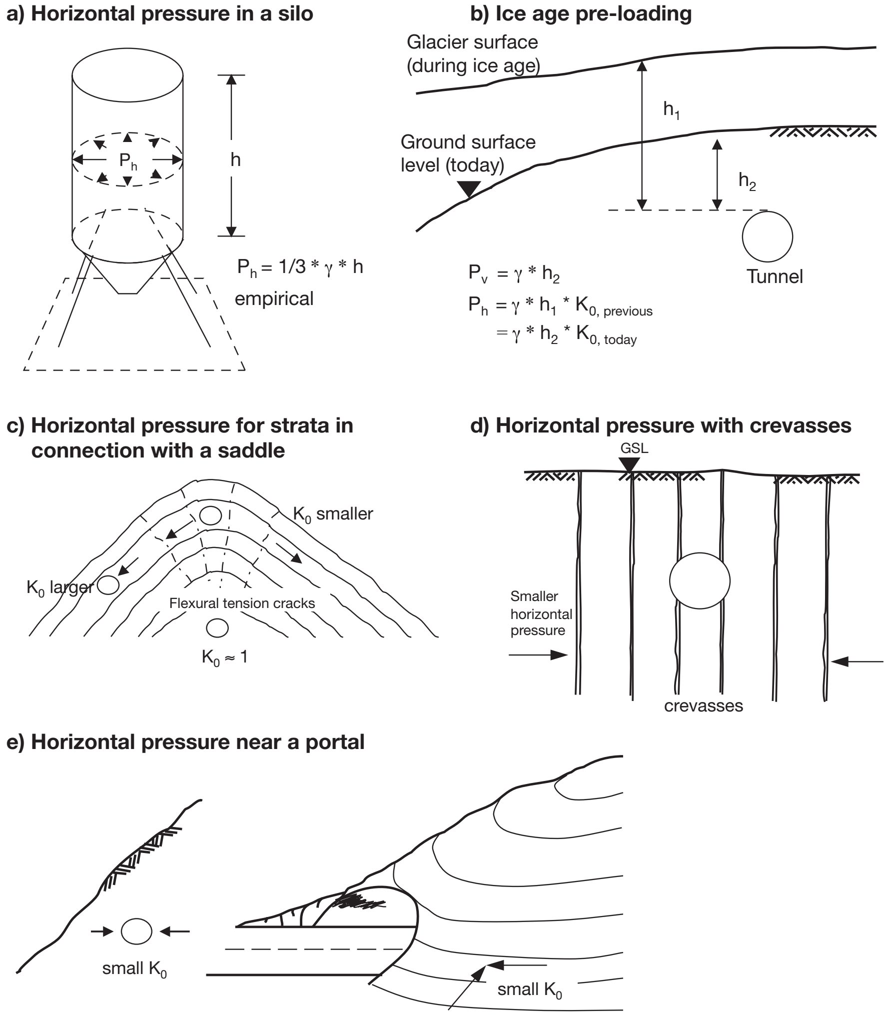


Figure 3.5 Coefficient of lateral earth pressure

- **Depth.** Close to the ground surface K_0 would be expected to be small due to weathering. In addition, for a high K_0 value the tension of the ground is missing, for example at a slope (Figure 3.5e).
- **Tunnel in groundwater.** If the tunnel is constructed within the groundwater, at least two components need to be considered when estimating the primary stresses. These components are the effective ground stress and the water pressure ($K_0 = 1$) as described in section 3.2.

For the reasons mentioned above, there is no definitive value for K_0. However, one can statistically define the range of K_0 as 0.1 to 3.0. For

Table 3.1 Typical values for K_0

Ground material	K_0
Sand	0.4–0.5
Clayey soil (between rock layers)	0.6–0.8
Slurry	1.0
Soft rock	0.4–0.6
Hard soil/rock	(0.2) 0.5–0.8 (1.2)
London Clay	0.6–1.5

normally consolidated soils, i.e. a soil that has not experienced greater stresses acting on it in the past than are acting on it now, K_0 can be estimated based on the internal friction angle, ϕ', of the material, for example $K_0 = (1 - \sin \phi')$. For overconsolidated clays, i.e. where the soil has experienced larger stresses in the past than it is experiencing now, K_0 is likely to be greater than 1.0. Some examples of typical values of K_0 are shown in Table 3.1.

3.5 Preliminary analytical methods

3.5.1 Introduction

It is impossible to take all the influences, parameters and boundary conditions that are dependent on the geology and construction phases into account in a calculation. Therefore, analytical models have been developed which simplify reality to such an extent that the remaining parameters can be dealt with in a calculation and at the same time lead to sensible results.

In the following discussion, three common analytical methods are briefly described; the bedded-beam spring method, the continuum method and the tunnel support resistance method. The assessment of which method to use, depends on the tunnel depth. In soft soil, two conditions can be defined as:

- shallow, $C<2D$, i.e. where the ground above the tunnel crown in assumed to have no bearing capacity;
- deep, $C>3D$, i.e. where the ground above the tunnel crown is acting as a support;

where C is the tunnel crown depth and D is the tunnel diameter.

$C<2D$: The excavation process creates a softening zone in the crown area, which for shallow tunnels in soft ground reaches the ground surface. As a result, no arching can develop over the crown. The ground in this area has no bearing capacity and acts only as a load on the tunnel lining. For the unsupported area, on average, an angle of 90 degrees is assumed at the tunnel crown (Figure 3.6a). This is a very conservative approach.

C>3D: For a supporting crown, the ground is capable of creating a supporting ring, i.e. the ground can form an arch and transfer loads around the tunnel void.

In the range 2D< C <3D, the ground above the tunnel crown can be acting either as a support or not depending upon the geological conditions, i.e. bedding arrangements.

Further details on the design of shield tunnel linings, segmental linings for example, can be found in ITA (2000). Further details on structural design models for tunnels in soft ground can be found in Duddeck and Erdman (1985).

3.5.2 Bedded-beam spring method

The tunnel support is idealized as an elastically supported circular ring. The elastic bedding is achieved through radial and potentially tangentially arranged springs. The spring stiffness simulates the support behaviour of the ground. The important parameters of the ground are the stiffness modulus E_s (which is included in the spring stiffness) and the coefficient of lateral earth pressure K_0 (which is included in the loading). The calculation is carried out elastically. As the ground is only represented by springs, the analysis cannot provide any information with regard to the settlement at the ground surface and to the possible stress and deformation behaviour of the ground (secondary stress situation). Figure 3.6a shows the model used within the bedded-beam spring method with an unsupported crown, the so called 'partially bedded method' for shallow tunnels (ITA 1988). For deep tunnels the bedded-beam spring method is generally not used because even with a supporting crown area, the supporting nature of the ground is not sufficiently taken into account.

The bedded-beam spring method is the fastest and simplest calculation method. Therefore it is often applied even though it has limited potential for interpretation with respect to the real situation due to the many simplifications made. It is often used to determine thickness and required reinforcement of the supporting circular ring following the results of a more sophisticated calculation method. The usage is mainly for shallow tunnels in soft ground or weak rock.

3.5.3 Continuum method

The ground, in which the tunnel is constructed, is idealized as a continuum, i.e. there are no discontinuities in the material. The method assumes that the ground is an infinitely large thin section with a hole at the centre (Figure 3.6b). This calculation method allows the interpretation of the deformation and strains in the ground. In addition, this method allows the construction phases to be simulated. The elastic modulus, E, is required as a parameter for the ground. The structural system can be established for both an

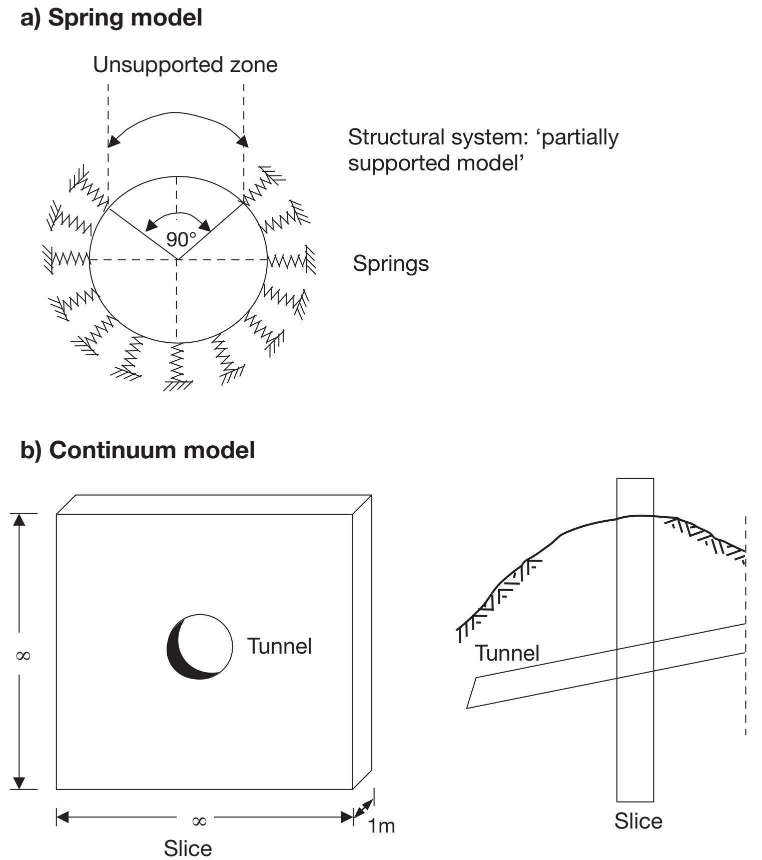


Figure 3.6 Calculation models for tunnels in soft ground

unbedded as well as a bedded crown area. However, the assumptions for this calculation method for deep tunnels are often unfavourable; with increasing depth, the load on the tunnel lining grows linearly and with this its thickness. In combination with an elastic calculation, this leads to a potentially unrealistically large lining thickness. In the calculation, the load acting on the tunnel lining is limited, i.e. the total overburden pressure is not considered. Instead, only the weight of the disturbed zone, which develops over the tunnel crown as a result of the tunnel construction, is used in the calculation. The biggest difficulty lies in the estimation of the height of this disturbed zone and thus the load for which the thickness of the tunnel lining has to be calculated. Estimating the size of the disturbed zone, which acts as the overburden on the tunnel, is based on experience, engineering judgement and the ground characteristics. The method is used in shallow tunnels in weak rock as a partial continuum method and for deep tunnels in weak rock as a continuum method with a bedded crown area.

An example calculation using the continuum method is provided in Appendix B. Further details on continuum methods can be found in Duddeck and Erdman (1985).

3.5.4 Tunnel support resistance method

For the support resistance method, it is assumed that the tunnel support constrains the deformations of the ground, i.e. it provides an internal pressure (resistance) against the ground. The resistance is taken as the pressure inside the tunnel in the calculation and is defined as P_T (Figure 3.7a). The pressure inside the tunnel is dependent on the deformations. This 'thought' model (tunnel support as an internal pressure) can be applied to deep rock tunnels. The tunnel support resistance method is also a continuum method and in addition to the elasticity modulus, the cohesion and the friction angle of the ground are required.

The design criterion for this method is the limitation of the deformation of the ground. The connection between the rock deformation and the tunnel support resistance can be shown pictorially using the Fenner-Pacher curve, as shown in Figure 3.7b (w is the settlement of the tunnel crown).

$w < w_{crit}$: The more the ground deforms (distresses) before the tunnel support is placed, the lower the load that has to be carried by the tunnel lining and the higher the self supporting element of the ground. The required tunnel support resistance reduces with increasing deformation.

$w > w_{crit}$: When the deformation reaches a certain amount, it results in softening and weakening of the ground fabric. To construct a stable tunnel beyond this point, increasing support resistance is essential with increasing deformations.

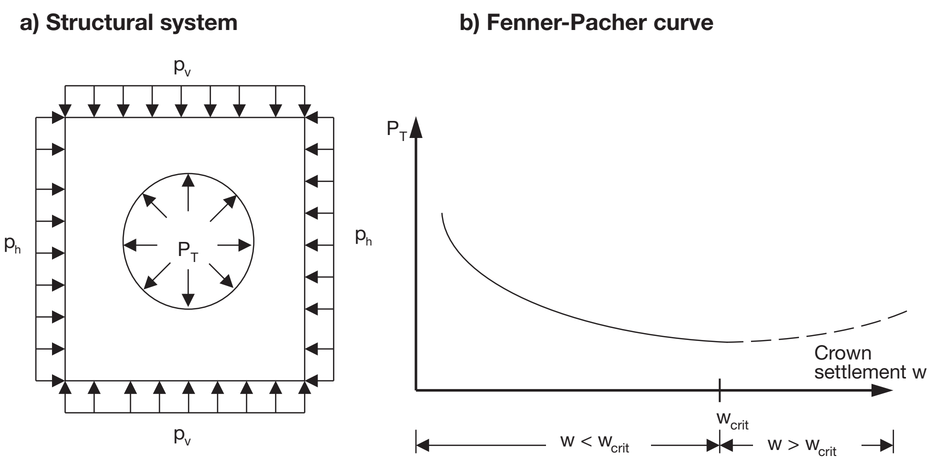


Figure 3.7 Tunnel support resistance method

Thus, there is a deformation value for the ground at which the required tunnel support resistance is minimal. This deformation should be reached when all the stress redistribution has finished. By keeping the deformation to w_{crit}, it would be possible to have the optimal support system both from an economical and rock behaviour point of view. The relationship between the support system resistance and the deformation is dependent on the geology. This means that for every ground there is a different Fenner-Pacher curve and a different critical deformation. This leads to a number of problems.

- First: how big is w_{crit}? If a lot of experience exists in comparable geological conditions with similar underground construction methods, it could be possible to put a quantitative boundary on the critical deformation. However, in an unknown ground this is nearly impossible.
- Second: even if the critical deformation is known, the difficulty remains to ensure that the construction phases result in a final value of w_{crit}. Many of the factors that influence the development of the deformations are not linear and are time dependent, for example the curing of sprayed concrete when using sprayed concrete lining (see section 4.3.2). Furthermore, there is the problem of checking the rock deformations with measurements. This is particularly relevant for the rock deformation ahead of the tunnel drive. (The topic of deformation measurement is looked at in section 7.3.)
- Third: the tunnel support is not calculated but assumed as an inner pressure. Hence there are no internal forces. Therefore the problem exists to translate the w_{crit} and the associated required support resistance into a sprayed concrete lining thickness and related reinforcement.

The support resistance method is consequently not suitable for an analysis in the traditional sense (structural system with load → determination of internal forces → proof of stresses). The ground is the main support element and is in the forefront of the analysis. The tunnel support system, in this case a sprayed concrete lining, supports the disturbed boundary areas of the void and is, by comparison with the methods for soft ground, of lower importance.

