Civil Engineering for Underground Rail Transport

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Foreword

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Thirty-five years ago Bill Cassie (Professor William Fisher Cassie) used to tell his students in Newcastle that one of the tasks of civil engineers was to put services under the ground, including transport. The surface was for people on foot and, in future, the movement of vehicles would be underneath towns, as with the movement of water and sewage.

He was ahead of his time; he had written one of the first British textbooks on soil mechanics and created the first programme in traffic engineering in the UK. His vision has been a long time coming, in spite of London having built the first urban underground railway in the world. The spiralling cost of tunnelling has not helped.

However, as we enter a new decade and approach a new century we are surely entering a new era. There is a renewed enthusiasm for rail transport and, in spite of a parallel increase in interest in surface light rail, environmental pressures are changing the balance of advantage towards underground construction for urban as well as inter-urban systems. London Underground, which had seemed to have stopped building new lines, is again engaged in new developments. Much of the link between the Channel Tunnel and London will be under the ground.

British engineers have been at the forefront of new developments in recent years and *Civil Engineering for Underground Rail Transport* benefits from their experience not only in London and on the Channel Tunnel but also in Hong Kong, Singapore and Taipei. Of course, they are not alone. Several Continental European experts played a part there, and their contributions also cover experience in their own countries, including in Budapest, which followed London by building the first Continental urban underground railway more than one hundred years ago.

This book is about civil engineering, but civil engineering with a very definite and unique purpose – underground rail transport. The contributions reflect that uniqueness.

Preface

The editor wishes to record his gratitude to the authors of each chapter. Many of them are leaders in their specialist fields and were very busy during 1988/89 when tunnelling and underground construction has been booming throughout the world: at the same time they have had to write their chapters. To all of them the editor has been, and the reader will be, grateful for setting out their expertise with such apparent enthusiasm.

The editor is most appreciative of Dr Tony Ridley for writing the foreword. Dr Ridley is a civil engineer who has been intimately involved in the construction of underground rail works for much of his professional life, first with the metro systems of Newcastle-upon-Tyne, England and Hong Kong and latterly with Eurotunnel as Managing Director–Project. Before this post he was Managing Director of London Underground. As a leader in UITP, the International Union for Public Transport, he is very much aware of the importance of underground works in the development of metros and main-line railways.

Finally the editor thanks all those who have agreed to allow the authors of chapters to quote from their own relevant works and so to widen the sources of information available to the reader.

Glossary

Note: This glossary is not comprehensive but defines words used by specialists in the UK, which may not be familiar to other English speakers.

ACI American Concrete Institute

BSI British Standards Institution

Back grouting or *contact grouting* Filling of voids immediately after placing the lining.

Competence of rock A measure of its capacity to resist deformation under load.

Formal linings Tunnel linings of preformed metal or concrete. *See* Chapters 13 and 14.

Formwork Has replaced 'shuttering' to describe timber or metal surfaces which retain wet concrete to form a desired shape.

Mass Transit Railway Defined in BSI Specification 8100 as a 'railway for the rapid movement of high passenger load densities in urban areas'. The predominant form of Mass Rapid Transit, that is 'Mass' up to 80 000/h/direction, 'Rapid' up to 35 kph.

Heavy Rapid Transit – Metro 25 000–80 000 h and 35 kph.

Light Rapid Transit – LRT Up to 25 000 h and 25 kph, mainly at grade and not separated from other traffic.

Pre-metro Mainly separated LRT with elevated or underground sections.

Pack or *packer* A wooden or more usually steel plate, but occasionally a concrete block used to fill a gap. In grouting, a packer is inserted and expanded to seal the hole for injecting grout only below the level of the packer. *See,* in Chapter 4, the 'tube à la manchette', a special type of packer.

Plant, machinery, equipment Words, each of which may be applied to the same item by different people. But the words are not interchangeable in all cases; for example, *machinery* contains moving parts but *equipment* may not.

SPT Standard Penetration Test. The number of blows of a specified weight falling through a specified distance onto a special rod resting on the soil to produce a specified penetration.

Step plate junction Where two tunnels, lined with plates of different diameters, meet special vertical plates are required to close the vertical faces – so forming a step. Often *in-situ* concrete is used instead of plates to avoid the heavy cost and time delay in making special plates.

Tubbing Used rarely in the UK, although in common English use elsewhere, to describe the usually circular metal or pre-cast concrete plates to line tunnels or shafts. The equivalent word used in the UK is *segment*.

Chapter 1

Introduction

Jack Edwards

Contents

1.1 The growing need for underground rail transport

The building of railways underground is being resumed on a large scale for both main-line railways and urban mass transit systems. The first category has been enlarged by the demand for high-speed railways and the second has grown with the renewed interest in light rail systems, often in cities that once had streetcars or tramways as well as conventional metro systems.

New high-speed lines are being planned or built in many countries, including Japan, Taiwan, Australia, the United States, West Germany, France and the United Kingdom. The experience of the early Tokaido Line in Japan showed that, despite care being taken to minimize noise, high-speed trains had an unpopular environmental impact. The same view appeared recently in England. Putting the traffic underground in areas near houses is likely to be a growing feature of high-speed lines, even through the construction cost is much higher.

Completion of the monumental Seikan Tunnel in Japan and commencement of construction of the Channel Tunnel between France and England have further stimulated interest in underground rail transport.

Many new light rail systems are being planned and built along disused railtracks, central reservations of roads and corridors that have been allocated and kept on city plans for such purposes. It is often the case that, although such routes can be run radially outside the city centre, surface travel into and across the centre on reserved areas becomes impracticable for environmental reasons, or perhaps financial ones if the area is densely built-up. In some places modern light rail cars are being operated along streets in city centres where all other wheeled traffic is prohibited. That cannot always be done, particularly in old towns with narrow streets having many changes in direction, and even if such constraints are not so severe as to make it impossible, they will make the service slower, which will detract from its popularity and also worsen rather than relieve road traffic congestion. It is in those locations and where city centres have been built on hills that the light rail system must run underground. The need for low-cost construction of underground lines that are inherently much more expensive than those on the surface makes the use of good planning and economical construction methods essential. This book outlines such methods.

1.2 How underground systems are justified

The first step towards building an underground section of railway should be the provision of funds by an institution (often government, city or transport authorities) for a study to determine whether the building of such a railway is justified. Justification may be on financial, economic, heavy travel demand or whatever grounds the authorizing institution decides is appropriate. Such feasibility studies take into account all relevant topics such as demand, environmental aspects and cost. It may be preceded by a pre-feasibility study undertaken, for example, as part of a national transport study that has identified a corridor along which sufficient passengers are likely to travel but has not evaluated other aspects. Without the benefit of a feasibility study, a railway may be of the wrong type, or in the wrong place, and may generate public dissatisfaction and an unnecessarily heavy financial burden.

The feasibility study team should include a transport economist, a transport planner, an urban planner, a railway planning engineer, a civil engineer, an economist, a financial analyst and, depending on the location, a range of specialists such as architects, archeologists, landscape architects, environmental impact analysts, mechanical engineers, electrical engineers and others. The objective of the team will be to recommend a route that, on balance, has the most advantages and the least disadvantages. The team needs, therefore, experienced staff to assess the relative merits of features not obviously comparable (for example, environment, costs, convenience and economic benefit). Above all, the team must have an experienced leader who can ensure that all factors have been identified and the assessment of each has been done to achieve a proper balance and weighting. The study report should include, as well as the recommendation for the route, the parameters (for example, track gauge, radius, gradients, vehicle size and speed) on which the location is based.

The outcome of the study should be the location of a route and stations with a financial and economic assessment on which the controlling authority can decide whether to proceed with its construction. Following a decision to proceed, it is most desirable (but rarely achieved) that detailed design and construction should be quickly approved. City development is not usually slow and never precisely to a detailed plan. Rural land-use plans can equally be changed without warning. Delays always lead to increased cost and usually to changes in railway design.

The importance of a sound feasibility study and even a specific pre-feasibility study has long been stressed by the World Bank and other lending agencies as a requisite for the evaluation of a potential urban mass transit system. This policy has developed a need for an internationally recognized methodology for such studies. The British Overseas Development Administration has funded a study of existing systems to assess to what extent they had relieved urban transport problems. The study required the development of a mathematical model to evaluate the effectiveness of the systems and their economic worth. That work has, in turn, led to the development of software for a Metro Appraisal Guide which can be used in a pre-feasibility study for a new system. The development of this software was a feature in the Seminar on Rail Mass Transit for Developing Countries on 9–10 October 1989 in London. The Guide will enable transport planners and other specialists to produce a report likely to be acceptable by the standards of the lending agencies, which should enable the authority to decide whether to proceed further with the implementation of the scheme. Details of the Report and Guide are given in the Further Reading at the end of this chapter.

Chapters 2 and 3 describe the planning of new routes and the location of stations after the feasibility study stage. The need for the railway planning engineer to continue to work with the urban planner and the transport planner must be emphasized. A new line will have a considerable influence on the subsequent urban development along its route. The urban planner must therefore be involved directly in the railway planning. The transport planner will have to calculate likely traffic flows along the route and at each station using computer-based methods. Both planners should participate in the study of alternatives and selection of the optimum route and station locations. Even though such work may have been done at the feasibility study stage, it will need to be continued in more detail in the design stage. Similarly, the design parameters should be reviewed and expanded. Disused railway routes and stations may be in locations no longer suitable even if they were originally in the best places. The temptation to follow an old route, when its chief merit is low cost, may have influenced too much the decision in the feasibility study and such a decision should be reviewed in the design stage. Some relevant books and papers for further information on these topics are listed in the Further Reading. It should be borne in mind when reading Chapters 2 and 3 and these references that most of the features of design and construction of underground railways for mass transit are also applicable to suburban and main-line railways.

1.3 Civil engineering development

The techniques of civil engineering have changed significantly in the last fifty years and appreciably in the last twenty. No doubt change will contine as new electrical and mechanical developments (comparable with hydraulic rams and electronic controls in recent years) are applied to fundamental methods. Also, of course, entirely new techniques (for example, ultra-high-pressure water jets for excavation) will be developed. Advances in chemistry and physics have also brought great benefits: resins, concrete additives and ground-treatment methods are examples. Such changes will continue in both tunnelling and other methods of construction. While the purpose of all developments is fundamentally to achieve cheaper or safer methods, some may have such a significant effect as to make underground construction financially viable in circumstances where previously it was not. This change may also be brought about in combination with other factors (for example, a large increase in the price of land).

1.4 Readership and scope of this book

Main-line railway tunnels will almost invariably be built by railway administrations having departments dealing with all other aspects of railway construction and operation. Tunnels for metro and light rail lines may be needed for existing systems by administrations not having such wide experience and specialist engineering staff. Both types of railway staff will find a large amount of information about underground construction in this volume.

The readers are assumed to be graduate engineers with some experience but none in specifically underground works. The book may also be interesting to undergraduates wishing to gain an idea of the type of work in this field. As a reference book, quantity surveyors, geologists, railway administrators, railway operators, lawyers and arbitrators should find particular aspects relevant to their work.

1.5 Other topics

Some light rail and metro systems will be new, and engineers becoming involved in the planning and design may look for help on aspects other than the underground works. Ground-level and elevated rail tracks will be required together with stations, offices, control rooms, depots and workshops. Information on associated electrical and mechanical equipment affecting the civil engineering work, and an outline of organizational and operational methods that greatly influence building and civil engineering provisions, will be subjects about which the engineer will need to know. It will also be helpful for the engineer to have at an early stage an understanding of some of the ways in which light rail systems can be financed and contracts for their construction arranged. These matters, which are beyond the scope of this book, will be the subject of a later companion volume.

Other subjects are related closely to underground construction. Two examples are safety and fire protection. These subjects are of primary importance and related both during construction and in the permanent works. The construction of underground works is potentially hazardous and must always be controlled by a person experienced in the type of work at each site and supervised independently by another experienced person. Nevertheless, the impact on and needs of each method of construction differ, so that they are best dealt with in the relevant chapters. The chapters dealing with stations refer to the effects of safety requirements and fire protection on the permanent works.

The comments in Section 13.1.9 (Procedure and contractual basis), although made specifically about tunnel contracts are relevant to the other civil engineering contracts where the works to be constructed are influenced by soil conditions. Contract types, conditions of contract, finance, organization and management of construction are typical of other aspects of underground construction that are important. Some relevant books and papers are listed in the Further Reading at the end of this chapter.

1.6 Arrangement of this book

The book covers the planning, design and construction of underground structures. Many of the chapters are related to several others, so that at least some related chapters are kept near to each other. The chapters are arranged as follows:

1. Planning
2. Ground treatment
3. Cut and cover (open-cut) construction
4. Driven tunnels
5. Stations

Ground treatment must be considered when planning the route and station locations as well as in the selection of construction methods, so it is placed between the chapters on planning and construction. Station design follows the chapters on tunnels since it is influenced by the method of construction used. Other chapters on details influencing station design follow.

'Rail Transport' is included in the title because 'railways' may be interpreted as solely steel wheels on steel rails. In practice, the support and guidance method, especially on metro and light rail systems, may also be rubber wheels on concrete track, magnetic levitation, air cushion or some combination of systems. Most of this book will be applicable to whatever method is used to support and guide the vehicle. Chapter 5 describes the different track types.

Ground treatment is covered in Chapter 4 and some topics related to ground treatment are in other chapters. Micro-piles, which may be used to form a continuous row making a water cut-off, are described in Chapter 8. A slurry trench or wall used for the same purpose is dealt with in Chapter 7.

Section 6.1 describes the process and techniques necessary for good design, and much of this is applicable to methods of construction other than reinforced concrete.

Loadbearing piles, referred to in Chapter 6, are formed using plant and methods similar to those described in Chapter 8.

The need to monitor existing buildings when underground works are constructed adjacent to or underneath them is stressed in Chapter 3. Methods of doing so and their importance are also dealt with in Chapters 7 and 8.

Chapter 13 includes the selection of the appropriate type of tunnel, but all the other chapters on tunnels need to be read to obtain an appreciation of the complex factors influencing the selection of the correct type of construction method and the appropriate lining.

The construction of stations by cut and cover methods is described in Chapter 6 and in driven (bored) tunnels in Chapter 20. The facilities to be provided within the station together with an indication of their space requirements are given in Chapter 21. It should be noted that this book does not describe how the facilities within the station are built, since conventional reinforced concrete, masonry (including brick and concrete blocks) and structural steelwork are used. Chapter 22 must be read with Chapter 21 to obtain a full appreciation of what has to be fitted within the station shell. The layout of the station may be influenced considerably if the civil defence requirements described in Chapter 23 have to be incorporated. In planning the detailed layout of a station all the chapters mentioned in this paragraph need to be studied as well as the principles in Chapter 3.

1.7 Costs

Costs are indicated in some chapters but, as circumstances differ between countries, the figures must be understood as only an approximate guide to the level in 1989. For comparison between currencies, nominal exchange rates may be used of:

£1 = US $1.70
= French francs 10.20
= West German deutschmarks 3.30
= Japanese yen 230

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Chapter 2

Civil engineering aspects of route planning

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Contents

2.1 General

2.1.1 The civil engineer's role

In planning the route for an underground railway the civil engineer will not be confined simply to setting an alignment for the tunnels between the traffic objectives. There will be a comprehensive plan which will include consideration of alternative alignments, both horizontal and vertical, station layouts, the most suitable type of construction, a study of ground conditions and obstructions, working sites, the type of track form, rolling stock depots and equipment generally, as well as cost estimates and an outline construction and equipping programme.

2.1.2 The civil engineer and the traffic planner

The civil engineer will become involved in the route planning when the traffic planner has broadly identified the objectives and the corridors that need to be served together with an assessment of the potential traffic usage of the route. It is also necessary for the traffic planner to identify station locations and interchange points with other lines or modes of transport.

There is likely to be a need for a continuing dialogue between the traffic planner and the civil engineer to establish a satisfactory compromise between the ideal solution in terms of fulfilment of traffic objectives and what can be provided at an acceptable cost and within an acceptable time scale.

2.2 The initial steps

2.2.1 Horizontal alignment

The first objective will be to make the horizontal alignment as direct as possible, thereby giving the least length of construction necessary to meet the traffic requirements. A direct alignment will best meet the objective of efficient passenger transportation, permit highest train speeds and avoid problems related to severity of curvature.

In heavily built-up areas the most direct alignment is often, in practice, not achievable, even for a railway in bored tunnels. This is because of the prevalence in modern cities of tall buildings on deep foundations and also the likelihood of existing tunnels (including underground railways). The straightest and most direct practicable route should still be sought, even though this may involve considering many differing alignments. The viability of an underground railway depends on its ability to attract traffic in competition with other forms of passenger transport. The advantages in terms of speed and directness must therefore be exploited to the full, and this means that devious and highly curved routes should be avoided.

2.2.2 Vertical alignment

The second objective will be to make the new railway as near to the surface as possible. This is primarily because access to the system for passengers at stations will be more convenient and less expensive in capital cost and also in running and maintenance costs for such facilities as escalators or lifts.

There are five vertical horizons in siting a railway:

1. Elevated railway
2. Tracks at grade
3. Open cutting
4. Cut and cover construction
5. Bored tunnel

Typical urban railways may contain lengths in any or all five horizons. Horizons 1–3 are beyond the scope of this book, but the traffic planner and civil engineer need nevertheless to be fully aware of them as options because of their advantage in lower construction cost albeit, perhaps, at some environmental disadvantage.

2.2.3 Cut and cover construction

This is the term used for the provision of a shallow structure formed by first making an excavation from the surface in which the box or structure is then built. Backfilling and final reinstatement of ground surface then follows. There are many different variations in both the sequence of construction and the materials and techniques that can be used. Although there will be exceptions, this type of construction will generally result in the level of the rails being between 4 and 10 m below ground level. The structure formed can conveniently accommodate at least two tracks.

This form of construction has several advantages. Being shallow, access for passengers between ground level and platforms is relatively easy and, if initial cost is of most importance, fixed staircases rather than escalators could be provided. Cut and cover construction for both running tunnels and stations is generally cheaper than providing the same facilities in bored tunnels and also avoids the need for specialist tunnelling labour and other resources, which are often in short supply. Being near the surface, it is usually easier to provide for ventilation without having to construct large, deep shafts. Some underground railways rely on the train movements to circulate air and simply provide short open sections between lengths of cut and cover construction with no mechanical ventilation necessary. There is generally a requirement for emergency exits from tunnels, and in cut and cover construction these involve less building work and give easier access to the surface.

However, there are also disadvantages with this method. There will usually be extensive surface disruption along the route during construction, with buildings requiring either demolition or complex temporary supports unless the railway is built under a highway or open ground. If under a highway there will almost certainly need to be diversions of sewers and drains, gas and water mains, together with electrical and telecommunication cables. With a shallow railway there is more likely to be problems with the transmission of noise and vibration from trains to the neighbouring properties than with those operating in deeper bored tunnels.

Cut and cover construction need not necessarily be confined to existing highway routes but can also be linked to building new highways. Also if the construction of the railway coincides with redevelopment its structure can often be used to support the above-ground development.

2.2.4 Bored tunnels

Bored tunnels are structures constructed beneath ground level and only require occupation of the surface of the ground at working sites situated along the line of the route. In heavily congested built-up areas it is therefore a very convenient way of constructing an underground railway. There will be greater flexibility in planning the line of route because obstructions to a direct alignment will probably be limited to tall building foundations, sewers and any other existing tunnels. Access to the traffic is usually easier and stations can be provided close to the locations that passengers require. The only need for surface disruption will be at station locations and at working sites, with the need to demolish buildings reduced to a minimum.

However, tunnels are expensive compared with construction at or near ground level. Station works are particularly expensive because of the need for larger-diameter station tunnels, cross passages, concourses and lift or escalator shafts to provide means of access from street level to station platforms. It will also be necessary to provide for ventilation plant, escalators, lifts (especially if provision is to be made for disabled passengers) and pumping machinery.

A further problem with tunnelling is ground settlement, which will always occur to a greater or lesser extent when driven tunnels are constructed. Larger-diameter tunnels, which are sometimes formed by initially building a smaller tunnel and then enlarging it perhaps in more than one stage, will cause greater settlement, as the effects of each tunnelling operation are additive.

Waterproofing of running tunnels is not usually necessary although large quantities of water can cause problems, particularly with electrical equipment and track circuits used in the signalling systems. However water ingress at stations is undesirable because of the effect on the architectural finishings, which can quickly deteriorate and become shabby. It can be difficult to prevent water entering the tunnels through joints in the linings although modern grouting techniques are often used successfully. Waterproofing of cut and cover structures is much easier because waterproof membranes, applied to the external face of the structure, can usually be accommodated in the designs.

2.3 Surveys

The civil engineer will need to arrange a number of surveys and these can be broadly summarized as follows:

1. A topographical survey
2. A soil survey
3. A survey of ground obstructions

2.3.1 Topographical survey

The topographical survey will be used for initial planning of the track alignment, and a convenient scale for this work is 1:1250. In Britain plans to this scale are generally available from the Ordnance Survey. At a later stage these plans would need to be updated for use in authorization of the line, in Britain generally through a Parliamentary Bill. A convenient way of doing this is by carrying out an aerial survey and using this in the first place to correct the standard Ordnance Survey sheets.

An added advantage of this method is that station layout scheme drawings need to be prepared to a larger scale than the route plans, 1:500 generally being a convenient size. These larger-scale plans can readily be plotted from the aerial survey. A further advantage is that, ultimately, contract drawings and documents for the invitation of tenders for the construction works will need to be prepared to a similar scale, and again this can be done using data obtained from the aerial survey.

In other countries where these are not commonly used scales it will be necessary to prepare plans for the planning of the track alignment to a scale which enables individual parcels of land to be identified. For the planning of station works, a scale which permits the width of passageways, platforms, staircases, etc. to be accurately represented should be used.

2.3.2 Soil survey

The soil survey will be necessary to determine the type of ground through which the railway will be constructed. This will normally be done in two stages, the first being a desk study of existing soils information during the early stages of route planning. Sources such as major construction works in the vicinity, developments, wells, etc. can be a useful source of soil information. Local authorities may also be helpful together with geological surveys or local museums.

The second stage will consist of sinking a series of boreholes along the line of the route, from which samples of the soil will be recovered. These samples will be subjected to various tests in the laboratory to determine the soil properties. The spacing of the boreholes will depend on expected uniformity of the strata to be encountered. However, the problems that have occurred in tunnelling work due to insufficient soils data would seem to indicate the tendency in many projects to underestimate the number of boreholes that should be made. A more detailed reference to soil surveys is included in Chapter 4.

2.3.3 Ground obstructions

A knowledge of existing structures, services, pipes or mains along the route is essential, and this will need to be put together by the civil engineer after consultation with local authorities and the authorities for sewers, drainage, gas, water, electricity and telecommunications, as well as those responsible for historical archives. Shallow obstructions such as pipes, mains and cables are usually much more significant when cut and cover construction is being contemplated, whereas deep-level driven tunnels are more usually affected by sewers, other tunnels and obstructions such as deep piles.

It will also be necessary to enquire about future proposals of the planning authority and the authorities for sewers, drainage, gas, water, electricity supply and telecommunications. This will enable provision to be made for future development or construction of new facilities after the railway has been completed and running. It might be worth considering incorporating ducts for cables or foundations for future buildings into the railway construction works, as these can often be formed very easily at the time of building the railway but with much greater difficulty and possible disruption to the operation of the railway if left until later.

The construction of deep foundations close to or over driven tunnels can cause distortion and unacceptable stresses in the tunnel linings and needs careful consideration.

2.3.4 Ground conditions

The ground conditions, obtained from the soil survey, will govern the types of construction to be used and therefore the cost. The presence or otherwise of groundwater is particularly important, whether the construction is by cut and cover methods or by driven tunnel.

Driven tunnels are easier to construct in cohesive soils and rocks while those in granular soils usually require special techniques which can be expensive. In more recent years slurry shields have been developed which greatly assist the construction of tunnels in water-bearing granular soils which previously might have required the adoption of working under high air pressure. Where tunnel depth is not governed by other constraints (e.g. between stations) the civil engineer will usually attempt to site the tunnels within the most suitable soil strata.

2.4 Curvature and gradients

The two fundamental constraints in determining the alignment of a railway are the minimum track curvature and the maximum gradient. The absolute values of these will depend on the characteristics of the rolling stock to be employed.

2.4.1 Curvature and speed

Curvature is important because it determines the value of any speed limitations, and therefore in addition to there being an absolute minimum radius of track curvature there will also be a desirable minimum radius depending on the speed at which it is proposed to operate the trains. The relationship between the maximum permissible speed and the radius of curvature is:

$$V_m = 11.27 \sqrt{\frac{R(E+D)}{S}} \tag{2.1}$$

where V_m = maximum permissible speed (km/h),
R = radius of curvature (m),
E = actual cant (mm),
D = maximum allowable cant deficiency (mm), and
S = distance between the centres of the rails (mm).

For 1432 mm standard gauge of track (and allowing for a rail head width in the order of 70 mm) the above relationship becomes

$$V_m = 0.29 \sqrt{[R(E+D)]} \tag{2.2}$$

Since train speeds near to stations will necessarily be lower than on the mid-station runs, a lower value for desirable minimum radius of curvature may be more appropriate on the approaches and exits from stations than on the rest of the line. For higher-speed lines the maximum operating speed may be constrained not only by curvature but by the need to limit the speed of air movments in the tunnels and stations to an acceptable level, notwithstanding the provisions that will have been made for draught relief.

An early decision will need to be made on where the specification for the railway is to be placed on the spectrum extending from 'light rapid transit' through 'heavy

Table 2.1 Characteristics for underground railways

	Light rail	*Metro*	*Suburban rail/ regional metro*	*Main-line railway*
Maximum speed (km/h)	55–90	75–100	80–130	100–200
Cars per train	1–4	2–10	4–10	8–16
Length of train (m)	15–80	30–150	80–260	150–300
Station spacing (m)	300–800	500–2500	1500+	2000+
Minimum radius of curvature (m)				
Desirable	150–400	250–500	300–1000	500–2000
Absolute	15–60	50–100	60–120	75–200
Maximum gradient (%)				
Desirable	2–4	1–2	1–2	1–2
Absolute	5–6	2–4	2–4	2–4

rapid transit' to 'main-line railway'. The decision will be based partly on the passenger-carrying requirements, partly on the capital likely to be available for construction and partly on engineering considerations. Table 2.1 illustrates the range of characteristics which are typical for the various classes in underground rail transport.

2.4.2 Limiting values

Adoption of the specification for the rolling stock and for the line will determine the limiting values for curvature and gradient. There are, of course, likely to be locations where values for radius of curvature between the desirable and absolute minimum have to be adopted, but these should be avoided wherever possible, because it will be at the expense of permanent speed restrictions, possible noise problems and, quite probably, continuing higher maintenance costs for both track and rolling stock. Similarly, gradient values between the desirable and absolute maximum should be avoided if possible because of lower operating speeds and the increased risk of maintenance problems and train failures.

Within the station itself the track should be straight so as to avoid a gap between the platform edge and the floor of the car. In cases of difficulty, a very slight curvature may be introduced, but for sighting purposes the curvature of the platform edge should be convex, not concave.

2.4.3 Gradients

Track through stations should preferably be level and certainly with a track gradient not steeper than about 0.2% (but dependent on the characteristics of the rolling stock), to avoid the possibility of a train starting to move through accidental release of the brakes. Stations should preferably be sited on a hump with an upward gradient approaching the station and a downward gradient on exit. This profile assists with braking and deceleration on the approach and acceleration on leaving the station. There are also considerable energy savings in adopting this type of profile. For a heavy rapid transit line a typical gradient each side of the station would be 2% over a length of about 150–200 m.

2.5 Sensitive areas

Many underground railways will pass either close to or even under sensitive areas or buildings such as cathedrals, churches, monuments, theatres, etc. The civil engineer will try to keep the alignment as far away as possible from such sites, but this is not always feasible. The two points of concern are ground settlement and transmission of noise and vibration.

2.5.1 Settlement

Settlement is caused by the construction of the tunnels and its magnitude depends on the type and speed of construction and ground conditions. The effects of settlement are also dependent on the type of building or structure and its foundations. Predicting the amount of settlement is not easy, but there are several ways of keeping it to a minimum but not eliminating it. Settlement caused by the construction of underground railways is not generally a major problem, provided proper techniques and precautions are employed in the execution of the works.

2.5.2 Noise and vibration

The transmission of noise and vibration from the tunnels to the buildings in the vicinity is usually only a local problem – generally associated with a theatre, a building housing sensitive scientific instruments, etc. There are a number of techniques that have been successfully used to reduce vibration transmission, all of which require a special type of construction, usually locally near the sensitive area.

2.6 Working sites

2.6.1 Running tunnels

An important part of the early planning is the choice of working sites from which the railway can be constructed. With cut and cover type construction the whole length will be a working site. However, techniques such as the construction of only the side walls and roof before reinstating the ground (usually a roadway) can enable occupation to be limited perhaps to only a few weeks on any one part of the route. This method requires sites at intervals where excavation can continue below the completed roof and proceed as if it were a tunnelling project.

With a driven tunnel working sites will be required to enable shafts to be sunk to the depth at which tunnelling can commence. The sites need to be large enough to enable the excavated spoil to be handled and the construction materials such as tunnel linings to be lowered down the shaft and transported to the face. Space will be needed to store materials and perhaps some spoil if restrictions on its transportation and disposal are expected and will also need to be provided for servicing the works, which includes maintenance of plant used in construction and accommodation for the workforce.

The civil engineer will always try to avoid the need for temporary works which have no use in the finished project. The first choice for working sites would therefore be directly on the alignment of the tunnels, thereby avoiding temporary connecting passages. Also, if a working shaft can ultimately be used for other purposes, such as tunnel ventilation and/or emergency exit, further savings can be made.

Working sites should have good road access and preferably should be open spaces, thereby avoiding the need to demolish buildings. Public open spaces are often convenient and can be attractively landscaped on completion of the works before being handed back to the local authority.

2.6.2 Station works

It is often convenient to have separate working sites for the main tunnel drives and for the station works. The station working sites will be needed for construction of escalator shafts, low-level concourses and passageways, and will probably be used as a base for much of the equipping of the stations.

2.7 Running tunnels

The dimensions of the tunnels are of fundamental importance and will depend on many factors.

Figure 2.1 A typical covered way in concrete construction. The side walls are formed with continuous piles and the roof with precast beams. Dimension between side walls is approximately 7.3 m

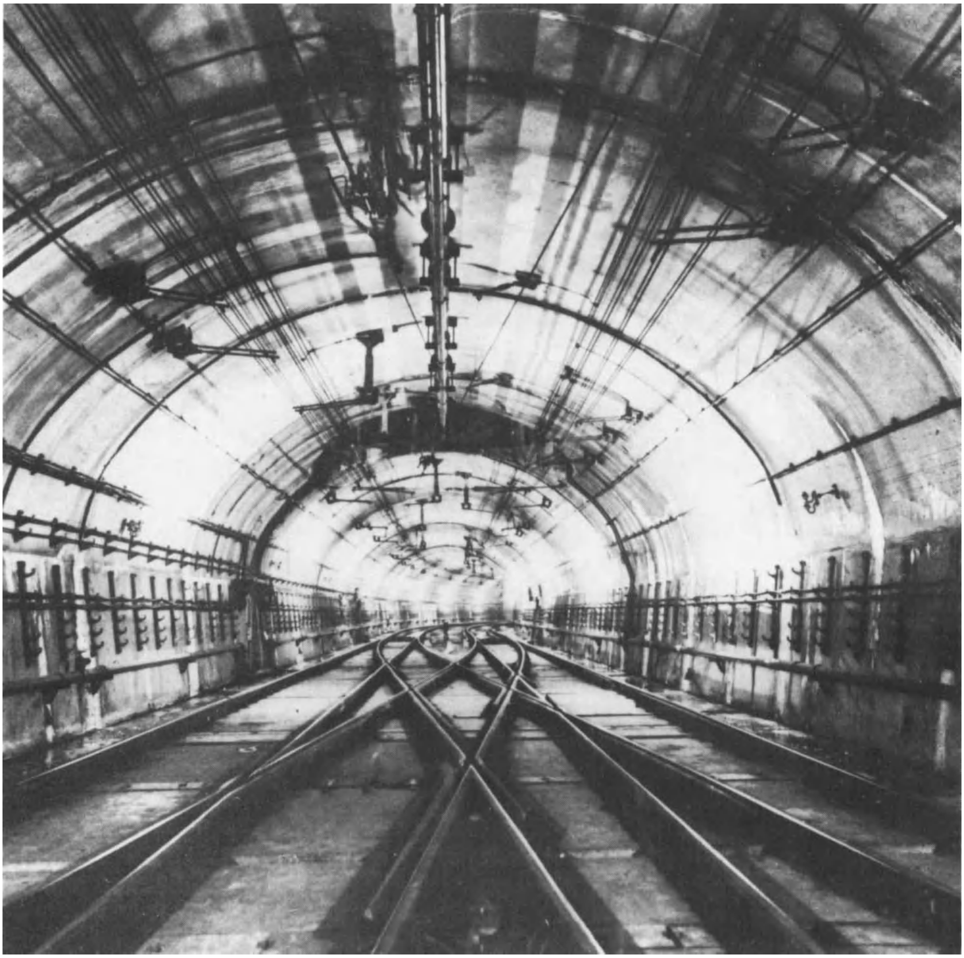

Figure 2.2 A typical double-track tunnel driven through rock with an *in-situ* concrete lining. Enough space is provided for overhead current collection and the track is concreted in and not ballasted

2.7.1 Dimensions for cut and cover tunnels

With cut and cover type construction it is normal to have two tracks within the same structure. The internal dimension between the side walls will then depend on the clearance required between the rolling stock and the wall.

Maintenance work in tunnels is usually carried out during the night, when trains are not running, and therefore the clearance between rolling stock and walls need not make provision for a person to stand while a train passes. Cables will normally be carried on the side walls and a minimum clearance can be provided between the cables and the rolling stock. If the track is supported on ballast and not concreted into the invert an extra allowance has to be made for movement of the track under traffic. A further consideration for side clearances is whether or not a side walkway is to be provided for detraining passengers in an emergency situation. Some railways prefer to detrain passengers through the ends of trains with centre doors provided at the ends of each car and use the track as the emergency walkway. This

Figure 2.3 A running tunnel constructed with bolted cast-iron linings. Tunnel diameter is 3.8 m, suitable for small rolling stock

is usually only feasible with concreted track, when a reasonable surface can be provided for passengers to walk along. A covered way of internal width of some 7–8 m would generally provide space for two tracks on ballast but without side walkways (Figure 2.1).

The internal height of the covered way above rail level will depend on the size of rolling stock and whether overhead or current rail electrification is provided. This might vary from about 3 m for small rolling stock with current rails to about 5 m for large rolling stock with overhead electrification (Figure 2.2).

2.7.2 Dimension for bored tunnels

Driven tunnels are most commonly of circular cross section and generally it is economical to have a separate tunnel for each track. Clearance considerations are the same as for covered ways. The circular section has advantages in that it is widest at the centre line, where cables are usually carried and where side walkway space would be required if provided for. There are also disadvantages in that as the diameter of the tunnel increases, more space is formed below track level which is generally of little use. A tunnel of about 4 m internal diameter would be suitable for small rolling stock with current rails, track concreted into the invert and no side walkways (Figure 2.3), whereas about 6 m internal diameter would permit large rolling stock with overhead electrification and side walkways. It will generally be found that provision of a circular tunnel for overhead electrification will automatically create enough space for side walkways.

Where junctions are required the diameter of the running tunnel will have to be increased in stages until it is possible to construct two normal-size tunnels to meet the largest diameter in the junction. These structures are called 'step plate junctions' (Figure 2.4). Where a connection is required between the two running lines a constant-diameter crossover tunnel can be provided which would be large enough to accommodate the two through-tracks side by side and the connection between them. With large rolling stock which requires bigger running tunnels, or if overhead electrification is provided, the diameter of a crossover tunnel may be prohibitively large. In this case the crossover would be formed with two step plate junctions, or four if a scissors crossover is required.

To avoid the creation of unwanted space below the tracks large-diameter tunnels can be constructed either with a flat bottom or with an elliptical cross section, both of which considerably reduces the amount of excavation necessary.

2.7.3 Tunnel shields and linings

The relationship between the tunnel shield or tunnelling machine and the tunnel lining is important; the external diameter of the lining being the critical dimension. In soft ground the linings can either be expanded against the surface cut by the

Figure 2.4 A step plate junction accommodating a simple turnout formed with bolted cast iron linings. The largest diameter (in the foreground) is 9 m and the smallest 3.8 m

Figure 2.5 A running tunnel, 3.8 m diameter, constructed with expanded concrete linings. The wedges for expanding the rings can be seen on each side

shield or machine or they can be bolted together, leaving a small gap between the outside of the lining and the excavated surface which then has to be filled with grout. Expanded linings are normally adopted when the ground conditions are good and the excavated surface will retain its shape for sufficient time to allow the ring of linings to be expanded out and support the ground. Bolted and grouted linings will be necessary where the ground conditions are poor. Tunnels constructed with expanded linings can usually be built much more quickly than those using bolted linings and hence are cheaper (Figure 2.5). Careful design can permit both expanded and bolted linings to be used with the same shield and thus take advantage of using the most appropriate methods in variable ground conditions without interruption to the progress of the work.

2.7.4 Dimensions for other tunnels

The dimensions of station tunnels will depend on the length of trains to be used and the width of platforms required. Similarly, the size of escalator shafts will depend on the number of machines to be provided. The sizes of other tunnels will vary according to estimated passenger flows, emergency evacuation requirements and ventilation considerations. The capacity of passages and stairways in stations is dealt with in some detail in Chapter 3.

2.8 Terminal facilities

This is an important consideration, and will depend on the intensity of train service to be handled. If two platforms are sufficient then simple scissors crossovers may be adequate (Figure 2.6). If three or more platforms are required the layout will be more complicated.

With cut and cover construction the layout is less important because the cost of providing space for crossovers, etc. is less significant. However, with driven tunnels the cost of providing a large-diameter crossover tunnel is significant, and if a third platform is needed the additional track connections will require step plate junctions to be provided.

An alternative is a terminal loop, where trains run continuously rather than reverse. A considerable length of plain running tunnel can be constructed for the cost of a crossover tunnel. Consideration should also be given to the possible need for future extension of the railway before deciding on the terminal arrangements.

Figure 2.6 A 9.5 m diameter cast-iron lined tunnel providing space for a scissors crossover at a terminal station. The longitudinal high-level gantry carries the cables through the single tunnel. The secondary lining in the soffit directs seepage water to the sides where it is led away to the drainage system

2.9 Drainage

2.9.1 Gravity drains and pumping

It will be necessary to provide drainage facilities throughout the railway. The most convenient method is to install gravity drains in the tunnel invert running to the low points where sumps would be provided. This will require the tunnels to be constructed on gradients of not less than about 0.25%. The sumps would normally be equipped with automatic pumps which would discharge the water to public sewers or drains usually via an adjacent station or ventilation shaft.

The drains and sumps need to be designed to deal with seepage through the tunnel linings and any other source of water. Sump capacity should be large enough to avoid the need to service pumps in the event of a failure during the traffic day. A capacity of, say, 48 hours would generally be adequate.

2.9.2 Flooding

Some cities are subject to the risk of flooding, either from storms or from rivers overflowing their banks. In these circumstances care needs to be taken at the planning stage to ensure as far as possible that large quantities of water cannot get into the tunnels. Considerable damage can be caused to equipment leading to disruption and even suspension of train services, and clearing up after flooding is an expensive and lengthy operation.

Chapter 3 deals with the precautions that can be taken at stations, but care must also be taken at ventilation and draught-relief shafts to site openings high enough to ensure that they are above flood level. At tunnel portals it may be necessary to make special provisions such as watertight doors or bund walls around the approaches to the tunnel. Flooding from such occurrences as a burst water main can often be limited by quite simple precautions at the detailed design stage.

2.10 Ventilation

Ventilation is an important consideration as far as the comfort and safety of passengers and staff is concerned. It has even more significance in countries which experience very hot or humid weather conditions, and this will generally determine whether or not air conditioning of trains and stations should be provided.

It may be that the running of trains through the tunnels will cause sufficient air movement and circulation. However, heat is generated in the tunnels by the conversion of electrical energy, and it will probably be necessary to extract this heat. This would normally be done by providing ventilation shafts midway between stations connected to fresh air above ground level and equipped with mechanical fans. These should be arranged so that they can be switched to either extract the air from the tunnels or introduce fresh air from the atmosphere into the tunnels.

The purpose of providing fans which can operate in both directions is to assist with the control and removal of smoke from the tunnels in the event of a fire or smouldering below ground. Great care needs to be exercised in operating the fans in the most suitable mode in such circumstances.

Ventilation shafts sited between stations can also be utilized for the evacuation of passengers from tunnels in the case of a derailment or other emergency by providing suitable staircases. They are also a means of access to the tunnels for the emergency services should an incident occur requiring their attendance.

The running of trains through the tunnels causes a piston effect and considerable quantities of air will be moved both in front of and behind them. Consideration needs to be given to keeping the air velocity to a comfortable level in stations by the provision of draught-relief shafts which connect the tunnels to fresh air above ground level. Where there is more than one line at a station, connecting tunnels may be necessary to balance the air pressures.

The quantity of air induced into the stations from the running tunnels can be considerably reduced if cross passages are provided between the twin running tunnels at regular intervals. The distance between these passages will be governed by the length of trains being operated. They also provide a useful means of access for maintenance staff requiring to pass from one tunnel to the adjacent one. It will be necessary to provide ventilation to staff accommodation sited below ground and possibly also to plant rooms, particularly escalator machine rooms.

2.11 Rolling stock depots

An important requirement for an underground railway, or indeed any railway, is the provision of depot facilities where repairs and day-to-day maintenance of the rolling stock can be carried out. It is usually convenient to combine this facility with stabling accommodation where trains can be left when not required for service. Some railways site their operational control centre at the depot.

The size of the depot will be governed by the number of trains on the railway, usually with an extra one or two sidings for engineers' trains which would be used for general maintenance work. The depot would be provided with some sidings having access pits between the rails and also with lifting facilities, where cars could easily be separated from their bogies. This would be done either by providing overhead cranes or by jacking. In the depot, trains would be inspected and any routine maintenance works carried out together with routine component changing. Accommodation for the workforce and a material stores would usually be provided.

These facilities would normally be sited in covered accommodation, and it is desirable also to cover the stabling sidings. This is particularly important in countries which experience cold weather with ice and snow. A further advantage of covering the stabling sidings is the additional security that can be provided compared with trains left in sidings in the open. Another precaution against the effects of cold weather is the provision of point heaters on track laid in the open. It would be unfortunate if a railway built totally underground, and therefore protected from the weather, were temporarily immobilized through frozen train equipment and snowbound points within a depot on the surface.

The layout of the depot should be such that trains either leaving it for service or entering it after being taken out of service can pass through the train-washing machine, which would ensure that the exteriors of the trains are washed at least daily. Internal cleaning and sweeping out of the cars would be carried out in the stabling sidings. Depots should ideally be double-ended, so that in the event of a derailment or other incident at the commencement of traffic which blocks one entrance it will still be possible to use the other entrance to provide the service on the railway.

A typical depot for housing 20 eight-car trains would occupy a space of about 3–5 ha. For planning purposes an area of 250 m^2 per car could be used.

Depending on the length of the railway, depot facilities may be provided at one or both ends of the line. On longer lines, a depot situated some distance short of the end of the line may be preferable for operating purposes. The cost would be prohibitive to site these facilities in a bored tunnel but, on completion of depot works constructed on the surface or in open cut, it may be feasible to raft over to enable development to take place, thereby making use of the land taken for the railway works.

2.12 Costs

2.12.1 General

Costs associated with an underground railway fall into three categories. First, there is the initial capital cost of construction, second, the ongoing maintenance and running costs and, third, the renewal costs when the assets reach the end of their useful life. These three areas will be considered below, although running costs will be excluded, as they are outside the scope of this book. All the costs quoted are at 1988 price levels and are in pounds sterling.

It must be emphasized that conditions can vary considerably from one country to another and between cities within the same country, and the costs quoted should be seen in that light. Nevertheless, they should give an order of magnitude for those requiring such information.

2.12.2 Capital

The most important factor affecting the cost of an underground railway is the method of construction. Cut and cover methods can potentially be the cheapest, but this will depend largely on the cost of moving pipes, mains, cables and sewers and any expenses associated with creating the space such as traffic diversions, etc. As an example, a 4 km railway extension in London was built in the 1970s by cut and cover methods. The cost of services and sewer diversions was about 25% of the total cost of the railway, but nevertheless it was still cheaper to construct using this method. Another cost advantage of cut and cover construction is that crossovers and junctions can be provided relatively cheaply compared with the same facilities in driven tunnels.

A further advantage is that station works are usually simpler, and even if escalators are considered necessary to transport passengers to and from the platforms, they can usually be installed in the space already provided by the cut and cover method. With driven tunnels separate shafts need to be constructed, with enlargements for machine chambers at both the top and the bottom. In some circumstances a possible compromise could be shallow-bored tunnels and cut and cover stations.

Of course, there will be many locations where cut and cover construction is out of the question and therefore driven tunnels will be essential. The cost of tunnelling is dependent more on the method to be used rather than on the depth of the tunnels. The method will largely be governed by the ground conditions.

In tunnel construction there will be an initial cost of sinking shafts and providing tunnelling shields or machines. There will then be the cost of driving the tunnels, which will depend on tunnel diameter and speed of construction. Tunnels driven in good cohesive soils, suitable for expanded linings, can be constructed much more quickly than those requiring the use of bolted linings and hence are much cheaper. Linings will normally be made from concrete or some form of cast iron. Again, concrete is usually cheaper but is not always suitable for the particular conditions. In more recent years ductile iron linings have been developed, and these can be economical, particularly in the larger diameters.

Running tunnels
The cost of driven tunnels per kilometre of route (i.e. twin tunnels with a single track in each) would range from about £6 million to £45 million. The lower figure would be appropriate for small-diameter tunnels driven in good cohesive ground and using expanded concrete linings, while the higher figure would include larger-diameter tunnels driven in poor ground requiring special tunnelling techniques. These costs are for civil engineering only and do not include those of land.

The cost of cut and cover tunnels (i.e. a single structure containing two tracks) would lie between £5 million and £12 million per kilometre. The lower figure would be for construction in good ground above the water table while the higher one would allow for water-bearing ground with substantial temporary works and services diversions. Again, these costs are for civil engineering only and do not allow for land acquisition.

Stations
The cost of stations is more complicated, and will depend on depth, number of lines, length of platforms, position of ticket hall and, of course, ground conditions. For a typical station constructed in driven tunnels with two tracks about 20 m deep, platforms 140 m long and a subsurface ticket hall with escalators the cost would be between £18 million and £45 million, according to ground conditions. These figures are for civil engineering only and again do not allow for any land acquisition. For a similar station but shallow and constructed by cut and cover methods the cost would lie between £6 million and £15 million.

Other costs
To enable a comparison to be made, the cost per kilometer of a surface railway would be some £2 million to £4 million. Similarly, for an elevated railway the cost might range from £5 million to £10 million per kilometre.

The cost of equipping the tunnels with track, signalling and electrical supplies will add about £2 million to £4 million per kilometre of route (i.e. two tracks) to the previously mentioned figures. Equipping stations, including architectural finishings, might range from £1 million or less for a simple surface station to £15 million for a deep-level station with escalators. A rolling stock depot for 30 six-car trains, fully equipped and constructed on the surface, could cost between £20 million and £40 million, excluding land.

2.12.3 Maintenance

Tunnels require little maintenance. Inspections should be carried out at intervals of about twelve years for cast iron-lined tunnels and about four years for others, and any damage to or deformation of the linings or structures should be noted.

Particular attention should be paid to ingress of water. It may be necessary to repair caulking or to take precautions such as grouting if water ingress is considered to be unacceptable. Such inspection and maintenance might cost about £1500–8000 per kilometre per annum, depending on the age of the structures.

The drainage system needs to be maintained, with the sumps being cleaned out regularly. The frequency will depend on experience gained on the rate of build-up of silt.

The most expensive item will be the maintenance of the permanent way. The track geometry will need to be adjusted as wear takes place and regular attention to fastenings will be necessary. It is normal practice to inspect the track at regular intervals, depending on the volume of traffic. This might vary from every day to once a week, and will normally be done at night during non-traffic hours.

Assuming an inspection frequency of once every 48 h, with work carried out at night and an annual traffic of some 15–20 million tonnes, then the annual inspection and maintenance cost per kilometre of route (i.e. two tracks) would be in the range of £9000–14 000 for track concreted into the tunnel invert and about £13 000–20 000 for track laid in ballast.

A further maintenance consideration is cleaning the tunnels. Dust will collect in the tunnels from the passengers, train brakes, wear of the rails and train wheels and from external sources via stations and tunnel portals. This dust tends to be flammable and can ignite from a spark. It is therefore necessary to keep the tunnels clean, and this can either be done manually or by mechanical means such as a tunnel-cleaning train. Experience of the rate of build-up of dust will determine the cleaning frequency, Normally this would be not less than annually.

2.12.4 Renewal

The life of the assets will vary. The tunnel structure can be expected to have at least 100 years of life. Some brick-lined tunnels in London are now 125 years old and are still performing adequately, with little sign of deterioration and very low maintenance costs. The drainage system, if maintained adequately, should also have a long life, although pumping equipment might need renewing at about 25-year intervals.

The life of the track components will vary according to the amount of traffic carried. In general, rails should have a life of at least 30 years and hardwood or concrete sleepers perhaps 50 years. If the track is poorly maintained or laid on sharp curvature these lives could be much less.

Assuming that track has an average life of about 30 years the annual cost of renewal of ballasted track would be between £11 000 and £17 000 per kilometre. For track concreted into the tunnel invert the annual renewal cost would be between £22 000 and £30 000 per kilometre. These figures can vary considerably, according to local conditions and practice, but do give an indication of the magnitude of the expenditure. They assume that the renewal work will be carried out outside normal traffic hours (generally at night) without substantial interruption to normal train services.

2.13 Construction materials

The consequences of fire below ground is likely to be much more serious than above ground, and it is therefore essential that care be taken in selecting materials to be used in the construction and equipment of the railway. Wherever possible, metal, concrete or other inert, non-organic materials should be used. Any suspect materials should be thoroughly tested and advice sought from fire experts.

2.14 Programme

2.14.1 Construction time and critical activities

The preparation of a programme for the construction of an underground railway is an important part of the planning process, and will be carried out by civil engineers using their knowledge of construction methods and techniques. It is possible that the railway is being built specifically for a special function or event, and will be required to commence operating on a particular date. Such a time constraint could very well govern the type of construction adopted and adds emphasis to the need for early programming considerations.

With cut and cover construction the time required for diverting underground services is likely to be significant, and detailed discussions will need to be held with the authorities responsible for sewers, drains, gas and water mains, electricity and telecommunication cables to determine how the works will best be carried out. Telecommunications services in particular can be very time consuming to alter. It may be possible to do some of this work in advance, perhaps while the detailed civil engineering design is being completed, so that when the main work commences on site much of the diversions will have been done.

With a railway constructed in deep-level bored tunnels it is usually the station works that take the longest to complete, and these should therefore be planned to commence as early as possible. Driving of the running tunnels is rarely a critical time factor, although full consideration should be given to the availability of both shields or tunnelling machines and tunnel linings.

Another part of the works that is often critical as far as construction time is concerned is the rolling stock depot, and again early commencement of these facilities is advisable.

2.14.2 Equipping

In preparing the programme the civil engineer will need to allow time for the equipping of the tunnels and stations. Installation of trackwork will normally proceed as soon as the tunnels are sufficiently far advanced, followed by the signalling system and the running of cables through the tunnels for lighting, power supplies, etc. Equipping of the stations is also a time-consuming activity and should be integrated as far as possible with the construction works. Architectural finishings to floors, walls and ceilings will follow close behind the civil engineering works, and it is advisable to commence the work on escalators or lifts as early as possible, as the installation and testing time for these items can be considerable. It is usually possible to undertake such works as staff accommodation, ventilation plant installation, lighting, signing, public-address systems and ticket-issuing and control facilities concurrently with the escalator works.

Chapter 3

Civil engineering aspects of station location and design

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Contents

3.1 Location

3.1.1 The dilemma

Designers of underground railways face a major dilemma when deciding where to locate the stations. On the one hand, they wish to site them so that they are convenient for the intending passengers, and this means that the preferred locations are in the most densely developed areas of the city. It is here that the largest numbers of passengers are available, and it is by serving these areas that the underground railway can make its greatest contribution to ease congestion on the city streets.

On the other hand, by choosing sites which maximize the catchment area for the stations, the designers are inevitably increasing the engineering problems of constructing the railway. The shortage of space in the centres of large cities has necessitated the construction of multistorey buildings with deep foundations and the proliferation of underground cables and pipes carrying the essential public utilities in the roadways. Except in the few cases of truly modern cities, the roadways themselves are at their narrowest in the central area.

This, then, is the problem faced by designers: they must build stations in the most difficult and congested locations.

3.1.2 Passenger access

Favoured positions for stations are beneath surface-level junctions where they can benefit from the pedestrian routes radiating from the intersection. If entrances are sited in each quadrant of a typical four-way junction, passengers do not have to cross road-traffic streams when entering or leaving the station.

It is also beneficial to locate stations close to large office, shopping or residential developments where easy access by considerable numbers of people will encourage high ridership on the railway (Figure 3.1). Even better accessibility can be achieved if stations and new developments are designed and built as an integrated whole. When correctly planned, this produces attractively short access to the station with protection from any inclement weather conditions. However, these advantages will only be fully realized if there is good cooperation between the railway authority and the developer in dealing with the technical, programming and legal interfaces between the railway and the development.

3.1.3 Transport interchange

In a medium-sized or large city no single means of transport can meet the needs of all the population in all areas of the city. Each mode has a role to play, and therefore the overall efficiency of the total transport system is improved if there are convenient interchange points between the modes (for example, between bus services and the railway, and at major carparks).

In addition to improving the service for the general public, collaboration between the different forms of urban transport is mutually beneficial to each of the cooperating modes. For this reason, it is good policy to site underground railway stations close to existing transport interchange facilities, such as major bus stations. If redevelopment or reconstruction is possible then an integrated design can be produced to give the maximum convenience to the travelling public.

Figure 3.1 An entrance to an underground station incorporated into a high-quality commercial building. The corner site allows two entries to be provided; one on each of the two roads

3.1.4 Utilities

City streets and footways are frequently used as convenient routes for a wide range of services such as:

Water supply;
Drainage;
Gas;
Electrical power; and
Telecommunications

Even the smallest engineering works are complicated by the presence of these utilities, and they must be protected or diverted so that the services are maintained during construction.

Building an underground railway station is a major task, involving substantial disruption to underground services, particularly if the construction is by the cut and cover method. Less disturbance is caused if the station is formed by driven tunnels and the only works constructed at surface level are passenger entrances and ventilation shafts.

The accurate identification of all utilities at a station site is a time-consuming activity, but one which must be rigorously undertaken to avoid later costly emergency works and delays. Records kept by utility companies do not necessarily have the dimensional detail needed by the designer of the underground railway, and site surveys are essential.

In some cases the existence of a telecommunications cable may not be acknowledged by the government authorities because of its importance for national security. The designer should be aware of this possibility, and be sensitive to any guidance provided by the authorities.

The time and cost of diverting any existing utilities are factors that must be taken into account when deciding on station locations, but it is impossible to avoid underground services entirely. Of those that are encountered, the most difficult to alter are large sewers and drains operating under gravity.

Altering the invert level of a drain by a significant amount may have far-reaching consequences. Raising the level decreases the upstream gradient and capacity, resulting in a slower flow which might encourage sedimentation. Reducing the level may require extensive lowering downstream so that a falling gradient is maintained; while the diversion of a drain round a station site will inevitably result in a longer route and therefore a reduced gradient.

These factors will not always cause major problems at every site where there are existing large drains, and the difficulties of maintaining gravity flow will always be easier in areas where the natural slope of the ground is significant. However, when severe problems are encountered it may be necessary to install pumps or inverted syphons to bypass the new obstacle created by the underground station. Flushing arrangements may also be needed to clean drains that have been relaid to a shallow gradient.

3.1.5 Method of construction

The disruptive effect of cut and cover construction has been mentioned. Access to the whole ground surface of the station site is necessary and this effectively limits the location of stations to roads, open spaces such as parks, or occupied sites which are due for demolition and redevelopment.

If a station is to be formed by tunnelling there is less restriction on possible sites, but there is not complete freedom of choice. In general, the construction of tunnels will affect the overlying strata so as to cause subsidence of the ground surface. The extent of the disturbance depends on the nature of the ground and the method of tunnelling; while the consequential effect of subsidence is determined by the nature of the buildings and their foundations. Deeper tunnelling causes less ground-level subsidence than shallow tunnelling, and therefore allows greater choice for the location of stations. The principal disadvantage of deep stations is their greater cost and the inconvenience to passengers of the extra time needed to reach the trains.

3.2 Layout and main dimensions

3.2.1 Principal elements

A typical underground station has three main elements:

1. The platform area;
2. The concourse containing ticket-issuing machines and access control; and
3. Offices and equipment rooms not open to the passengers.

These are normally arranged in two levels with the platforms at the lower level and the concourse above. Offices and equipment rooms are usually accommodated at

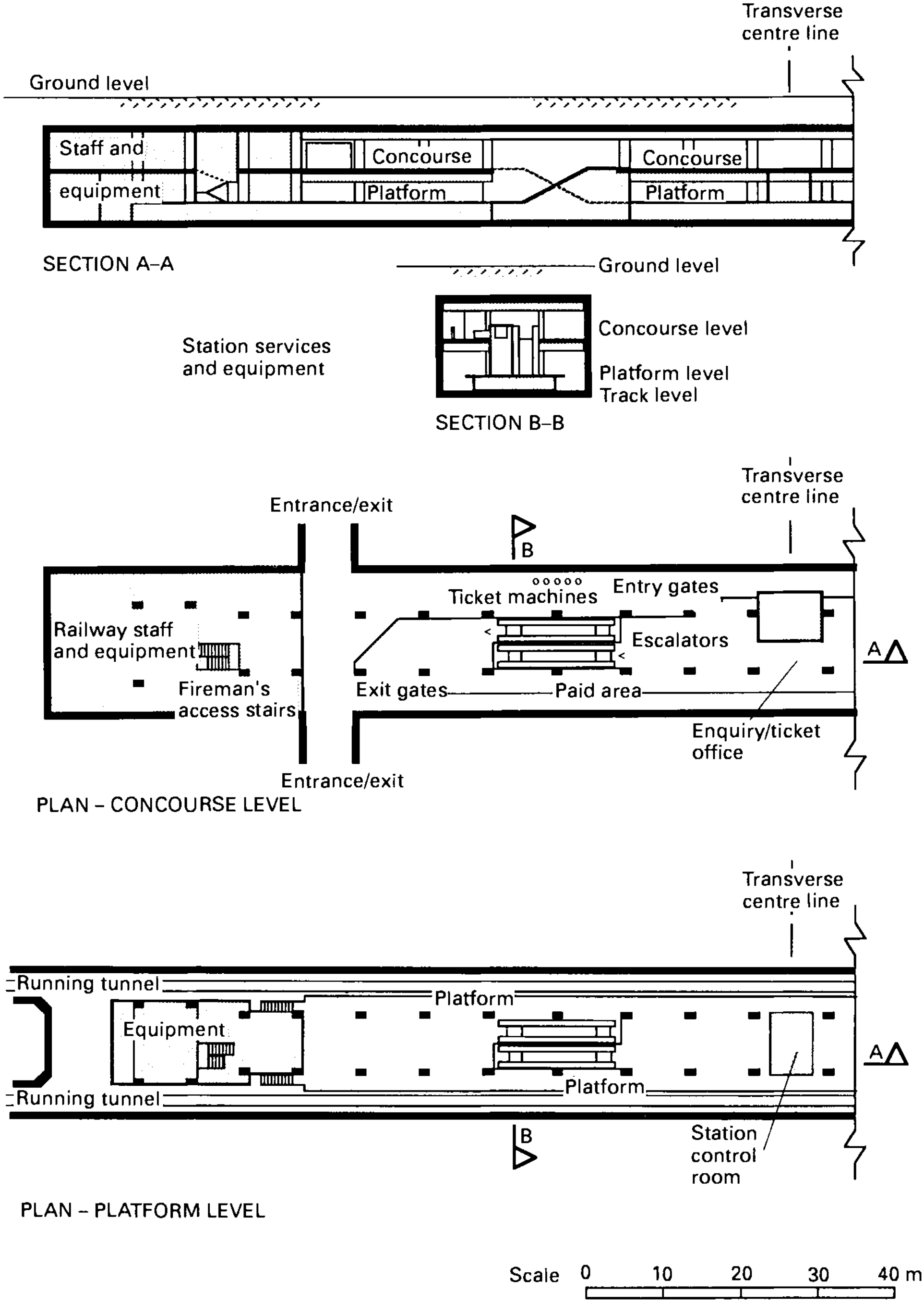

Figure 3.2 Plan and sections of an island platform station constructed by the cut and cover method. The passenger concourse containing ticket machines and entry gates is located at the first level below ground. Escalators provide access to the platform at the lower level. Staff offices and equipment rooms are sited at both ends of the station

the ends of the station at both concourse and platform levels. Escalators, stairways and lifts (i.e. elevators) are provided for the use of passengers between the concourse and platforms, and there are also connections between the non-public areas on the upper and lower levels for station staff and maintenance personnel.

Platform lengths are determined by the size of train and are typically about 140 m long for a heavy metro, although this can extend to 180 m on some systems. An overall length of about 250 m provides space at the end of the platforms for electrical switchgear, transformers, emergency generators, signalling equipment and other plant (Figure 3.2).

The internal width of a cut and cover station is largely determined by the need to accommodate the tracks, platforms and escalators at the platform level, and is usually in the range of 18–22 m. The variation is mainly due to the differing numbers of escalators needed to serve passengers at heavily loaded and lightly loaded stations, and the different widths of platform that are required.

The internal height of a cut and cover station is typically 9.5 m from rail level to the underside of the concourse roof slab, but its depth below ground level is variable and is determined by local site conditions. A 2 m depth of cover over the roof slab provides a suitable allowance for pavement construction and small utilities, but greater depths may be necessary to avoid the permanent rerouting of large drains. The overall depth from ground level to rail level is usually in the range 13–25 m.

Two different platform layouts are possible with cut and cover stations:

1. Side platforms (i.e. platforms on either side of two adjacent running tracks) (Figure 3.3); and
2. Island platforms (i.e. central platforms with the running tracks on either side).

Figure 3.3 A side platform station constructed by cut and cover methods with the steel beam supporting the roof visible over the tracks

Figure 3.4 A single track and a 3 m wide platform accommodated within a 6.5 m diameter bored tunnel

Similar arrangements are possible with tunnelling methods, but it is often uneconomic to form a large enough void to accommodate two tracks and two platforms, and therefore a common solution is to build two enlarged tunnels, each containing one track and a platform (Figure 3.4). The two are then linked by secondary tunnels to escalators and stairs giving access to the concourse area. However, this is by no means a universal solution, and even in difficult ground conditions spacious caverns can be formed to create attractive and convenient stations. The objectives of the railway authority and their financial constraints are major determinants of the station layout and style.

In cities with narrow streets and buildings having deep foundations it may be necessary to locate one track and its platform below the other to keep within the available construction width. This may be done by either tunnelling or cut and cover construction.

With cut and cover construction, the overall dimensions of the station at platform level effectively determine the size of the concourse level. This generally provides adequate space for passenger circulation, and the efficient disposition of ticket gates and vending machines, together with an area which accommodates offices and equipment rooms that are closed to the public (Figure 3.5).