The advantages of the theory of support resistance are as follows:

1. The full overburden can be assumed. The stress redistributions and overstressing in the ground can be determined, which is not possible with the other analytical methods. In the support system resistance method, the creep of sprayed concrete is taken into account. This means that the calculated deformations are greater and closer to reality compared with the methods using elastic analysis.
2. The choice of the calculation method therefore also depends on the type of ground: soft ground or rock. Principally it has to be decided

how much self-support the ground possesses, i.e. whether one builds uneconomically (dimensioning the tunnel support too large) or unsafe (assuming the self-support of the ground to be too large). This depends mainly on how valid the estimation is. *It should also be noted that without estimation no structural analysis functions in underground construction!*

3.6 Preliminary numerical modelling

3.6.1 Introduction

The previous section described simple analytical methods, which can be used to estimate the stresses in the tunnel lining or the required lining thickness for a given deformation of the ground. In recent years, as a result in the increases in computing power and the fact that there are many commercial packages available, the use of numerical models has increased significantly. This section describes the use of some of these numerical models. However, it is not the intention of this section to fully describe how to carry out tunnel analyses, but to briefly describe some aspects of the problem. The reader is directed to other books such as Potts and Zdavkovic (1999 and 2001) for further details. The benefits of numerical methods over analytical or closed form solutions (as described in section 3.5) are highlighted by Potts and Zdavkovic (2001) as being able to:

- simulate the construction sequence;
- deal with complex ground conditions;
- model realistic soil behaviour;
- handle complex hydraulic conditions;
- deal with ground treatment, for example compensation grouting;
- account for adjacent services and structures;
- simulate intermediate and long-term conditions;
- deal with multiple tunnels.

It must be remembered, however, that numerical analyses are only as good as the user's experience, the input values and the numerical modelling package. They are not a panacea and must be treated as any other engineering tool. Many assumptions are still required in order to produce workable analyses and these require good engineering judgements and a clear understanding of their implications.

Numerical methods can be divided into different types depending on the computation methods adopted in the software package. For modelling continua, such as soils, the most common numerical methods adopted for analysing tunnelling projects are the finite element method or finite difference methods. For modelling discontinua, such as rocks, the most common numerical models adopted are the discrete element method and the boundary

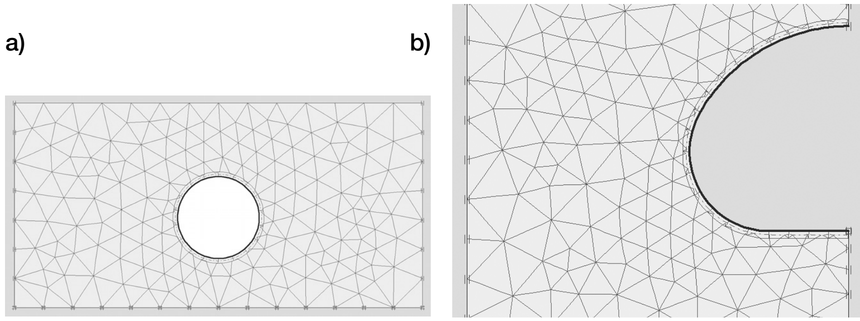


Figure 3.8 Example 2-D meshes using finite elements, a) 2-D plain strain analysis of a bored tunnel construction, and b) 2-D plain strain analysis (with symmetry associated with the tunnel centreline) of a tunnel being constructed using NATM, as produced using the PLAXIS® software (courtesy of Wilde FEA Ltd)

element method. It should be noted that there is overlap between these latter methods and the modelling of continua.

Tunnelling is a three-dimensional problem. Even with more powerful computers, however, these models can be computationally demanding. In addition, three-dimensional models for tunnelling problems are not easy to set up, even though modern commercial software packages are making this easier for routine problems. This means that two-dimensional models are still very common. Adopting a two-dimensional model for a tunnelling problem immediately implies that a number of assumptions are needed with respect to the construction process. In particular, the fact that the three-dimensional arching effect, which is so important for the behaviour of the ground to allow economic tunnels to be constructed, cannot be modelled directly.

There are a number of ways to represent the tunnel in 2-D models. When modelling shallow tunnels or if the ground surface response is key to the analysis, then a plane strain analysis is required. Typical finite element meshes for the 2-D plane strain analyses are shown in Figure 3.8a and b.

3.6.2 Modelling the tunnel construction in 2-D

In 2-D there are a number of ways of modelling the construction method. These include the following:

‘GAP’ METHOD

In this method a predefined void is introduced into the finite element mesh that represents the total ‘volume loss’ expected. The ‘gap’ is greatest at the crown of the tunnel and zero at the invert (Rowe *et al.* 1983).

CONVERGENCE–CONFINEMENT METHOD

This is the most suitable method for tunnels excavated without a shield or TBM, e.g. NATM. This was demonstrated by Karakus (2007), who looked at various methods in order to determine which ones best represented the 3-D effects of tunnelling in 2-D analyses. In this method the proportion of unloading of the ground before the installation of the lining construction is prescribed, i.e. the volume loss is a predicted value. The parameter λ is used to define the proportion of unloading. Initially λ is zero and is progressively increased to 1 to model the excavation process. At a predetermined value of λ_d the lining is installed, at which point the stress reduction at the tunnel boundary is λ_d times the initial soil stress. The remainder of the stress is applied to create the lining stress, i.e. the stress imposed on the lining is $(1–\lambda_d)$ times the initial soil stress (Potts and Zdavkovic 2001).

PROGRESSIVE SOFTENING METHOD

This was developed for NATM (or sprayed concrete lining) tunnelling by Swoboda (1979). The method involves reducing the ground stiffness in the heading by a certain amount. The lining is installed before the modelled excavation is complete. The method can cope with crown and invert construction or side drifts.

VOLUME LOSS CONTROL METHOD

This is similar to the convergence–confinement method, in this case, however, the expected volume loss at the end of construction is prescribed. This is useful if the volume loss can be estimated with a reasonable degree of certainty, and is also useful for back analysis of tunnelling operations. In this method the support pressure at the tunnel boundary is reduced in increments, and the volume loss generated can be monitored. Once the prescribed value is achieved, the lining is installed. Depending on the stiffness of the lining, further deformations and hence volume loss may occur, so it may be that the lining is installed before the prescribed volume loss is reached to allow for this additional value.

It is also important when setting up the model to use appropriate boundary conditions, both for far field conditions, for example the restraints applied at the edges of the mesh area, including hydraulic conditions, and the near field conditions associated with, for example the lining. Normally, in a simple 2-D plane strain analysis, the restraints to movement at the far field conditions are that the ground surface is not restrained from moving, the base of the mesh is restrained vertically and horizontally and the edges are restrained horizontally, but not vertically.

MODELLING THE LINING

As mentioned previously, in 2-D the analysis does not recognize the 3-D support from the lining already installed behind the face, into which the stresses arch. So called wished-in-place lining occurring in a single increment in the analysis is common, for example when using the volume loss or convergence–confinement approaches. There are two ways of modelling the lining using solid element or shell elements. Solid elements are standard elements used for representing most materials within finite element meshes and hence there are a wide range of constitutive models available for these elements. However, solid elements have the problem that the element shape can be an issue (defined by the aspect ratio of length to width). Linings are relatively thin in relation to the tunnel diameter and therefore a large number of elements are required to maintain acceptable aspect ratios. Shell elements in contrast have zero thickness and curved shell elements can be used to model tunnel linings. This removes the problem of aspect ratio and allows more flexibility with respect to the mesh definition. There are many issues to consider when modelling tunnel linings, particularly segmental linings, and the reader is encouraged to read more detailed literature on this subject, for example Potts and Zdavkovic (2001).

3.6.3 Modelling the tunnel construction in 3-D

3-D numerical analyses allow the possibility of modelling the tunnel operation more realistically, particularly the behaviour of the ground ahead of the tunnel face and the 3-D arching effects that occur around the tunnel face. Although these analyses are more costly in terms of computation time, it is still not possible to model accurately every aspect of the tunnel construction in detail and assumptions are still required. However, modern software packages do offer the possibility of doing this type of analysis with relative ease. It must be remembered, however, that it is important to understand what you are doing and to consider the limitations and assumptions that are made in these analyses. Figure 3.9 shows an example of a finite element 3-D mesh.

An example of three-dimensional numerical modelling was reported by Ng *et al.* (2004) who carried out a series of three-dimensional finite element analyses to investigate multiple tunnel interactions for sprayed concrete lined tunnels in stiff clay using ABAQUS®. In parallel, Lee and Ng (2005) studied the effects of tunnels on an existing loaded pile using three-dimensional finite element modelling, again using ABAQUS®. Bloodworth (2002), following on from previous work by Burd *et al.* (2000), conducted a detailed study to investigate the effects of new tunnelling on existing structures in 2-D and 3-D. This work highlights many of the issues associated with simulating both the tunnelling operation and the realistic modelling of, in this case, the buildings at the ground surface. The numerical

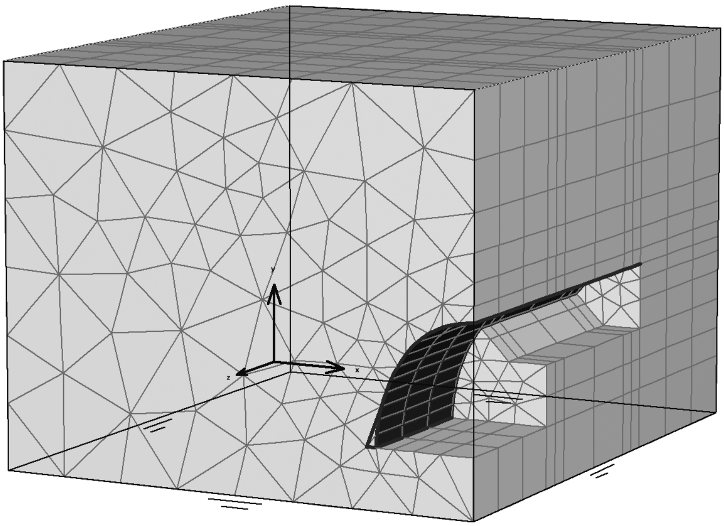


Figure 3.9 Example 3-D analysis of a tunnel being constructed using NATM, as produced using the PLAXIS 3-D Tunnel® software (courtesy of Wilde FEA Ltd)

modelling generally over-predicted the damage to the buildings at the ground surface. It was suggested that this was due to the level of detail that could sensibly be modelled within the numerical models whilst working with the computer power available at that time.

3.6.4 Choice of ground and lining constitutive models

One of the most critical aspects of any numerical modelling is the choice of constitutive models for the ground and the lining, i.e. how the material behaviour is simulated. Many people still use linear elastic or elasto-plastic (e.g. Mohr-Coulomb) constitutive models to analyse the soil behaviour. However, it has been shown by many researchers that the soil model has a large impact when modelling tunnelling operations. The construction operations involve unloading as well as loading stress conditions and so any model must be able to cope with this. In addition, the strains around tunnelling operations are often small and for soft ground this means that the stiffness of the material is extremely nonlinear. Therefore, if pre-yield behaviour dominates the ground response, it is essential to model the nonlinear elasticity at small strains. The reason for people choosing simpler constitutive models for the ground is generally related to the choice of input parameters. The more sophisticated the soil model the more parameters that are required. Obtaining these parameters from available site investigation information can be difficult and often requires assumptions to be made.

Ideally, to model the ground behaviour successfully, it is important to consider its nonlinear stress-strain behaviour, variable K_0 values, anisotropy and consolidation characteristics.

For modelling sprayed concrete lining there is a need to model the age- and time-dependent behaviour of the material, as well as its nonlinearity. Typical models include (Thomas 2009a):

- *Hypothetical Modulus of Elasticity (HME),* which uses reduced values of elastic stiffness for the lining to account for 3-D effects, ageing of the elastic stiffness, creep and shrinkage. It is largely an empirical based model.
- *Age-dependent elastic models*, which, as an alternative to the HME model, models the ageing stiffness explicitly.
- *Age-dependent nonlinear mod*els, which takes into account the non-linear stress-strain behaviour of concrete when loaded to more than 30% of it compressive strength. As sprayed concrete can be loaded heavily early on, i.e. at low strength, this nonlinearity could be relevant.

Further details of modelling sprayed concrete can found in Thomas (2009a).

4 Ground improvement techniques and lining systems

4.1 Introduction

This chapter is divided into two sections. The first describes techniques of improving and stabilizing the ground, with respect to both strength and also permeability. The second describes the various lining techniques commonly employed in tunnel construction. It should be noted that many of the stabilization techniques and lining methods are intimately linked with the tunnel construction methods described in Chapter 5, so the reader is advised not to treat these chapters in isolation, but to treat both as part of the tunnel construction process.

4.2 Ground improvement and stabilization techniques

This section describes a number of techniques that can be used to improve the stability of the ground to aid construction of the tunnel, and in soft ground to reduce/control ground displacements and hence mitigate the effects of the tunnelling operation on adjacent structures.

With respect to settlement control, it is obviously better if the choice of tunnel alignment avoids the necessity of using settlement control measures (ITA/AITES 2007). Increasing the depth of the tunnel to provide a larger cover depth will reduce the magnitudes of the displacements reaching the ground surface and shallow subsurface structures including existing tunnels and services. It is important to choose an alignment for the tunnel so that the tunnel passes through the strata which have the most favourable mechanical properties. Choosing the smallest cross section for the tunnel can help as this provides a more stable face. This may mean, in the case of transportation tunnels, choosing between a single larger diameter tunnel and a twin-tube tunnel. Twin-tube tunnels are often recommended for safety reasons as the second tube can act as an emergency exit in case of an accident, such as fire. If a TBM is used, choosing an alignment that is as straight as possible is beneficial. However it may be necessary to use artificial ground improvement measures, and some of the more common techniques are described below.

Many of the techniques described in this section can generally be applied either from the ground surface or from within the tunnel during construction. The latter will obviously slow the rate of advance of the tunnel.

4.2.1 Ground freezing

Although perceived as a relatively expensive last resort, in cases where something goes wrong and no other solution is available, this can be a powerful technique as it can be used across the whole range of ground types, depending on the groundwater flow rate. In fact, in shallow tunnelling where access can be gained from the ground surface, it is used relatively frequently (Pelizza and Piela 2005).

The freezing method is only applicable when the ground contains water, ideally still, fresh water. A ground with a moisture content greater than 5% will freeze. Water can be added via a fire hose, a sprinkler system, a borehole or injection device to raise the moisture content in the ground. The principle of ground freezing is to use a refrigerant to convert *in situ* pore water into a frostwall, with the ice bonding the soil particles together.

As a rule, if used from within the tunnel, freezing lances are installed from the tunnel in the direction of tunnel excavation as the frozen ground should create an arching mechanism (Figure 4.1). The lances are situated in the crown and, if necessary, at the springline. In order to achieve a closed frozen body, the distances between the lances are limited, e.g. 1 m, in combination with a length of 20 m or more. It is important that the frozen areas overlap to provide an impermeable barrier. Cooling fluid is pumped through the freezing lances. Examples of cooling materials are brine (salt solution) with a temperature of –50 °C to –20 °C, or liquid nitrogen which evaporates at –196 °C.

For excavations from the ground surface, a cylindrical freeze wall is formed around the periphery of the planned excavation or a layer of ground above the tunnel roof is frozen. The refrigerant pipes are equally spaced at approximately 1 m apart and, in order to ensure a continuous freeze wall, they need to be accurately drilled with minimal deviation.

Advantages of ground freezing:

- The strength of ground can be increased.
- An impermeable barrier is created. (Although it should be noted that if the freezing process is conducted from within the tunnel as opposed to from the ground surface, it is normal only to extend the frozen ground from the crown to the tunnel springline (or above) and hence this just extends the flow path for the water and does not make a completely impermeable barrier: See Figure 4.1. It is normal in this case to use ground freezing in combination with pressurized tunnelling.)
- It is non-toxic and noiseless.