If a station is constructed by tunnelling methods the size of the concourse area is not necessarily affected by the dimensions of the platform level, and because of the high cost of tunnelling, concourses formed by this method will usually be less generous in size than when cut and cover construction is used.

Figure 3.5 A spacious concourse in a modern station constructed by the cut and cover method, well equipped with ticket-issuing machines and enquiry office

More attractive designs are possible when the ground conditions allow the formation of larger voids by mining or tunnelling techniques. These opportunities depend on the presence of a reliable rock stratum or the ability to enhance the strength of the existing ground.

3.2.2 Side platforms

The arrangement of platforms on either side of a pair of centrally placed tracks usually arises because cut and cover construction is used for the running tunnel between stations. With this method both tracks are placed side by side within one box-like structure. To avoid unnecessary expense the overall width of the box is kept to a minimum, consistent with the railway structure gauge. It is therefore convenient to maintain the tracks in the same close position as they pass through the station and to locate the platforms one on either side of the pair of tracks.

One consequence of this arrangement is that the escalators, stairs and lifts to the concourse level must be placed against the outer wall of the station structure. The upper ends of the escalators may then interfere with the preferred siting of entrance passageways or stairs into the concourse from street level. Another disadvantage of side platforms is that they require their own escalators and stairs. There is no possibility of one set serving both platforms, as is the case with an island platform arrangement.

3.2.3 Island platforms

The most generally favoured arrangement of platforms in an underground station is a combined central or island platform between the two tracks. This is the natural design when the running tunnels between stations are formed by bored tunnels. The two separate drives must be kept apart and thus the tunnels are conveniently located so that the tracks can run on either side of an island platform within the station.

Escalators, stairs and lifts are located at one or more points along the centre line of the platform and are used jointly by passengers arriving at, or departing from, both faces of the platform. Therefore there may be some opportunity to reduce the total provision of such facilities compared with a side-platform arrangement. Because the tops of escalators are located in the central area of the concourse they do not inhibit the siting of entrances and exits from street level. These can be located at any point on the periphery of the concourse to suit the local conditions.

The opening in the concourse floor slab can be the minimum size necessary to accommodate the tops of the escalators and stairs, or it may be decided to introduce a larger opening as a particular architectural feature. This creates an enhanced feeling of space and a point of interest for passengers arriving at the station, but the safety aspect needs to be considered (see Section 3.2.6).

3.2.4 Interchange stations

When stations are located at the intersection of two or more routes, the layout should be designed to facilitate the interchange of passengers between the services. The numbers of people wishing to change from one train to another at an interchange station will depend on the configuration of the railway system, and it is important to establish which will be the predominant flow of transfer passengers.

The most convenient interchange occurs when the arriving train stops at one side of an island platform and the departing train leaves from the other. This so-called 'cross-platform transfer' should be used for the largest interchange movements.

A typical cut and cover interchange station involving two routes (say, A and B) is constructed with three levels below ground:

Level 1: Passenger concourse;
Level 2: Route A track separated by an island platform from Route B track; and
Level 3: The second tracks of routes A and B separated by an island platform.

A similar general arrangement can be provided in stations that are formed by tunnelling, but a true cross-platform transfer is not possible. Passengers move between adjacent platforms at the same level by means of cross adits driven at intervals between the platform tunnels. The need to maintain a safe separation between tunnels also means that the vertical distance between different levels must be greater than with a cut and cover station.

Other layouts may be used for interchange stations (e.g. with all tracks laid at the same level). In this case the transfer between platforms which are not adjacent to each other may be made through an overlying concourse or by passageways constructed above or below the platforms.

Interchange stations are frequently located in the central business district or other intensively developed parts of the city. Their design is therefore tightly constrained by existing buildings, highways and public utilities, and presents one of the main challenges to the civil engineer.

3.2.5 Flotation

An attractive feature of cut and cover stations is their spaciousness compared with stations formed from a series of interlinked tunnels. This particular characteristic can, however, be a disadvantage in water-bearing soils because the buried station structure may have an overall density similar to that of water. There is then the risk that the box-like structure will not maintain its correct level but will tend to rise.

The weight of the structure divided by that of the displaced volume of water must be greater than unity. Ratios of 1.03 and 1.07 are typical of the factors of safety adopted for the construction phase and the completed stations, respectively. In calculating these factors, conservative assumptions are made about the density of materials and construction tolerances. The effects of live loads, skin friction and other overlying buildings are ignored.

3.2.6 Safety

The safety of passengers and staff is of paramount importance, and should be a principal objective of the station designers and operators. This is not just a matter of installing safety equipment. Warning devices, firefighting equipment and emergency communications are all essential for the protection of passengers and staff, but a station's layout and structural design determines whether it is intrinsically safe or dangerous. Inadequate fire resistance of the structural elements and incomplete compartmentation create a potentially hazardous situation, as do narrow platforms and passageways and long, continuous escalators or flights of stairs.

During peak periods many thousands of people will enter and leave a mass transit station, and it is important that realistic estimates are made of the flow of passengers before the design is finalized. Inadequate provision can be rectified later only at a high cost. When large numbers of people are moving in confined spaces and frequently hurrying there is always the worry that a minor mishap, such as one person stumbling and falling, could result in many others falling and being unable to regain their feet quickly because of the pressure of the crowd behind them. In these circumstances serious injury and death may be caused by passengers being trampled underfoot or asphyxiated under a heap of bodies.

If the capacity of passageways, stairs and escalators is adequate, the press of people will be less and a slight accident to one person should not result in a major event. Typical design capacities are given in Section 3.3.4.

The chance of any one individual falling is minimized by:

1. Avoiding unexpected changes in level;
2. Ensuring that stairs and ramps have safe dimensions and slopes;
3. Avoiding long flights of stairs; and
4. Providing generous space at the head and foot of escalators and stairs.

Escalators themselves are frequently constructed with four flat steps at their upper and lower ends so as to provide a more gradual transition for passengers mounting and dismounting. Suitable standards and dimensions for stairs and ramps are suggested in Section 3.3.4.

In addition to ensuring that all passenger routes through the station are safe for the normal day-to-day operations, it is necessary to provide evacuation routes which may be used in an emergency to lead passengers away from the immediate danger and out of the station. Any passenger or member of staff, wherever they might be within the station, should be able to turn away from the hazard and find at least one means of escape. Some evacuation routes might pass through corridors and stairs in those areas of the station that are normally reserved for railway staff.

Local emergency and rescue services should be consulted when the evacuation criteria are being decided and agreement reached on the maximum times that should be allowed to empty the platforms, concourse and other areas. It is usual to specify a scenario which assumes a combination of adverse factors such as:

1. Peak-period conditions;
2. A delay to train services prior to the emergency which results in an accumulation of passengers waiting on the platforms;
3. The arrival of a fully loaded train shortly after the emergency occurs;
4. One escalator from the platform out of action and can only be used as a fixed stair.

It may be that more than one scenario is necessary to cover the possible range of circumstances that could apply when an emergency occurs.

To arrive at a safe design it is advisable to make conservative assumptions about the speed with which emergency procedures are implemented. The movement of escalators cannot be changed instantly from the downward to the upward direction to help evacuation, and it may be impossible to prevent the first train arriving at an affected station from stopping there and adding to the number of people who then must use the escape routes.

The most hazardous place in a station is at the edge of a crowded platform as a train arrives, but because of the obvious danger most people are very careful and accidents are rare. However, some urban railways are equipped with screens and sliding doors along the edge of the platform which open when the train has arrived and come to a halt and close before it moves off again. In addition to being an obvious safety feature, the screens also reduce the amount of air which is lost from the station into the running tunnels, and this can be an important consideration in tropical or other countries, where air conditioning is necessary for passengers' comfort in stations.

There has been some concern about the reliability of platform-edge doors themselves and also the ability of the rolling stock to stop at the precisely required spot. If these difficulties can be overcome, the sense of security that platform doors provide will encourage their wider use.

The most likely cause of an emergency in an underground station is fire, and although staff training and the provision of equipment are essential for fire prevention, detection and fighting, it is obvious that inflammable materials and those which produce toxic fumes when heated should not be used in the structure or finishes of a station. Furthermore, good civil engineering design is able to mitigate the effects of a fire if one occurs.

Early consultation with the local firefighting authority is necessary to decide the appropriate design standards for the compartmentation and evacuation/access routes and the type and scale of equipment that is installed for prevention, warning and firefighting. In discussion with the professional firefighters it should be borne in mind that they will have the dangerous duty of dealing with any major disaster that may occur, and their requirements cannot be treated casually.

The practice of arranging the interior of a building into a series of fire-resistant compartments is well established and can readily be applied in the non-public areas of stations. In this way, safe routes for evacuation of passengers and staff can be created, and access provided for the professional firefighting personnel. The containment of a fire also delays its spread so that only the equipment in its immediate vicinity is damaged.

The fire-resistance period of each compartment should be decided by considering the safety of passengers, staff and firefighters, the quantity of combustible material, and the importance of the plant and equipment. It is unlikely that a period of less than 2 h would be adopted for most parts of an underground station, and some critical areas might have a significantly greater resistance.

Special consideration must be given to the main structural members, since any collapse due to fire damage would be disastrous – far greater than the loss of individual items of plant and equipment. For this reason, longer fire-resistance periods of 4 h or more may be adopted for the main structural elements.

The principle of compartmentation cannot be fully applied in the public areas of the stations (i.e. the concourse and platforms) because their main characteristic is to allow the easy and unimpeded movement of passengers. Fortunately, the quantity of combustible material in these areas should be low; provided that the correct materials have been used in the architectural finishes and the installed equipment, and that rubbish is not allowed to accumulate. Although the risk may be low the effect of any fire that did occur in the large public areas would be serious, and it is prudent to take the maximum precautions that are possible. These can include the compartmentation of any ceiling voids to restrict the movement of high-level smoke and the construction of downstand beams around the openings in the concourse floor slab to contain rising smoke in the platform area.

3.3 Entrances and passenger facilities

3.3.1 Free-standing entrances

Entrances should be convenient and attractive so as to encourage people to use the underground railway and to make an immediate impression of quality. It is therefore important that they are located so that the maximum number of potential passengers are assured easy and safe access.

Overhead cover may be constructed to protect passengers from rain or strong sunlight, depending on the climate, and to cover the upper end of any escalators installed to carry passengers from street to concourse level. However, this protection is not essential provided that the escalators are designed for external use, and may in fact be undesirable because of its physical and visual intrusion in the city street. Lifts (i.e. elevators) will be necessary if it is the general policy of the railway authority to provide facilities for disabled passengers, and will require some entrance structure above ground level.

In countries that experience heavy rainfall such as that occurring in typhoons or similar tropical storms it is necessary to protect the passenger entrances and all access points to the underground railway against the effects of local flooding of the streets. This is normally achieved by raising the threshold of the entrance by an amount up to 50 cm in order to prevent the inflow of water (Figure 3.6).

Raised thresholds are not recommended as a general practice because they are inconvenient for passengers who have to mount one or two steps before starting their descent into the station. If the disabled have to be accommodated, it is also

Figure 3.6 A free-standing entrance adjacent to a ventilation shaft structure. The threshold is raised above the street level to prevent the entry of flood water

necessary to provide ramps as well as stairs; and there must always be some concern that the transition from an up-ramp to a down-ramp will be difficult for wheelchair users to negotiate safely. It is much more preferable if the local stormwater drainage system can be improved and maintained so that rainwater is not allowed to accumulate in the vicinity of the entrance. However, this is not always feasible where very intense storms occur, and if temporary flooding is possible an alternative to raising the entrance threshold is to make provision for floodboards that may be placed in position across the entrances in response to an imminent threat.

The appearance of a free-standing entrance structure above ground may be modest and unobtrusive or ornate and imposing. Its style should be influenced by the surrounding buildings and activities. Large entrance structures are not likely to be feasible in a central business district.

3.3.2 Entrances within other buildings

In response to the shortage of space in city centres for free-standing entrances it is common practice to incorporate station entrances within other buildings. If these are large office or retail establishments there is the additional advantage that the people visiting them will be given convenient access to the underground railway (Figure 3.7).

Figure 3.7 An underground station concourse integrated with retail shopping facilities

Although it is not impossible to construct a new entrance to a station from within an existing development, it is more usual for an entrance to be incorporated during the design and construction of a new building. Such opportunities occur when a site adjacent to a station is being redeveloped either during the construction of the underground railway or at a later date.

There are a number of technical and legal interfaces to be identified and agreed between the railway authority and the developers when entrances are incorporated within another building. Not all of these are directly relevant to civil engineers but they must decide whether any part of the entrance should be integral with the building structure, or whether it can and should be structurally independent. There is no reason, in principle, for preferring one arrangement to another. What is important is that the designers of the station and the building both have a clear understanding of the interreaction between the two structures. Station designers must satisfy themselves that the structural design of the building does not pose any threat to the fabric of the entrance and to the passengers and staff. They must also review the drainage arrangements and be assured that flood water and waste water from the building cannot be diverted into the station entrances.

The construction of a cut and cover station outside the limits of the roadway usually provides the opportunity for the joint development of the station and a commercial, residential or mixed-use building. In this case the structural integration is complete, since the overlying building is wholly or partly supported by the station.

Figure 3.8 A set of three escalators serving a wide island platform in a modern station constructed by cut and cover methods

3.3.3 Provision for lifts, escalators and stairs

The preferred method of moving passengers between the concourse and platform levels is by escalator. However, these must be supplemented by lifts (elevators) if provision is being made for physically disabled persons. When more than one set of escalators are required to carry the anticipated peak flow of passengers they are best located so as to encourage an even distribution of passengers along the length of the platform.

A typical metro station should have two sets of escalators to serve an island platform. Each set should comprise a minimum of one reversible escalator and one flight of stairs. This provision would only be suitable for lightly loaded stations. Moderate or heavy flows will warrant additional escalators and stairs to meet their particular needs for both normal operation and emergency evacuation (Figure 3.8).

In stations that are formed by tunnelling, the escalators are normally accommodated in inclined tunnel drives connected to cross adits giving access to the platforms. In cut and cover stations the escalators are supported at their upper end by the concourse slab and at their lower end at a point below platform level (Figure 3.9). An intermediate support may also be provided. The total dead and live load of an escalator with a 5 m rise is likely to be in excess of 250 kN.

Stairs provide the means of connection between the upper and lower levels of the non-public areas (i.e. between the offices and equipment rooms on the concourse

Figure 3.9 A set of three escalators accommodated within a 7.5 m diameter inclined shaft

level and the equipment rooms at platform level). These may also be used for the emergency evacuation of passengers, or if they are not conveniently sited for that purpose, additional stairways should be installed.

3.3.4 Platforms and passageways

The widths of platforms are determined by considerations of safety and numbers of passengers in the peak period. A minimum of 3 m is usually maintained between the platform edge and any continuous structure or fitment. This may be reduced to 2.5 m where the obstruction is small and isolated.

Under normal operating conditions a space of 1 m^2 should be allowed for each passenger waiting to board a train. Thus it is possible to calculate the necessary width of the platform from a knowledge of the peak arrival rate of passengers and the service interval of trains. It is usual to allow a 0.5 m zone adjacent to the platform edge, which should not be occupied before the train has arrived and come to a standstill.

Emergency situations must also be considered. These may involve the disembarkation of a full trainload of passengers onto a platform which already contains an accumulation of passengers. Under such conditions the minimum safe area per person is about 0.2 m^2. The adopted platform width should not be less than the minimum required for normal and emergency conditions, and should be at least 3 m (or 2.5 m at an isolated obstruction).

Corridors may be provided for either one- or two-way flows of passengers. The latter have a lower capacity per metre width than the former, but where the flows are moderate it will be more economical to provide the less efficient two-way corridor. Typical flow rates are as follows:

One-way flow: 85 passengers per minute per metre width;
Two-way flow: 70 passengers per minute per metre width.

Recommended minimum widths for corridors in public areas are:

One-way flow: 1.8 m
Two-way flow: 2.4 m

A narrower width of 1.2 m is appropriate for corridors which are used only by staff.

3.4 Plant rooms and provision for services

3.4.1 Heavy equipment

A variety of plant, ranging from telecommunications equipment to standby electrical generators, may be accommodated within an underground station. When compared with the live load in the concourse and platform areas, the loads imposed by some of this equipment are quite onerous, and all designs must be checked to ensure that they are adequate to sustain the static and dynamic loads generated by the actual plant that is installed.

Full details of the equipment will not normally be known when the structural designs are first being prepared, and notional loads must be adopted to determine the principal structural sizes. Electrical power equipment is the heaviest plant to be accommodated, and the following values are typical of the uniformly distributed and concentrated loads assumed in the initial design:

Distributed load: 20 kN/m^2
Concentrated load: 20 kN on a square of 300 mm side.

These values may be compared with the corresponding figures of 10 kN/m^2 and 20 kN for other plant rooms containing lighter equipment such as telecommunications gear and with 6 kN/m^2 and 20 kN for the public areas of the station.

3.4.2 Noisy equipment

It is desirable to maintain a low level of noise in the public areas of stations for the general comfort of passengers and so that announcements made over the public-address system are audible. A standard of about NR 50 may be adopted for the concourse. Within some of the plant rooms the noise level may be as high as NR 90, and it is necessary that this should be attenuated so that it does not adversely affect the public areas. Sound-absorbent materials may be applied to walls and ceilings, but the material and construction of the walls and floor slabs themselves can make a major contribution in preventing the dispersal of noise throughout the station. In some parts of the station the sound-insulation qualities of the walls may be more important than their structural strength; increased thicknesses or stiffness may be introduced to limit the transmission of sound.

3.4.3 Ventilation equipment

Environmental control systems, including chillers and filters, may be regarded as standard equipment for underground stations, particularly in tropical countries. In addition to the space required for the air-handling units, provision must be made for ducts to distribute treated air throughout the station and to return vitiated air to the treatment plant. These ducts may be located at ceiling level in the concourse and platform areas and below the platforms themselves.

Three ventilation shafts are normally required at each end of the station in order to:

1. Provide a supply of fresh air;
2. Dispose of vitiated air;
3. Provide draught relief.

Draught-relief shafts reduce the fluctuation of air pressure within the station as trains approach or leave. Each shaft is likely to have a cross-sectional area in excess of $10\,m^2$ (Figure 3.10).

Figure 3.10 A ventilation structure incorporating inlet and exhaust shafts from an underground station, and accommodating chiller units for the environmental control system

The three shafts may be combined into a single structure or incorporated into another building provided that the openings are arranged so that there is no interference between their separate functions. They must also be clear of any window or other opening in an adjoining building.

It is necessary to take into account the variation in air pressure caused by the movement of trains and the ventilation fans when designing the lighter structural elements of the station.

3.4.4 Provision for services

Many of the civil engineering details of an underground station are influenced by the requirements of the electrical and mechanical equipment. Some of the main items of equipment have already been mentioned, but the full range will include most of the following:

1. Power supply (traction substations, station substations, standby generators, batteries, switchgear and cables);
2. Communications (telephones, radio, clocks and passenger-information displays, closed-circuit television, public-address systems);
3. Escalators and lifts (elevators);
4. Fare collection (ticket-issuing/validating machines, entry and exit gates, cash-handling equipment);
5. Environmental control (air conditioning and ventilating equipment, ducts, tunnel vent fans);
6. Building services (lighting, low-voltage distribution, fire-detection and firefighting equipment, water supply, sanitation). In addition, there may also be signalling equipment sited within a station, even though signals will normally be controlled and monitored at a central site.

The civil engineering design must take account of all this equipment by the provision of all necessary plinths and openings, the reservation of routes for ducts, cables and pipes, and the building-in of brackets and supports. The location of some equipment, notably ticket-issuing machines and entry gates, cannot be firmly determined during the civil design phase. In fact their positions will probably need to be altered during the operation of the station as the passenger flows vary from year to year (Figure 3.11). For this reason, the civil engineering detailing should not inhibit the resiting of such equipment and the rerouting of cables. The necessary adaptability can be provided either by a false floor (i.e. a space beneath the surface finish) or 100–200 mm of non-structural filling over the loadbearing floor.

3.5 Construction

3.5.1 Diversion and protection of utilities

Water and gas supply pipes and drains and sewers are generally located below ground. Electricity supply and telecommunications cables may be situated either above or below ground, but it is becoming increasingly common to locate them below ground, particularly in well-developed city centres.

Drains and sewers with a diameter of 1 m and above present a major obstacle to construction. It is often impractical to support them across the large excavation that

Figure 3.11 Ticket-operated entry and exit gates newly installed in the concourse of an existing station

is required for the construction of a cut and cover station, and they are frequently at a depth which would conflict with the completed station. In such cases they must be diverted around the station and all subsidiary drains reconnected.

Smaller drains and sewers are often supported across an excavation. The choice of whether to support or divert will depend upon the local circumstances, and particularly the depth of the drains relative to the upper level of the finished underground structure. Underground utilities close to a driven tunnel are only at risk where they cross the settlement trough caused by tunnel driving.

Large-diameter pipes and cables susceptible to damage by settlement may be supported by treating the ground beneath them to limit the settlement. An alternative is to excavate down to the utility and hang it on adjustable supports. The length of the supports is altered as the ground subsides, thereby maintaining the utility at its correct level. Overhead cables are diverted where they would interfere with construction plant and worksites.

3.5.2 Protection of adjacent structures

All underground works result in some displacement of the surrounding earth, either by the loss of ground, release of naturally occurring stresses or the movement of groundwater. The magnitude and extent of the movements depend on the nature of the ground and the method of excavation and construction of the underground works.

The degree of possible damage to adjacent structures depends on the size and form of the building, the extent and type of foundations and the degree of settlement. Structural form and materials are important factors in determining a building's response to a given degree of settlement. Loadbearing masonry and brick structures have an inherent flexibility which tends to produce minor rather than major damage when small settlements occur, but they may not have the ultimate capability to resist the relatively greater settlements that can be sustained by a well-designed modern frame building. However, there are no general rules that can be applied, and each case must be examined by the civil engineer.

Thus, before the design of an underground station can be finalized it is necessary to study all available information on the design and construction of buildings that are likely to be affected, and to survey their present structural condition. The need for underpinning or other protection can then be assessed.

Underpinning may be used to extend the foundations of a structure to a stratum which is either undisturbed by the construction or is only affected to a small degree. An alternative is to enhance the characteristics of the soil using ground-treatment methods, either beneath the building or between the building and the new construction. With bridges in particular, it may be feasible to jack up the superstructure to compensate for the subsiding foundations and so maintain the correct levels and avoid settlement loads. To ensure that the jacking is carried out safely and effectively it is necessary to monitor the settlement to ensure that the correct uplift is given in step with the subsidence.

In addition to measurements of surface settlement a number of other parameters are commonly measured during the course of underground works. e.g.

Subsurface settlement
Horizontal surface movement
Horizontal subsurface movement
Groundwater level and pore pressure
Ground heave in the bottom of open cuts
Strains in struts, anchors, etc.
Movement of buildings.

The monitoring programme should be designed so that it provides all necessary information for the safe and economical construction of the works, but excessive data collection should be avoided. For this reason, it is important to ensure that the responsibilities and organization for the prompt analysis and evaluation of data are well established before fieldwork commences. If this is not ensured then there will always be the risk that crucial information may be overlooked. The objectives of the monitoring programme must be clearly defined and the incoming data categorized by its function, so that any information concerned with the immediate safety and progress of the works may be separated from long-term research data and evaluated promptly.

A well-designed programme might, for instance, include the careful monitoring of the excavation and construction of the first station in a new system so as to provide valuable data when deciding whether or not protective measures are needed for structures that are adjacent to the second and subsequent stations. It would enable the risks associated with the tunnelling methods and prevailing ground conditions to be quantified.

Preparations for the building condition surveys should be started at an early date because it is a time-consuming process to identify the ownership of each building, search for the design records and arrange access to the premises. Then, immediately prior to construction, a detailed survey should be made which records all defects in the buildings, so that any subsequent damage caused by tunnelling or excavation can be correctly attributed to the construction of the station. Without a building condition survey it may be difficult to prove that some defects existed before the underground works were carried out.

3.5.3 Road-traffic diversions and temporary support

Cut and cover construction in a city centre is very disruptive to traffic because of the large excavations in the roadways. With bored tunnelling there is less disorder, but there is still a need to provide worksites and to accommodate the construction traffic bringing plant and materials and removing spoil.

The traffic capacity of a city street depends principally on its width and the parking regulations that are enforced along its length. Lane capacities of 1500 passenger car units (pcu's) per hour can be expected on one-way roads with generous dimensions and effective restriction of parked vehicles. Much lower lane capacities are achieved with two-way roads, but the crucial factor in a network of city roads is usually the capacity of the at-grade road junctions. These are normally the principal elements influencing the overall capacity of the network, and it is these very locations which are often the favoured sites for an underground railway station. Closing a junction for the construction of a station can reduce capacity by tens of thousands of vehicles per day, and the traffic previously using the junction must divert to other routes avoiding the site.

The majority of road systems in large cities are carrying traffic flows close to or even in excess of their practical capacity. It follows that any reduction in that capacity caused by closing a length of street or a junction will have a significant and adverse effect on the levels of congestion, and may create problems in areas that are relatively remote from the location of the works. It is a complex problem to determine how traffic will reroute in response to a road or junction closure and to develop a plan of diversions which will minimize the overall disruption. The best method requires reliable data on existing traffic flows, origin-destination patterns and an inventory of road and junction characteristics and dimensions. This information is then used with a computer-based model with the capability of representing the interrelationship between traffic flow and speed over an extended network composed of roads with differing widths and junctions with varying types of traffic control.

Before the model can be used to study the effects of road closures it must first be calibrated so that it can satisfactorily reproduce the existing traffic flows on the unaffected network. An exact match between the model's predictions and the counted traffic cannot be expected, if only for the fact that the counts of actual traffic are themselves subject to daily and seasonal variation. Reliable data on origin and destination patterns are particularly important in calibrating the model so that it can be used with confidence for testing alternative diversion plans. If a complete set of origin-destination data is not available, some computer models contain routines that will estimate the missing information. However, it is best to calibrate the model with a full set of directly observed data if at all possible.

Developing the traffic-diversion programmes with the calibrated model is a process of gradual refinement, starting with the examination of broad conceptual plans and leading to detailed schemes. A series of schemes is needed that can be

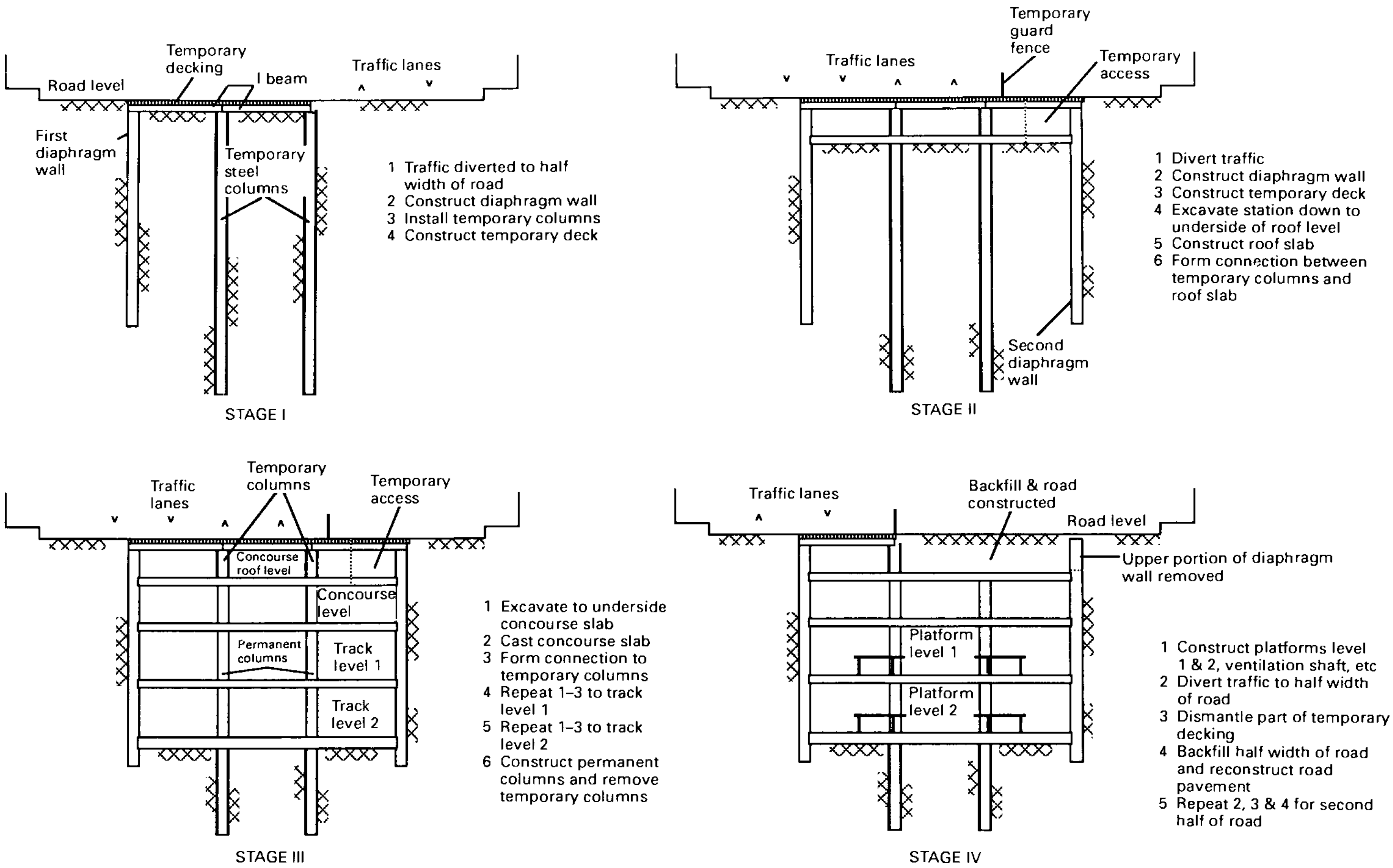

Figure 3.12 The sequence of work involved in constructing a cut and cover station from the top down

introduced as the construction work progresses as to minimize the disruption and to ensure that emergency services have continuous access to all localities, including the construction sites. They must be prepared in great detail and in full collaboration with the highway authorities and the police, and must incorporate all necessary signing, signalling and advance publicity and information.

Evaluation of traffic-diversion schemes is helped by the statistics on vehicle delays and travel distances produced by the model. These data can be used in an economic analysis of the alternative plans so that some aspects of the disruption caused to the general traffic may be quantified and expressed in monetary terms.

One way of minimizing the interruption to traffic is to use a 'top-down' method of construction for cut and cover stations. This permits most of the work to take place under temporary decking, thereby allowing road traffic to be maintained during most of the building period. The usual sequence is to close one half of the roadway and install:

1. One side wall and the halves of both end walls;
2. Temporary columns; and
3. Road decking.

The process is then repeated on the other half of the roadway, with traffic running on the recently installed decking (Figure 3.12).

When this is complete, traffic may be allowed to use the full width of decking while the station is excavated and constructed. Finally, the decking is removed and the void between the upper surface of the station roof and road level is backfilled and the road is reinstated. This, too, is undertaken in stages, so that at no time is the road totally closed to vehicular traffic.

Chapter 4

Ground treatment

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Contents

4.1 Introduction

Ground treatment is often part of the method of construction of cut and cover structures and tunnels, and the aim of such treatment is to improve the strength and/or watertightness of soil. This chapter will deal with four main methods:

1. *Grouting by injection*: this process allows filling in the voids in the ground (e.g. the fissures of a rock or the intergranular voids of sandy materials).
2. *Jet grouting*: the process produces an *in-situ* soil-cement.
3. *Compaction grouting*: the soil is compacted by an intrusion into the ground under the high pressure of mortar bulbs.
4. *Freezing*: the water contained in the soil's voids is frozen by means of cooling probes.

Other soil-improvement methods such as *in-situ* mechanical soil mixing, vibroflotation, vertical drains, electro-osmosis and dynamic compaction are not covered here as they have very restricted fields of application in the context of underground rail transport projects. Procedures for reinforcing soils using steel supports, micropiling, nailing or bored poling are not generally described under the heading 'Soil treatment'.

4.2 Grouting by injection

This treatment consists of drilling holes through the area to be treated and introducing the grout under pressure through them.

4.2.1 Impregnation and 'claquage' (splitting)

Before describing the more usual methods and products in greater detail, it is essential to define two fundamental concepts: impregnation and 'claquage'.

Impregnation
Impregnation can be said to have been achieved when the fluidity of the grout and the grouting pressures have caused the grout to penetrate into all the voids in the ground while removing the water without, however, displacing soil or widening existing fissures. Perfect impregnation is a complete substitution of the free water contained in the soil by a grouting material without distortion of the soil's structure. When this type of result is sought, the treatment is referred to as 'impregnation grouting' or 'permeation grouting'[13]. Grouting materials capable of producing such impregnation are described as 'penetrating'.

'Claquage' (soil splitting)
When the applied grouting pressure exceeds the value of the lowest principal stress in the ground at the point of injection, an artificial fissure is opened in a plane perpendicular to this minimum principal stress. This phenomenon is referred to as splitting, fracturing[13], hydrofracturing or, frequently, 'claquage'. This term was coined by the French specialists who first published on this subject.

In loose soils, a 'claquage' appears as the presence of plates, tongues or lenses of grout material with variable spacing and thickness which follow the surfaces of least resistance of the soil or the contact surfaces between different layers. In rock, it is seen as a widening of existing fissures.

When large quantities of grout are injected in the form of 'claquage' the fractures created have a tendency to be horizontal and, hence, to give rise to uplifts without any beneficial effect being obtained. In loose soils, however, controlled and limited 'claquage' is frequently efficient, producing a tightening and compacting effect. it is used almost always in conjunction with impregnation grouting.

4.2.2. Drilling grout holes

Boreholes for grouting are of small diameter (40–100 mm), carried out using site-investigation or drilling rigs working in rotation or in rotopercussion. Unless special, and hence costly, precautions are taken, drill holes for grouting are subject to deviations which are normally of 2–3%. The risk of deviation is reduced when a casing is used which is stiffer than a drill string and when the drill holes are vertical. On the other hand, the risk is increased when the drill holes are horizontal or subhorizontal and when soils are heterogeneous and/or contain boulders. These risks place a practical limit on the length of drill holes for grouting. In urban sites, lengths of 40 m for vertical holes and 25 m for horizontal or subhorizontal ones are rarely exceeded. The maximum spacing between drill holes is generally within the ranges shown in Table 4.1.

Table 4.1

Type of soil treated	*Soil permeability range*	*Maximum spacing between grout holes* (m)
Fine sands	Under 10^{-5} m/s	0.8–1.3
Sands, sand and gravels	10^{-5} to 10^{-3} m/s	1–2.0
Gravels	Over 10^{-3} m/s	2–4
Rock with fine fissures	1–20 lugeon	1–3
Rock with open fissures	Over 20 lugeon	2–4
Contact grouting	–	2–3
Cavity grouting	–	3–10

4.2.3 Grout hole packer systems

Methods

The various existing grouting methods are best described by AFTES [1], Filliat [12] and Cambefort [15]. The most usual are:

1. Connection to the top of grouting holes: mainly used for backfilling cavities;
2. Downward stage grouting: the safest and most efficient method in rock (Figure 4.1);
3. Upward stage grouting: this is applicable in rock only in cases where the walls of the drill hole are stable;
4. 'Tube à manchette' method: this most efficient method in soil is described below;
5. Grouting through drill rods: this has very limited application.

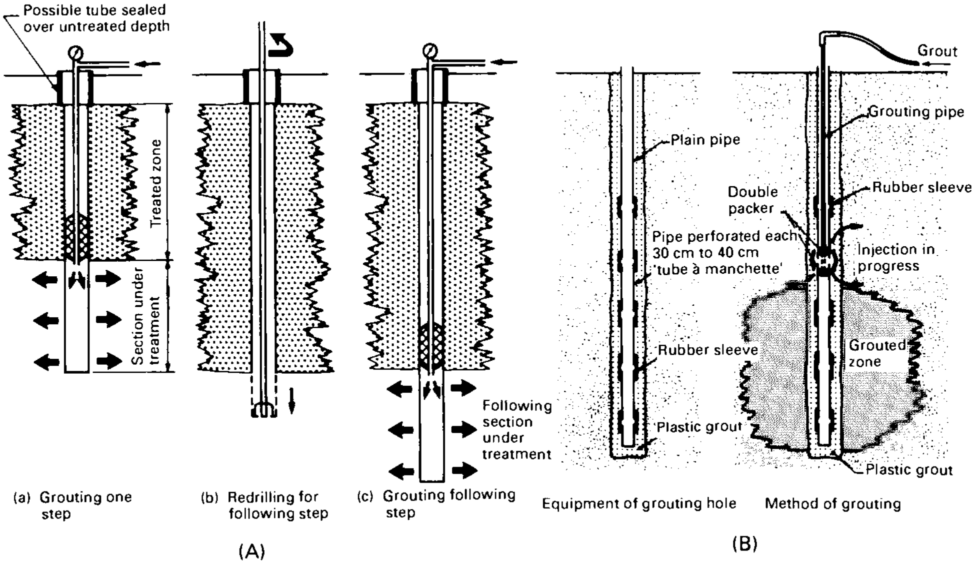

Figure 4.1 Grouting methods. (A) In rock, the downward stage grouting method is the safest but involves the cost of redrilling for each step, which are usually 1 to 3 m deep. (B) In soil, the 'tubes à manchettes' method gives the best results; it allows multi-stage grouting without the need for redrilling

'Tube à manchette' This method was first developed in France and is referred to in the English-speaking world by its French name (Figure 4.1). A plastic or metal tube of approximately 1½ in diameter is provided at intervals along its length with vents to the outside which are equipped with non-return valves or 'manchettes'. These tubes are placed in a drill hole and bonded to the soil by special grout referred to as a 'sleeve grout'. The grouting then takes place via a double packer which allows any given 'manchette' to be isolated. The grout penetrates into the soil, having first broken through the sleeve grout.

This method has the major advantage of allowing several successive phases of grouting at whatever level without any redrilling. From an operational standpoint, it allows drilling and grouting works to proceed quite independently.

'Tube à manchette' grouting gives excellent results in loose soils. It is not, however, applicable in rock since the confinement would prevent opening of the 'manchettes'.

Grouting through drill rods In Japan, this type of method is used in conjunction with chemical grouts and is referred to as LAG (limited area grouting). The chemicals used are sodium silicate with a very fast action hardener. The very short setting time is such that the fluid coagulates in the annular space between the drill string and the soil before it can rise to the surface. This blockage acts as a packer and effectively allows pressure grouting through the drill string.

Chemical grouting through the drill string requires the use of rapid-setting gels. These gels have the major disadvantage of very limited penetrability in fine soils. They can thus not ensure a homogeneous impregnation of the soil.

4.2.4 Grouting materials

Grouting products are described in the literature but many are only of academic interest. We will mention only those of practical interest. The reader will find more detailed descriptions in AFTES' recommendations [1].

Cement grout
Pure cement grouts are unstable except with high concentrations of cement (cement/water ratio greater than 1.5) and, hence, with high viscosities. This instability of fluid grouts with low cement content considerably limits their power of penetration into cracks and intergranular voids. This is due to clogging which results from the sedimentation of the grout during its progress through the soil. The instability of these grouts also causes problems of pumpability. Therefore these types of grout are of no interest for soil grouting.

Bentonite/cement grout
The bentonite (clay)/cement mixture is the universal and economical base grout for soil treatment. It can be used to impregnate open soils with permeabilities of 10^{-3} m/s or more and the treatment of rock with open fissures of 1 mm minimum in width.

In fine loose soils, bentonite/cement grout is used systematically in a preliminary and preparatory phase before the grouting of chemical products. The aim of the preliminary phase is to fill the large voids and the open or preferential passages. These grouts are also used to form controlled 'claquage' before or after chemical grouting. The aim here is to compact the soil. Sleeve grouts used for sealing 'tubes à machettes' are also bentonite/cement grouts whose composition is adapted to their function.

Special cement-based grout
Grouts for special applications can be prepared by the addition to the bentonite/cement grout of various products such as:

1. Inert granular material (stone dust, also known as 'filler'): this thickens the grout and is used for backfilling voids or large fissures.
2. Sodium silicate: this creates an acceleration in the stiffening of the grout. It is used in very open soil or in soils where there is significant water circulation.
3. Aluminium powder or tensio-active agent which produces 'foam grout'. This is also used for backfilling voids.

Mineral grouts with enhanced penetrability
The penetrating power of bentonite/cement grouts can be improved by:

1. Improving the apparent viscosity obtained through the addition of a fluidizing agent;
2. Reducing the tendency to bleed by adding dispersing or water-retaining agents which prevent the grout from losing its water (under the effect of the grouting pressure) into the porous surfaces encountered during grouting. The process of bleeding, which hinders the grout's progress, is thus reduced [3].
3. Reducing the size of the particles in the grout; this is achieved by the use of superfine cement giving a particle size of around 10–15 μm.

Finally, it is worth noting that in 1985 mineral grouts based on micro silica appeared on the market with a particle size of approximately 1–5 μm. These microparticle grouts allow impregnation of soils with a permeability of 10^{-5} m/s [6].

Silica gels
A sodium silicate solution mixed with appropriate hardeners constitutes a grout with very low initial viscosity which hardens into a gel. For reasons of economy, silica gels are the most widely used chemical grout product. Their use becomes necessary when the permeability of the soil is too low to allow penetration and impregnation by cement/bentonite grouts. They are, therefore, appropriate in soils where the coefficient of permeability is less than 10^{-4} m/s. As with other chemical products, silica gels are almost invariably used in conjunction with preliminary cement grouting.

For treatments where the principal objective is to improve impermeability, a soft, low-concentrated gel is preferred since it has the advantage of being less costly and provides a low initial viscosity. For consolidation grouting, a concentrated solution producing a hard gel is selected, although it has the drawback of greater cost and a higher initial viscosity, which makes it less penetrating.

Resins Resins are used in the relatively infrequent case where impregnation of the soil is sought, but its permeability is too low to allow penetration by silica gels. The most common are:

1. *Acrylic resins*: their initial viscosity is close to that of water (1 mPa.s). Their mechanical strengths are, however, low and they are used essentially for impermeability treatments. These resins can be mixed with sodium silicate or polymers, and this results in a product with slightly higher viscosity but also with higher strength.
2. *Phenolic resins*: these provide high mechanical strengths with initial viscosities lower than that of the silica gels.
3. *Polyurethane resins*: these have the unique property of producing hard or soft foam when in contact with water. They are used mostly for the filling of large voids in which water is circulating and for the plugging of water passages.

Comparison of the materials Figure 4.2 defines the areas of application of the products indicated above as a function of the permeability of the soils to be treated. From the point of view of direct cost, the grouts and gels are classified as follows:

Product		*Cost index*
Bentonite/cement grout		1 to 1.6
Microparticle mineral grout	– semi-hard	3, 5
	– hard	6
Silica gel	– soft	2
	– semi-hard	8
	– hard	12
	– very hard	16
Resin	– acrylic	33
	– phenolic	40
Polyurethane foam		20

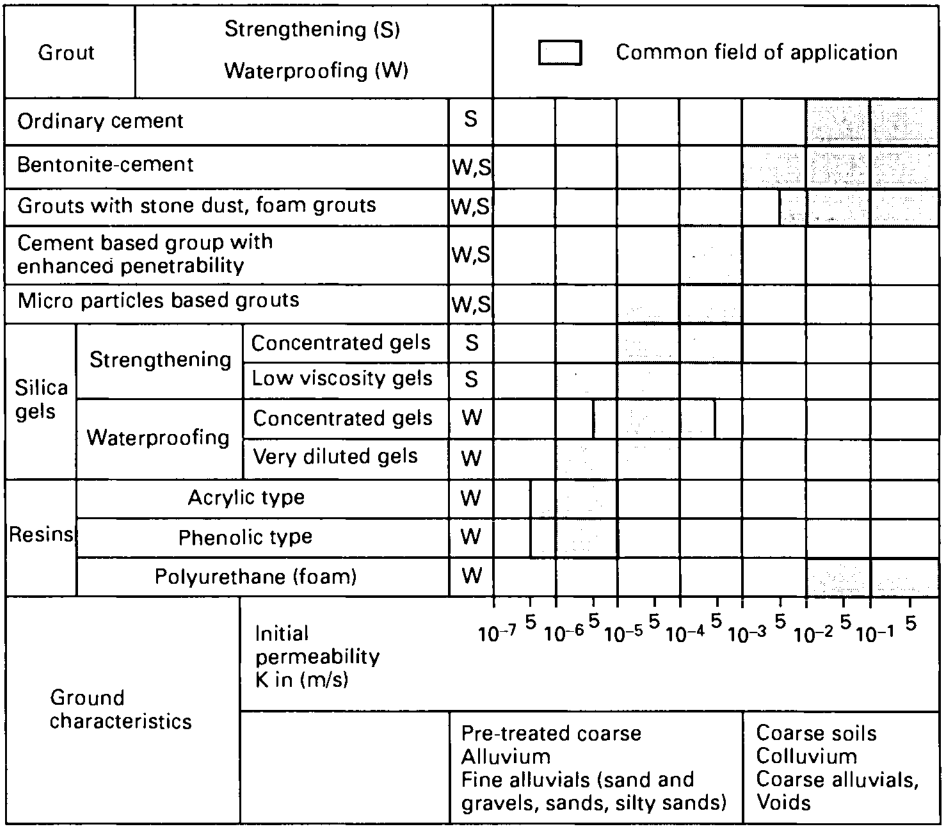

Figure 4.2 Grouting products. Field of application of grouts in granular souls. (Document by AFTES recommendations, translated and updated by the author)

The above cost indices are proportional to the direct supply cost of materials necessary for 1 m^3 of grout or gel and do not include the cost of mixing and grouting. Cost index 1 is equivalent to 120 French francs on 1 January 1989. The cost index for polyurethane is calculated on the basis of a foam having a volume twenty times that of the original liquid resin.

4.2.5 Grouting pressure/flow rate

The grouting pressure P and the discharge of grout Q are related. When the discharge is increased, so is the pressure. What is normally referred to as 'grouting pressure' is, in fact, the pressure at the top of the borehole (sometimes with a correction for hydrostatic head related to grouting level). This is not the same as the actual pressure on the ground, which is impossible to measure in practice.

The two can be very different, particularly in the case of 'tubes à manchettes' grouting due to the obvious reasons of head loss in the line, the packer and the voids or fissures in the ground. Any assessment of actual pressure of the grout at a point in the ground by reference to the measured grouting pressure is generally misleading. Similarly, an attempt at an analytical assessment of the measured grouting pressure *P* liable to cause 'claquage' is likely to be very inaccurate.

The above observations favour an experimental approach to the two parameters *P* and *Q*. At the outset of a project a number of trials are carried out in which the grouting discharge *Q* is progressively increased. A clear discontinuity in the *P* versus *Q* curve is indicative of the grouting pressure producing 'claquage'.

For impregnation grouting, a discharge *Q* will be chosen to produce a grouting pressure *P* as high as possible while remaining below that experimentally shown to produce 'claquage'. In very fine soils, this restriction may lead to very low discharges of around 3 l/min. Open soils, such as gravel, may allow much greater discharges of around 20 l/min.

Comparison of the grouting pressures observed in different points of the soil mass is an effective means of locating weaker zones in need of an additional phase of grouting. This comparison is, however, only meaningful if the discharge is the same and constant in each case.

The necessity to measure *P* as a function of *Q* reveals one of the major drawbacks of a grouting system, much favoured by North American engineers, in which a high-discharge pump feeds a closed-circuit grout line from which several boreholes are supplied.

The importance of the control of *P* and *Q* has led employers and specialists to seek improvement in the monitoring of these grouting parameters in practice. The general trend is towards fitting grouting equipment with remote control and transducers which, through the use of microcomputers and adequate software, lead to the automation of the process and give a permanent record of grouting parameters. Beside reducing the risks of human error, such systems allow a day-to-day 'mapping' of the grout take and grouting pressure. This information, complemented by other data provided during the drilling of the grout holes, makes it possible reliably to detect weak zones requiring further phases of re-injection.

This growing sophistication is, obviously, at a cost. Experience has, however, shown that careful monitoring while the work is being carried out is one of the most reliable forms of quality control for ground treatment by grouting.

4.2.6 Application of grouting by injection

Statistically, soil treatment by grouting is the most appropriate method in most cases. In any soil conditions, correctly performed grouting treatments can, after treatment, guarantee an *in-situ* soil permeability of the order of 10^{-6} m/s. This level of permeability is generally sufficient for our applications. However, a substantial improvement in soil cohesion can only be achieved in sandy or sandy/gravelly soils with a low silt and clay content.

Grouting by injection is rarely efficient in soft clayey soils, peats or mud except in the form of 'claquage'. In this case, some degree of compacting and water expulsion through soil tightening can occur, but no significant improvement in strength is achieved.

4.3 Jet grouting

4.3.1 The jet grouting process

This process was developed in Japan in the 1960s and patented. The first application in Europe was in the late 1970s.

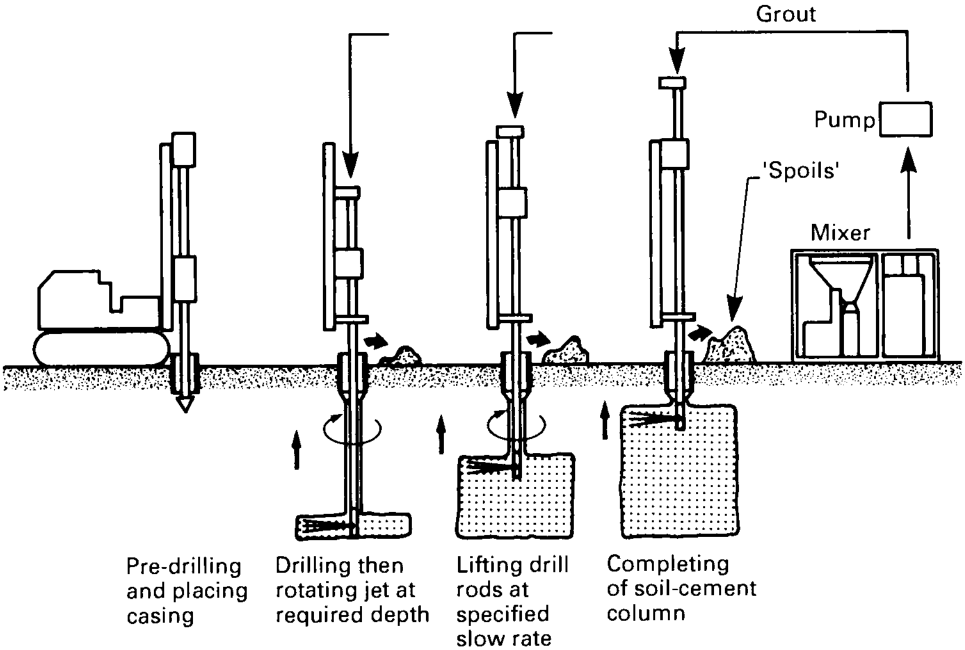

Figure 4.3 Jet grouting. The soil is broken up and simultaneously mixed to a cement grout by a rotating very high-speed jet

Jet grouting, as shown in Figure 4.3, uses a thin high-velocity jet of liquid. As the jet rotates, the soil is cut and disintegrated hydrodynamically. The soil, thus disturbed, is mixed simultaneously with the grout. In this process, the grout does not penetrate the soil either by impregnation or by 'claquage'. It is simply thoroughly mixed *in situ* with the water and soil particles within a cavity created by the destructive action of the rotating jet. No grouting pressure is applied. The objective is to obtain a cylindrical mass of soil–cement. The construction of secant cylinders of this type enables the formation of a screen or mass treatment to be achieved. The soil/grout/water mix produced by jet grouting has a volume greater than that of the soil treated. Therefore the process produces a surplus of this mix which must rise to the surface in the annular space around the drill string. This surplus soil/cement liquid is often called 'slime' or 'spoils'. It is important to monitor the regular upward flow of these 'spoils' since any obstruction would lead to an undesirable pressurizing of the treated soil which could initiate a 'claquage'. The surplus which reappears at the surface must be removed to a pit. This is a major proportion of the cost of the process and can be as much as 20–30%.

The various type of jet grouting methods are:

1. *Single jet*: this uses a single high-velocity jet of grout and can produce a column of 600–700 mm in diameter.
2. *Double jet*: an annulus of compressed air is injected around the jet of grout. The effective destructive radius of action of the jet is increased by approximately 150–200 mm compared with single jet.

3. *Triple jet*: the cutting jet is pure water surrounded by an annulus of compressed air. The injection of cement grout can be either as an annulus or as a separate jet. The method allows the construction of columns of approximately 1–1.2 m in diameter in clayey soils with SPTs less than 8 and of 1.2–1.4 m in diameter in sandy soils with SPTs less than 80.

4.3.2 Application

Jet grouting can be used in all granular soils in which the SPT value does not exceed 8 for clayey soils and 80 for sandy ones. Firm clay is difficult to break up by the process and, hence, tends to produce a heterogeneous treatment composed of lumps of clay within a grout matrix. Figure 4.4 shows the limits of the process's application according to Dupeuble[16].

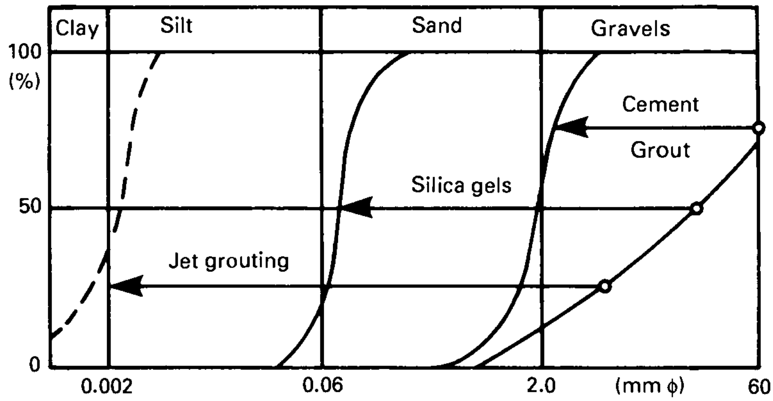

Figure 4.4 Application of jet grouting. This allows the treatment of silty soils for which grouting by injection is inefficient

The process loses its efficiency with depth. While jet grouting treatments have been carried out down to a depth of 26 m on the Singapore metro (Figure 4.7) and to 50 m in Japan, it is generally considered that the use of the process is not viable below about 20 m.

In soils suitable for jet grouting a considerable improvement in soil strength can be achieved. In sands, typical strengths of 5–15 MPa at 28 days are attained for a quantity of cement of 400–800 kg per theoretical cubic metre of soil treated. In peats, compression strengths of 0.5–0.6 MPa at 28 days can be achieved using 800 kg of cement per cubic metre of soil. Treatment will give a permeability of the order of 10^{-8} to 10^{-10} m/s at 180 days.

Jet grouting is most specifically suited to applications where a large increase in soil strength is required: e.g.

1. Underpinning;
2. Improvement of bearing capacity;
3. Grouted bracing slab (Figure 4.12);
4. The treatment of silty soils, soft mud and other impermeable soils in which grouting by injection would not provide a significant improvement in the strength and where soil freezing would be a costly solution.

The method is of less interest where only an improvement in impermeability is required. In this case, grouting by injection produces sufficient impermeability ($K = 10^{-6}$ to 10^{-7} m/s). It is generally less costly and allows treatment to be carried out to a greater depth.

4.4 Compaction grouting

4.4.1 Method

Compaction grouting was first used in the USA in the 1960s. It is appropriate in loose soils and involves forcing a pumpable, very dry mortar into the soil under high pressure. The aim is to obtain a compaction of the soil by compression resulting from the formation of bulbs of mortar in the ground. Impregnation and 'claquage', as defined above, are to be avoided.

The grout material contains only 15–30% water and has a slump smaller than 50 mm. Its particle size distribution must be within a fairly narrow band[18]. The addition of cement is not always necessary. On the whole, between 5% and 10% of the theoretical volume to be treated is usually placed as grouting material.

4.4.2 Application

In practice, compaction grouting is appropriate only when the soil is sufficiently loose and compressible. *In-situ* pressuremeter tests are the most suitable method for defining in which soils the method is applicable and for monitoring and quantifying the improvement provided by the treatment. Generally, a soil is considered suitable for compaction grouting if its pressuremetric limit pressure is less than 0.5 MPa. The percentage of grout take decreases as the pressuremetric limit pressure increases. Here, Standard Penetration Tests (SPT) are not particularly informative and correlations between SPT and pressuremeter tests should be interpreted carefully. Nevertheless, it can be said that the method should only be considered in sandy soils with SPTs below 10–15 blows or clayey soils with SPTs below 4–6 blows.

The effect of treatment by compaction grouting may be threefold:

1. A compaction effect due to the densifying of the *in-situ* soil;
2. The effect of the presence of the mortar improving the overall shear strength of the soil. In this case, the mortar would contain cement;
3. A consolidation effect by forced drainage through the mortar, which, in this case, should be porous.

It can be seen that the effects of compaction grouting are rather similar to injection grouting by 'claquage'. However, compaction grouting enables high horizontal stresses to be applied, and such stresses are the most beneficial. In grouting by 'claquage', horizontal stresses are limited due to the rapid appearance of horizontal fractures. These produce useless vertical stresses (except in the case of deliberate uplifting). For this reason, compaction grouting is sometimes referred to as 'static horizontal compaction'. Grouting by 'claquage', however, has the advantage that it can be used in more compact soils.

Compaction grouting is used mainly in the following applications:

1. Underpinning of existing structures;
2. Recompression of soil immediately after the passage of a tunnelling machine [21];
3. Slope stabilization;
4. Improvement of soil liquefaction potential under seismic load;
5. Improvement of bearing capacity in loose fill.

4.5 Ground freezing

4.5.1 Method

This consists of freezing the water contained in the soil using freezing probes sealed in boreholes passing through the area to be treated. Freezing is achieved by circulating within the probes either liquid nitrogen or brine produced by a refrigeration plant. The boreholes are fitted with probes made of two coaxial steel tubes connected by a special head, which enable the freezing liquid to circulate in the smaller tube and to rise through the annular space between the two tubes. The probe is thermally insulated along any length outside the treatment area.

Liquid nitrogen, which has a very low temperature (−196°C), freezes the water more quickly than brine (−55°C). The wall of ice is usually formed in 2 or 3 days using liquid nitrogen compared with 3 or 4 weeks using brine. Lower temperatures can be maintained in the soil when liquid nitrogen is used, which means that higher strengths can be achieved if necessary. Furthermore, the speed with which the ice wall is formed also prevents the phenomenon of water migration towards the ice under formation. This causes increases in volume of the frozen ground and can lead to heaving.

In some fine soils, freezing can destroy the structure of the soil and reduce its strength after thawing. The speed of formation of the ice using liquid nitrogen considerably reduces this. Note, however, that in some countries sufficient quantities of liquid nitrogen may not be available (e.g. the Hong Kong Mass Transit Railway). Freezing using brine is often the cheapest solution except where small volumes are to be treated.

A combination method, where the ice is formed with liquid nitrogen but the freeze is maintained with brine, is sometimes the best solution from both an economical and a technical point of view. This is particularly true in difficult soils or when the freeze is to be maintained over a long period of time. When water is circulating in the soil, prior treatment by grouting is necessary in order to avoid heavy loss of cooling power.

4.5.2 Application

Freezing makes it possible to give high strength to the soil. While being a function of soil type, this strength is essentially dependent on the temperature. Unconfirmed compression tests on samples can give strengths of around 2 MPa at −5°C and 8 MPa at −20°C. Over a relatively short period of time, the effective strength is, however, substantially reduced by creep. This needs to be taken into account in the design.

Due to its high cost, soil freezing is limited to applications where no other method can provide a satisfactory result. Such applications have been further eroded in the last decade as a result of the development of jet grouting.

The situations in which soil freezing remains applicable are essentially as follows:

1. Colluvium or alluvium made up of blocks of coarse gravel within a silty or clayey matrix. Grouting by injection cannot provide an improvement in strength and the blocks prevent the jet grouting techniques from breaking up the soil;
2. Silty or clayey soils under a high water head;
3. Temporary cutoffs which could cut underground flowpaths but not produce unwanted raising of the water table in the long term due to a damming effect[30];
4. Areas which are ecologically very sensitive;
5. In very fine soils where the volume to be treated is small and the freeze needs to be maintained for a short period only.

Due to its restricted field of application, soil freezing is little used compared with other methods. Examples of its application can, however, be found on numerous metro projects such as those in Paris, Stuttgart, Frankfurt, London, Oslo[28], Zurich[24], Brussels[25], Barcelona and Duisbourg[30] (see also Chapter 10).

4.6 Working levels

4.6.1 Drilling from below the water table

Unless the ground to be treated is cohesive and has low permeability (such as porous or slightly fissured rock), drilling from below the water table level will tend to produce a large ingress of water and eventually of eroded soil. To avoid this problem, the use of stuffing boxes at grout hole heads is necessary. Drilling operations are therefore significantly complicated. Furthermore, when this situation occurs at the tunnel face, a temporary concrete head wall is necessary. Similar complications may also occur:

1. When drilling from the ground surface into an artesian ground layer;
2. When treating from ground surface layers of soils in which excessive losses of compressed air are experienced.

4.6.2 Treatment from the tunnel face

Ground treatment carried out from the tunnel face is usually by successive steps. For reasons of drilling accuracy, the length of each step is normally limited to about 20 m.

Figure 4.5 shows a typical example of this kind of treatment on a section of Line D of the Lyon subway under the Fourvières hills. The treatment was limited to the

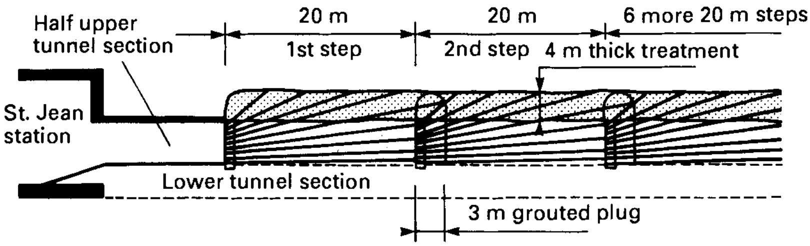

Figure 4.5 Longitudinal section of a typical ground treatment carried from the tunnel face, by successive 20 m steps. Line D under the Fourvières hills (Lyon Metro, 1987)

upper half section of the 10 m wide tunnel located in unstable soil ($K = 10^{-4}$ to 10^{-6} m/s) under a 20 m water head. The lower tunnel section located in consolidated fine sand ($K = 10^{-5}$ to 10^{-7} m/s) only needed some drainage and was excavated after completion and lining of the upper half section. Each treatment step was carried out through an 80 cm thick concrete head wall. Drilling lengths were 23 m for a 20 m step, allowing a 3 m ground treatment plug at the end of each step.

Apart from the drilling difficulties described above, treatment carried out from the tunnel face has other disadvantages:

1. Limited working space;
2. Greater difficulties of drilling horizontal or subhorizontal holes as compared to vertical ones;
3. Stoppage of excavation during ground treatment operations, resulting in possible standing time of either tunnelling or ground treatment resources;
4. Costly treatment as a result of the above difficulties.

Consequently, such treatment is usually carried out only when:

1. Either the depth is excessive (say, over 30 m); or
2. Access at ground surface is impossible; or
3. Soil freezing is used. In this case, drilling from the tunnel face minimizes drilling length, thus allowing better accuracy of freezing tube installation.

Some tunnel machines also have built-in grouting capabilities which allow treatment to be carried out over a few metres ahead of the face, through the machine front face. In practice, such arrangements can, however, only allow for grouting of the squeeze or compaction types using either bentonite/cement grout or quick-setting chemicals.