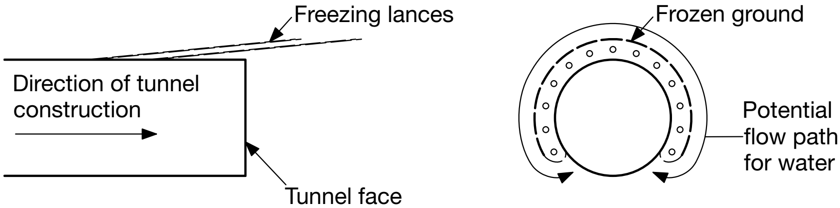


Figure 4.1 Potential flow paths when in-tunnel ground freezing is used at the tunnel crown

- It is totally removable (unlike grouting) – although there can be an adverse reaction in some soils.

Limitations:

- The time required to achieve ground freezing can be many weeks depending on the ground and groundwater conditions.
- Flowing water causes heat drain and can prevent the ground freezing. The limiting flow rate depends on the type of freezing being used (see below). For example, if a two phase brine freezing process is used, a maximum flow rate of 2 m/day can be tolerated, whereas for a direct process using liquid nitrogen, the maximum flow rate is 20 m/day.
- The boreholes must be accurately positioned to create a continuous frozen zone.

Care must be taken as there is the potential for the ground to heave during the freezing process and subsequent settlement at the end of the freezing process (ITA/AITES 2007). Ground heave is related to the frost susceptibility of the ground. In coarse grained soils, the frost susceptibility is low as the permeability is high. This means there is less heave because the water can drain as the freezing progresses. Conversely, in fine grained soils the frost susceptibility is high as these materials have a low permeability and therefore there is more heave as the water does not drain during the freezing process. Ground heave can be limited by controlling the speed of the freezing process and the sequence of freezing.

There are two methods of ground freezing:

- Two phase method (closed) – *Figure 4.2a*. In this method, a primary refrigerant (ammonia or freon) is used to cool a secondary fluid (usually brine).
- Direct process (open) – *Figure 4.2b*. In the direct process liquid nitrogen is used to freeze the ground. The nitrogen is passed down the freeze pipes and then allowed to evaporate into the atmosphere. This direct process is good for short-term or emergency projects. Liquid nitrogen is likely to be the only effective method for freezing pore water in fine grained soils.

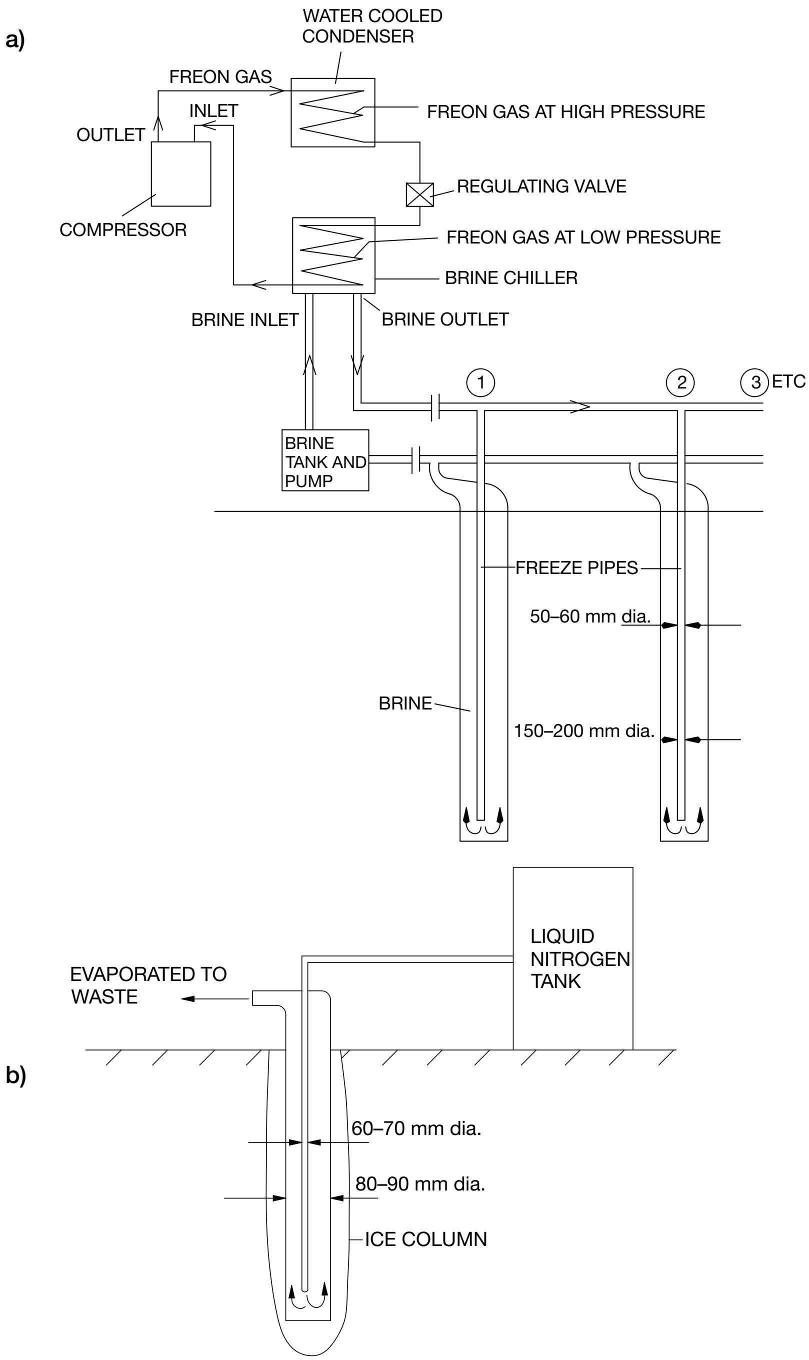


Figure 4.2 Ground freezing methods, a) two phase method, b) direct process (after Harris 1983)

When designing the freezing system, it is important to determine the thermal characteristics of the ground to be frozen and the freezing point of the groundwater. It is also important during the freezing operation to monitor the process carefully. Thermocouple strings can be used to monitor the ground temperatures between the freezing elements and to monitor the refrigerant temperature. Kuesel and King (1996) suggest a simple method of ensuring closure of the frozen ring in free-draining soils (high permeability soils) when constructing shafts by using a centrally placed piezometer. The piezometer is used to measure the water pressure within the ground. As the freeze front advances it pushes water, which expands on cooling, out of the pores between the soil particles. Once the frozen ground forms a complete ring, there is no means for the water to exit from the area and hence the pressure measured by the piezometer increases.

Figure 4.3a shows the ground freezing tubes around the perimeter of the tunnel portal. Figures 4.3b and c show the excavation of the frozen ground at the tunnel face.

Ground freezing has also been used during jacked box tunnels (section 5.10) in Boston, USA on the 'Big Dig' project in 2001 to allow jacking of box sections under live rail tracks (see section 5.10.3.2 for further details).

The technique has also been used to rescue TBMs that have become flooded due to adverse ground conditions, for example on the 2.6 m internal diameter Thames Water Ring Main in London, UK (Clarke and Mackenzie 1994). On one of the drives an open face machine with a backhoe excavator became inoperable due to water inundation when it hit an unexpected water-bearing sand stratum at a pressure of 4.5 bar. In order to remove this machine and restart the drive using an EPB machine, ground freezing was found to be an economical solution. The machine was approximately 55 m below ground surface level and a 7.6 m diameter shaft was excavated using an underpinning method (see Figure 4.26), i.e. just above the water bearing sand stratum. It was then plugged with a concrete base, and precautions were taken to control the high water pressures during the installation of the freeze lances. The freeze lances were drilled vertically from within this shaft to a level 5 m below the tunnelling machine, i.e. 61.3 m below ground surface level, to prevent vertical water flow during the excavation and recovery of the machine. The two stage freezing system was employed in this case using ammonia as the primary refrigerant and brine as the secondary refrigerant. The primary freezing period took four and a half weeks and the average temperature of the ground was –12 °C. The original machine was successfully removed and a new EPB machine completed the drive.

An example of the recovery of a tunnel where a collapse occurred was in Hull, UK (Brown 2004). In this project a 100 m long section of a 3.6 m diameter tunnel associated with a new wastewater project collapsed and ground freezing using liquid nitrogen was used to stabilize the ground and provide an impermeable barrier to allow reconstruction to take place.

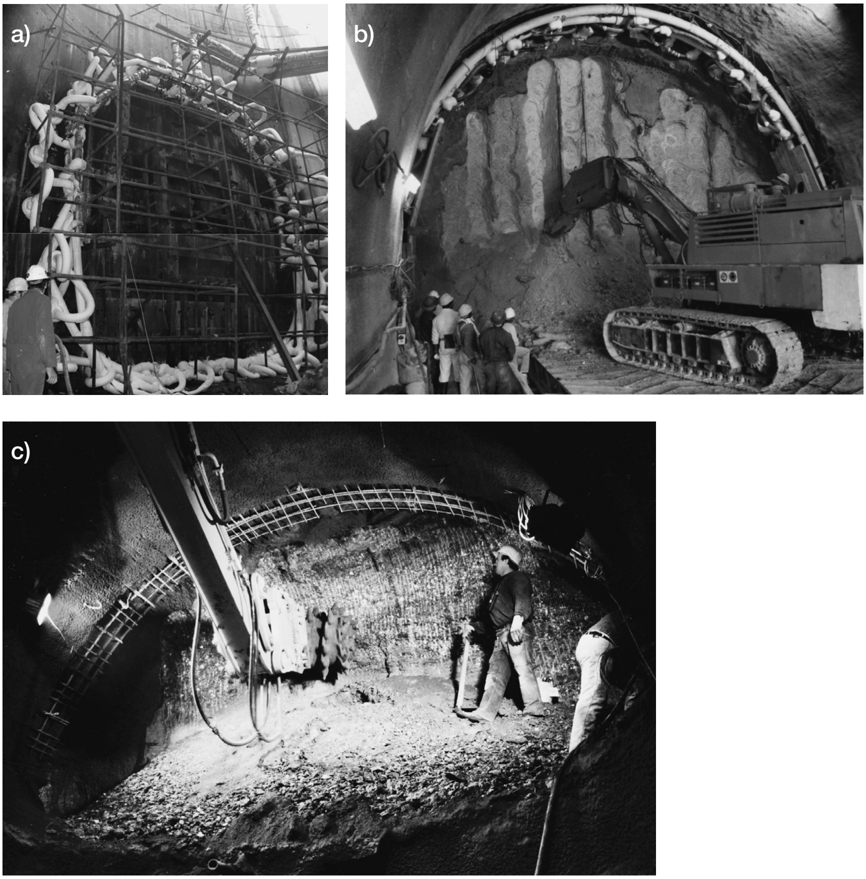


Figure 4.3 Examples of ground freezing used in tunnelling, a) horizontal freezing to rescue a broken down TBM in Cairo, b) freezing at the portal of a 13 m diameter road tunnel in Du Toitskloof, South Africa, through decomposed granite, c) excavation of the tunnel crown through frozen sand and gravel in Dusseldorf (courtesy of British Drilling & Freezing Co. Ltd)

The tunnelling took place through predominantly alluvial granular deposits at a depth of 15.5 m. On this project the maximum consumption of liquid nitrogen over any 24 hour period was 165,000 litres.

Further details on artificial ground freezing can be found in Harris (1995), Holden (1997) and Woodward (2005).

4.2.2 Lowering of the groundwater table

If groundwater lowering can be achieved successfully, a marked improvement is possible in the ground properties. However, groundwater table lowering is, even as a time limited measure, not always possible. Under running streams, in settlement-critical inner city areas, in areas where there may be an influence on existing water supply aquifers, or in areas where there could be a potential adverse effect on the flora, this measure should not be used. Furthermore it requires intensive installations for holding the extracted water, which may have to be treated before it can be disposed of.

In permeable strata where the permeability, k, exceeds about 10^{-3} cm/s, or where an aquifer can be dewatered below less permeable strata, the level of the water table over a wide area can be drawn down by pumping from boreholes and deep wells. These processes are widely used in open excavations and are suited also to cut-and-cover tunnels and shallow bored tunnels, for example Lainzer Tunnel LT31, Vienna, see section 8.3 (Megaw and Bartlett 1982).

There are two principle methods of groundwater lowering: wellpoints and deep filter wells. Wellpoints, although one of the most versatile methods of dewatering, are limited to dewatering to a depth of about 6 m (limited by the effective vacuum lift of a pump), although staged wellpoints can be used to go deeper, but a greater excavated plan area is required. Wellpoints are installed at between 1 to 3 m intervals by wash boring, i.e. using high pressure water jetting to form the borehole, but the spacing depends on the permeability of the ground. Figure 4.4a shows a typical arrangement for a wellpoint system. Wellpoints can also be used from inside the tunnel. In this case they should be directed upwards.

Deep wells can be used to dewater to greater depths. These consist of 300 mm or greater wells sunk at an average spacing of 3 m or more to below the level required for the dewatering. A filter is used at the base of the well around perforated suction pipes, above which a submersible pump is located (Figure 4.4b). It is important to establish a detailed conceptual model from the site investigation and pumping test data, preferably with distance/drawdown/time results. Further details on the design of wells can be found in Woodward (2005).

Drawdown of the groundwater level can cause consolidation settlements in the surrounding ground and hence affect adjacent structures, and therefore it should be closely monitored. The extent of the drawdown zone depends on the depth of the well and the type of ground.

Further information on groundwater lowering and dewatering can be found in Preene *et al.* (2000), Cashman and Preene (2001) and Powers *et al.* (2007).

4.2.3 Grouting

Grouting involves the process of injecting a material into the ground with the following two principal objectives:

- to reduce the permeability of the ground;
- to strengthen and stabilize the ground. In soft ground this leads to an increase in its 'strength' and in jointed rock in its 'stiffness'.

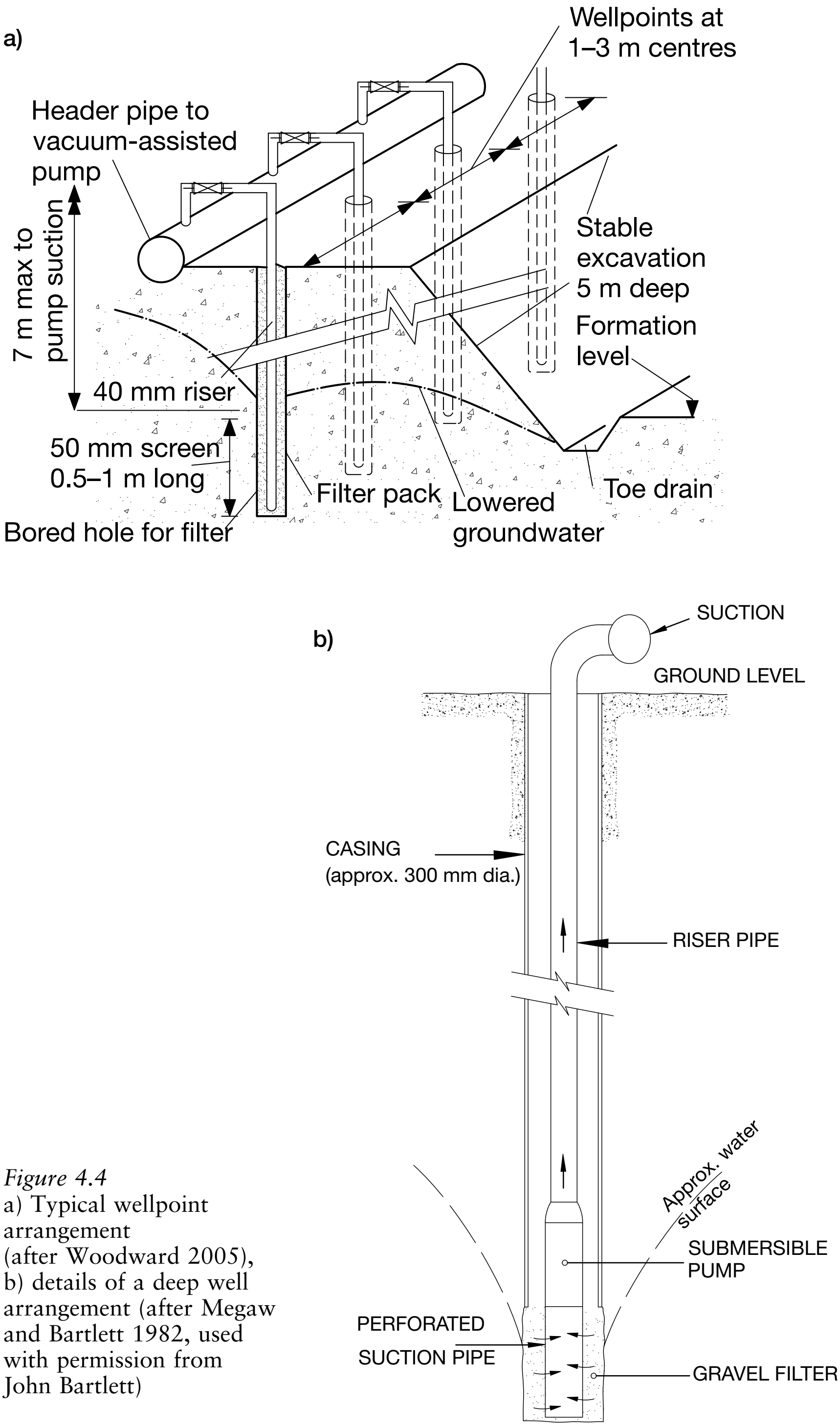


Figure 4.4
a) Typical wellpoint arrangement (after Woodward 2005),
b) details of a deep well arrangement (after Megaw and Bartlett 1982, used with permission from John Bartlett)

Grouting operations can be carried out either from the ground surface (or from within an adjacent shaft to the tunnel operation) or from within the tunnel construction itself. They can also be applied to locally stabilize the foundations of structures likely to be affected by the tunnelling works in the form of settlements.

For tunnel grouting, the grouting holes are drilled ahead of the advancing tunnel in a pattern of diverging holes at an acute angle to the tunnel axis to form overlapping cones of treated ground. For drill and blast tunnels the holes can be drilled at the face (Muir Wood 2000). For TBMs the holes can be drilled forward from the rear of the machine, to avoid affecting the cutter wheel, but direct grouting of the face through the cutter wheel is also possible. Grouting using a shield TBM can also be carried out through the shield, both towards the face and also radially. However, great care is needed as there is a risk of grouting-in the machine. In addition, grouting can be conducted radially through the lining to fill any voids. Figure 4.5 shows some examples of grouting during tunnel construction.

Percussion and rotary drilling are used to install the grout tubes. The grouting tubes may be simple open-ended tubes, possibly fitted with an expendable tip to prevent blockage during installation, or perforated tubes which allow grout to be injected over a specific length.

The use of a tube-a-manchette (TAM) or sleeved tube makes successive injections at specific locations possible. Perforations at appropriate intervals

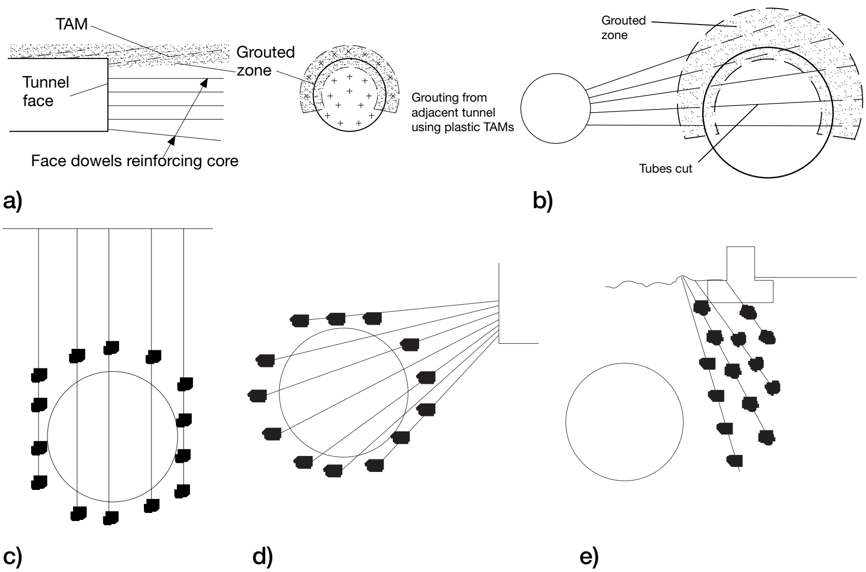


Figure 4.5 Examples of grouting tunnels during construction, a) from within a tunnel, b) using an adjacent tunnel (after Woodward 2005), c) from the ground surface, d) from an adjacent shaft, or e) as protection to adjacent structures (after Baker 1982, used with permission from ASCE)

along the tube are closed by an external elastic sleeve which can be opened by the internal pressure of the grout. The grout is passed to the injection point by a movable separate internal tube. The grout is contained within the location of the perforation using seals either side of the end of this internal tube. Figure 4.6 shows details of a tube-a-manchette (called a sleeve port pipe in the US).

There are several types of grouting technique and these can be described as permeation grouting, jet grouting and compaction grouting.

PERMEATION GROUTING (CHEMICAL GROUTING)

This technique fills the voids in the soil with either chemical or cement binders with the intention of not disturbing the fabric of the ground. The range of particle sizes over which it can be applied is from sands (0.06 mm) to coarse gravels (60 mm). Further information on permeation grouting can be found in Karol (1990).

JET GROUTING

This technique uses high pressure jets to break up the soil and replace it with a mixture of excavated soil and cement. The range is wider than for permeation grouting, extending from clays (< 0.002 mm) to fine gravels (10 mm). Jet grouting may be used in pre-bored holes or the 'jets' can be self-drilled. Once the jet has reached the required depth, it is rotated and the jetting fluids are pumped at high pressure to the jetting tip as the system

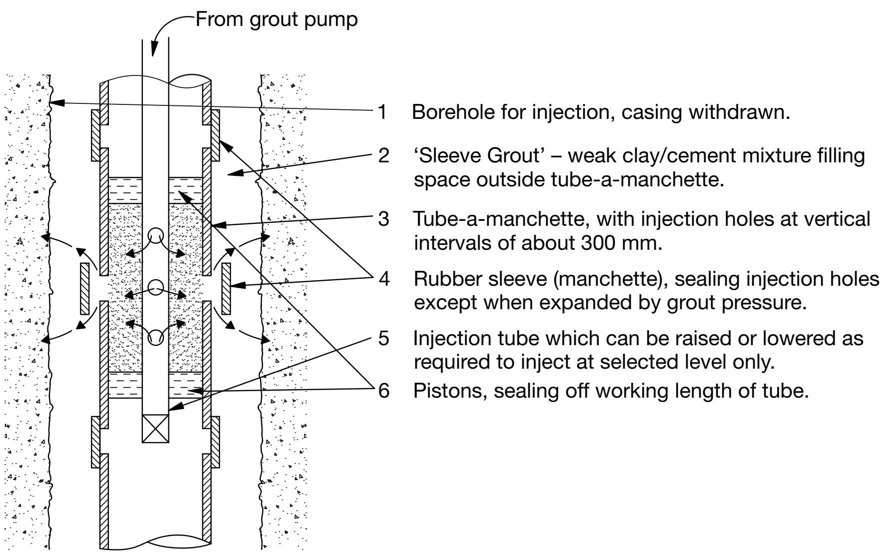


Figure 4.6 General arrangement of a tube-a-manchette (after Megaw and Barlett 1982, used with permission from John Bartlett)

is withdrawn from the hole at a controlled rate to form an *in situ* column (Woodward 2005). There are three basic jetting systems, a single jet which uses just grout, a double jet system involving grout with an air shroud and a triple jet system where the grout is discharged through one hole and just above this is a second jetting point where an air-water mixture is injected. It should be noted that if the hole blocks with debris, a sudden pressure can build up with bursting pressures developing, which can damage adjacent services and even flood cellars with grout. This is a real risk and the operator has to carefully monitor the return flows and pressures. The system used depends on the ground type, with the single system being suitable for sands with $N_{SPT} < 15$ (where N_{SPT} is the standard penetration test blow count, as described in section 2.3.2.2), and the other systems used for finer grained soils (Woodward 2005). If the jet is not rotated then more of a 'panel' shape is produced rather than a column. Further details on jet grouting can be found in BSI (2001a).

COMPACTION GROUTING

This technique differs from both permeation and jet grouting in that it is a ground improvement technique rather than a ground treatment technique. Compaction grouting is essentially the injection of a low slump (typically 25–100 mm) grout, i.e. stiff grout, such that an expanding bulb forms. This expansion causes deformation and densification around it and ultimately improves the ground. The method is carried out by either drilling or driving small diameter casings (89–114 mm typically) to the required depth, withdrawing the rods or knocking off the drive point and then pumping the grout to the bottom of the hole (Essler 2009). The range of applicable soils for this method is similar to permeation grouting ranging from sands (0.06 mm) to medium gravels (30 mm).

It should be noted that rock grouting differs from the above techniques as it is neither the material interstitial pores that are grouted as in permeation grouting nor is the material body destabilized as in jet grouting or compaction grouting, but instead the fissures and fractures are filled (Essler 2009).

Grout types can be split broadly into two categories, suspension grouts and chemical solution grouts. There are several requirements that a grout should meet in terms of its basic properties as listed below (after Whittaker and Frith 1990).

- *Stability* – grouts should remain stable during the mixing and injection processes and not separate prematurely in the case of suspension grouts, or set prematurely if it is a liquid grout.
- *Particle size* – for a suspension grout this sets the lower limit of the grain size of the soil that it can penetrate.

- *Viscosity* – this is basically a measure of its ability to penetrate soils. Other flow properties and the gelling time determine the maximum injection radius.
- *Strength when set or gel strength* – this depends on whether the grout is being used to strengthen the ground or reduce its permeability.
- *Permanence/durability* – the grout, when set, should resist chemical attack and erosion by groundwater.

Suspension grouts basically consist of cement slurry with a cement/water ratio of approximately 0.1 to 0.4, and an optional clay component. The purpose of the clay is to reduce the cement consumption and to improve the stability and viscosity of the suspension. Sand can be added to grout suspensions when large fissures are to be injected. Additives such as plasticizers (comprising metal salts, such as lithium, sodium and potassium salts) can be used in suspension grouts to prevent the clay particles flocculating (i.e. clumping together) and this will give different properties to the grout. Suspension grouts are best suited to injection into fissured rocks and granular media with large voids and porosity (down to a particle size of approximately 0.2 mm). A suspension grout containing fine to coarse sand, cement and a plasticizer is technically known as a mortar, which can be used to plug large fissures and cavities (Whittaker and Frith 1990).

Chemical grouts usually consist of solutions and resins which form gels. They reduce the permeability by void filling and strengthen the ground. These grouts have a major advantage over suspensions in that they can be injected into very fine grained soils, since some liquid grouts, such as resin types, have viscosities approaching that of water (down to a particle size of approximately 0.02 mm). The strength of chemical grouts is generally low compared to cement grouts.

The most common types of grouts are either cement bentonite (suspension grout) or silicate based (chemical grout). The type of grout depends on both the ground type and the grouting technique adopted. For filling large voids, materials such a pulverized fuel ash (PFA, a waste product from coal-fired power stations) can be used.

Further details on grouting techniques and grout materials can be found in Xanthakos *et al.* (1994) and Moseley and Kirsch (2004).

4.2.4 Ground reinforcement

There are three distinct types of ground reinforcement methods (Whittaker and Frith 1990, Woodward 2005):

ROCK DOWELS

These are reinforcing elements with no installed tension. They consist of a rod, faceplate and nut (a conical spacer is sometimes used if the angle between the dowel and the face plate differs significantly), and can be made from deformed steel bars, glass fibre or plastic, depending on whether a permanent or a temporary installation is required. The rod is usually embedded in a mortar or grout filled tube, although resin capsules are also used extensively. Dowels can be used as a systematic reinforcement of the ground or in hard rock can be placed at discrete locations to prevent unstable parts of the ground falling into the excavation. (Note: in Austria and Germany cemented rock dowels are commonly know as 'SN-anchors', named after 'Store-Norfors', the Norwegian city where they were first used. In the US dowels are known as nails.)

Another development is inflatable rock dowels (Swellex®, Atlas Copco). These consist of folded steel, closed at the end, and inflated by water. The steel expands and is pressed against the wall of the borehole providing close contact between the dowel and the ground, resulting in no need for grout or resin.

ROCK BOLTS

These are reinforcing elements which are tensioned during installation. They consist of a rod and mechanical or grouted anchorage (resin capsules or cement) coupled with some means of applying and retaining the rod tension. Mechanical fixings are suitable for hard rock, whereas grouted, fixed length bolts can be used in most rock types. The length varies between 2 to 8 m for resin capsule grouted bars, and 3 to 20 m for an expanding shell fixing on a bar. Figure 4.7 shows some diagrams of typical rock bolts and dowels. (Note: in some countries the term 'bolt' is also used for untensioned systems.)

ROCK ANCHORS

These are reinforcing elements which are tensioned following installation and are of higher capacity and generally of greater length than rock bolts. They consist of high strength steel tendons usually in the form of cables to which is fitted a stressing anchorage at one end and means of transferring a tensile load to the cable at the other end. These can be used in most rock types. Double corrosion protection is required for permanent anchors and conducting proof loading tests of each anchor is normal during tensioning. As mechanical anchors slacken with time, and hence could allow movement of the ground, fully bonded anchors should be used.

There are four generally accepted mechanisms by which rock reinforcement can improve the stability of the ground (Whittaker and Frith 1990).

1 By stabilizing individual blocks of material that may detach due to gravity in relatively competent and well-jointed rocks, by using rock bolts with an anchorage force capacity greater than the weight of the block.

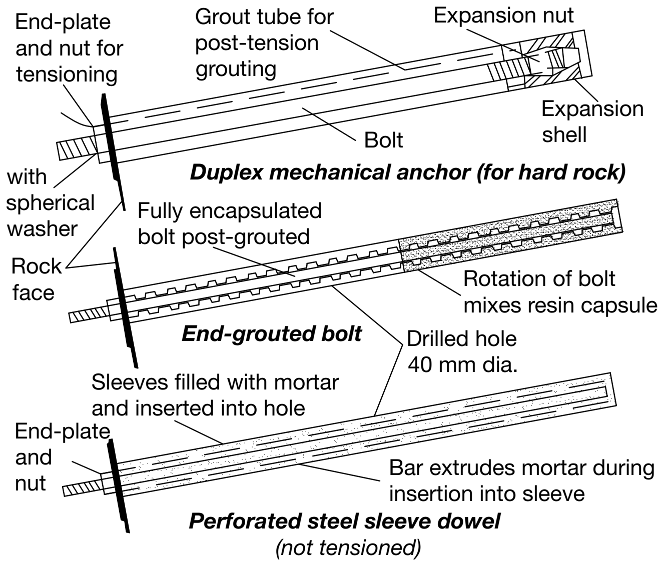


Figure 4.7 Examples of typical rock bolts and dowels (after Woodward 2005)

2 By using tensioned or untensioned bolts to maintain the shear strength of the ground along discontinuities in weaker fractured ground conditions.
3 By using fully grouted untensioned rock bolts in laminated or stratified rocks to preserve the inter-strata shear strength.
4 By using tensioned rock bolts installed relatively quickly after excavation to improve the degree of confinement or the minor principal stress (this is normally perpendicular to the tunnel wall) in overstressed rocks.