4.6.3 Drilling from the ground surface

Underground railway projects in urban areas are often located at relatively shallow depths, which makes it possible to envisage a treatment from the ground surface. When feasible, this is almost always preferred, since it avoids most of the difficulties connected with treatment from below the water table level described above. It also allows the ground treatment to be carried out in advance of and, therefore, independently of the tunnelling operations.

These advantages usually more than compensate for the obvious disadvantages:

1. Careful investigation and localization of possible underground utilities;
2. Temporary restrictions to surface traffic imposed by working platforms at street level;
3. An extra dead drilling length through overburden.

4.6.4 Drilling from galleries and shafts

Other, less common, solutions may be adopted:

1. Drilling from an access shaft, the difficulties in this case being similar to those encountered from the tunnel face;
2. Drilling from high-level pilot galleries, as at Auber Station – RER, Paris;
3. Drilling from an existing upper-level tunnel, as at Place Saint-Augustin – RER, Paris.

Table 4.2 Approximate comparative costs of ground treatments

Type of ground treatment (B/C = bentonite/cement)	*Price index[a] per m³ of soil* A^b	B^c
(A) GROUTING BY INJECTION		
Fissured rock, using B/C grouts only	100	270
Fissured rock, using B/C grouts with enhanced penetrability	130	330
Porous or microfissured rock, using silica gels	200	430
Open alluvials, using common B/C grouts only	230	600
Alluvials, using B/C grouts with enhanced penetrability	270	660
Fine soils, using microparticle mineral grouts	310	700
Fine soils, using silica gels and associated B/C grout	340	730
Very fine soils, using acrylic type resin	530	930
Very fine soils, using phenolic type resin	580	980
(B) JET GROUTING	670[d]	900[d]
(C) COMPACTION GROUTING	45	[e]
(D) FREEZING: initial phase of ice formation	950	1500
maintenance of the ice walls (per day and per cubic metre of soil)	35	35

[a] Index 100 is equivalent to 300 French francs as of 1 January 1989. Price index applies to one cubic metre of *in situ* soil treated.
[b] Column A corresponds to treatments from above the groundwater table: ground surface, or nearby gallery, 3 m minimum diameter, grout hole length not exceeding 30 m.
[c] Column B corresponds to treatments from below the groundwater table: tunnel face, pilot tunnels, adjacent gallery or shaft, grout hole length not exceeding 30 m.
[d] Depth not exceeding 20 m.
[e] Unusual case.

4.7 Costs

Table 4.2 shows costs for ground treatment in various conditions. The price indices are taken from specialist contractors. They are estimated averages and are to be used as such. These prices are worked out per theoretical cubic metre of soil treated, and include mobilization, drilling, grouting, materials and energy.

Two main types of working conditions are considered:

1. Drilling carried out from a level above the water table: ground level or possible existing high-level sizeable gallery;
2. Drilling carried out from a level below the water table: tunnel face, pilot tunnel or adjacent gallery.

These price indices highlight some typical cost aspects of ground treatments:

1. Costs of grouting by injection are higher in soil than those in rock. The main additional cost factors are: drilling using temporary casing, 'tubes à manchettes' installation, necessary multi-stage injections, higher percentage of voids to be filled.
2. The cost of grouting by injection increases significantly when soil permeability decreases. This is a direct result of the higher cost of more penetrating grouting products.
3. Grouting using cement grouts with enhanced penetrability and microparticle grouts is, in general, cheaper than using the traditional combination of bentonite/cement grouts and silica gels. These products, which appeared on the

European market in the mid-1980s, are tending to replace silica gels in the 10^{-5} to 5×10^{-4} m/s soil permeability range.
4. The jet-grouting price per cubic metre of soil treated is high when compared to using combined bentonite/cement grout and silica gels. In practice, the disparity of prices is, however, smaller, since in soil conditions where both methods are appropriate and competitive the necessary thickness of a jet-grout treatment is usually smaller than that of silicate grouting.
5. Freezing costs are very much time dependent. The price indices show that the maintenance cost of the freeze during a 4–6 week period is equivalent to that of initial ice formation. Consequently, in soil conditions where both freezing and resin grouting are applicable, freezing may have an economical advantage only in the case of short drives, where excavation and lining can be carried out quickly.
6. The price index of compaction grouting is low. The improvement provided by this method is not, however, comparable to the other above-ground treatment methods.
7. The cost of treatment carried out from a level below the water table is 50–150% higher than that of the same treatment from the ground surface. This results from the difficulties described above.

4.8 Applications

4.8.1 Tunnels

Free-air tunnelling
In unstable soils, mass ground treatments often provide adequate solutions for tunnels where the use of tunnel machines or compressed air is impracticable or where the cost of their mobilization and installation is disproportionate to the section length to be built. Figure 4.6 shows a typical example of such a situation. An 800 m section of the Caracas metro under San Martin Avenue consists of a 10 m diameter tunnel at a shallow depth in heterogeneous soil. The soil is made up of loose gravelly and silty clayey sands (permeability 10^{-5} to 10^{-6} m/s), overlying an uneven weathered schist substratum. Mass grouting was carried out using 'tubes à manchettes' and successive phases of bentonite/cement grout and silica gel. The systematic recording of drilling parameters facilitated the control of the treatment in very variable layers.

Compressed air tunnelling
Compressed air tunnelling methods may require complementary localized ground treatments:

1. At the start of a drive where a short free-air section is necessary for the placement of an airlock;
2. At a section end over a few metres' length when connecting with station walls;
3. Where air losses are anticipated;
4. Where the lack of overburden and the tunnel size do not allow the application of appropriate air pressure.

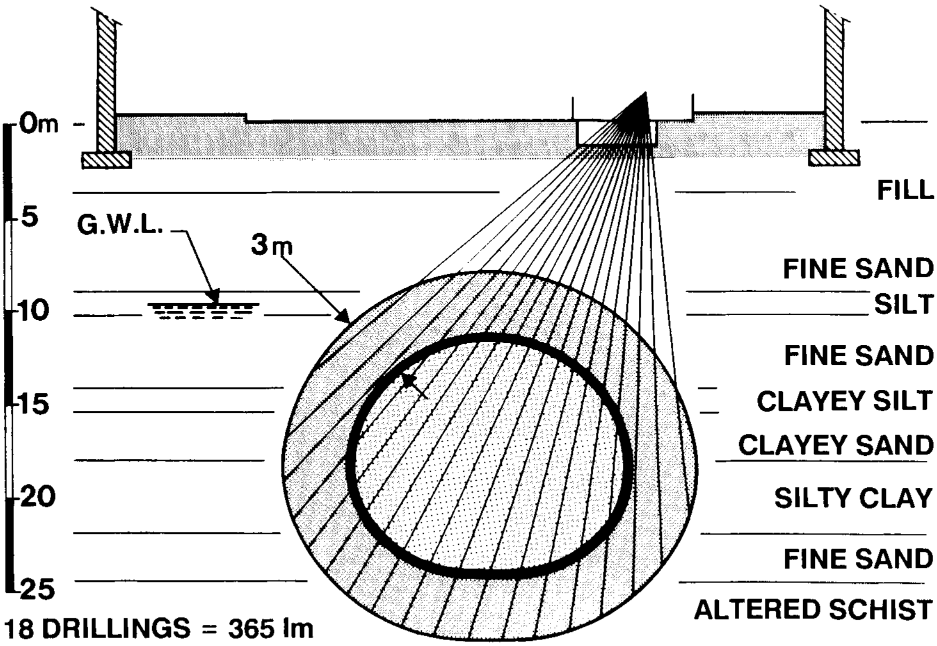

Figure 4.6 Treatment by grouting over a 800 m length for a 10 m diameter tunnel at a shallow depth, using combined bentonite–cement grouts and silica gels injected through 'tubes à manchettes' (Caracas metro, 1986)

Tunnel machines

When using tunnel machines, starter sections and end sections at station of construction shaft connections usually require localized soil treatments. Ground treatment of very loose soils prior to machine tunnelling may prove to be economical, as was the case on the Singapore Mass Rapid Transit System (Figure 4.7). Jet grouting treatment was used in connection with two superimposed tunnels located under Robinson Road in muds and very loose clays (SPT = 0–2). This prior ground treatment allowed a saving in the cost of compressed air which would otherwise have been necessary in conjunction with the tunnel machine used.

Access shaft

In adequate soil conditions, ground treatments of a cylindrical shape can allow the sinking in free air of hand-dug shafts lines with successive cast *in-situ* concrete rings. Where feasible, hand digging together with grouting may be attractive when:

1. The small diameter and the large depth of the shaft, the restrictions of the working platform or ground obstructions render uneconomical or impractical the use of sheetpiles, secant bored piles or diaphragm walls;
2. The thicknesses of the unstable layers are small in relation to the total shaft depth.

Figure 4.8 shows an example of ground treatment of the above type for an access shaft in the Hong Kong Mass Transit Railway. An interesting aspect of this

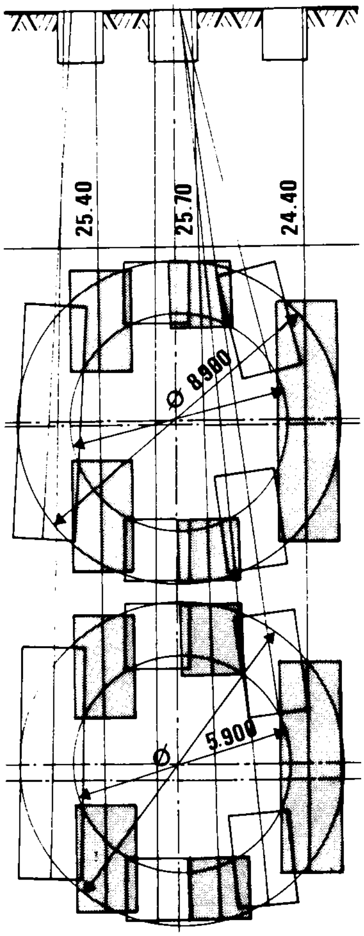

Figure 4.7 Jet grouting prior to machine tunnelling in mud and very loose clays (SPT = 0 to 2). (Robinson Road, Singapore Mass Rapid Transit System, 1985)

application is the combined simultaneous use of two different grouting methods:

1. Jet grouting, well adapted to the upper soft marine deposits layers, which was carried out first;
2. Silica gel grouting, very suitable for the lower completely decomposed granite layers, carried out after completion of the jet grout treatment.

Surface settlement

In soil, the most simple and common form of grouting used to reduce surface settlement is the injection of cement grout through preformed holes or valves incorporated into the tunnel lining. This so-called 'contact grouting' fills gaps and recompacts loosened soil near the lining contact. It must be carried out quickly and as close as possible behind the tunnel face and is usual in most tunnelling methods.

Consolidation grouting or compaction grouting may be used following tunnel excavation to recompress larger masses of destressed soil in a zone extending a few metres above the tunnel crown. Such grouting, however, may be difficult and disruptive to carry out from the tunnel and may be impractical from the ground

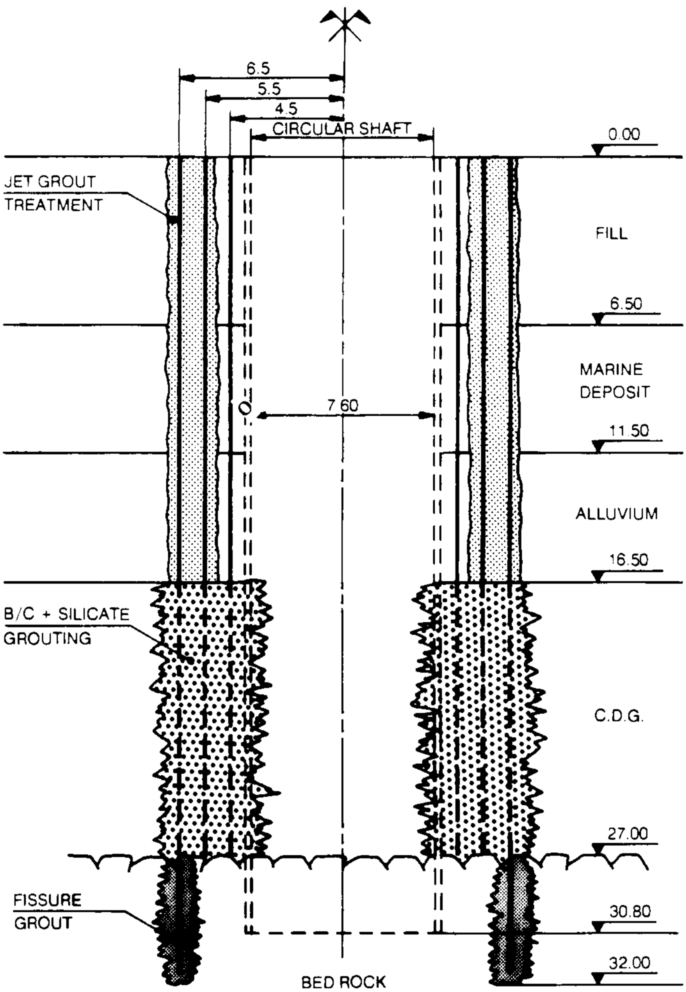

Figure 4.8 Circular hand-dug shaft sunk under the protection of a cylindrical ground treatment. Jet grouting is used in the upper marine deposits, while silicate grouting is carried out in the lower completely decomposed granite. Jet grouting was carried out before silicate grouting. (Contract 402, Island Line, Hong Kong Mass Transit Railway, 1984)

surface. An example of compaction grouting from ground surface used for this purpose can be found on the Baltimore subway project[21]. Where soil cohesion can be improved by grouting, ground treatment prior to tunnelling is an efficient means of reducing surface settlement.

Figure 4.9 shows an example of this on a 40 m section of the Duisburg Metro Tunnels located at a shallow depth below an existing building in Hedwigstrasse. Settlement was minimized using a ground-treated arch 1.2 m thick. Treatment was carried out through horizontal grout holes drilled from the tunnel face of the adjoining sections. Hard silica gels injected through 'tubes à manchettes' conferred a minimum crushing strength of 2 MPa to the treated alluvials. Tunnels were then

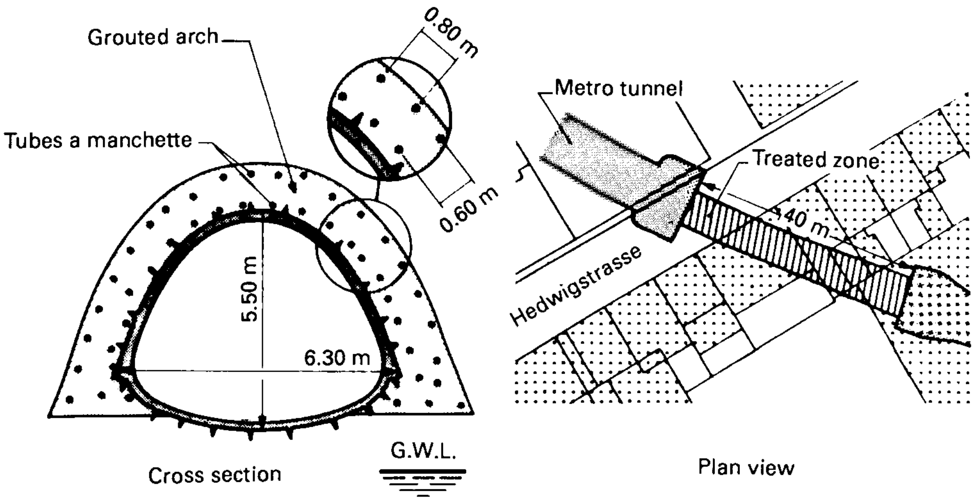

Figure 4.9 A grouted soil arch 1.2 m thick used on a tunnel section located at a shallow depth in alluvials below existing buildings. Hard silica gels and 'tubes à manchettes' were used. (Hedwigstrasse, Duisburg Metro, 1988)

excavated in stages of 1 m lengths which were immediately shotcreted after excavation.

4.8.2 Cut and cover

Injected raft

Where the side walls of a cut and cover tunnel cannot be keyed into an adequate impervious layer at a practicable depth, injected rafts may provide an economical answer. This was the case on the Cairo Metro, where a thick layer of pervious sands ($K = 10^{-3}$ to 10^{-4} m/s) was found below 10 m in depth (Figure 4.10). A grouted

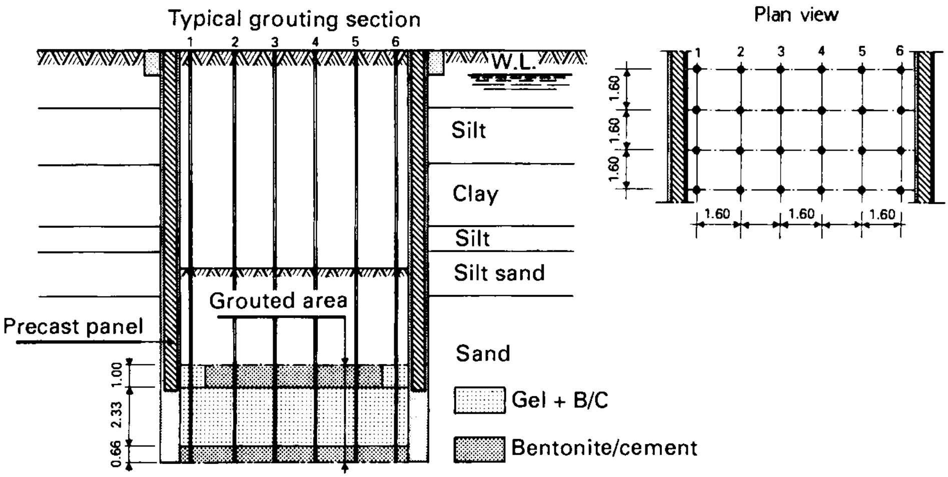

Figure 4.10 Typical grouted raft in sands, 3 m thick, used on the Cairo Metro cut and cover tunnels and stations, over a total length of about 4.5 km (1983–1986)

raft, 3 m thick, was used to close the box. 'Tubes à manchettes' on a 1.6 × 1.6 m grid were used with three successive grouting phases: (1) bentonite/cement grout; (2) silica gel; (3) a final squeeze with 'claquage' by bentonite/cement grout.

The cut and cover tunnel was subdivided into 60 m cells by temporary cross walls. The control of the treatment before bulk excavation was carried out by pumping tests inside each cell using pumping wells and piezometers. The method was used systematically on tunnels and stations over a total length of approximately 4500 m.

Groundwater cutoff

Linear ground treatments can be competitive for groundwater cutoffs compared with slurry walls, diaphragm walls or sheet pile walls, particularly where:

1. The soil is hard and/or includes obstructions, thus resulting in high-cost slurry walls and diaphragm walls, and unfeasible sheet pile walls;
2. A water cutoff extension at depth is necessary below the toe of a diaphragm wall or sheet pile wall;

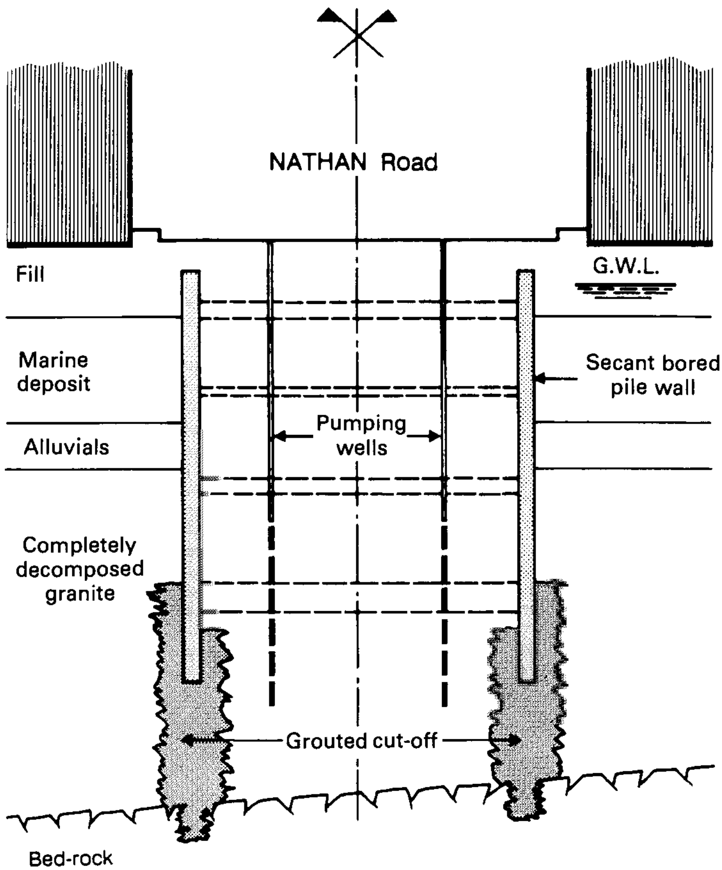

Figure 4.11 Secant-bored pile wall extended by a grouted cutoff down to the impervious bedrock, allowing the dewatering of the box without significant drawdown of the surrounding water table. (North Nathan Road, Hong Kong Mass Transit Railway, 1978)

3. The layer to be sealed off is covered by a thick layer of dry or watertight overburden.

Figure 4.11 shows an application of grouted cutoff on a North Nathan Road station of the Hong Kong Mass Transit Railway built in 1978. On this cut and cover section, extending the secant bored-pile side walls at depth into hard, completely decomposed granite was not feasible. The secant-pile walls were thus extended down to the impervious granite bedrock using a grouted cutoff. The grouting was carried out using combined bentonite/cement grout and silica gels injected through 'tubes à manchettes'. Pumping wells allowed dewatering of the box without a significant drawdown of the surrounding groundwater table.

Diaphragm wall underpinning
Where the bedrock level is higher than the designed raft level it is often economical to underpin a diaphragm wall toe instead of deepening it at great cost into hard rock. To do this, localized ground treatment behind and beneath the diaphragm wall toe is necessary to allow safe excavation below the toe.

Improvement of passive reaction at the toe
In very soft soils, such as muds or soft clays, the design of structural retaining walls may be complicated due to the low available passive soil pressure at the lower part of the walls. In this case, the calculated large inward displacement of the wall toe during bulk excavation leads to designing walls with large inertia in conjunction with heavy propping.

An economical solution may consist of a ground treatment which provides soil strengthening at the inner face of the structural wall just below the designed formation level. Figure 4.12 shows a jet grouted slab which temporarily propped the diaphragm walls at a depth in Newton Station on the Singapore Rapid Transit System.

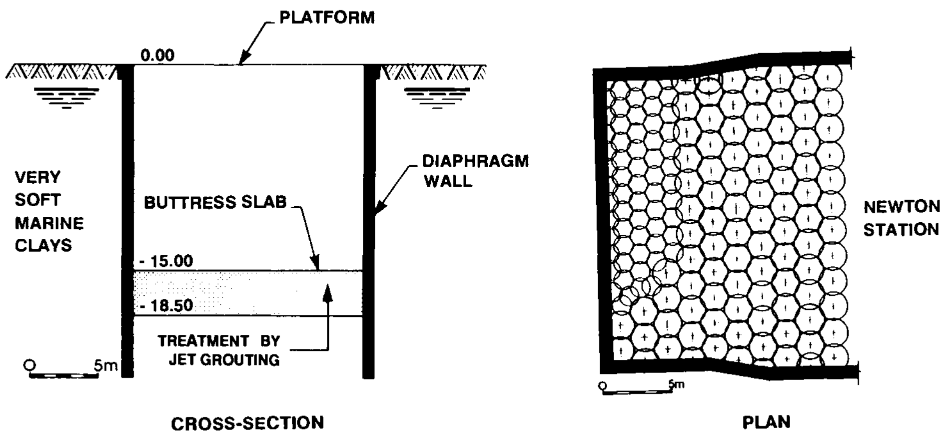

Figure 4.12 A jet grouted strutting slab in soft marine clays (SPT: 0–5, cohesion: 10–40 kPa, water content: 50–100%). This grouted slab allowed bulk excavation without toe displacement of the instrumented diaphragm walls. (Newton Station, Singapore Mass Rapid Transit System, 1984)

4.8.3 Underpinning

Where prior underpinning of adjoining sensitive structures is necessary, ground treatment can provide a solution, as described by Greenwood [2].

4.9 Environmental aspects

The possible toxicity of grouting products is a primary concern with respect to the environment. Some chemicals (i.e. acrylamids and products including heavy metal molecules such as chrome-lignin) are now banned because of their toxicity, although they have proved efficient from an engineering point of view.

Toxicity is not, however, the only concern regarding safety and the environment. Products may also be irritating, noxious, corrosive, inflammable or explosive, hence requiring protection for the workforce. A report on this subject has been published by CIRIA [11]. The grout itself may be subject to long-term decomposition due to soil agressivity and change in pH induced in the ground may also be of concern. Defective workmanship is another factor when considering pollution. The spilling of products on ground surfaces, the uncontrolled discharge of water when cleaning grouting pumps and pipes, mixing errors resulting in incomplete setting and uncontrolled grout losses through preferential ground paths are all potentially harmful.

In each particular project, the pollution risks must be assessed in relation to the sensitivity of the environment. The presence of underground water circulation, or the proximity of groundwater pumping wells for industrial or domestic use, lakes and rivers, are all very relevant to such a study. However, sodium silicate, the most common type of low-viscosity chemical grout, has been extensively used in the urban context of more than fifteen metro schemes, including those in London, Paris, Vienna, Hong Kong, Cairo and Caracas. There have been no reports of incidents of significant pollution or environmental damage from any of these.

Nevertheless, caution is advisable regarding temporary ground treatments where saving on silica gel costs may lead to the use of mixes with a low reagent content or a low-cost reagent. The cause of concern lies in the lack of stability of such gels and their limited capacity to neutralize the soda completely. This type of gel may, therefore, introduce into the ground a certain amount of free soda and an unwanted basicity. When in doubt, the use of mixes with a high soda neutralization ratio is recommended.

Resins of the acrylic, phenolic and polyurethane types are stable products after polymerization or chemical reaction. Consequently, they are considered safe in non-agressive soils.

The possible injection of unmixed components due to faulty workmanship remains a hazard with almost all chemical grouts. It is, however, a relatively controllable one. Polyurethane is an exception in this respect, since it reacts with groundwater.

The growing general concern for the environment has led to the development of mineral grouts with enhanced penetrability such as ultra-fine cement and microparticle mineral grout. These are tending to replace silica gels when soil porosity permits.

In the context of pollution, jet and compaction grouting are perfectly satisfactory, since their potential polluting effects are limited to the well-known one of cement. Ground treatments share with other underground works the risks of cutting off natural underground water flows, thus creating the possibility of a 'dam effect' where such flows exist.

4.10 Heave control

4.10.1 Heave monitoring

All ground treatments, with few exceptions, tend to produce heave. Systematic heave monitoring arrangements are therefore necessary. Levelling surveys, made at time intervals adapted to the type of ground treatment used, are adequate when the expected heave is unlikely to result in serious consequences. When working close to sensitive structures such as live railway lines permanent alert systems capable of detecting any heave in excess of predetermined safe values must be installed. Settlement gauging may consist of deep boreholes equipped with steel bars grouted in their bases. A simpler settlement gauging system may consist of a rotating laser beam encountering a network of photosensitive cells covering the area treated.

4.10.2 Factors producing heave

Grouting by injection

Uncontrolled fracturing is one of the most common causes of heave, since large grout hydrofractures (or 'claquages') act as jacks which, if fed with large quantities of grout, can produce extensive heave, even using relatively low grouting pressures. Permeation grouting without hydrofracturing can also, to a lesser extent, produce a heave caused by a local increase in pore pressure. This is explained by the fact that the pressure applied to the grout is, in turn, transmitted to the groundwater which is expelled from the treated zone. Cases of systematic decreases in the measured heave during stoppages of grouting work have been observed on large treatments and are evidence of this. The range of heave on a very large mass treatment can be of the order of 20–50 mm.

When adequately monitored, heave can be controlled to a certain extent by adjusting the grouting parameters. The parameters influencing heave during grouting are:

1. Rate of grout flow;
2. Distance between simultaneous injection points;
3. Limitation of grout take at each successive grouting phase;
4. Total amount of grout.

A common error when heave occurs is to regard only the grouting pressure as the cause. This often leads to a reduction of pressure limits to a point where the impregnation and microhydrofracturing cannot take place effectively.

Jet grouting

The jet grouting process, in principle, does not require pressure to be applied to the wall of the cavity in which the soil mixing takes place. However, the head produced by the continuous upward flow to the surface of surplus dense soil-mix creates a pressure significantly higher than the piezometric head. Pressurizing of the cavity under treatment is thus unavoidable. This phenomenon, which may produce heave, can be made worse by head losses and blockages of the upward soil-mix flow caused by collapses of the grout hole walls.

Heave of 300–500 mm was found in some locations where various jet grout treatments were used on the Singapore Rapid Transit System (1984–1985). In order to reduce heave produced by jet grouting, it may be necessary to pre-drill large-diameter cased holes at each jet-hole location to ensure the continuous upward flow of the soil–cement mix to the surface. The use of air facilitates this upward flow. In extreme cases, where practical, a reduction in jet column diameter may provide a solution.

Compaction grouting
Since compaction grouting is based on *in-situ* soil displacement, an amount of heave must be expected at some stage in the process. Heave monitoring is therefore an essential part of the quality control procedure.

Freezing
As mentioned in Section 4.5.1, heave may be produced by an increase in soil volume due to water migration towards the ice under formation. Fast freezing using liquid nitrogen tends to reduce the extent of this.

4.11 Soil investigations

4.11.1 Preliminary investigations

This subject has been purposely placed near the end of the chapter since a review of problems connected with the various treatment methods will lead to an appreciation of the prime importance of adequate preliminary investigation.

It is impossible to define a standard soil-investigation programme. The extent and type of an appropriate investigation will depend on the availability, quality and quantity of existing soil information and the complexity of the project and of the local geology.

A soil-investigation programme based on core sampling and *in-situ* and laboratory testing should specify and confirm the characteristics of each layer: particle size distribution, fissures, permeability, water content, compressibility and strength. Surveys based on, for example, seismicity resistivity may sometimes provide more global information where applicable.

A careful groundwater study is necessary, and large-scale pumping tests are often informative regarding overall permeability. Water and piezometric levels and possible variations are measured using a network of stand-pipes and piezometers which must be monitored for a sufficient length of time. Chemical analysis of groundwater samples at various depths and locations, essential for determining the suitability of construction materials, also makes it possible to detect potential grout unsuitability. When necessary, an assessment of possible groundwater flow and gradient must be made, since such flows can affect or be affected by the treatment processes (washing away of grout or freezing energy, giving a damming effect).

A detailed survey of underground utilities, tunnels and adjoining basements is an obvious requirement when designing drilling patterns. A survey of adjoining structures, their type of foundation, their age, possible incidents during or after their construction and existing fissures are all elements which enable accurate judgements to be made on possible heave effects or on the necessity of some form of underpinning.

Close-centre investigation holes are highly recommended, specially in areas of variable or heterogeneous layers or those containing cavities or obstructions such as boulders.