Rock reinforcement alone is unlikely to be appropriate if (Woodward 2005):

- the support pressure required is greater than 600 kN/m^2;
- the spacing of dominant discontinuities is greater than 600 mm;
- the rock strength is inadequate for anchorages;
- the RQD is low or there are infilled joints or high water flow.

Figure 4.8 shows some typical examples of the arrangement of rock bolts/dowels within tunnels.

An example of a typical specification for supporting blocks for short (5 m) spans within the ground is given below (Woodward 2005):

- minimum bolt length, 0.5 × span or 3 × width of an unstable block;
- maximum spacing, 0.5 × bolt length or 1.5 × width of a critical block, and 2 m when using mesh restraints;

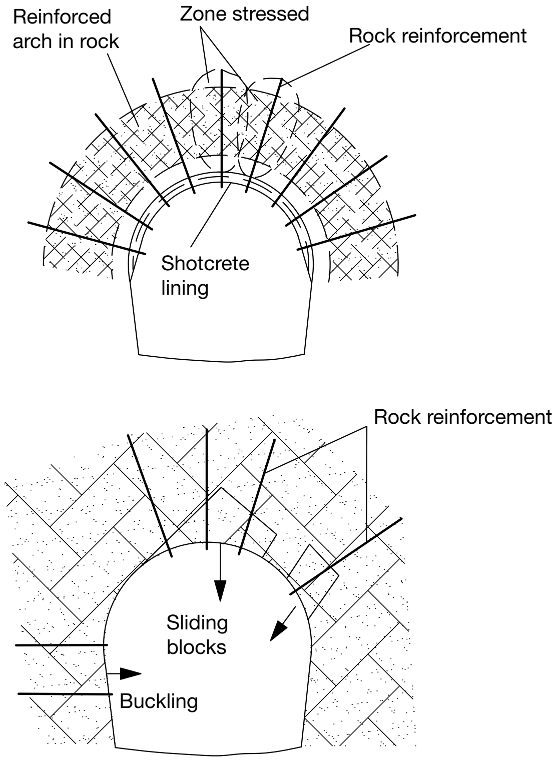


Figure 4.8
Typical examples of the arrangement of rock bolts/dowels within tunnels (after Woodward 2005)

- larger spans in fractured rock will require primary, secondary and even tertiary reinforcement.

(Note: dowels and bolts are also applicable for soft ground.)

Figure 4.9 shows an anchor installation associated with sprayed concrete lining in the Heidkopf Tunnel (HKT), Göttingen, Germany. This tunnel was constructed through sandstone and limestone and consisted of a twin tube, 2-lane road tunnel, 1720 m long (each tube) and a cross section of 88–129 m^2 (approx. width 12 m). Figure 4.10 shows the load testing of an anchor as part of the construction of the Lainzer Tunnel LT31, Vienna (see section 8.3 for further details of this tunnel).

4.2.5 Forepoling

This technique is aimed at limiting the decompression in the crown immediately ahead of the face (ITA/AITES 2007). Longitudinal bars (dowels) or steel plates (forepoling plates) are installed ahead of the tunnel from the periphery of the face, typically over the upper third or quarter of the

Figure 4.9 Anchor installation associated with sprayed concrete lining, HKT Tunnel, Germany (courtesy of ALPINE BeMo Tunnelling GmbH Innsbruck)

Figure 4.10 Load testing of an anchor, Lainzer Tunnel LT31, Vienna, Austria

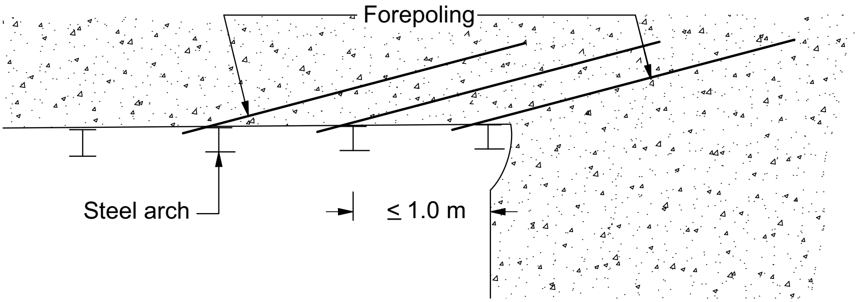


Figure 4.11 Basic arrangement of forepoling using dowels (after ITA/AITES 2007)

excavated profile. In rock, the plates or dowels driven ahead of the excavation are also known as spiles.

DOWELS

The material used for dowels is the same as that used for rock dowels (section 4.2.4). Dowels are also installed from within the tunnel, and positioned on lattice girders at an angle to the direction of tunnelling (Figure 4.11). If the ground is too dense or too hard, dowels are placed in pre-drilled holes. In this case the hole can be filled with grout before the dowels are pushed further into the ground. They are designed to protect the crown area against afterfall (i.e. falling blocks of material from the tunnel roof). The separation of the dowels is dependent, amongst other things, on the size of the blocks in the ground, and as a rule is greater than 20 cm. The length of the dowels is approximately 3 to 4 m (about three to four times the heading advance).

PLATES (SHEETS)

Forepoling plates, mainly made from steel, are pushed forwards individually, but close together in the same way as described for the dowels. Plates are used with coarse grained soils such as sands or gravels, which would fall through the spacing between dowels.

4.2.6 Face dowels

Face dowels can be used to improve the stability of an excavated tunnel face. The technique involves installing an array of dowels over the cross section of the tunnel face (Figure 4.12). The dowels should be made of material that can be easily excavated, for example fibreglass. Fibreglass dowels are particularly useful when using a TBM or roadheader as they can be cut through easily. However, fibreglass dowels can produce sharp,

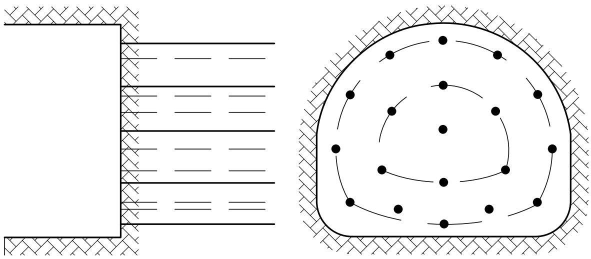


Figure 4.12 Schematic showing a typical arrangement of dowels used in the tunnel face (after ITA/AITES 2007)

and potentially dangerous, ends when excavated due to the brittle manner in which they break. Therefore, steel dowels can prove more useful even though the excavation process needs more care. Steel dowels can be easily cut using handheld cutters. Ideally, the dowels should provide a continuous stability as the excavation advances. This is achieved by the dowels being shortened as the face advances until a minimum length is reached. At this point a new set of dowels is installed in the face. These new dowels overlap with the previous dowels by a few metres.

Figure 4.13 shows an example of temporary face dowels being used during the construction of the LT31 Tunnel in Vienna, Austria. In this case the dowels were 12 m long and overlapped longitudinally by 5 or 6 m. There was also a plate (12 cm × 50 cm) attached to the end of the dowels that distributed the stress and avoided overstressing the sprayed concrete, which was relatively thin (approx. 10 cm) compared to the side walls (approximately 30 to 35 cm thick).

4.2.7 Roof pipe umbrella

In the roof pipe umbrella method steel pipes are drilled from within the tunnel in the direction of tunnelling around the perimeter of the tunnel roof, as described in the artificial ground freezing section 4.2.1. The steel pipes have a diameter of approximately 70 to 150 mm. After drilling, the holes are filled with grout. The spacing of the pipes ranges from approximately 20 to 50 cm. The length of the pipes is often 15 m or more. The previous umbrella overlaps with the subsequent umbrella by at least 3 to 5 m. Roof pipe umbrellas act like forepoling, i.e. they are supposed to protect the crown area against afterfall. However, due to their larger diameter and length, roof pipe umbrellas are a lot more robust than forepoling.

Figure 4.14 shows an example of forepoling (dowels) in association with sprayed concrete lining being used as a roof pipe umbrella on the LT31 Tunnel, Vienna (see the case study in section 8.3 for further details of this tunnel).

Figure 4.13 Installation of temporary dowels into the tunnel face during the construction of the Lainzer Tunnel LT31 in Vienna, Austria

4.2.8 Compensation grouting

Compensation grouting is a technique developed to control the settlement of structures in the vicinity of tunnels constructed in soft ground and is one of the most specialized forms of ground treatment. Settlements can occur around tunnels as a result of stress changes during construction and are discussed further in section 7.1. It is therefore not a stabilization technique to aid tunnel construction as such, but to avoid the tunnel construction adversely affecting adjacent structures. It can also be used as a method of maintaining or re-levelling structures or ground subject to on-going settlements, for example due to the consolidation of clay soils. Generally, compensation grouting is only considered after it has been determined that the ground displacements cannot be reduced to an acceptable level by increasing the support to the ground from within the tunnel during construction.

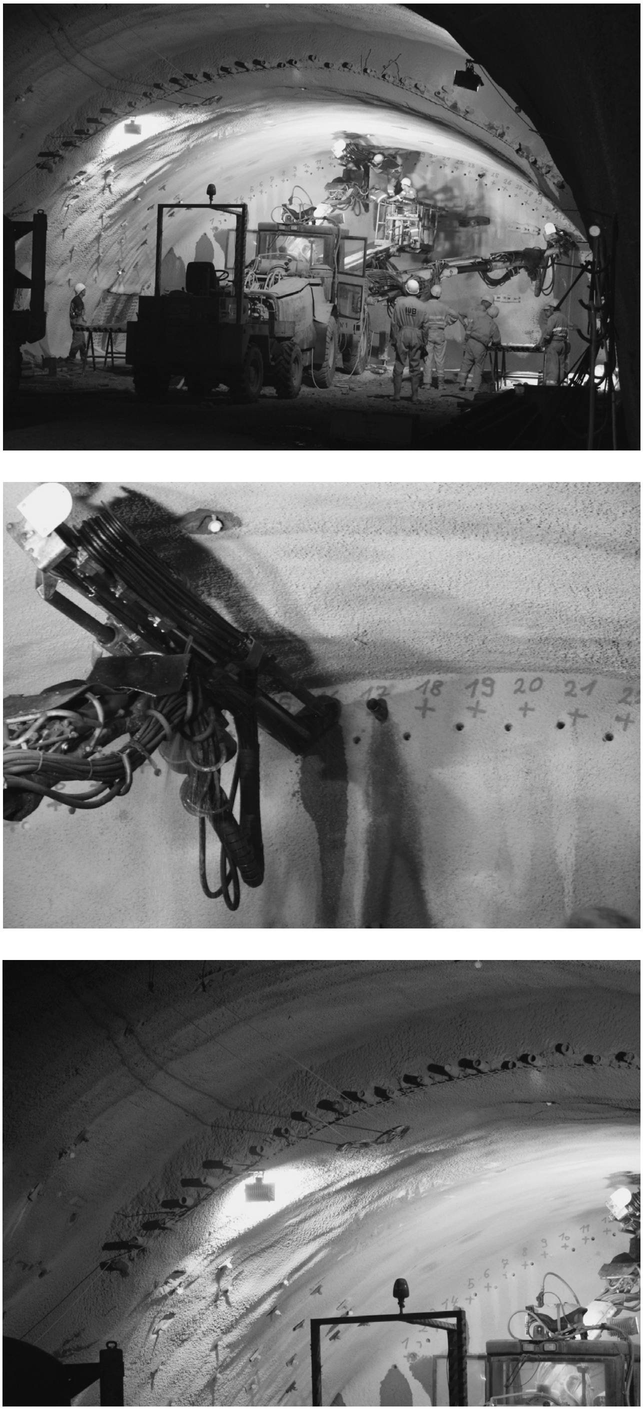


Figure 4.14
Forepoling using dowels as a roof pipe umbrella on the Lainzer Tunnel LT31 in Vienna, Austria

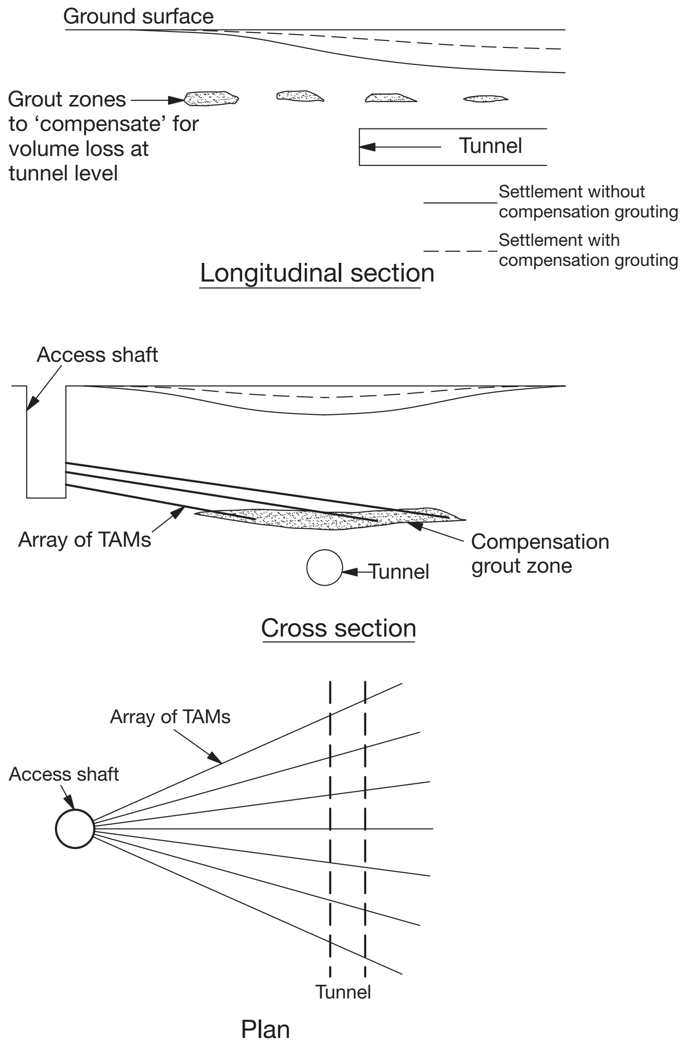


Figure 4.15 Compensation grouting for tunnel construction (after Woodward 2005 and Burland *et al.* 2001a)

This method involves injecting grout into the ground at a level between the tunnel crown and the structure to be protected (Figure 4.15). This is commonly done from grout holes drilled radially from a shaft. Tube-a-manchette (TAM) grouting (section 4.2.3) or similar techniques are used to inject the grout in controlled amounts after an initial preconditioning of the ground locally. Preconditioning means that grout is used to locally compress the soil, possibly fracturing the ground, so that subsequent injections have an immediate effect on the ground. Controlled volumes of grout are used to 'compensate' for the occurring ground displacements. The volumes involved are usually low, for example 20–100 litres, to ensure that grout does not travel out of the compensation grouting zone. The required grout depends on the ground efficiency, i.e. the ratio of grout volume to be injected to ground volume change. Typically, the ground efficiency in London Clay might be better than 50%, in a soft clay 10–15% and in sands 20–30% (Essler 2009).

It is important that there is real time feedback during the grouting operations from instrumentation on the structures being 'protected' so that the compensation grouting can be accurately used. This is a good example of the need for a clear observational approach (section 7.3.3). Injecting the grout into the ground stops the ground moving downwards, which is essentially a jacking operation. Therefore a reaction force is necessary to generate this support, which is directed downwards. Hence, it is important to position the drill holes at a level so that this downward force will not adversely affect the tunnel heading (and the subsequently installed lining). Careful control is also needed when using this technique near existing tunnels as there is a danger of the outward forces generated by the grout injection causing deformation or damage to the existing tunnel lining.

4.2.9 Pressurized tunnelling (compressed air)

If a tunnel is constructed within the groundwater, special precautions have to be taken or tunnel construction methods chosen which prevent water from getting into the tunnel as this would make the works impossible. One such method is by using air under high pressure within the tunnel during construction. Air pressure can be used to control water flow, and hence stability, below the groundwater table and is one of the oldest pressurized face support methods used in tunnelling.

The disadvantage of constructing a tunnel under air pressure is that, in order to maintain the air pressure at the face, all the materials and spoil, as well as the workforce, have to be passed through an airlock system. The maximum working pressure and the time that workforce can spend working in compressed air have to be strictly controlled. Originally the whole tunnel length was put under air pressure, i.e. from the face to the pressure chamber, which was most often near the starting shaft. However, developing from the idea of no longer wishing to put the whole tunnel under pressure, pressure bulkheads (airlocks) started to be placed closer to the rear of the tunnel face (Figure 4.16).

Pressurized air is compressed into the tunnel by using compressors installed on the surface until the required overpressure is established (atmospheric overpressure acts in addition to air pressure). In order to achieve this, the necessary amount of air has to be determined prior to, and during, tunnel construction. The danger of 'blowouts' has to be considered and, as far as possible, minimized.

Figure 4.16 Airlock arrangement on a pipe jacking project for a new sewer in Germany (see section 5.11 for a description of pipe jacking)

BLOWOUTS

A blowout is where the pressurized air finds a pathway to the ground surface, blows out suddenly and the pressure at the face drops and can no longer be maintained. This can result from the seepage of air into the ground and the consequent loosening of the ground matrix (for example lifting of sand). This can increase the porosity enormously resulting in a sudden loss of pressure within the tunnel. Obviously, this situation is particularly dangerous for tunnellers.

WORKING UNDER PRESSURE

The normal air pressure at the surface of the earth, or more precisely at sea level, is 1013 millibars, which is around 1 bar or 100 kN/m^2.

The critical level for working under high pressure begins with an atmospheric overpressure of approximately 1.0 bar. If someone is put in a situation where the pressure is higher than atmospheric, the amount of soluble nitrogen (N_2) increases in the body. If the outside pressure is suddenly reduced the nitrogen comes out of solution instead of being exhaled and this can result in the formation of bubbles in the blood. The bubbles can result in blockage of the arteries. It is possible that the excess nitrogen can settle in the joints and result in pain. Complaints, which can be traced back to both phenomena, often only occur after many years. The illness is known under many different names, for example caisson or decompression illness (this is the same condition that can be experienced by divers).

In order to avoid endangering tunnellers medically, the decompression chamber times on a tunnel construction site are strictly regulated. Different regulations and guidelines exist in various countries, for example in the UK 'The Work in Compressed Air Regulations 1996' (UK Government 1996) and in Germany the pressurized air regulation of 1972 in the Bundesgesetzblatt Teil 1, Nr. 110, last updated in 2008 (air pressure regulation).

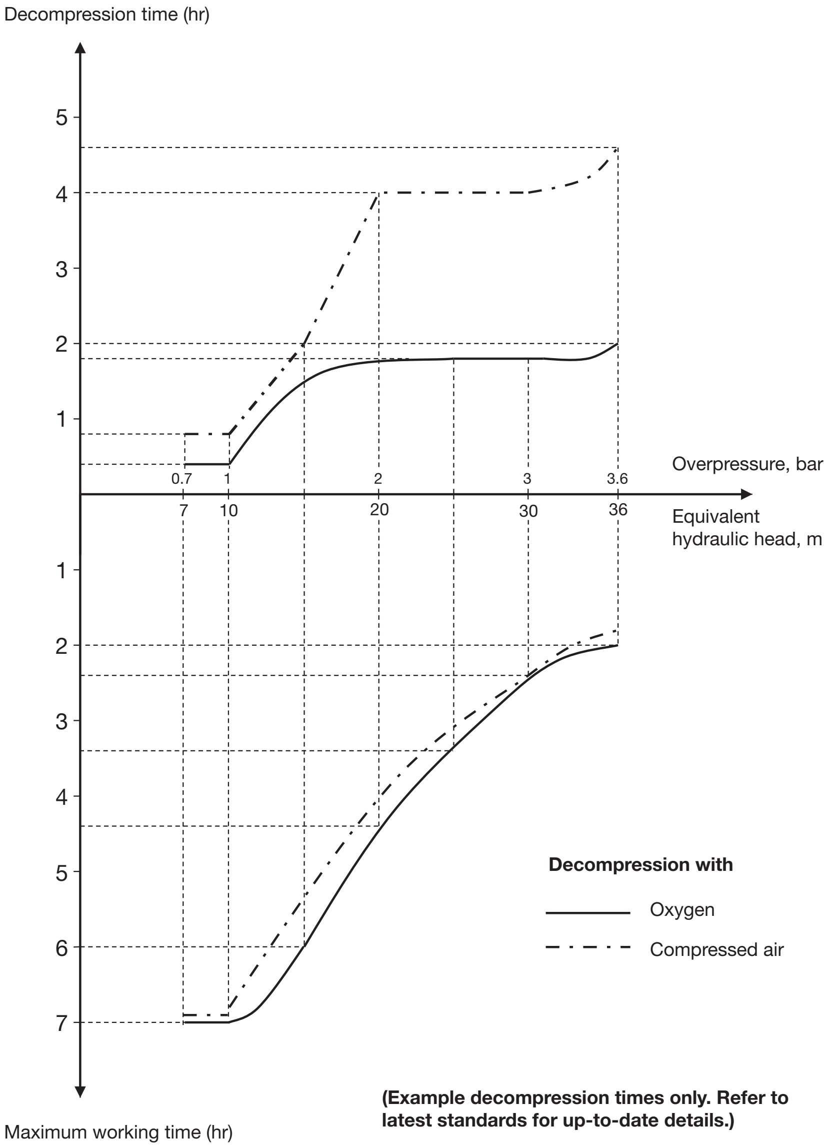


Figure 4.17 Simplified decompression times after Bundesgesetzblatt (2008), Germany

Generally the situation can be summarized as:

- the higher the air pressure, the longer the decompression time;
- the longer the working exposure, the longer the decompression time.

Figure 4.17 shows, in a simplified form, the recommendation of the Bundesgesetzblatt, Germany. Maximum working and decompression times are shown up to an overpressure of 3.6 bar. The use of a decompression chamber is required from an overpressure of 0.7 bar. Decompression using oxygen through face masks has now become the norm with the exhaled oxygen being discarded outside the airlock. This makes it easier to exhale any excessive nitrogen. The level of oxygen inside the airlock is carefully monitored to ensure no build up of oxygen as this poses a fire risk.

Material and personnel are put through the airlock separately. The personnel airlocks must consist of two chambers. In the UK, when the working pressure is above 0.7 bar a medical lock must be provided for the recompression and treatment of any person showing symptoms of decompression illness (UK Government 1996). In some countries this is only necessary if no suitable hospital facilities are available in the vicinity of the site. After decompression the personnel are required to stay on the construction site for 30 minutes as most of the symptoms appear within this timeframe. With the aid of precautionary measures it is possible to practically rule out any delayed symptoms. Although there is an opinion these days that working under high pressure is no riskier than other tunnelling works, there are health and safety implications and consequently these risks have to be managed (see section 6.1.15).

4.3 Tunnel lining systems

4.3.1 Lining design requirements

The design of a permanent tunnel lining solution is influenced by the full range of project-specific operational requirements, i.e. not only the ground loading and water control. These include the electrical and mechanical operation and maintenance and safety aspects such as resistance under fire (Legge 2006).

Design issues that need to be considered are:

- functionality, for example lining and material type and lining thickness, material properties, constructability;
- durability, for example corrosion, adverse chemical reactions, fire;
- appearance, for example the effect of water, cracking, deformation, surface texture.

CHARACTERISTICS OF LINING BEHAVIOUR

A list of common characteristics that pervade all lining systems are (after Kuesel and King 1996):

- The processes of ground pre-treatment (e.g. grouting), excavation and ground stabilization (e.g. rock bolting) alter the pre-existing 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. The loads acting on a tunnel are not well defined, and its behaviour 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. Defining the loads on the tunnel lining is one of the most challenging aspects for a civil engineer on a tunnel project.
- Tunnel lining is a four-dimensional problem. During construction, the ground conditions at the tunnel heading involve both transverse and longitudinal arching, or cantilevering from the excavated face. All the ground properties are time-dependent, 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 the lining installation is an important variable. In addition, some tunnel linings such as sprayed concrete lining (shotcrete) can itself have time-dependent characteristics. In the case of sprayed concrete lining the stiffness is extremely time-dependent and the effects of early creep have to be taken into account before the lining reaches its full strength.
- The most serious structural problems encountered with actual lining behaviour are related to the absence of support rather than to the intensity and distribution of the load, for example inadvertent voids left behind the lining. However, exceptions exist and poor ground conditions could result in additional, unexpected loading on the lining.
- In most cases in hard rock, the bending strength and stiffness of structural linings are small compared with those of the surrounding ground. The properties of the ground therefore control the deformation of the lining, and changing the properties of the lining will not significantly change this deformation. It is important that the lining has adequate ductility to conform to the imposed deformations, and adequate strength to resist bending stresses is therefore secondary. The lining therefore forms a flexible ring confined by the ground.

4.3.2 Sprayed concrete (shotcrete)

Sprayed concrete is concrete which is conveyed under high pressure through a pneumatic hose and projected into place at high velocity, with simultaneous compaction (DIN 2005). Sprayed concrete can also be called 'shotcrete' and both terms are used in this book.

Sprayed concrete is an effective material for tunnel linings as (after Thomas 2006):

- it is a structural material that can be used as a permanent lining;
- it can be applied as and when required in a wide range of profiles and it can be adjusted to suit a wide range of ground conditions. Sprayed concrete is particularly suited to lining shafts, junctions, non-circular tunnels and tunnels of variable shape;
- it is soft when sprayed, but rapidly increases in stiffness and strength, thereby providing an increasing amount of support to the ground with time. This helps to limit movements in the ground, but also allows a degree of stress re-distribution to occur;
- it is possible to mechanize the shotcreting process, thus providing potential health and safety benefits.

Sprayed concrete consists of water, cement and aggregate, with various additives. The mix, compared to conventional cast concrete, has more sand, a higher cement content, smaller sized aggregate and more additives. This leads to a faster increase in strength and other properties with age, a lower ultimate strength and more pronounced creep and shrinkage behaviour. The creep behaviour may be important, particularly when loaded at an early age, and could become 'overstressed', i.e. loaded to a high percentage of its strength (Thomas 2006). This behaviour has to be considered very carefully when modelling the tunnel. Furthermore, it has to be taken into account when analysing observed displacements as the sprayed concrete will deform initially without any stresses being induced. It is critical to judge when the displacements are exceptionally large and the tunnel is in danger of collapse. There is no threshold value for the collapse and it depends on each situation. Measurements from previous cross sections can be consulted (if available), but often it is down to the experience of the civil engineer. This is discussed further in section 5.7 on NATM tunnels and section 7.3.4 on in-tunnel monitoring.

Additives such as microsilica have been found to improve durability of sprayed concrete linings and this forms a more dense concrete. Steel fibres can also be used as reinforcement for sprayed concrete and further details on this can be found in Thomas (2009a).

Waterproofing of sprayed concrete linings can consist of sheet membranes where complete watertightness is required. Where criteria for watertightness are less onerous then spray-on membranes or simply the inherent impermeability of concrete itself can be used to prevent water ingress. However, it should be noted that sprayed concrete is not as watertight as cast concrete as the joint between the sprayed concrete layers and construction advances are not completely watertight.

The specification of sprayed concrete works is straightforward since there are several published guidelines, for example ÖBV (1999) and

EFNARC (1996). Target strengths should always be specified for the early age period, i.e. less than 24 hours. It is important that the sprayed concrete gains sufficient strength to carry the anticipated loads at all ages (Thomas 2006).

There are two ways of producing sprayed concrete: the *dry mix* process and the *wet mix* process (Thomas 2009a).

The *dry mix* process uses a mixture of naturally moist or oven dried aggregates, cement and additives, which is conveyed by compressed air to the nozzle where it is mixed with water and liquid accelerator. The water/cement ratio is controlled by the 'nozzleman' during spraying.

A few reasons for using a dry mix process are (Thomas 2009a):

- higher early-age strength;
- lower plant costs;
- small space requirement on site;
- more flexibility during operation, i.e. it can be available as required as there is less equipment cleaning needed.

This means it is suited to projects requiring small to intermediate volumes of sprayed concrete and where there are space constraints on site (Thomas 2009a). The main disadvantages of the *dry mix* process are the higher levels of dust (health and safety) and the potential variability of the product due to the influence of the nozzleman.

The *wet mix* process involves conveying ready-mix (wet) concrete to the nozzle by either compressed air or pumping. Liquid accelerant is added at the nozzle. This is either controlled by the nozzleman (old system) or at a separate accelerator pump. The water/cement ratio is fixed when the concrete is batched outside the tunnel (Thomas 2009a).

There is a trend to use the *wet mix* process for a number of reasons (Thomas 2009a):

- there is greater quality control, i.e. less human variability;
- higher outputs can be achieved compared to the *dry mix* process as in the wet mix process robotic spraying techniques are required due to the weight of the nozzle;
- less rebound of the sprayed concrete off the excavated surface;
- less dust;
- it is easy to keep records of the exact mix and quantities sprayed due to the use of ready-mix batches and robotic spraying.

This means that the wet mix process is suited to projects requiring large volumes of sprayed concrete at regular intervals. In terms of cost, there is very little difference between the two processes (Thomas 2009a).

Figures 4.18 and 4.19 show some examples of sprayed concrete application during the construction of the LT31 Tunnel in Vienna, Austria (see section 8.3 for further details).

Figure 4.18 Sprayed concrete application of the invert of a side wall drift during the construction of the Lainzer Tunnel LT31 in Vienna, Austria

Figure 4.19
a) Manual spraying and
b) Spraying equipment used during the construction of the Lainzer Tunnel LT31 in Vienna, Austria

Table 4.1 Comparison between single and double shell linings (after Legge 2006, from Sala 2001, used with permission from Alex Sala)

	Single shell	*Double shell*
Advantages	• Reduced lining thickness • Smaller excavated profile • Reduced total costs	• Inner lining installed well behind the face • Less effect on excavation process • Greater ability to control quality
Disadvantages	• Groundwater control required • Reduced watertightness compared with double shell linings • Groundwater is in direct contact with the permanent lining	• Blocking of waterproofing and build-up of water pressure behind lining • Location and repair of leaks when watertight membranes are used can be difficult (leaks within watertight concrete are easy to identify, i.e. they are visible, and hence easy to repair) • Additional cost of approx. 5–25%

Modern lining designs for sprayed concrete may not be finalized before construction, i.e. not 'fully engineered' at the detailed design stage, and they are refined during construction following the assessment of monitoring results (see the observational method in section 7.3.3).