4.11.2 Drilling parameter recording

Recent developments in drilling parameter recording methods[32, 33] bring a solution to the necessity of close-centre investigation. In principle, the method consists of drilling holes using high-production rigs equipped with gauges and transducers which allow the continuous recording of various drilling parameters such as (Figure 4.13):

1. Speed;
2. Load on drilling tool;
3. Rotation speed;
4. Flushing fluid flow;
5. Percussion vibrations.

The values recorded are stored in a computer memory. Immediate readout of the measurements is available through the use of microcomputers, provided that the correlations of soils characteristics have been previously established. The high

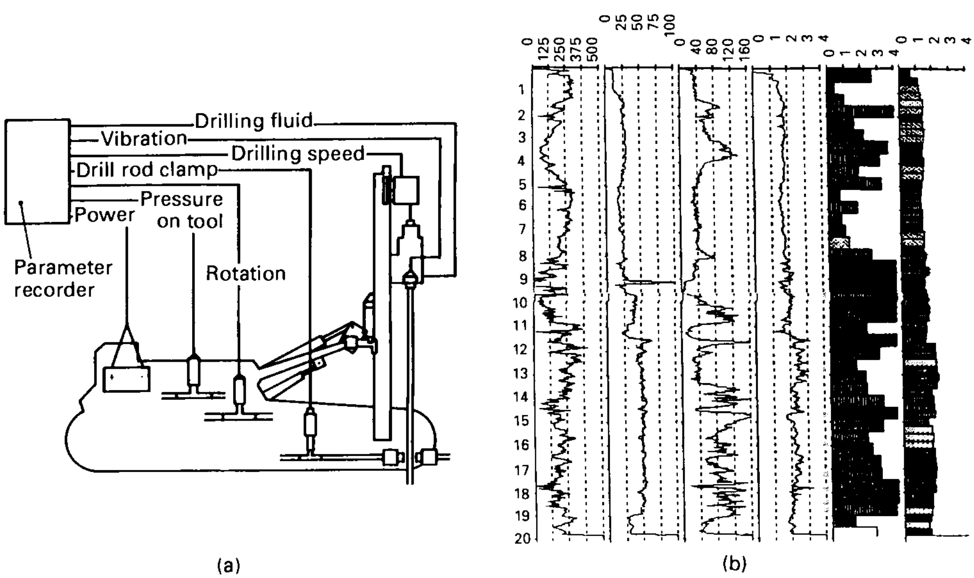

Figure 4.13 Drilling parameter recording. (a) Typical diagram of drilling parameter recording on an hydraulic drilling machine; (b) output of recorded parameter. (Borehole 2296 L, Caracas Metro, 1986)

production rate of such rigs (of the order of 100 m/shift/rig) allows a large number of probe holes to be carried out quickly at a reduced cost with rapid results.

It would be therefore wise to test this type of destructive method of soil investigation during the preliminary investigation stage. Where clear correlations with cored boreholes can be established such a method will provide the possibility of more accurate soil profiles at limited time and cost.

This type of investigation will also be very valuable during the carrying out of soil treatments. The use of rigs equipped with drilling parameter recording capabilities when drilling the grout holes during the grouting process provides an initial survey of the complete treatment zone at no additional time or cost.

Where a split-spacing grouting method is adopted, the drilling of secondary grout holes can give significant information on the first phase of treatment and guidelines for the second. This method was used extensively during the grouting for the Caracas Subway.

4.12 Large-scale grouting trials

In cities such as Paris or Hong Kong, with extensive experience in ground treatment among clients, engineers and specialist contractors, large-scale grouting trials have become unnecessary. Such trials are, however, of great value when and where there is little or no local experience concerning the soil layers encountered at the location.

In the absence of such trials, the feasibility and efficiency of ground treatment methods, assessed from results of soil investigation, may just become a subject of unproductive disputes between experts. Furthermore, the cost of large-scale trials can easily be justified by the magnitude of an underground railway project.

4.13 Contractual aspects

The specialization of the above techniques, their constant evolution and sophistication, the necessity of rapid adaptation to ground variations during the course of the works call for flexibility in contractual relations and specification plus, at times, a climate of confidence between client and specialist. When contracts are based on remeasurement, as is often the case, a very detailed bill of quantities, including representative unit prices and covering a wide variety of contingent works, will be beneficial to these relations and, consequently, to the project.

Too-rigid specifications can be a handicap to successful treatments. Contract documents imposing, among other things, unstable cement grout instead of stable betonite/cement grout, arbitrary low pressure limitations, closed grout circuits or lengthy application of refusal pressures are potential causes of failure of a ground treatment.

Unfortunately, this controversial subject is not covered by internationally recognized documents. The recommendation for grouting works published by the AFTES [1] is, to date, one of the few publications covering the subject of grouting by injection at length.

4.14 Conclusions

As we have seen, soil treatment can, in many cases, provide an economical technical solution. The methods described should therefore not be considered as long and expensive rescue operations. Rather they are methods of construction that should be planned well in advance of the project as an integral part of it. The feasibility of the different methods is hence to be studied at a very early stage with extensive preliminary subsoil investigation and, if needed, full-scale tests.

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Chapter 5

Track for underground railways

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Contents

5.1 Introduction

Railway tracks in the open generally comprise a system of welded rails laid on cross sleepers of either timber or concrete which, in turn, are supported by stone ballast on a prepared formation. There are other track systems used in the open, but these tend to be for special applications.

Track supported by ballast has formed the basis of the railway system for at least 150 years, and so there is great experience of track design, installation and maintenance. It can be very tolerant to deficiencies in track maintenance standards as well as to vehicles in poor running condition.

Maintenance techniques have reached a high standard of development associated with mechanization of most of the operations. Ballasted track has been and continues to be used successfully for underground railways and is particularly applicable in the case of reinstatement of track in tunnels in which the route had been previously closed.

However, there are many reasons why traditional ballasted track is not always the best choice for an underground railway. Its greatest diasadvantage is the considerable construction depth required under the rail foot. This, in turn, controls the size of the tunnel bore and hence tunnelling and spoil-removal costs.

Track construction depth, and hence tunnel bore size, can both be reduced significantly by replacing the ballast by a structural slab, which in some cases can also incorporate the sleepers. Rails are attached to the slab via an elastic support system. The support systems vary in form, depending on the configuration of the tunnel invert and traffic type.

A feature of most track systems is the need to incline the rail seat areas such that each rail is inclined inwards towards the track centreline. The amount of this incline depends on the vehicle tyre profile, but is typically 1 in 20. On curves, the complete track is rotated such that the outer rail is raised with respect to the inner.

For both ballasted and non-ballasted track special measures may be required to reduce noise in tunnels and ground-borne vibrations transmitted into adjacent buildings. This can considerably complicate the track structure and require the use of significantly greater tunnel bores.

In view of the above, track type can have a very significant influence on the design of an underground railway system, especially the size and form of the tunnel. Therefore, it is important that the design of the track should be finalized at a fairly early stage of the railway design procedure.

However, there can be specific cases in which tunnel design and size can be dictated by local conditions, and hence track design would need to be specifically configured to suit these particular cases.

5.2 Ballasted track

The design principles of ballasted track in tunnels are generally based on those for tracks in the open air. However, a significant difference is that the ballast can be laid on a rigid concrete invert rather than a built-up granular formation. There are exceptions when track in the open is constructed on a rock formation. In this latter case design constraints can be similar for both open-air and tunnel tracks.

Either timber or concrete sleepers may be used, but concrete sleepers are generally preferred, especially if there is either a suitable precast concrete factory in the neighbourhood or good-quality timber is not easily available. Concrete sleepers may be either prestressed concrete monobloc construction or reinforced concrete twin-block.

In the case of concrete sleepers, rails may be clipped directly to the sleeper using an elastic fastening with a resilient pad interposed between the rail foot and sleeper rail seat. Alternatively, metallic baseplates may be bolted onto the sleepers and the rail, in turn, be clipped or clamped to the baseplate. With timber sleepers, baseplates are normally installed, especially if softwood sleepers are used. It is, however, possible to have direct-fastening systems similar to those used for concrete sleepers.

In general, it is preferable to have a flat surface to the invert which is also parallel to the axis of the sleepers, onto which ballast is laid. In small-diameter tunnels the width of the invert may be such that it is necessary to use comparatively short sleepers, as standard length sleepers could foul the curved portion of the tunnel lining.

This implies that, generally, ballasted track is not necessarily compatible with circular-bore tunnels unless the bore diameter is significantly increased. Hence rectangular section tunnels are more compatible with ballasted track, and the track can be designed and constructed to standards similar to those used in the open air with the same track components. Rectangular section tunnel inverts are flat, apart from the necessity of providing falls to drains and so give a good formation onto which to lay ballast.

Track should be designed such that mechanized maintenance procedures may be used, and normally this requires the provision of a minimum of 200 mm of ballast under the sleeper. At least 250 mm should be provided if concrete sleepers are used on a concrete invert, otherwise attrition of the ballast will occur between these two hard surfaces.

Long welded rails are well suited to underground railways due to the reasonably stable air temperatures in tunnels compared with the open air. The possibility of track buckling is virtualy eliminated and small-radius curves may be used safely. However, it is essential that the temperature at which rails are clipped down is specified so that high-compression rail forces are avoided.

Care must be taken that rails are electrically isolated from one another for signalling purposes and that both rails are insulated from earth if DC traction operation is used. This latter requirement is to reduce the risk of reinforcement corrosion in the tunnel structure or corrosion of metallic pipes carrying services.

5.3 Non-ballasted track

Non-ballasted track is a system in which track ballast is replaced by a rigid structural support, usually made of concrete and often forming part of the tunnel invert. Ballast elasticity is normally simulated by elastomeric pads inserted between the underside of the rail foot and the support system.

This type of track has been used in both the open air and in tunnels, but is especially suited to tunnel application. It has been employed since the earliest days of underground railways, and a notable example of early use of non-ballasted track is in the tube lines of London Underground Ltd.

Tracks without ballast have been the subject of much study and development, and today many proven systems are available. However, there is considerable variation in complexity and costs of such tracks, and this is especially so in the case of the rail-fastening system.

A major objective of this development has been to design a track structure whose construction depth between the underside of the rail foot and the tunnel invert is significantly less than an equivalent design of ballasted track. Further objectives have been concerned with the ease and rate of construction within tunnel confines as well as reducing to a minimum the need to maintain track level and alignment.

There are essentially three types of ballastless track in use:

1. A concrete slab or longitudinal plinth bonded to the tunnel invert. The rails are fastened to the slab using either resiliently mounted baseplates or some form of attachments which hold the rails directly onto the slab. A resilient pad is interposed between the underside of the rail foot and the slab. The slabs or plinths may be precast outside the tunnel and then bonded to the tunnel invert. Alternatively, the slabs may be cast *in situ* (Figures 5.1 and 5.2).
2. The use of an *in-situ* formed concrete slab into which either precast concrete sleepers or blocks are incorporated. These sleepers may either be rigidly cast into the base slab or have some resilient materials between the sleepers and the slab. Rails can be fastened directly to these cast-in sleepers using traditional fastenings. Alternatively, baseplate systems can be used.
3. Precast or *in-situ* concrete slabs or beams which are resiliently supported on the tunnel invert. Rail fastenings may be similar to the systems used for the other types of track. This track is normally restricted in use to areas which may be affected by vibrations transmitted through the ground. (Applications of the system are dealt with later.)

5.3.1 Rigid slabs directly coupled to the tunnel invert

For most applications an *in-situ* reinforced concrete track slab is cast directly against the tunnel invert. Shear connectors should project from the invert into the concrete slab to ensure that relative movement cannot occur between the slab and the invert. Such movement may cause localized attrition of the contact surfaces. An

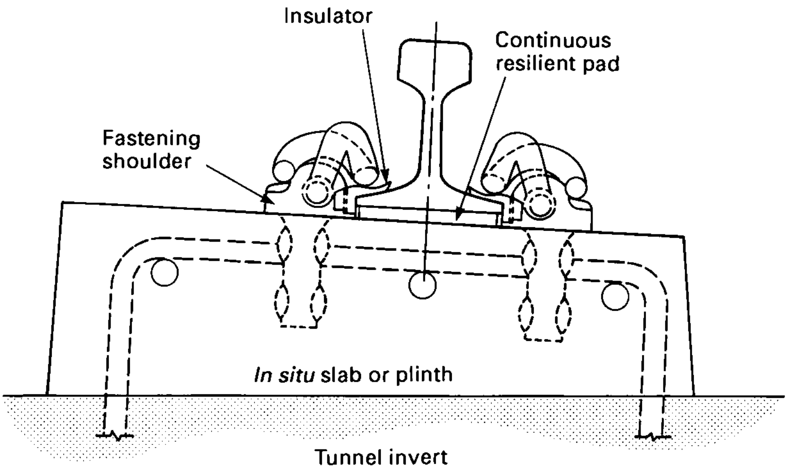

Figure 5.1 Continuous-rail support system

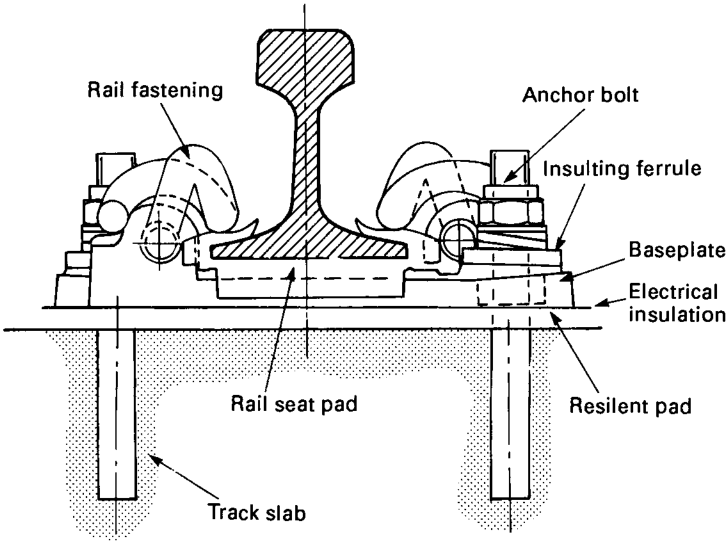

Figure 5.2 A typical baseplate

alternative application is to use precast concrete base units set on the invert with bitumen emulsion cement or cement grout.

The rail-fastening system supporting the rail on the track slab is normally designed to give track-stiffness properties similar to those which would be expected from well-maintained ballasted track. This normally means a rail head deflection of between 1 mm and 2 mm under the most frequently occurring axle loads. However, it is important that the system is capable of carrying the heaviest expected axle load without damage.

Baseplates in cast iron or steel are inserted between the rail foot and the slab and normally carry two resilient pads, one underneath the baseplate, the other under the rail foot. The primary resilience may be provided by either one or both of these two pads (Figure 5.2).

Baseplates are located on the track slab by two or more bolts which are either glued into holes in the slab or cast directly into the top surface of the slab. These bolts pass through the holes in the baseplates and provide a positive lateral and longitudinal location of the baseplates with respect to the slab. They also accommodate the vertical movement due to wheel loading. In order to prevent fretting between the bolts and the baseplate due to this movement the baseplate holes are lined with nylon or other polymeric ferrules.

Baseplates are held down onto the track by screwing down nuts on the anchor bolts. Resilient washers or helical springs are usually inserted between the nuts and the baseplates. This spring permits a degree of controlled vertical movement without the baseplates losing contact with the baseplate pads.

The nylon ferrules can be made with the bolt holes eccentric with respect to the ferrule axis such that rotating the bush in the baseplate hole moves the baseplate laterally with respect to the slab. This movement can provide a degree of gauge or alignment adjustment. Rails are fastened to the baseplates by one of the commonly available elastic rail clips similar to those used on ballasted track.

For most applications there is a requirement that the rail be electrically insulated from the slab. With baseplates the insulation can be in two stages, i.e. between the rail foot and baseplate and between baseplate and slab.

In the former, insulation is achieved by use of a resilient pad (which is made of insulating material) under the rail foot and an insulator between the rail fastening and the rail foot. In the latter an insulating pad and the nylon ferrule can provide adequate electrical insulation.

As an alternative to the use of baseplates, the rails may be fastened directly to the track slab with only a resilient pad between the underside of the rail foot and the slab. This pad may provide either continuous support to the rail or may be in discrete lengths located at the rail-fastening positions. In both cases a standard elastic rail fastening is used with the shank of the fastening body glued into the base slab.

Electrical resistance of the rail to earth is provided by a combination of the resilient pad and an insulator between the fastening and the rail foot.

5.3.2 Sleepers cast into track base slab

As an alternative to producing a slab and then adding independent fastenings, sleepers may be cast directly into the track base slab. These can be manufactured outside the tunnel and supplied to site with a complete rail-fastening system. A major advantage of this encasing process is that the sleepers provide a positive and accurate fixing of the track gauge as well as fixing the inward tilt of the rails. The level of the *in-situ* concrete need not be accurately controlled and there is no need to drill holes in the base slab.

For normal applications timber, monobloc concrete and twin-block sleepers may be used. However, with timber sleepers problems can arise due to long-term shrinkage of the timber, causing the sleepers to work loose in the concrete.

Monobloc sleepers may be of either prestressed or reinforced concrete. The strength of the sleeper need only be designed to cater for handling up to the point of encasement. After having been cast in, the sleeper is intended to behave integrally with the *in-situ* concrete.

A very important feature of cast-in sleepers is the need to ensure that they are firmly bonded into the base concrete to make a structurally integrated system. To achieve this, reinforcement is left protruding from the sleepers, and this reinforcement is incorporated into the *in-situ* concrete. Also hoop reinforcement should be provided around the sleepers.

Additionally, it can be worth providing an exposed aggregate finish to the sleeper surfaces which are to be embedded. The *in-situ* concrete will bond well to these surfaces. Concrete twin block sleepers should be dealt with in a similar way and, in addition, the tie bar linking the blocks can be cast in.

An alternative to casting sleepers rigidly into the base slab is to provide a resilient layer between the precast sleeper and the *in-situ* concrete. This is dealt with in more detail in a later part of this chapter.

It is important to provide adequate resilience between the rail foot and the sleepers, and a resilient rail seat pad at least 10 mm thick may be required. Alternatively, baseplates similar to those described for rigid slab bases may be mounted on sleepers prior to casting in. Comments regarding electrical insulation apply equally to embedded sleepers.

5.4 Resiliently mounted tracks for vibration isolation

A major problem with underground railways in urban areas can be the transmission of ground-borne vibrations, due to wheel-to-rail contact, into adjacent buildings. This can be especially acute when railways pass close to flats, concert halls or theatres, when the rumble from trains becomes obtrusive. This rumble arises from vibrations travelling through the ground and exciting various components in a building structure, and these structural vibrations can radiate sound. There is generally no problem from noise generated in a railway tunnel being heard in adjacent buildings apart from when building basements are close to the running tunnels.

One of the best ways of controlling the generation of ground-borne vibrations is to ensure that the train wheels are smooth, concentric with respect to the axles and do not have defects such as wheel flats (Figure 5.3). Similarly, the track should be smooth and without corrugations or discrete irregularities such as dipped welds or joints. However, no railway system is perfect, and some or all of the defects may be present.

Other features of the track can contribute to the generation of ground-borne vibrations, but experience has shown that one of the best ways of controlling transmission of these vibrations is to provide adequate elasticity to the support of the track system.

An increase in track support resilience reduces vibration generation by several processes, but probably the more important ones are that dynamic wheel-to-rail contact forces are reduced, as is the basic natural frequency of the vehicle's unsprung mass when coupled to the track spring. This reduction in natural frequency ensures that attenuation of ground-borne vibrations occurs at frequencies above $\sqrt{2}$ multiplied by the system's natural frequency.

Therefore an increase in track mass and a reduction in its support stiffness reduces the system's natural frequency and hence the generation of ground-borne

Figure 5.3 A wheel flat

vibrations above certain frequencies. This track mass may be augmented by increasing the mass of the track components which are situated above the track spring. The resiliently mounted components of the track mass can be considered as being coupled to the effective unsprung mass of the vehicle.

For most practical applications, the vehicle's unsprung mass per axle plus the spring-supported track mass associated with this axle and supported on the track's resilient support system may be considered as a one degree of freedom mass-spring system. This assumption is normally adequate for calculating the system's natural frequency, i.e.

$$f_n = \frac{1}{2\pi}\sqrt{\frac{K}{M}}$$

and the frequency above which ground-borne vibrations will be reduced:

$$f^a = \frac{1}{(\sqrt{2})\pi}\sqrt{\frac{K}{M}}$$

where K is the track support stiffness and M is the vehicle's unsprung mass plus the coupled sprung track mass.

5.4.1 Vibration isolation of ballasted track

It has been demonstrated above that a decrease in the track stiffness and an increase in the track mass which is resiliently supported could reduce the frequency above which attenuation of ground-borne vibrations occurs. In the case of ballasted track this can be accomplished in four ways:

1. Supporting the complete ballasted track on reinforced concrete trays which, in turn, are resiliently carried with respect to the tunnel invert by rubber spring blocks;
2. Providing a resilient mat between the ballast and the invert;
3. Installing resilient pads to the underside of the sleepers such that the sleepers and rails are resiliently supported;
4. Using very resilient pads between the rail and sleepers.

5.4.2 Ballasted track in resiliently supported concrete trays

A system which effectively provides a standard ballasted track system but has excellent vibration attenuation characteristics is one in which the ballasted track is carried on resiliently mounted trays. These trays are of prestressed concrete and are supported on the tunnel invert by rubber springs. In view of the large track mass comprising sleepers, rails, ballast and concrete trays, it is possible to produce a system having a comparatively low natural frequency and hence is able to attenuate ground-borne vibrations down to low frequencies.

The use of ballasted track means that noise levels in the tunnels can be reasonable and standard track components may be used. However, the depth required for trays and rubber springs means that construction depth below the rail foot is significantly greater than for just ballasted track. Hence the cost of the tunnel is greater and the cost of trays and springs has to be added.

5.4.3 Resilently mounted track – ballast mats

Ballast mats are made of resilient materials which are installed between the underside of the ballast and the top surface of the tunnel invert. They can be manufactured from a range of materials, but, to be successful, should be capable of providing rail head vertical deflections of at least 3 mm when loaded by axle loads which are typical for the route. The environmental conditions between ballast and invert are severe and specially designed and manufactured mats are required.

Sharp pieces of ballast can penetrate the surface of the mat and 'short circuit' the resilient support. Hence either the mat must be intrinsically tough or the resilient layer should be protected by a tough facing material on the top of the mat. The invert surface should be smooth to ensure good bedding down of the mat. However, because of the large area of mat which supports the ballast, nominal contact pressures between the mat and the ballast are comparatively small.

In some tunnels the invert surface is always wet because it can perform a primary function of directing water into the drains. Under these conditions ballast mats can be permanently saturated. Also, any spillage of oil or chemicals is likely to find its way through the ballast onto the mats. It is essential therefore that the dynamic behaviour of the mats should not be affected by these conditions. Additionally, they should not be degraded physically.

Good attenuation of ground-borne vibrations can be achieved because the effective mass which is carried on the resilience provided by the mats is large. In this case the mass of the ballast can be added to the sleeper and rail mass to give the effective total track mass. An additional advantage of ballast mats is that they protect the invert and ballast from attrition between the ballast and invert, and in some cases it is possible to reduce the ballast depth as a consequence. A major disadvantage of ballast mats is that they can be comparatively expensive due to the need to cover the large area of the invert. Another arises because the mats are buried under the ballast. It is therefore very difficult to inspect the mats without digging out panels of ballast down to the mats. This can be difficult to arrange in an intensively operated railway system.

If it is found to be necessary to change ballast mats because either they have not been properly matched to the dynamic environment or they have been physically degraded it will be necessary to lift out the rails and sleepers and remove the ballast. The ballast mats can then be changed and the track restored. Due to the comparatively large movements of 3 mm which take place in the ballast, the track may require some months before it settles down. Meanwhile, some fettling of level may be required.

5.4.4 Resiliently mounted ballasted track – soffit pads

Soffit pads are installed on the underside of sleepers and can either cover the whole or part of the underside of timber or concrete sleepers. Pads are available for monobloc or twin-block sleepers.

Soffit pads have the advantage that they reduce track support stiffness without significantly changing other track properties. Normally the pads provide an elastic deflection at the rail head of between 2 mm and 3 mm when loaded by the standard traffic of the route, but greater deflections may be provided if the track is adjacent to sensitive buildings.

Generally, vibration insulation by soffit pads is not quite as good as that provided by ballast mats because the resiliently supported mass does not include the ballast.

The main difference is that lower frequencies are not attenuated to the same degree as with ballast mats, but at higher frequencies the attenuation is very similar.

Installation is straightforward. Soffit pads can be attached to the sleepers outside the tunnel and the rails and sleepers installed as either standard track panels or as individually placed components. The environment under sleepers is hostile and the pads are loaded to significantly higher contact pressures by the ballast than in the case of ballast mats. The water and chemical environment is not so severe because most liquids will drain through the ballast. Because of high contact pressures, very tough facing material is required to prevent indentation and penetration of the resilient material by the ballast stones. However, soffit pads reduce hard contact between sleeper and ballast, and in the case of concrete sleepers can reduce attrition between ballast and sleepers. Therefore probably ballast depth will be reduced. Track experience indicates a possible reduction in the frequency of track-maintenance cycles compared with standard concrete sleeper track. If it is necessary to change soffit pads this can be done without disturbing the ballast, and so is a more straightforward operation than is the case for ballast mats.

It is conceivable that both soffit pads and ballast mats may be used in environmentally sensitive areas, but care must be taken to investigate the extra degree of freedom which the additional resilient layer introduces.

5.4.5 Resiliently mounted ballasted track – resilience under rail foot

Another method which can give a reduction in track stiffness is to introduce low-stiffness resilient pads directly under the rail foot. The pads are significantly softer than would be required for normal train operation. However, these softer pads are not generally as effective as ballast mats or soffit pads in reducing ground-borne vibrations because only the rail mass may be considered to be coupled to the vehicle's unsprung mass.

There is also a practical limit to the acceptable live load deflection of the pad, partly because there are limits to the deflection of the pad to give it long life and partly because large deflections can cause problems with the rail-fastening system. These problems can be fretting between metallic components and fatigue of the fastenings.

In some instances the lateral movement of the rail head can become excessive due to lateral wheel-to-rail forces, especially on curved track. Baseplates on sleepers can be used to improve track resilience without some of the objections listed for rail seat pads. In this case a standard rail seat pad can be used between the rail foot and the baseplate, while a thicker and more resilient pad can be installed between the baseplate and the sleeper. Increased resilience under the rail foot absorbs high-frequency impact forces from wheel flats but has little effect on the transmission of low-frequency ground-borne vibrations.

5.4.6 Non-ballasted track – floating slabs

The floating slab is the non-ballasted equivalent of ballasted track on resiliently mounted trays or ballast mats. Floating slabs usually comprise prestressed concrete slabs supported on the tunnel invert by either discrete rubber springs or on resilient mats. In the latter, grout is used under the slabs to ensure uniform bedding of the resilient mat to the invert and the slab. Slabs can vary in length from 5 m down to one or two sleeper pitches. Rail fastening to the slab is similar to the systems used on the standard rigidly supported slabs.

Floating slabs can also be cast *in-situ* directly onto a resilient mat, and one consequence is that longer slabs may be used than for precast slabs. With floating slabs, attenuation can be good down to comparatively low frequencies because of the large coupled masses comprising the slab plus the vehicle's unsprung mass on the supporting springs. However, care needs to be taken to avoid resonant conditions which can be generated by the vehicle in the slabs and can give rise to a signifiant increase in noise levels which can be radiated from the surface of the slabs.

Floating slab systems are expensive because the slab construction depth is generally greater than is required for rigidly supported slabs. Provision has also to be made for the rubber springs.

5.4.7 Non-ballasted track – resiliently mounted blocks

An intermediate stage in effective vibration isolation compared to the floating slab system is to mount monobloc or twin-block sleepers resiliently onto a rigid track slab base. Alternatively, individual resiliently mounted blocks under each rail may be used. To provide this resilience, a flexible rubber or rubber-bonded cork pad is attached to the underside of the block or sleeper either by being glued or placed in a rubber boot which envelops the lower part of the sleeper (Figure 5.4). The track is assembled with rails attached to the sleepers and is positioned for line and level. Pockets are provided in the track slab into which the sleepers are positioned. The sleepers are then grouted into the pockets.

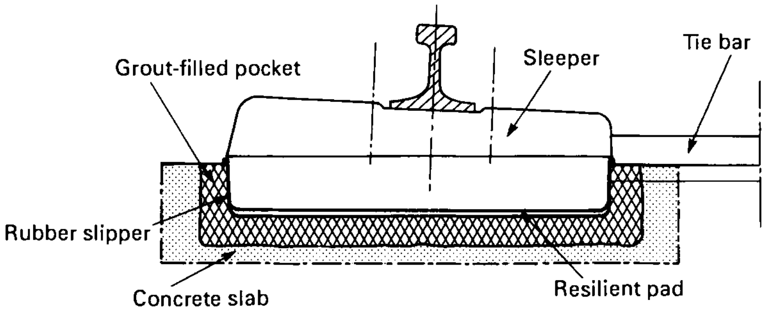

Figure 5.4 A resiliently mounted twin-block sleeper

An alternative system is to use a pourable elastomeric compound which either incorporates the resilient pad or can be used in place of the pad (Figure 5.5). Resiliently mounted block track normally has baseplates for rail attachment to the blocks.

Track deflections of between 1 mm and 2 mm may be achieved, but vibration isolation is not normally as good as can be obtained by the floating slab track, especially at lower frequencies. Costs are generally less than for floating slab track, especially if standard sleepers are used. Construction depth can be slightly less.

5.4.8 Non-ballasted track – resiliently mounted rails

The limitations regarding the degree of resilience which is practical using standard baseplates have been given previously for the case of ballasted track. Generally,

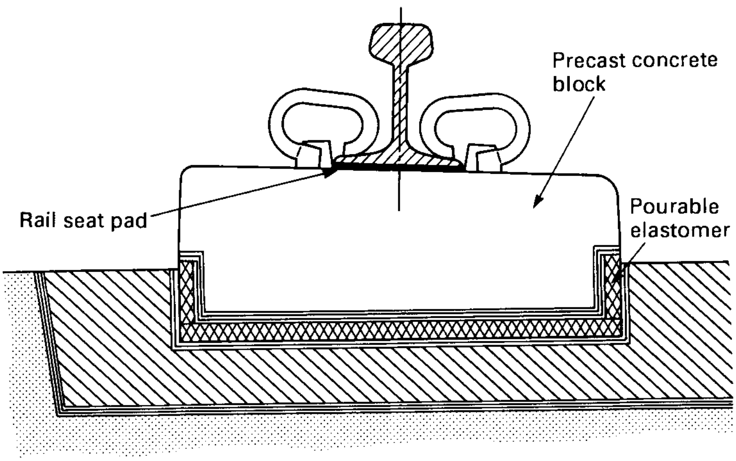

Figure 5.5 A resiliently mounted concrete block

similar limitations apply when baseplates are used on rigidly mounted slabs. There have been some attempts to overcome these limitations, and one of the more successful designs has been the 'Cologne Egg' (Figure 5.6). In this case the rail is fastened to a baseplate which is supported on a lower plate by a rubber block acting in shear. Although the rubber block is comparatively flexible in shear, it is very stiff in compression. This high compressive stiffness of the rubber gives high lateral stiffness to the rail support. Hence it is possible to obtain a comparatively large vertical deflection of the rail while having an acceptably low lateral deflection.

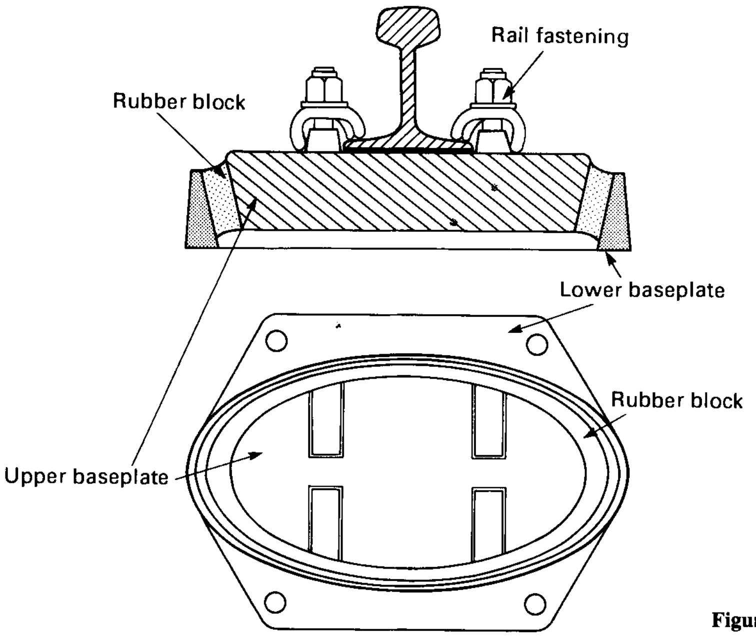

Figure 5.6 A 'Cologne Egg'

5.5 Switches and crossings in tunnels

Most of the above comments relating to plain line apply equally well to the installation of switches and crossings. However, it is very important, especially in environmentally sensitive areas, to ensure that any impact loading due to wheels traversing the crossing nose is kept to a minimum. This may be accomplished by good design of the crossing area using a manganese steel casting for the crossing. An alternative is to have a swing nose crossing which ensures continuous support to the wheel when on the crossing.