Traditionally, sprayed concrete linings have been constructed using a 'double shell' or two pass lining approach. This involves a sacrificial primary lining being installed followed by a permanent secondary lining (Legge 2006). The function of the primary lining is purely to stabilize the tunnel following excavation and avoid loose material falling on the workforce; it generally has no long-term load carrying design function. The primary lining can have a long-term function if the groundwater is not aggressive to concrete and the inner lining has purely the function to keep the tunnel watertight. Double shell linings have advantages in poorer and variable ground conditions because the primary lining can be installed quickly and more time spent on creating the secondary lining. The alterative to the 'double shell' approach is the 'single shell' or one pass lining, which forms the final lining. A single shell lining is installed with the advancing face and the initial support required to stabilize the ground following excavation is an integral part of this final lining. It is therefore important that the quality of the installed lining material is higher than for the primary lining of a double shell approach. However, the single shell system can potentially reduce the overall cost and time. Table 4.1 shows some advantages and disadvantages of single and double shell systems.

It should be noted that sprayed concrete can also be used to construct shafts. Further details on sprayed concrete can be found in Thomas (2009a) and Franzen *et al.* (2001).

4.3.3 Ribbed systems

Support systems based on steel ribs have been used for many decades. This technique involves rolled steel sections being placed around the circumference of the excavated tunnel profile at specified intervals. It is inevitable that there will be gaps between the steel ribs and the ground and it is important that these gaps are suitably wedged to prevent excessive deformations. The importance of ensuring that the loads carried by the steel supports are evenly distributed around the tunnel profile is well recognized. Point loading of the steel supports significantly reduces their ultimate load bearing capacity.

It is common these days to combine steel ribs with sprayed concrete. If a layer of sprayed concrete is applied prior to erecting the steel ribs, this helps to overcome some of the problems with wedging. A subsequent layer of sprayed concrete is then applied to integrate the steel ribs into the lining and provide additional stability.

The legs of the steel arches are often set into concrete blocks to help distribute the loads into the ground and prevent settlement.

Lattice girders, rather than rolled steel sections, combined with sprayed concrete are also commonly used these days.

Figure 4.20 shows the mesh being installed ready for shotcreting, in combination with lattice girders. This two-lane road tunnel is the 2nd tube of the Katschberg Tunnel, Austria, and was constructed through hard rock (gneiss). It had a width of approximately 12 m, a length of 4300 m and a cross section of 88–111 m^2.

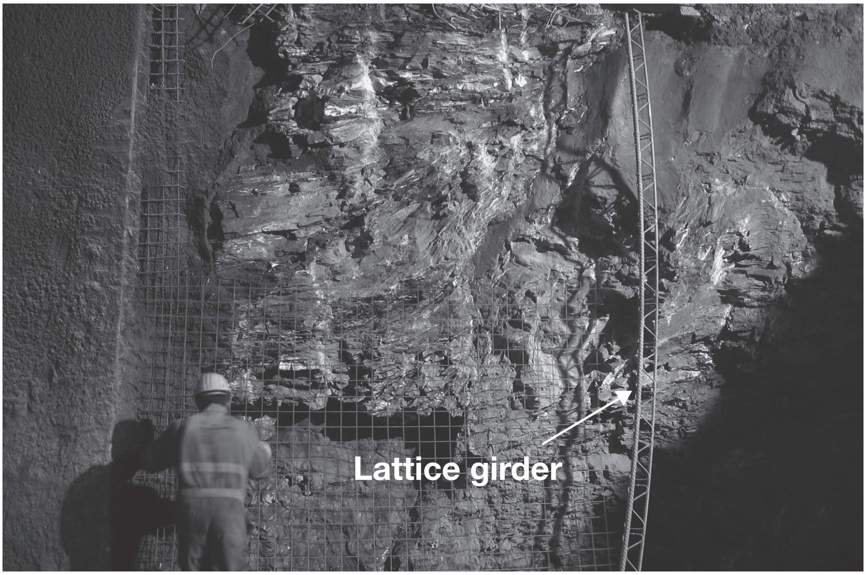


Figure 4.20 Mesh installation and a lattice girder arrangement

Figure 4.21
A load connection fitting for steel girders as used on the Gotthard Base Tunnel (Faido)

Figure 4.21 shows an example of a load connection fitting for steel girders.

4.3.4 Segmental linings

Segmental linings systems support the ground with a structure made up of a number of preformed interlocking structural elements. Together these elements form a continuous self supporting structure in the ground, which is most commonly circular in shape.

Although segmental linings are commonly used for soft ground conditions, the design principles are equally applicable to hard ground conditions. The permanent loading and the load developments with time are the main differences.

The design loads on segmental linings can be classed as either temporary or permanent. Temporary loads include demoulding, storage/stacking, transportation, handling, erection and grouting pressures. Permanent loads include external ground loads, external water pressure, imposed loads (traffic, adjacent foundation/pile loads), internal pressures (water pressure), external construction (adjacent tunnel construction) and flotation forces (King 2006).

Tunnels in soft ground are often designed to resist the full overburden of the ground and associated external water pressures. However, this is particularly conservative in stiff ground, such as overconsolidated clays. In overconsolidated soils it is often the ratio of horizontal to vertical stress (K_0) that is important (section 3.4), but this is difficult to assess as it is both time and construction related. It should be noted that the largest loads acting on the segments are often from the jacks moving the tunnelling shield or TBM, and hence this must be taken into account in the design.

Segmental linings may be connected together using bolts or dowels, or may have no physical connection, for example expanded linings. The choice of connections is closely related to the construction method for the tunnel (King 2006). The principle of expanded linings is shown in Figure 4.22. Figure 4.23 shows an example of an expanded segmental lining for one complete ring.

When using segmental linings with a tunnel boring machine (see section 5.5 on TBMs), the lining is erected in the tail of the TBM. An example of a segmental lining erector in the tail of an earth pressure balance machine (EPBM) (section 5.5.3.3) is shown in Figure 4.24. For a picture of a completed segmental lining, see Figure 5.31d.

Segmental tunnel linings are commonly made from unreinforced concrete, steel or fibre reinforced concrete, spheroidal graphite (cast) iron (SGI) (Figure 4.25), and steel. Table 4.2 indicates some of the main advantages and disadvantages of these different segmental lining types.

The durability of the segments is also a major design consideration. Clients these days require a 100 year design life or greater and this is in excess of commonly adopted building design codes (King 2006). It should be noted that this is the case for all tunnels regardless of their construction method. Maintaining tunnels is also costly and difficult due to limited access. It is therefore necessary to conduct a durability risk assessment to internal and external environments (see section 6.2 on risk assessment). The durability of segmental linings can be improved and the associated risks reduced in various ways. These include: avoiding cast-in metallic components in concrete linings, increasing the cover to reinforcement, minimizing the

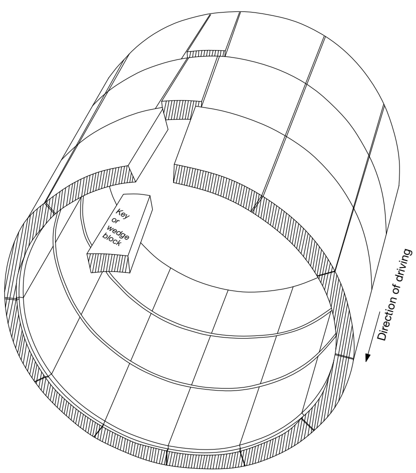


Figure 4.22
Principle of expanded wedge block segmental lining (after Whittaker and Frith 1990)

Figure 4.23
Example of an expanded segmental lining set for one complete ring

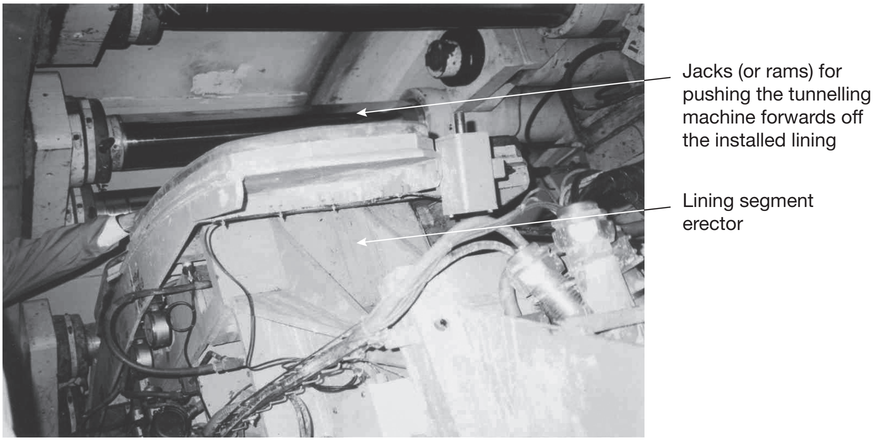


Figure 4.24 Segmental lining erector in the tail of an EPB tunnel boring machine (the photograph also shows the jacks for moving the tunnelling shield forward off the erected tunnel lining)

permeability and increasing the density (high cement contents are common in concrete lining segments), using coatings, removing reinforcement if at all possible and using unreinforced or fibre reinforced segments, and using cathodic protection (King 2006). It should be noted that steel fibres, when used as reinforcement, still have the potential to corrode.

Figure 4.25 Example of bolted SGI lining as used in London, UK (courtesy of ALPINE BeMo Tunnelling GmbH Innsbruck)

Fibre reinforced concrete segments were used on the Thames Tunnels as part of the Channel Tunnel Rail Link in the UK. Burgess and Davies (2007) state that the fibre-reinforced pre cast segments in this case made the manufacturing process easier as the space normally required for erecting the steel reinforcement cages was not required. The manufacturing process also employed steam curing which meant that the segments could be de-moulded after only 4 hours. Due to the 4.5 bar water pressure expected within the highly fissure chalk on this project, each segment had two rows of sealing

Table 4.2 Comparison of different materials used for segmental linings (after King 2006, used with permission)

Lining material	*Advantages*	*Disadvantages*
Unreinforced concrete	• Inexpensive compared to reinforced sections • Readily available • No corrosion concerns	• Low bending strengths • Low tensile bursting resistance • Low shear strength • Lead time for mould manufacture
Steel reinforced concrete (RC)	• High bending and tensile bursting resistance • High shear resistance • Readily available	• Expense of supply and fabrication of reinforcement cages • Corrosion considerations • Lead time for mould manufacture
Fibre reinforced concrete	• No major corrosion concerns • Moderate strengths • More ductile than unreinforced concrete	• Relatively new in tunnels and there have been difficulties getting the necessary approvals • Lower tensile and flexural capacity than RC • Lead time for mould manufacture
Spheroidal graphite cast iron (SGI)	• High tensile and compressive strength • Very high tolerance control • Lighter than equivalent concrete sections	• Expensive • Lead times for pattern manufacture • Repainting may be a possible maintenance requirement
Steel	• High tensile and compressive strength • Very high tolerance control • Lighter than equivalent concrete sections • Fabrication time shorter than SGI	• Expensive • Corrosion (repainting may be a possible maintenance requirement) • Mass production slower than SGI and tolerance control more labour intensive (except pressed steel – used for temporary works)

gaskets, one synthetic and one hydrophilic (see below). The tunnel boring machines used for the tunnelling on this project were slurry machines designed to be used in 'mix-shield' mode, whereby the face would be supported by a combination of pressurized slurry and a balancing air bubble. This had two advantages for the tunnel lining segments. The first was that the bentonite in the area behind the cutterhead (plenum) of the machine helped to reduce the jacking pressures on the segments and resulted in very little jacking damage. The other advantage was that the cutterhead torque was greatly reduced enabling a better control of the roll of the rings, thus avoiding any shearing failure of the bolts in the previously constructed ring.

WATERPROOFING

Waterproofing is important in tunnel lining construction to prevent excessive water flow into the tunnel. This is a particular problem if the tunnel is constructed below the groundwater table where it can act like a drain. Waterproofing of segmental linings has traditionally been by the use of caulking (applying a sealing material to the inside of the lining at the joints), but these days is generally achieved by the use of preformed gaskets. There are two basic forms (BTS/ICE 2004, King 2006):

- *compression seals* – these are manufactured from man-made rubbers (ethylene-propylene-diene monomer (EPDM) or neoprene) and are fitted around individual precast concrete or SGI segments;
- *hydrophilic seals* – these are made from specially impregnated rubbers or specially formulated bentonite compounds that swell on contact with water.

These waterproofing systems are not used for waterproofing the segments themselves, but to prevent water from penetrating between adjacent segments. The gaskets require a compression force to be applied to the lining as it is erected (compression seals more so than hydrophilic seals), which creates a line load on the segment that needs to be considered in the design.

TOLERANCES OF SEGMENTS

This needs careful consideration as they have practical implications for the constructability of the ring and performance of the gaskets. Herrenknecht and Bäppler (2003) recommend the following dimensions/tolerances:

- segment width ±0.6 mm;
- segment thickness ±3.0 mm;
- segment length ±0.8 mm;
- longitudinal joint evenness ±0.5 mm;
- ring joint evenness ±0.5 mm;
- cross-setting angle in longitudinal joints ±0.04°;
- angles of the longitudinal joint taper ±0.01°.