All methods of track support may be used, including those relating to vibration isolation. However, in the case of the more resilient systems care must be taken to ensure that large deflections do not interfere with the normal operation of the switches and detector gear or overstress the running rails.

5.6 Summary of advantages and disadvantages of track types

5.6.1 Ballasted track

Advantages

1. It is possible to restore line and level to the track using either simple hand tools or tamping and lining machines.
2. The ballast bed can provide a degree of acoustic absorption to reduce the noise in tunnels which emanates from wheel-to-rail contact.
3. Damaged or worn components may be easily renewed.
4. A range of proven techniques is available for reducing transmission of ground-borne vibrations.
5. Ballasted track is compatible with most vehicles.

Disadvantages

1. A track construction depth under the rail to the invert of at least 400 mm is required.
2. It is more compatible with rectangular section tunnels than with circular bores.
3. Drainage can be complicated, especially in wet tunnels. This can become a problem if the ballast becomes choked with slurry.
4. Track possessions are required for maintenance of track line and level. Considerable skill is required to keep the absolute position of the track correct. This is important where clearances between trains and tunnel walls are small.
5. Large quantities of ballast need carrying into the tunnel. It can be difficult to lift and carry out contaminated ballast.
6. Derailments can cause considerable damage to sleepers, especially if prestressed monobloc or reinforced twin-block concrete sleepers are used (Figure 5.7). Ballast needs to be up to the top surface of the sleepers to provide a degree of protection against derailed wheels. It may be necessary to use derailment-constraining devices.
7. Ballast does not provide a controlled resilience to the track structure because ballasted track tends to stiffen with respect to time and traffic loading. Tamping will generally restore some of the original resilience.
8. Considerable expense can be incurred if anti-vibration systems have to be incorporated.
9. An independent walkway may be required for detraining passengers in the event of an emergency.

Figure 5.7 Derailment damage to a concrete sleeper

5.6.2 Non-ballasted track

Advantages

1. A construction depth shallower than required for ballasted track may be attained. Hence smaller-sized tunnels are feasible.
2. Routine track maintenance can be reduced significantly, especially maintenance of rail level and alignment.
3. Transmission of ground-borne vibrations into adjacent buildings may be controlled by the incorporation of resilient material between rail foot and the tunnel invert.
4. Drainage inside the tunnel is reasonably straightforward and should require little maintenance.
5. Controlled support is given to the rail which should reduce the incidence of rail fractures.
6. Forms of non-ballasted track are available to suit all tunnel types and cross sections.
7. Derailment damage to the track can be minimized by suitable designs.
8. Slab design can provide a good walkway in the event of having to detrain passengers between stations.
9. Hard flat surfaces are easy to clean by train-mounted track cleaners.
10. The relative position between running track and electrical conductors is easy to maintain.

Disadvantages

1. Most non-ballasted track systems tend to be noisier than the equivalent ballasted track. This is because concrete is acoustically 'hard' compared to ballast.
2. Carrying and handling large quantities of wet concrete in tunnels can cause logistic problems during construction.

3. Great care is required in setting out and constructing the trackwork because subsequent adjustment may not always be feasible.
4. Reinforcement in the concrete track support system requires protection against earth leakage currents with DC traction.
5. Basic track costs are generally greater than for equivalent ballasted track.
6. Expensive and complex track systems may be required if high standards of isolation against ground-borne vibration are a necessity.

5.7 Alternative systems to steel wheels on steel rails

Although steel wheels on steel rails form the majority of underground railway systems, there is one principal and two potential alternative contending systems. The main contender uses rubber-tyred vehicles on a track system which provides both support and steering guidance. Surface rapid transport systems have also been developed in which the vehicles have been suspended magnetically. There are also monorail systems. However, neither of these two systems have yet been developed for underground application.

Both the rubber-tyred and magnetic levitation systems have been developed to produce a quieter operating system than steel wheel on steel rail, as well as reducing ground-borne vibrations. However, good modern orthodox track and vehicles can achieve satisfactory levels of quietness.

Steeper track gradients and increased train acceleration and braking are also possible with the alternative systems, but these increases cannot be achieved in practice because of the increased power requirement and the effect on passenger comfort and safety.

5.7.1 Tracks for rubber-tyred vehicles

A range of different vehicle types has been developed for rubber-tyred applications and these vary from full-sized bogie vehicles similar to normal metro cars (RAPT) down to small four-wheeled cars at Lille, which are operated fully automatically. Track design for this range is basically similar, with either steel running rails or longitudinal concrete beams to carry the rubber tyres. Support for these running rails can be provided either by cross sleepers in ballast or by concrete sleepers cast into base slabs. Steering guidance is normally provided by horizontal rubber-tyred wheels which bear on lateral guidance rails.

In the case of RAPT track (Figure 5.8) standard flat-bottom rails are fastened inside the rubber-tyre running rails. The flat-bottom rails provide two functions.

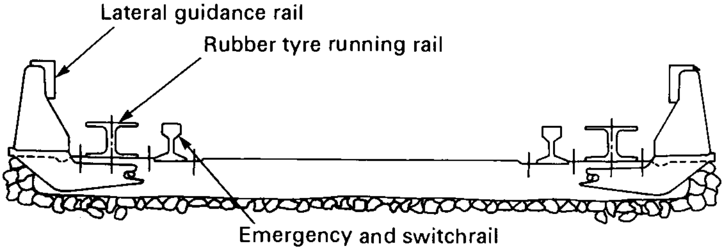

Figure 5.8 A rubber-tyred system (RATP, Paris)

First, in the event of a tyre puncture, a standard type of flanged wheel will come into contact with the rail and provide support for the train to continue its journey. Second, switches and crossings in the rail provide a switching function when the standard rail rises with respect to the running rail such that support is transferred to the standard rail. The train then operates as a standard-wheel vehicle through the switch and crossing.

Switching of the Lille system is achieved by additional horizontal rollers centrally located under the car which engage in a central switching rail, while in the Sapporo system both guidance and switching is accomplished from a central rail.

5.7.2 Monorail tracks

There are basically two types of monorail, those in which the vehicle body is suspended from a single beam and those in which it sits astride or straddles a single-spine beam. This beam is usually elevated. In modern monorails, the main spine beam is made in reinforced or prestressed concrete, while the carrying and lateral guidance wheels have rubber tyres. The principles of carrying and guidance are similar to some of the more orthodox rubber-tyred vehicles. Switching is normally accomplished by the flexing of a steel beam which replaces the concrete spine beam. Due to the relatively unobtrusive nature of the slim single-spine beam, monorails are especially suited to areas where aesthetics are important. Monorails have not so far been used underground.

5.7.3 Tracks for magnetically levitated vehicles

The principle of magnetic levitation has been applied to several types of trains, ranging from the 355 km/h West German Transrapid system down to the 45 km/h Maglev system at Birmingham Airport.

There are basically two types of support possible for the vehicle, magnetic attraction and repulsion. However, the most significant work has been done with the former.

In magnetic attraction, DC electromagnets are carried on the underside of the vehicle and these are attracted to laminated steel plates on the supporting structure (Figure 5.9). This magnetic attraction provides the support to the vehicle as well as

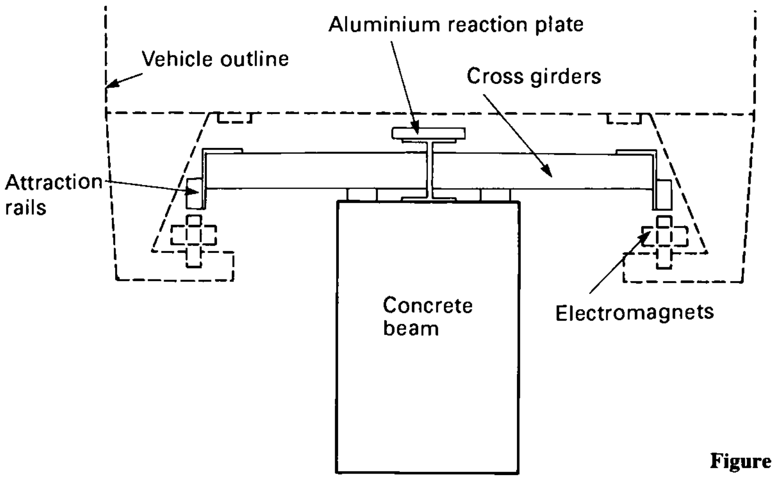

Figure 5.9

its suspension. However, this system is basically unstable, and it is necessary to have a control system for the electromagnets to keep a constant air gap of 15 mm between the electromagnets' pole pieces and the laminated plates. The same magnets provide lateral guidance.

Because there is no friction between the vehicle and track, traction and braking forces are generated by a linear induction motor carried by the vehicle and reacting along the track against an aluminium and steel reaction plate. Emergency braking is achieved by dropping the vehicle onto skid pads on the track. The vehicle has pads which engage with the skid plates. The same skid plates come into use in the event of a power or vehicle system failure.

High-dimensional track standards are required to ensure that the vehicle magnets do not normally make contact with the laminated plates. The track must also cope with the vehicle dropping onto the skid plates when both stationary and moving.

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Chapter 6

Cut and cover design and construction in reinforced concrete

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Contents

The design and construction of underground railways in cut and cover embraces a wide and varied range of foundation engineering techniques[1–3]. Fundamental to the art of foundation engineering is the need for the experience and understanding of three main areas of expertise – structural design, soil mechanics and construction methods[3,4]. Design cannot be separated from construction[5,6]. For urban sites cut and cover methods in vertically retained excavation are used almost exclusively (in contrast to construction in open cut with self-supporting slopes). The successful design and construction of such structures, particularly in constricted urban conditions, can present some of the greatest challenges to the civil engineer. Cut and cover construction is frequently used to commence or connect sections of bored tunnelling or where shallow depth or layout does not suit the latter. Where surface space is not critical, cut and cover (as opposed to permanent open cut) is still sometimes adopted – for example, to satisfy environmental or stability requirements[7]. While direct costs are major factors in the selection of construction methods, the effects of disruption from surface works can be dominant considerations, leading to a preference for bored tunnelling, particularly in urban environments[8]. The challenges presented to the designer are usually compounded by the need to address and incorporate a wide range of input from other disciplines. While typical design interfaces are listed, this chapter concentrates on reinforced concrete design in the context of its interrelationship with soil/structure interaction and construction methods.

6.1 Form and layout

6.1.1 General

The form and layout of reinforced concrete cut and cover structures associated with underground railways vary considerably. However, in conjunction with functional requirements, ease of construction is important. Simplicity of form and layout should be sought wherever practicable. Complicated shapes or voided sections, which ostensibly offer cost savings by minimizing materials, may not be economic in practice. They can be onerous to construct with consequential effects on programme and cost. Their presence may also create or exacerbate stress concentrations leading to serviceability or durability problems. Underground structures should be robust and require minimal maintenance.

Another major consideration of form and layout is the compatibility with different types and sequence of adjacent construction. Significant changes in cross section, foundation type or ground conditions require special consideration. Construction of adjacent or connecting sections at different times, particularly with regard to excavation and backfilling sequences, may also create incompatible settlements. It is not unusual to have various forms of cut and cover structures juxtaposed (Figure 6.1). For example, major variations in cross section are typical between stations and running tunnels. In all cases the time-dependent effects of ground movements during both the temporary and permanent conditions will need to be assessed.

6.1.2 Stations

The location and layout of stations are principal aspects of underground railways[9]. These two factors are influenced by operational requirements and

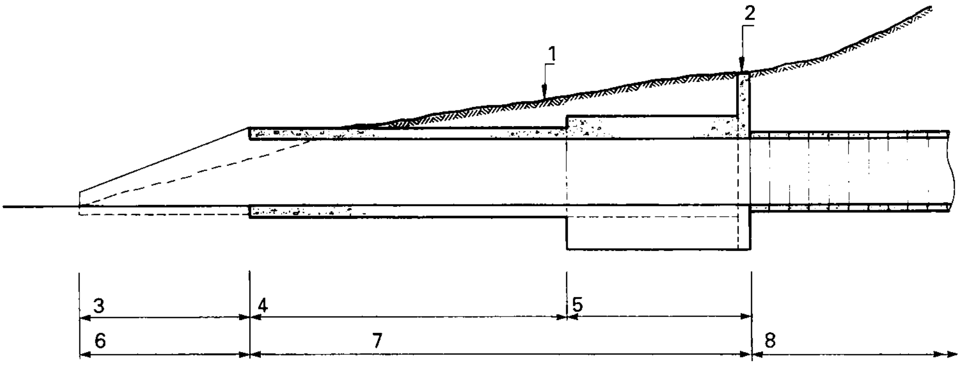

Figure 6.1 Variation in construction methods. 1 Original ground level; 2 temporary portal; 3 cantilever retaining walls; 4 box section in open excavation with self-supporting slopes; 5 box section in vertically retained excavation (e.g. with diaphragm walls); 6 open section; 7 cut and cover construction; 8 bored tunnels

constraints to the alignment of the running tunnels. It is desirable to make the depth of construction as shallow as possible[10]. This helps to minimize ground movements, time and cost of construction and the length of user access to and from street level. Shallow depth is also conveniently compatible with operational requirements, since less traction energy is expended and therefore heat generated with stations located between the tops of vertical curves. Trains approaching the station thus decelerate upgrade and conversely accelerate downgrade on leaving. Such a vertical alignment also facilitates maximum use of bored tunnelling in urban conditions. See also Chapters 2 and 3.

The designer will need to identify interfaces and incorporate input from them (see Section 6.7.2). Early in the design development, primary considerations are depth, spans, ground conditions and methods of construction. The last includes the nature of construction of adjacent works. The layout of the running tunnels, in particular, influences the general cross section. Twin-bored tunnels, immediately adjacent to a station, are usually associated with centre platforms. Layout with side or stacked platforms typically involves continuation with cut and cover tunnels. Typical arrangements of cross-sectional layout are illustrated in Figure 6.2. Deep and complex station construction is involved for those at intersections serving more than one line.

6.1.3 Running tunnels

The layout of cut and cover running tunnels is governed by the spatial relationship of individual tracks and the need to incorporate facilities such as crossovers and turnouts. Consequently, the reinforced concrete design and construction takes many varieties from simple boxes to large multi-span structures.

Portal frames
Portal frames may be constructed in battered excavation with strip footings or in retained cut using embedded walls. They are generally adopted for large spans (i.e. for two or more adjacent tracks) and where the trackbed formation provides adequate support without a structural base slab. Foundation design aspects for both

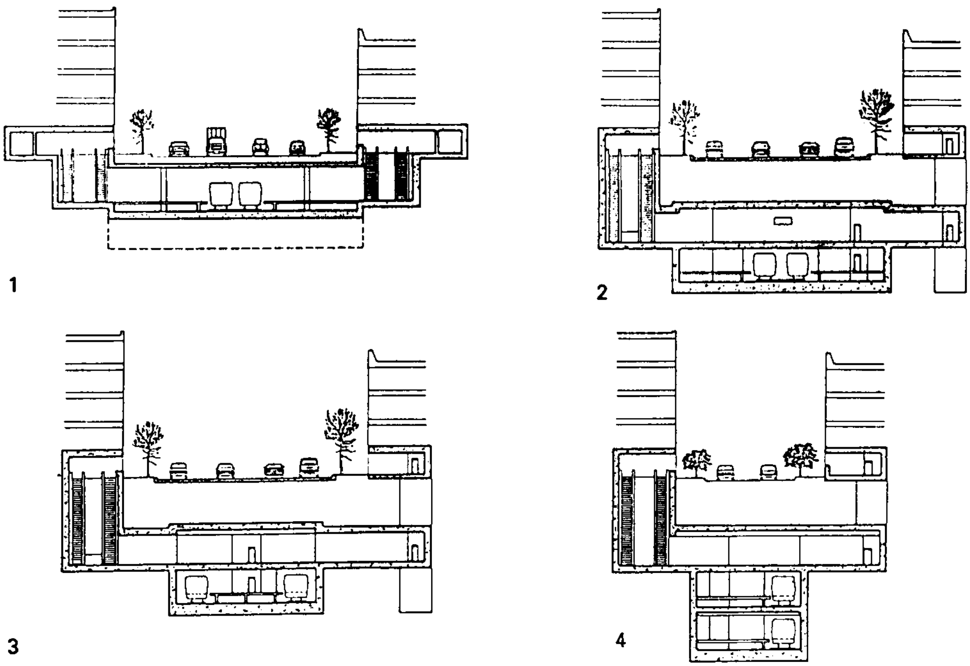

Figure 6.2 Typical cross-sectional layouts of stations. 1 Side platforms/shallow cut and cover; 2 side platforms/deep cut and cover; 3 centre platforms/deep cut and cover; 4 stacked platforms/deep cut and cover

the structure and the trackbed are discussed in Section 6.4. For strip foundations it is often desirable to limit the outward projection of the heel. This limitation must respect the requirements of allowable bearing pressure and settlements, but will help to reduce excavation and backfill quantities. Tapered walls assist in this process by reducing bending moments at the base (Figures 6.3 and 6.4). This structural arrangement, however, does add some complexity to the formwork, particularly with varying internal heights and spans and construction to small-radius curves.

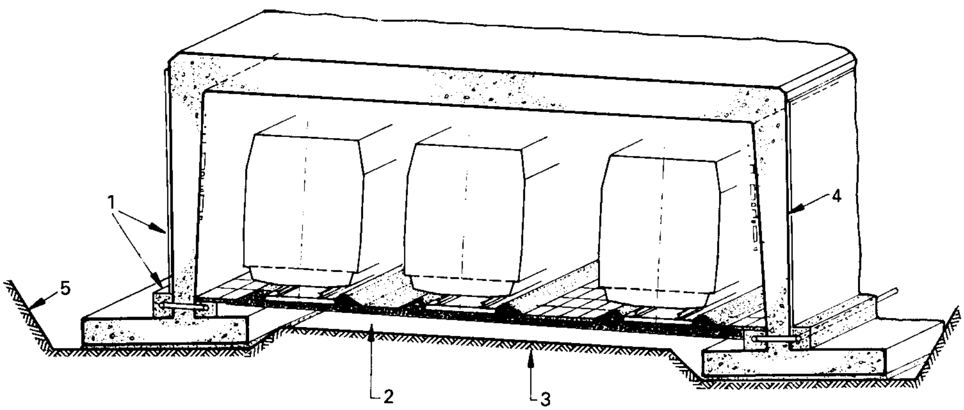

Figure 6.3 Portal frame construction. 1 Wall drainage (connected to trackbed drainage system); 2 trackbed; 3 sub-base; 4 tapered side walls; 5 temporary cut slope

Figure 6.4 Terminal loop cut and cover tunnels, Channel Tunnel, UK. (Note secant pile walls in the left foreground and tapered intermediate wall)

Box structures

Box structures are the most common form of cut and cover construction. The use of embedded walls for the permanent structure, as will be noted in Section 6.2.2, is becoming increasingly adopted for construction in vertically retained excavation. The soil/structure interaction relevant to such construction methods tends to create quite different design and detailing requirements compared to boxes formed completely with formwork (Figure 6.5). The differences are most marked when the base slab is suspended between embedded walls rather than being ground bearing. Suspended base slabs may be adopted in conjunction with permanent systems of peripheral support. The foundation conditions that determine their adoption are discussed in Section 6.5.3. Box structures with ground-bearing slabs may also be formed with embedded walls but are more generally constructed in battered excavation or utilizing temporary support such as sheet piling.

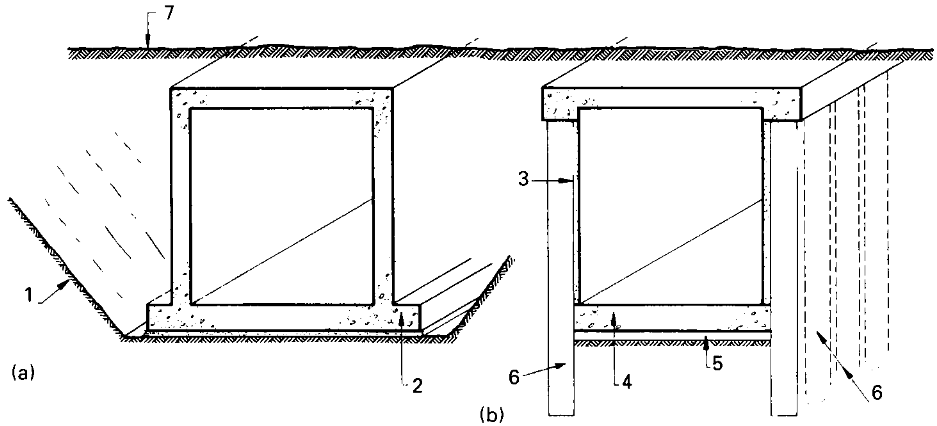

Figure 6.5 Cut and cover box structures. (a) Sloped excavation – walls formed with formwork. 1 Temporary cut slope; 2 outstand (may be used to increase resistance to flotation). (b) Vertically retained excavation – permanent embedded walls (formed from surface before main excavation). 3 Lining; 4 base slab (suspended if void incorporated); 5 void (optional); 6 contiguous piles (alternatives: diaphragm walls or secant piles); 7 original ground level

In battered excavation, boxes are more stable to construct than portal frames. The base slab of a box provides more restraint for the walls to cantilever than the strip footings of a portal frame. This factor may be exploited to allow early backfilling to the sides of a box prior to completion of the roof slab. The box form is also more robust in the permanent condition, being far less susceptible to sway effects. However, a principal consideration for box structures is stability against flotation. This applies both during construction and to the permanent condition. The resistance to uplift is potentially critical prior to completion of backfill above the top slab – particularly if construction is delayed at this stage. The dead loading of the box is then a minimum but groundwater levels could still be high. This is normally overcome by provision of appropriate drainage measures during construction. Drainage to the sides of the permanent structure is also generally desirable, but maintaining the factor of safety against flotation in the permanent condition by drainage is not recommended. An onerous reliance would then be imposed on the continuing effectiveness of the drainage system. Inspection and maintenance of the system are then vital requirements. It is preferable to incorporate more positive methods of control. Tension piles may be used where it is not practicable or economic to increase the direct dead weight of the structure. The factor of safety against flotation should not be less than 1.1 [11]. This lower bound factor should only be used where the maximum possible groundwater level can be established accurately and be based on the minimum dead weight of the permanent structure. The assessment should allow for possible removal of the trackbed and reduction of other imposed loads such as depth of backfill.

Cut and cover tunnels may also create a barrier to existing groundwater flow. This will include the potential effects on adjacent structures as well as uplift on the tunnel.

Where two forms of box structure interconnect, consideration of the compatibility of founding conditions will be necessary, as highlighted above in Section 6.1.1. Figure 6.6 illustrates a possible structural arrangement to overcome the potential discontinuity in foundation stiffness.

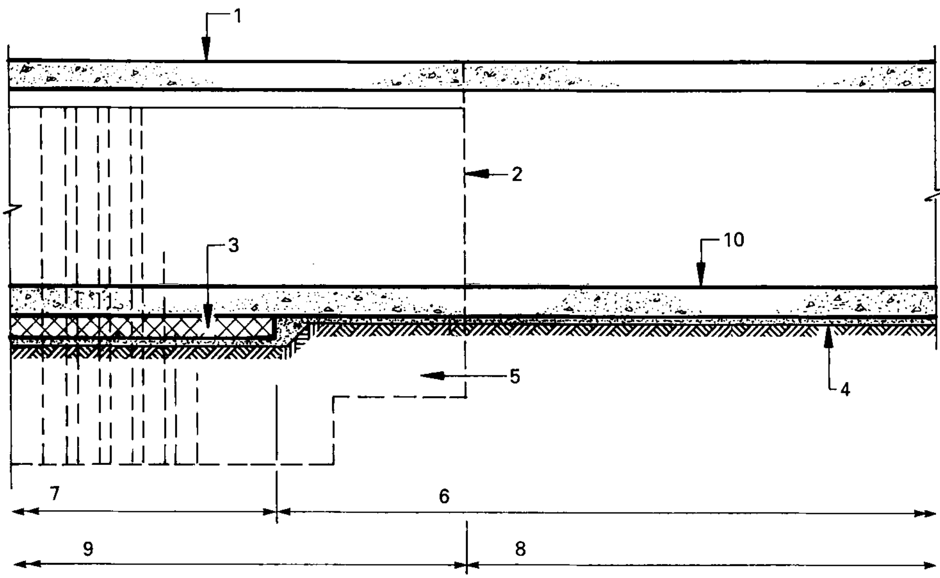

Figure 6.6 Different forms of abutting box construction. 1 Roof slab; 2 construction joint (must be made fully continuous); 3 void; 4 blinding; 5 possible reduction in wall embedment adjacent to change in construction; 6 ground-bearing base slab; 7 suspended base slab; 8 sloped excavation; 9 vertically retained excavation – embedded walls; 10 base slab

6.1.4 Ventilation shafts

The requirements for the design and construction of cut and cover ventilation shafts are similar to those for stations and tunnels. With cut and cover stations the associated ventilation shafts are simply incorporated into the overall scheme (see Chapters 2 and 22). The situation is somewhat different for shafts connected to bored tunnels. However, the requirements are still analogous to those associated with deep excavations for cut and cover tunnels particularly when the shafts are built in advance of the main sections to provide access for tunnelling machines. Line ventilation shafts between stations are often exploited for this purpose.

However, in the case of very deep ventilation shafts to bored tunnels, say in excess of 25 m, cut and cover techniques cease to be appropriate. Surface access or space for construction plant may also be a constraint since only relatively small areas at the surface are needed or may be available. In such circumstances construction employing tunnelling techniques may be adopted, such as raise boring or the underpinning method (see Chapters 8, 9 and 22).

6.2 Construction methods

6.2.1 General

The designer must have a good understanding of construction methods. Design cannot be separated from construction [5,6]. This applies perhaps more strongly in underground construction than in any other field of civil engineering, because of the fundamental interrelationship of structural design, soil/structure interaction and construction methods [12]. The vital importance of this relationship is stressed throughout this chapter. The designer needs to consider the constructional implications of the design from the start and throughout the process of its development.

6.2.2 Developments

There have been two main developments this century in the methods of cut and cover construction:

1. Modern methods utilize a great amount of mechanization [13]. This includes not only the wide range of plant for bulk excavation but also specialized plant for piled and diaphragm walls, the preference for large concrete pours and the use of travelling shutters and modular systems for falsework and formwork. Consequently there is greater emphasis on clear working space and speed of construction.
2. For excavation in retained cut the trend is to reduce the need for (and accordingly the cost of) temporary support systems by exploiting the ability of the permanent structure to provide such support during construction [6]. This applies notably to the technique of top-down construction and the use of embedded peripheral walls.

The development of water-jet excavation and the hydraulic transport of spoil can be applied to top-down construction (see Chapter 10), but the safety implications of introducing significant quantities of water to any excavation must be carefully assessed.

6.2.3 Design and construction

A principal requirement for the designer is to ensure the design is safe and buildable. Two basic factors are dominant. First, the structural design must be fully compatible with the construction methods adopted. Different methods, particularly in relation to the form and sequence of installation of temporary works, the type of plant used and construction programme, may create quite different design requirements. Second, it is impossible to undertake the process of excavation without movements, both horizontal and vertical, occurring in the surrounding ground (see Section 6.3.2). Significant ground movements may be caused, for example, by the installation of piles or diaphragm walls prior to any bulk excavation [6]. The potential effects of ground movements both on the railway construction itself and the adjacent structures and services must be taken into account. Cost and programme are critical aspects of any project and ease and speed of construction are consequently important considerations for both the designer and constructor. Speed of construction may also be very significant in reducing the short-term environmental impact[14].

If there are easier, cheaper or quicker methods than those proposed or assumed in the design which satisfy the specified requirements there will always be pressure to change the design. Major design changes are undesirable, particularly as the programme moves into the construction phase.

Harmonization of design with construction methods generally requires the design and construction teams to work closely together. This, unfortunately, does not occur frequently enough in practice, especially during the development of the Conceptual and Outline Designs. Usually, no significant input from the constructor becomes available until after the tender stage. This can even apply to 'design and build' contracts, irrespective of whether the design team is 'in house' or provided by an external consultant. The overall concepts are developed by the designer, who therefore has the onus to relate the design to construction methods. However, designers and constructors need to get together earlier and more often[15].

6.2.4 Importance of simplicity

In foundation engineering particularly, the need for simplicity should always be stressed. The following considerations are relevant and should be followed wherever practicable:

1. Adopt simple shapes and details.
2. Keep element and overall cross-sectional dimensions constant by avoiding small changes.
3. Avoid movement joints unless essential (see Section 6.6.1).
4. Allow freedom in location of construction joints and cater for large concrete pours.
5. Exploit the strength of the permanent structure to limit the need for temporary support (see Sections 6.3 and 6.4).
6. Specify materials which are locally available or in easy supply.
7. Facilitate fixing of reinforcement and placement of concrete. Sloping, skewed or curved surfaces should be avoided.
8. Maximize use of mid-range diameter reinforcement. Small bars need significant support and the large diameters need long lap lengths and are awkward to bend and fix.
9. Thick sections (in excess of 500 mm) are common for cut and cover structures. The space between the layers of reinforcement is convenient for fixing or welding – for example, to establish continuity for stray current protection[16] or cathodic protection against corrosion. Consequently, the extensive use of shear reinforcement should be avoided since, apart from the associated congestion and complication to reinforcement fixing, it severely obstructs access.
10. Detail distribution or early-age thermal crack control reinforcement on the outside of the main flexural steel. This not only eases fixing but is more effective in controlling crack widths at the surface, so giving greater protection and cover to the main reinforcement.

6.2.5 Top-down construction

A more specific insight into the relationship between design and construction is afforded by considering the technique of top-down construction. This should direct the designer from the start to appropriate evaluation of construction methods. The construction method involves the following sequence (see Figure 6.7):

1. Establish a level working platform close to that of existing ground. (In the case of sloping terrain the highest existing level may dictate, requiring placement of appropriate fill.)
2. Construct peripheral permanent reinforced concrete walls with specialist plant (e.g. diaphragm or piled walls).
3. Construct top slabs.
4. Backfill above completed top slabs as required to allow early release of surface area. This in turn enables early re-establishment of utilities, amenities and/or progress of development above the railway construction.
5. Subsequent construction continues beneath top slab and is thus substantially isolated from adjacent surface environment.

Advantages
The potential benefits offered by top-down construction relate to:

1. Environmental impact
2. Control of ground movements
3. Cost

These advantages are considered in turn below.

Environmental impact The construction of the top slab early in the overall sequence substantially isolates the adjacent environment from the effects of the subsequent excavation. Noise and dust pollution are thus significantly reduced. With the top slab acting as a shield, construction may also effectively proceed in severe weather conditions. This facilitates a cleaner operation and can offer the important benefit of a shorter programme overall. Also, as noted above, the top surface is freed, allowing early re-establishment or introduction of amenities (such as landscaping, gardens, roads or car parking). This is particularly relevant to urban conditions. While also a direct design consideration, the comprehensive control to limit ground movements inherent in this method of construction can also be a major environmental consideration – the safety and serviceability of adjacent structures and services may impose severe restraints on the permissible range of ground movements created by construction.

Ground movements Control of ground movements, including the associated potential effects on the water table, is often critical. The advantages afforded in this respect are:

1. The walls are propped rigidly by the roof slab prior to the bulk of excavation.
2. Short-term unloading of the substrata and consequently the elastic heave and associated ground movements (both within and outside the structure) are reduced. This benefit can be enhanced by loading above the roof slab prior to excavation beneath (either by backfill or commencement of development above).