SEGMENTAL LINING RINGS USED FOR SHAFT CONSTRUCTION

Shafts can also be constructed using segmental linings. One method of construction commonly employed, if the ground can remain unsupported for a suitable amount of time, utilizes an 'underpinning' technique (Figure 4.26a). This involves excavation starting at the ground surface by an excavator (depending on the diameter of the shaft) lowered into the construction area (Figure 4.26b). Trimming of the sides can be carried out

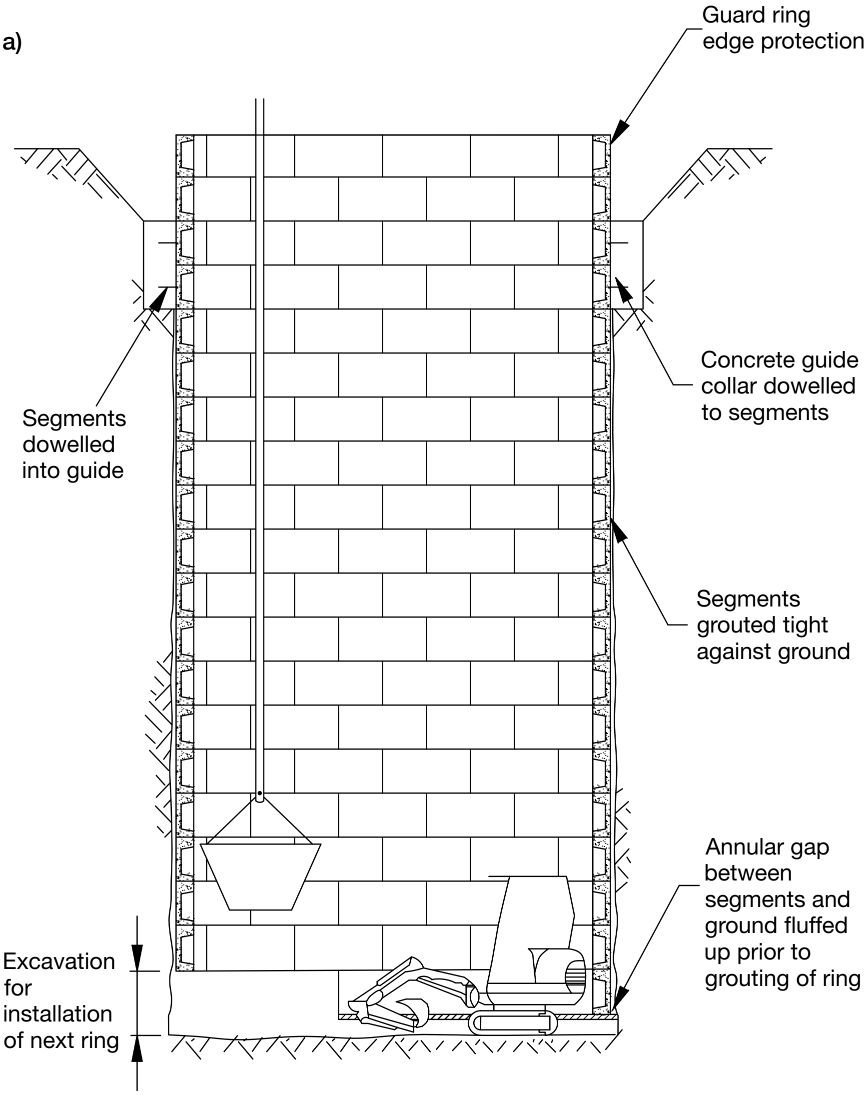


Figure 4.26 a) 'Underpinning' technique for constructing shafts (after BTS/ICE 2004, used with permission from Thomas Telford Ltd)

manually by using, for example, handheld clayspades (noting the health and safety issues associated with these devices, i.e. hand-arm vibration syndrome, see section 6.1 on health and safety). The excavation continues until a level is reached whereby a complete ring of segments can be installed, and the gap behind the lining is grouted immediately (Figure 4.26c). It should be noted that these segments may need to be installed by hand and so need to be of a manageable size and weight. The excavation is then

Figure 4.26 *(continued)* An example of the 'underpinning' technique used to construct a shaft, b) shows the excavation process, and c) shows the installation of the concrete segments

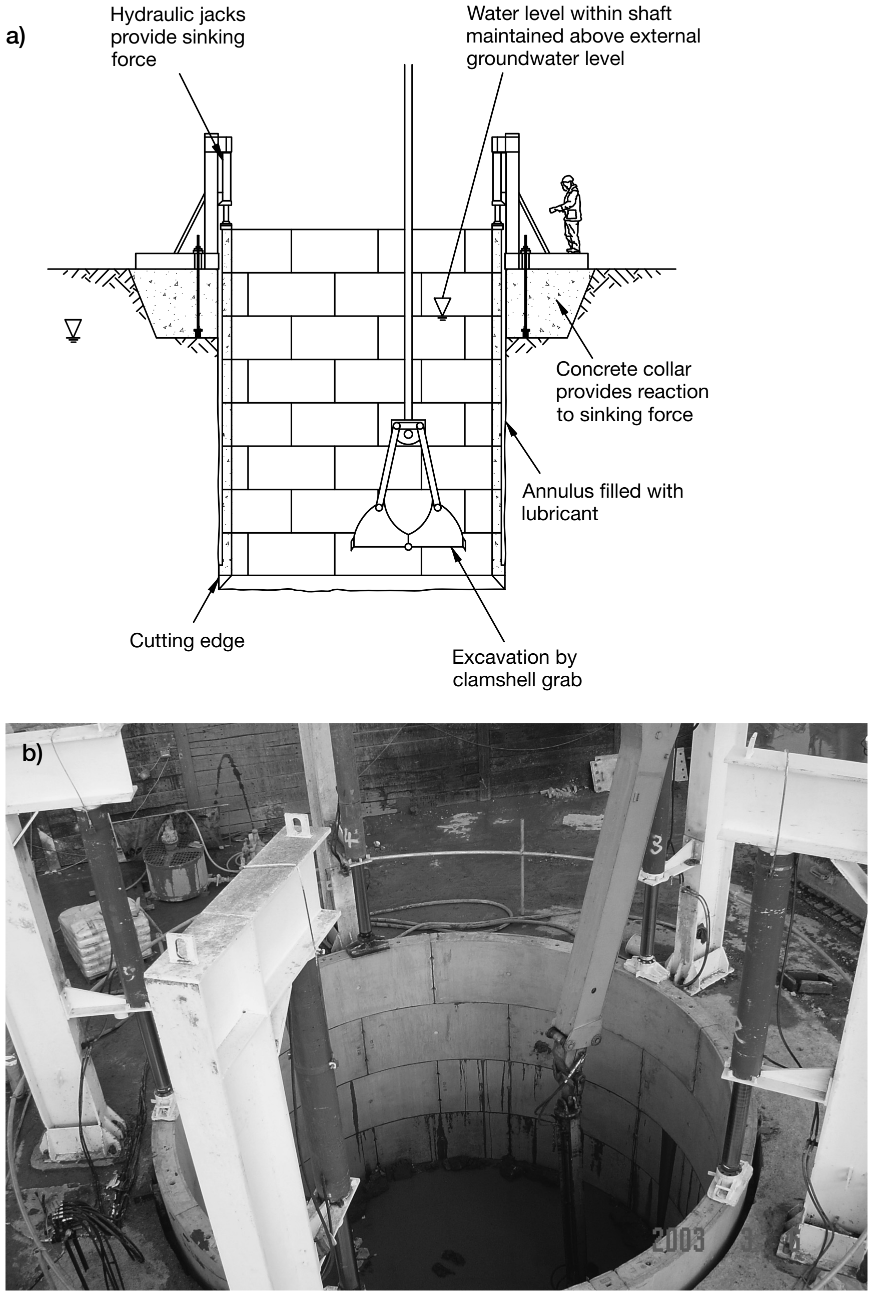


Figure 4.27 a) Caisson-sinking method to construct shafts (after BTS/ICE 2004, used with permission from Thomas Telford Ltd), b) shows the construction of a reception shaft for a pipe jacking operation as part of the Terminal 5 project at Heathrow, UK

continued until the next complete ring can be installed below the previous one. Further details of a deep shaft construction in London where this technique was used and the associated issues involved are discussed in Morrison *et al.* (2004).

An alternative technique which uses segmental linings is called the caisson-sinking method (Figure 4.27). This is generally used in ground where the stand-up time is poor or where base stability is of concern due to water pressure (BTS/ICE 2004). This technique uses a jacking process whereby a segmental lining is erected at the ground surface and then sunk using hydraulic jacks or weights (at the ground surface) to assist the self-weight of the shaft in order to overcome the ground friction. The excavation of the shaft may be conducted using an excavator within the shaft if the excavation is dry or from the ground surface using a grab on a crane in wet conditions. The friction on the outside of the shaft is reduced by using lubrication. Maintaining verticality of the shaft is critical during the sinking process as a small deviation from the vertical can cause an onerous loading condition.

4.3.5 **In situ** ***concrete linings***

In situ concrete linings can either be used in self supporting ground (for example rock) as the main lining, or as a second-stage lining where a temporary support system has already been placed during the excavation process (for example steel arches, sprayed concrete or rock bolting). In both cases the lining is cast *in situ* using a formwork system, which provides a gap between the ground, or initial support system, and the formwork into which wet concrete is placed (Figure 4.28a–c). The concrete lining can either be plain or reinforced. Once the concrete has reached a suitable early stage strength the formwork is 'struck', i.e. removed. In 'wet' ground, either a waterproof membrane is used between the initial support system and the cast *in situ* lining (Figure 4.29), or a watertight cast *in situ* lining can be used.

These linings are often used with 'system formwork' where travelling steel or wood forms are advanced, often as separate 'invert' and 'arch' forms, in tunnels with a suitable length of regular cross section and where the operation can be developed around a 24 hour cycle. Although expensive, 'system formwork' can become economically viable with extended use (Winter 2006). One benefit of using formwork is that it can be built as required to any shape and it is therefore highly adaptable. Thus it can be used in tunnels where there are junctions or tunnels of varying cross section.

Figures 4.28 and 4.29 show examples of formwork and falsework for the inner lining construction as used on the Heidkopf Tunnel, Germany (see Figure 4.9 for details of this tunnel).

These techniques can also be used for the construction of shafts. For example, a variation of the system formwork is 'slip form' lining system. In this case, the shaft is constructed and supported with the lining

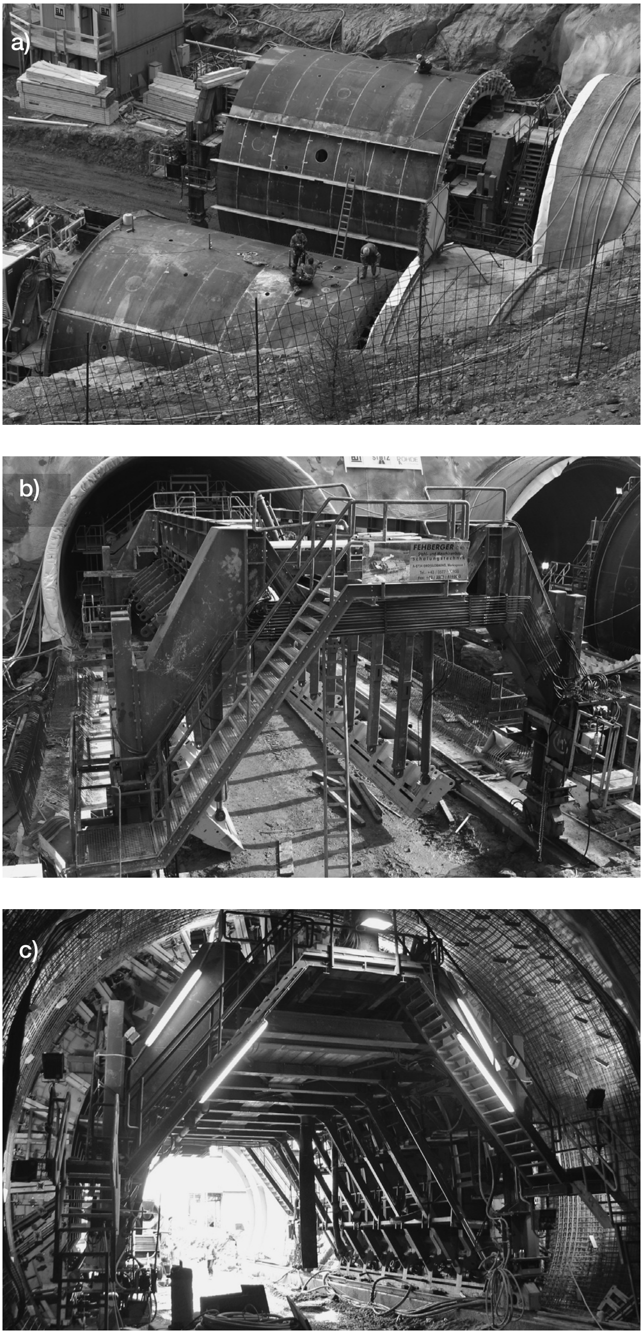


Figure 4.28 Construction of the inner lining, Heidkopf Tunnel, Germany, a) and b) setting up the shuttering at the tunnel portal, c) shuttering in the main tunnel

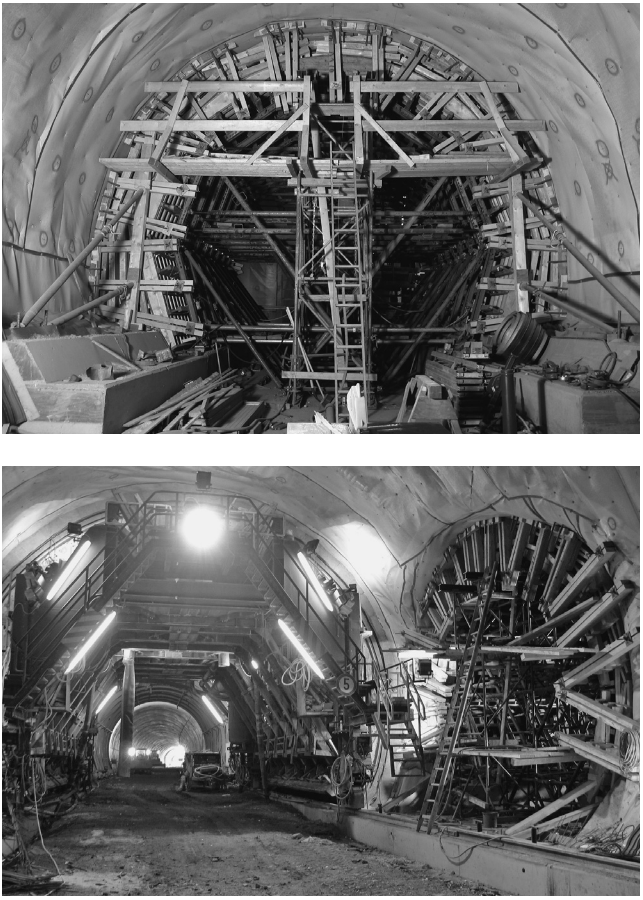


Figure 4.29 Inner lining construction, including waterproofing as used for the emergency cross-passage, Heidkopf Tunnel, Germany (courtesy of ALPINE BeMo Tunnelling GmbH Innsbruck)

construction starting at the bottom of the shaft and the formwork moving up continuously as the lining is cast *in situ*. The slip form technique can also be used in the tail of a tunnel boring machine. As the machine moves forward the concrete tunnel lining is formed continuously *in situ* behind the machine.

4.3.6 Fire resistance of concrete linings

Fire resistance of concrete linings is an important design criterion and the relevant standard for the design of structural linings should be used. In the UK the standard is Eurocode 2, BS EN 1992–01–02:2004 (BSI 2004b). In this standard there are at least three methods to determine the fire resistance of reinforced concrete members, including the '500 degree isotherm' method and the 'zone' method.

In all these methods the size and shape of the element together with the minimum thickness and cover to reinforcement influence the fire resistance. Allowance is also made for the moisture content of the concrete, the type of concrete and the aggregate used and whether any protection is provided (BTS/ICE 2004).

There are two basic options for fire protection of linings, either external or internal protection. External protection can be provided for relatively low temperature fires by the application of boarding or sprayed-applied coatings. Internal protection can be provided by adding polypropylene fibres to the concrete mix. In this case the polypropylene fibres melt and the resulting capillaries provide an escape path for moisture in the concrete, which can help to reduce spalling (Thomas 2009a). Further details on the fire resistance of concrete linings can be found in BTS/ICE (2004).

The importance of fire resistance was highlighted by the major lorry fire in the Mont Blanc road tunnel through the Alps in 1999. Although the fire caused significant damage to the tunnel, the immediate stability of the tunnel during the fire, which reached temperatures of up to 1000 °C, was not affected. In addition, three fires have so far occurred in the Channel Tunnel linking France and the UK in 1996, 2006 and 2008. All of these fires were caused by lorries catching fire on the heavy goods vehicle trains. All three caused damage to the tunnel lining meaning that repair and replacement of the damaged sections were required. The 2008 fire damaged 650 m of tunnel lining and cost approximately €60M to repair.