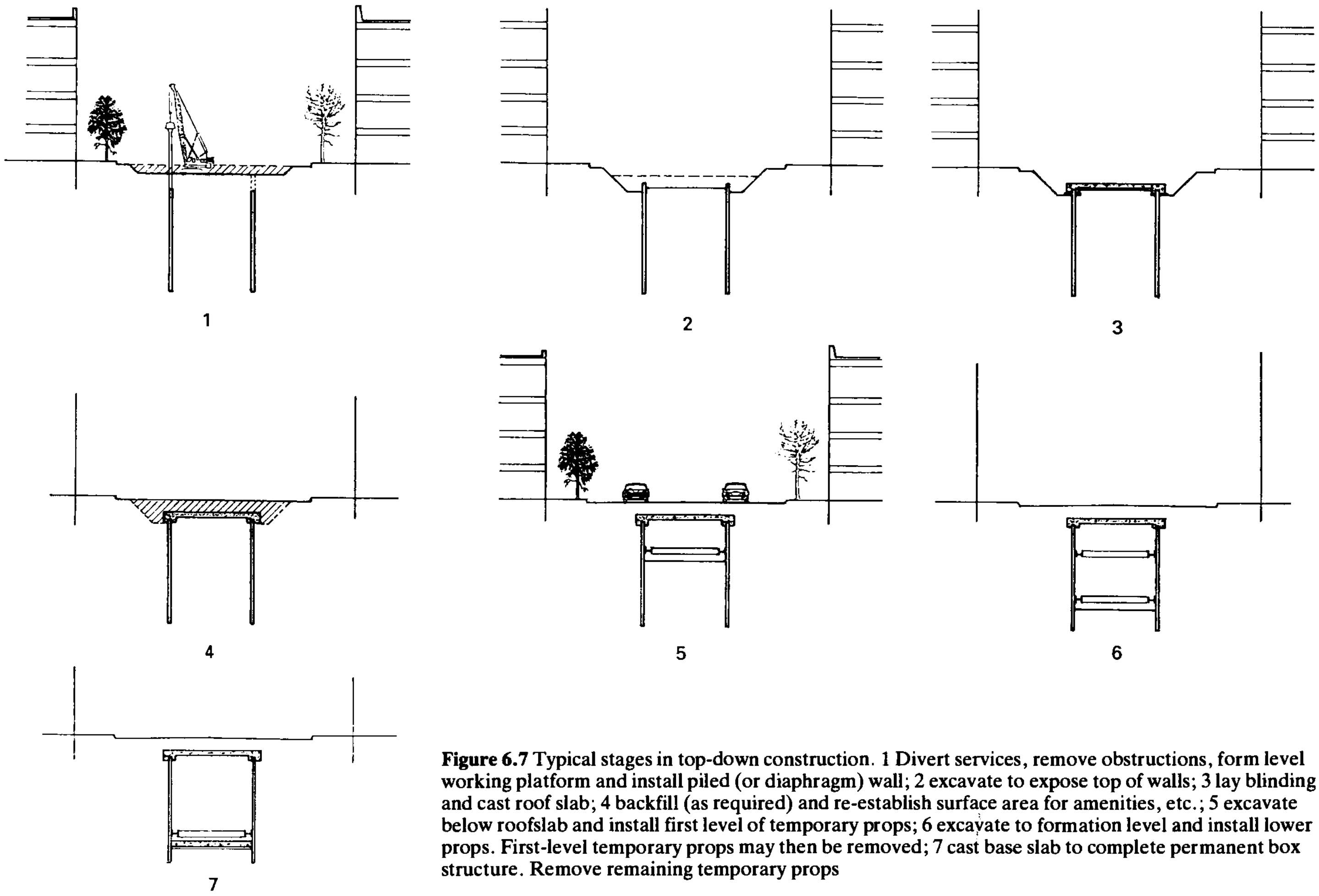

Figure 6.7 Typical stages in top-down construction. 1 Divert services, remove obstructions, form level working platform and install piled (or diaphragm) wall; 2 excavate to expose top of walls; 3 lay blinding and cast roof slab; 4 backfill (as required) and re-establish surface area for amenities, etc.; 5 excavate below roofslab and install first level of temporary props; 6 excavate to formation level and install lower props. First-level temporary props may then be removed; 7 cast base slab to complete permanent box structure. Remove remaining temporary props

3. The diaphragm or piled walls required for this construction method are inherently stiff, so helping to limit lateral deflections.
4. The cut-off to groundwater afforded by the embedment of the outer walls into strata of low permeability will minimize the drawdown effect on the water table beyond the excavation. This will consequently minimize the changes in effective stress and potential loss of fines which lead to settlement in the ground adjacent to the excavation. (This feature does not apply to contiguous piled walls.)

Costs In addition to the potential benefit to programme from severe weather conditions as noted above, economies may be derived from reduced requirements for temporary works. The shuttering to the top slab (and to intermediate slabs where present) may be directly supported on the blinded formation before excavation beneath. The need for the substantial amounts of falsework is thus eliminated. Also, and perhaps offering the greatest cost saving in materials and speed of construction, the requirements for ground anchors or additional temporary steelwork in the form of props and walings can be significantly eased[17]. The overall cost benefit will be offset by the higher cost of excavation beneath the permanent slabs and the general limitations imposed to access for construction activities.

Exploiting fully the potential advantages of top-down construction, as with any other technique, requires a compatible design which addresses soil/structure interaction, structural detailing and construction methods, plant and sequences.

6.3 Soil/structure interaction

6.3.1 General

Soil/structure interaction and the evaluation of the loadings applied to the structure and the associated ground movements are fundamental aspects of foundation engineering[7,18]. Even a structure founded in competent, hard bedrock is not completely unyielding in reality, although this may be assumed in simple analysis. Most cut and cover construction is undertaken in yielding ground. Soil/structure interaction is a complex subject. Indeed, even an embedded retaining wall may present an extremely complex example of this mechanism for which no exact design solution exists[19]. The complexity relates not only to the direct effects on the railway structure itself but also to the interaction with adjacent structures and services. This is especially relevant in urban locations but also applies to open sites where slope stability or large deformations may be of concern.

6.3.2 Ground loadings and movements

The estimation of the design of ground loading, particularly in the lateral direction, to be applied to cut and cover structures is influenced by the following:

1. Adequacy of site investigation;
2. Geotechnical evaluation and input, including soil parameters, geological history and groundwater.
3. Adjacent structures and services. The nature of their construction and location will influence the overall approach to soil/structure interaction. Generally, framed structures on piled foundations will be the least susceptible to the effects of ground movements generated by the adjacent cut and cover construction.

Surface structures, particularly those founded on isolated shallow pad footings, can be very sensitive to these effects[20, 22, 74].
4. Construction methods. Different methods, particularly in relation to speed sequence, rigidity of temporary works and types of plant, will have different effects;
5. Ground movements. Generally, the greater the restraint or limitation of ground movements, the higher the imposed ground loadings on the structure will be.

It is important to emphasize that each structure is unique. Judgment (based on observation and experience and understanding of actual structures) is essential in the assessment of appropriate ground loadings and possible ground movements.

For example, with regard to the soil strength parameters for effective stress analysis it should be noted that these cannot be directly obtained from laboratory tests or from *in-situ* tests prior to construction. It is the soil/structure interaction generated during and after construction that uniquely but unquantifiably determines the stress–strain relationships and loadings for each structure[19]. Ground movements, although they may be of minor importance in some circumstances, can be critical both in short- and long-term conditions. Base stability can be critical and should always be checked. Much useful general guidance on the considerations of ground movements and lateral loading (in particular in relation to effective stress design of embedded walls in stiff clays) is given by Padfield and Mair[19].

For embedded walls a major distinction must be made between those which are cantilevered or are singly propped and multi-propped (or effectively very stiff) walls. The former depend on active/passive modes of soil/structure interaction, which relate to the strain in the soil. Their stability is dependent on the passive resistance of the soil. This is not the situation with most multi-propped walls. These may undergo an initial phase where pressures close to active may be considered to act but the earth pressure on the retained side will always tend to build up towards an 'at rest' condition. A distinction should also be made between the earth pressure envelopes for design of the strutting and that relevant to the design of the reinforced concrete wall[23] (Figure 6.8).

Furthermore, it is necessary to note the important differences in design approach for stability of walls and their structural design[19]. The former relates a factor of safety to limiting failure conditions in the soil while the latter is based on stresses generated on the wall from often quite different earth pressure envelopes.

Another consideration regarding ground movements and loads relates to embedded walls in stiff overconsolidated clays. Conventional theory indicates that significant wall movement towards the passive side is necessary, particularly in clay soils, to generate full passive resistance. However, in overconsolidated clays minimal movement is likely to be required to achieve this since the reduction in overburden from excavation above tends to initiate conditions of passive failure in front of the wall[19].

6.3.3 Instrumentation and monitoring

Much effort can be expended in the prediction of the effects of construction. Whatever the level of detail in appraisal and the sophistication of analytical techniques, it is impossible to predict precisely ground movements and the associated structural deflections. While part of the art of foundation engineering is the judgment of the likely range of such effects, it is advisable to install a system of monitoring during construction. In some circumstances, as with swelling or

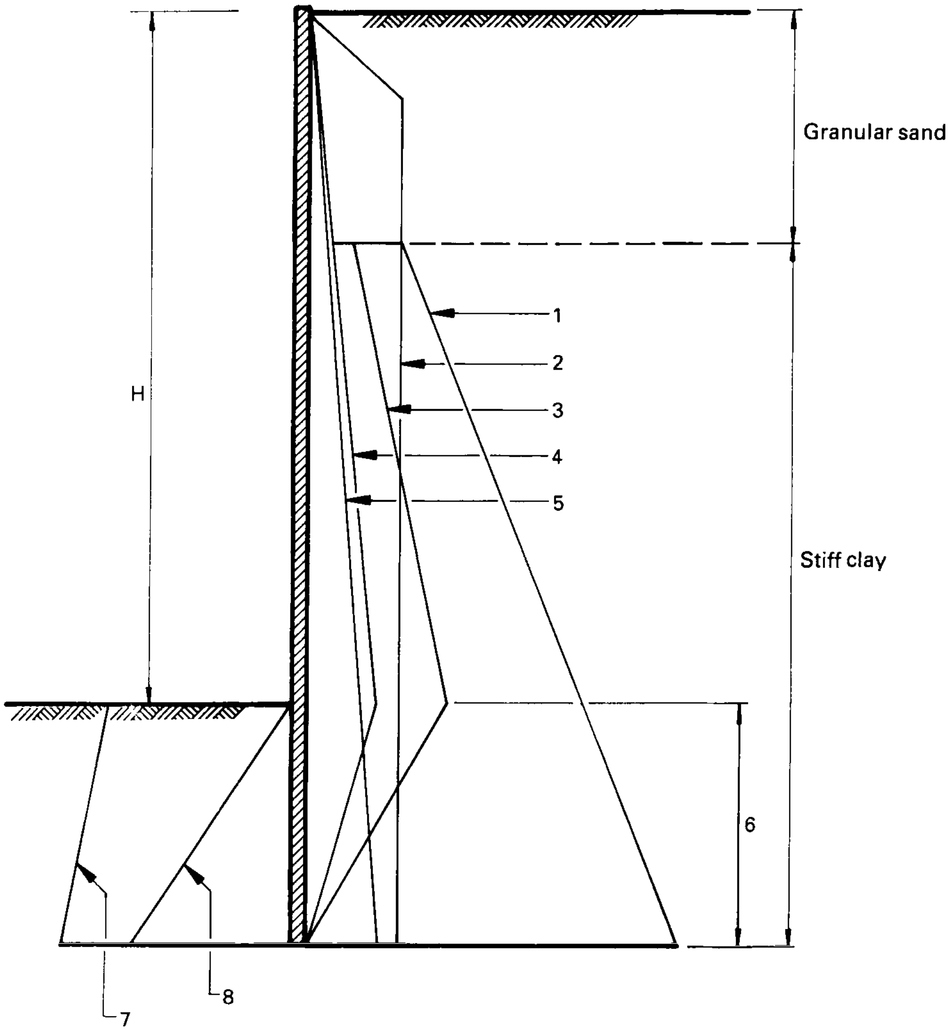

Figure 6.8 Idealized range of earth pressures associated with embedded retaining walls (pressures will be significantly modified by groundwater when present). 1 Pre-construction 'at rest' earth pressure ($K_0 \rightarrow 2$ or greater); 2 Terzaghi/Peck design strut load envelope; 3 indicative long-term design earth pressure for multi-propped walls, $K = 1$ (permanent structure); 4 indicative short-term design earth pressure for multi-propped walls, $K = 0.5$ (construction phase); 5 active earth pressure (e.g. $K = 0.37$ for effective stress parameters $\phi' = 24°$, $C' = 0$); 6 possible attenuation in design earth pressure diagrams (0.25 to $0.4H$) – depends on overall soil/structure interaction; 7 passive earth pressure – total stress ($2C_u + \gamma_d$); 8 passive earth pressure – effective stress (e.g. $K = 3.4$ for $\phi' = 24°$, $C' = 0$)

consolidation of clays, monitoring of movements should continue well beyond completion of construction. This is important where long-term movements could affect internal clearances, track alignment and drainage. Adequate extra tolerances must be included over the affected zone to accommodate the potential range of movement. The rate of movement may also be critical (see Section 6.5.2).

Simple, direct methods of monitoring such as level surveys, sight and plumb lines, and tape extensometers should be undertaken, regardless of other more sophisticated instrumentation. Such methods are quick and repeatable and normally adequately sensitive to indicate movement trends – although care is needed to ensure the integrity of datums. Reference points must be sufficiently remote from the construction to be unaffected by it. Ground movements arising from large excavations can extend well beyond the excavated area – vertical movement, for example, can occur as far out as three to four times the retained height [6].

The above, essentially optical, methods cannot, however, provide a comprehensive assessment of the strains (and loads) within the structure or the surrounding soil. Where this is necessary, these methods need to be used in conjunction with instrumentation. Such instrumentation may involve a combination of telltales, strain gauges, inclinometers and load cells. These instruments are located as appropriate in the new construction, the surrounding ground and adjacent structures. Comprehensive descriptions of equipment and techniques are given by Dunnicliff [24].

It is most important to maintain regular and complete written records from the monitoring system. No amount of data collected will serve much purpose if it is not evaluated adequately. Only timely assessment of movement trends will enable prediction of potential instability or unacceptable movement. There is a general paucity of meaningful results from instrumentation available for existing structures. Well-documented examples constitute a very small percentage of the effects of foundation construction worldwide. Instrumentation is either completely neglected or too little installed to generate useful data, and is also prone to damage during construction activities. However, it is important to balance the need for comprehensiveness with the practical considerations of data collection and interpretation. Accuracy and expediency in the collection and presentation of results requires good equipment and dedicated, skilled personnel. If too much instrumentation is installed the process of monitoring and evaluation becomes very onerous. At best, this will tend to delay evaluation and transmission of results which could be critical. At worst, instrumentation may be abandoned or effectively ignored. Clients are often resistant to provide funding, even though their projects benefit from the increased understanding derived from previous case histories. This short-term view is understandable, since, apart from the value of preventative evaluation, however important, it may be argued that much of the knowledge gained from the interpretation results will only benefit future projects. This dilemma must be overcome, for there are many gaps in the understanding of soil/structure interaction, particularly in the relationship between ground movements and applied loads. Peck [4] emphasizes the importance of the observational method and expresses concern about recent trends to concentrate unduly on numerical analysis. With the increased tendency for more congested sites and difficult ground conditions, the greater is the need for appropriate caution and to further our understanding.

6.4 Peripheral support

6.4.1 Basic costs and alternatives

The cheapest form of construction below existing ground level is in self-supporting excavation with no requirement to form a reinforced concrete tunnel structure which is subsequently backfilled (Figure 6.9). The principal requirements for this

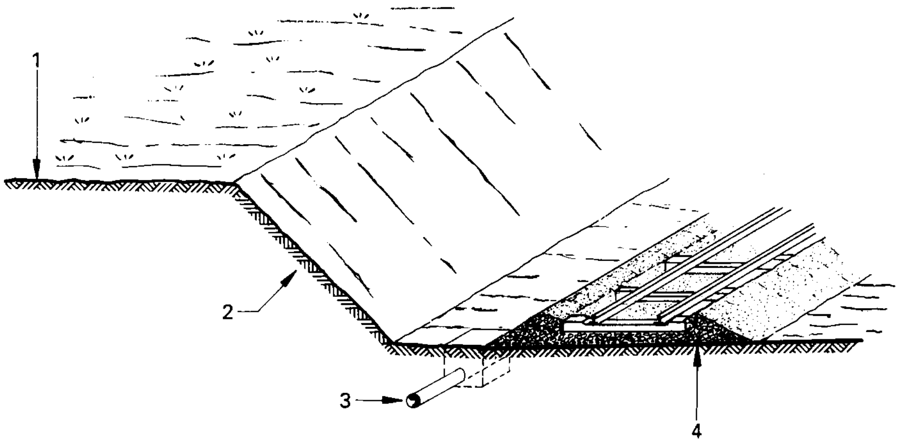

Figure 6.9 1 Original ground level; 2 permanent open cut; 3 drainage system; 4 sub-base (minimum slope 1 in 30 to drain)

permanent open-cut situation are to provide a stable formation for the trackbed and the slope stability of the excavation. However, even for rural sites, landscaping and other environmental issues can create the need for cut and cover tunnels. The overall stability of the surrounding land mass in which the cut is made may also necessitate their use, since the factors of safety of slips can be unacceptably reduced by the change in overburden pressure associated with permanent open cut [7]. Such situations require consideration of the temporary conditions created during construction, particularly with regard to ground movements and sequences of excavation and backfill. The long-term effects may also be critical.

Reinforced concrete structures formed in battered open cut are generally the cheapest form of cut and cover tunnels (Figure 6.10). More typically, and especially for urban locations, the proximity of existing buildings, roads and services necessitates cut and cover construction in retained cut. Here the direct costs are increased, sometimes substantially, by the use of specialist plant and the requirements for temporary support. Incidental costs associated with the disruptive effects of surface works may be very high and the overall selection of construction methods is often dominated by the need to minimize them. In this context it is often

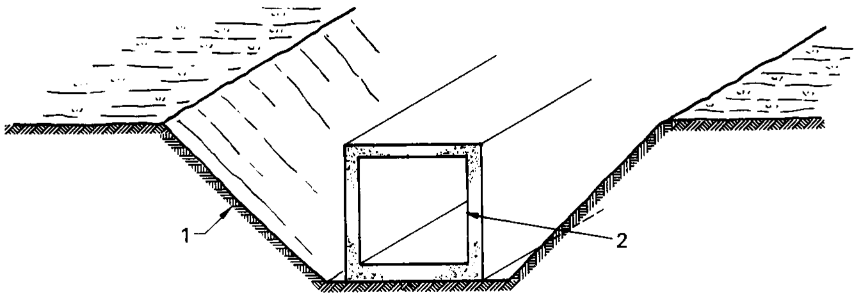

Figure 6.10 1 Temporary battered excavation (self-supporting); 2 reinforced concrete structure (subsequently backfilled)

better to maximize the extent of bored tunnelling and adopt cut and cover construction only where necessary, using methods which reduce the duration of surface disruption such as the top-down technique[8].

The choice of peripheral support is dictated by:

1. Space constraints. Different techniques and plant need different operating space – both vertically and horizontally;
2. Type and availability of construction plant. This will differ for various countries and local practice may strongly influence the usual order of cost and selection of plant. (An indicative evaluation of comparative costs for various types of peripheral support for the UK is given by Puller[25]. Exceptions will always occur – for example, Los Cortijos Station in the Caracas Metro was constructed within temporary diaphragm walls due to the influence of local practice and availability of local materials.) See also Chapters 7 and 8;
3. Ground conditions and the need to control movements;
4. Programme.

6.4.2 Temporary support systems

The most common methods involve the use of steel sheet piling and king piles with lagging using struts or ground anchors as necessary (see Sections 6.4.4 and 6.4.5). Less frequent is the use of temporary reinforced concrete diaphragm (or piled) walls. The benefits of such walls are usually exploited to provide the permanent support as well. Some reinforced concrete cut and cover stations have been built within excavations supported by temporary diaphragm walls (see the above reference to the Caracas Metro). Temporary diaphragm walls are more typically used to provide support to create the portals between sections of bored tunnels and cut and cover works prior to backfilling of the latter.

Other forms of temporary support include ground anchors and the various methods of ground treatment such as chemical injection, soil stabilization and injection grouting (see Chapter 4). It should be noted that temporary works (particularly walls) are often left around the structure after its completion and may present obstructions to future works. It is therefore important to maintain detailed records of temporary works installations.

6.4.3 Permanent support systems

As noted in Section 6.2, the current trend is to incorporate the requirements for temporary support with the permanent works. There are three main systems for reinforced concrete walling which involve different types of plant – diaphragm walls, secant piles and contiguous piles. These techniques are described in more detail in Chapters 7 and 8.

Diaphragm walls

Due to their inherent watertightness, diaphragm walls can be used to form a cut-off to groundwater by embedment into the stratum underlying the foundation. If this stratum is of low permeability, or rendered so by ground treatment, control of groundwater can be very effective. This has obvious benefits during construction. The depth of embedment can also be utilized in the control of wall deflections and stability. Furthermore, increasing the embedment and exploiting the inherent stiffness of the wall may offer attractive reductions in temporary support such as props or ground anchors. In the permanent structure, the embedment can enable the use of a suspended base slab (to reduce or eliminate hydrostatic or ground pressures). (See Section 6.5.3.) However, the consequences of such construction stopping the natural transverse flow of groundwater – particularly if the wall extends into an impermeable stratum beneath a permeable one – must be carefully evaluated, as noted in Section 6.1.3.

Secant piles
Walls of reinforced concrete interlocking (secanted) piles may be formed using a Benoto grab-rig. Recent developments in high-torque auger rigs now offer the technique with alternative plant. The former employs a full-depth oscillating casing which eliminates the need for bentonite support slurries. Compared generally to diaphragm walls, however, the benefit of watertightness, also afforded by secant piles, is offset by:

1. Greater constructional space requirements adjacent to existing buildings;
2. Slower rate of construction;
3. The circular surfaces make structural connections more difficult and may be a less acceptable exposed finish;
4. More construction joints.

Contiguous piles
These are generally installed with conventional short flight-auger bored piling rigs and can offer a fast and economical form of construction. Even with ideal ground conditions, a general disadvantage of contiguous piles is their lack of watertightness.

Slurry piles
A variant of piled walling using conventional augers is the incorporation of slurry piles[26]. Here discrete female piles formed with a bentonite/cementitious slurry are first installed. Reinforced concrete piles, which form the structural elements of the wall, are then bored between them. An interlocking alternate sequence of secanted reinforced concrete and slurry piles to form a watertight wall is thus created. This technique can offer fast and economic construction, but is only suitable for appropriate ground conditions and generally depths of excavation to about 10 m.

6.4.4 Strutted excavations

This common form of temporary support has many variations, although this may be provided in the case of top-down construction, at least partially, by the permanent slabs. The choice of support depends on structural form and layout, ground conditions, control of ground movements and construction methods. The loads applied, particularly for deep excavations in heavily overconsolidated clay, can be substantial. Accordingly, temporary strutting (or ground anchors) may be a major element in the overall cost of construction. While respecting the paramount criteria for safety, consideration should be given to the possible reduction or elimination of temporary support. This may be achieved by exploiting the inherent strength of the permanent works (and, where applicable, the higher short-term strength of the soil) in conjunction with the speed and sequence of construction.

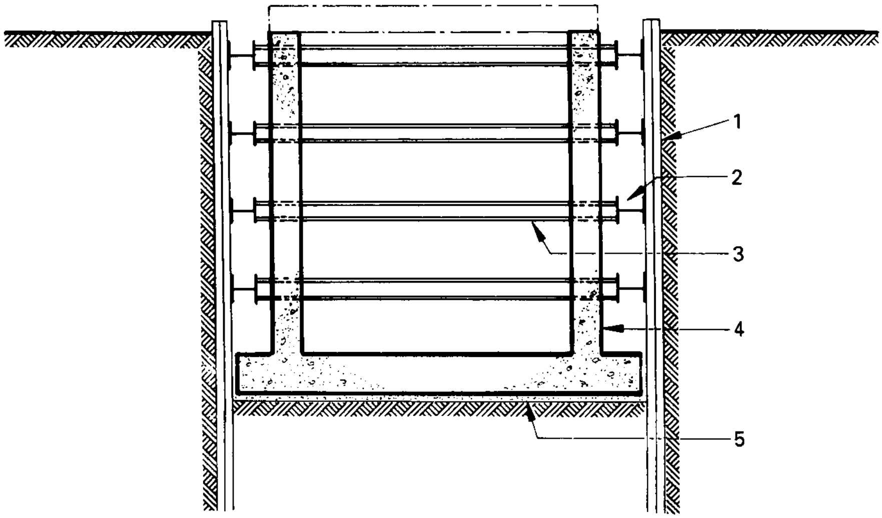

Figure 6.11 1 Temporary wall (e.g. sheet piling or soldier piles and lagging); 2 waling; 3 strut; 4 permanent reinforced concrete structure; 5 blinding (may be thickened and lightly reinforced as necessary to act as temporary strut)

The arrangement of temporary strutting will depend on the size and shape of the excavation. Fully braced systems are utilized for deep, narrow excavations (Figure 6.11). Raking props are more appropriate for those which are wide and relatively shallow. For the latter, two typical sequences are employed:

1. The props are supported on a central berm and a stable L-shaped wall constructed along the excavated periphery (Figure 6.12).
2. Alternatively, berms are left adjacent to the peripheral embedded walls and the central sections of the foundation slab constructed. The walls are then uniformly

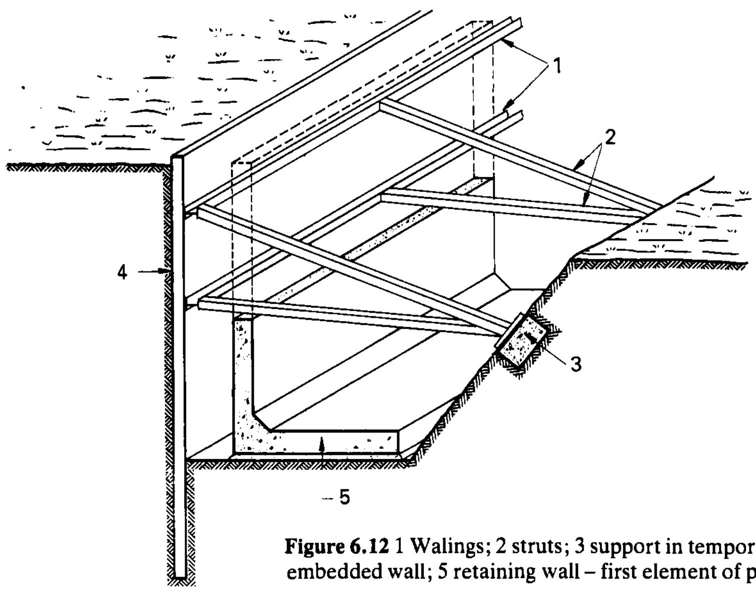

Figure 6.12 1 Walings; 2 struts; 3 support in temporary berm; 4 temporary embedded wall; 5 retaining wall – first element of permanent structure

propped from the completed central sections while the side berms are excavated and the remainder of the foundation slab constructed (Figure 6.13).

As noted, there can be major benefits in cost and creation of clear working space to be derived from the reduction of such temporary works. Two main factors in achieving this are the stiffness and embedment depth of the peripheral walls and the utilization of the permanent slabs for lateral support. A good example is the use of top-down construction, which exploits both factors. The advantages of this method are discussed in Section 6.2.5.

The design of strutted excavations is well documented[27,28]. For deep excavations a comprehensive treatment of the traditional approach is given in Peck's state of the art report[18]. Due to the uncertainties of the magnitude and possible variations in ground pressures, loading envelopes are applied in the design to create a robust support system. Factors of safety should be generous, considering the grave implications of a failure in such temporary works, and should be based on the overall loading envelope. They should also allow for the effects of temperature and accidental impact by construction plant. The overall support system should have adequate redundancy to guard against accidental damage and progressive

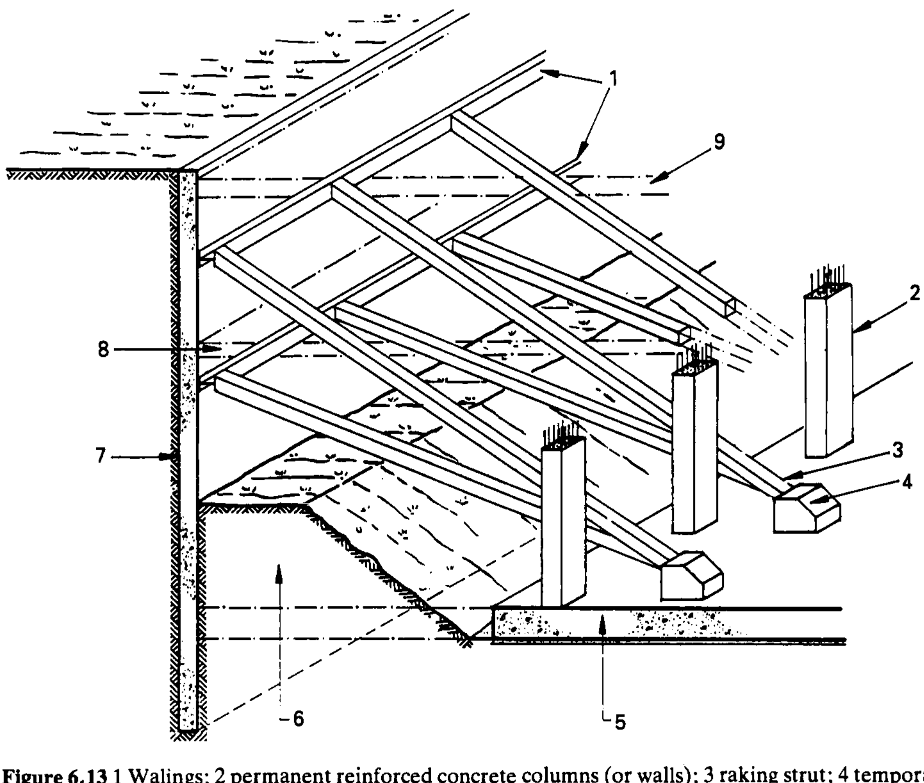

Figure 6.13 1 Walings; 2 permanent reinforced concrete columns (or walls); 3 raking strut; 4 temporary support; 5 base slab (constructed in sequence from bottom up); 6 side berm; 7 permanent reinforced concrete embedded wall; 8 intermediate slab (constructed in sequence from bottom up); 9 top slab (constructed in sequence from bottom up)

collapse. The use of compact sections for struts will reduce the need for bracing which, if extensive, may significantly impede access space for construction. Heavy sections, however, which eliminate bracing entirely, will increase the direct cost of the temporary works and require awkward, high-capacity plant for their installation and removal. A balance therefore needs to be made between bracing and strut size.

A similar situation relates to the vertical spacing of struts, where a compromise between section weight, wall loading and construction access is involved. For access a minimum clearance of 3 m is desirable to allow passage of standard front-bucket excavators. Smaller plant is available but with the disadvantage of reduced speed of construction. An important rule is to make the struts stronger than the walings and make the latter continuous, at least over a number of struts. This should avoid or at least give early warning of potential progressive strut failure under unforeseen loading conditions. The excavation can remain supported even if the walings deflect significantly in bending, but if the struts are overloaded first the buckling collapse mode can be sudden and progressive.

6.4.5 Ground anchors

An alternative method to strutting for peripheral support is afforded by ground anchors (Figure 6.14). The preference to use these over other forms of support varies widely between countries and is strongly influenced by local practice and ground conditions. They offer the advantage of creating the clear working space so desirable for construction. This benefit may be negated by high relative cost compared to other forms of support and slow installation times. Other disadvantages relate to interference with adjacent underground facilities and the limitation of ground movements. It is generally more difficult to control ground movements with anchors compared to propping the walls. Indeed, since the action of ground anchors initiates ground strain they may actually increase overall movement particularly if anchorage is mobilized too close to the wall[6,19]. While

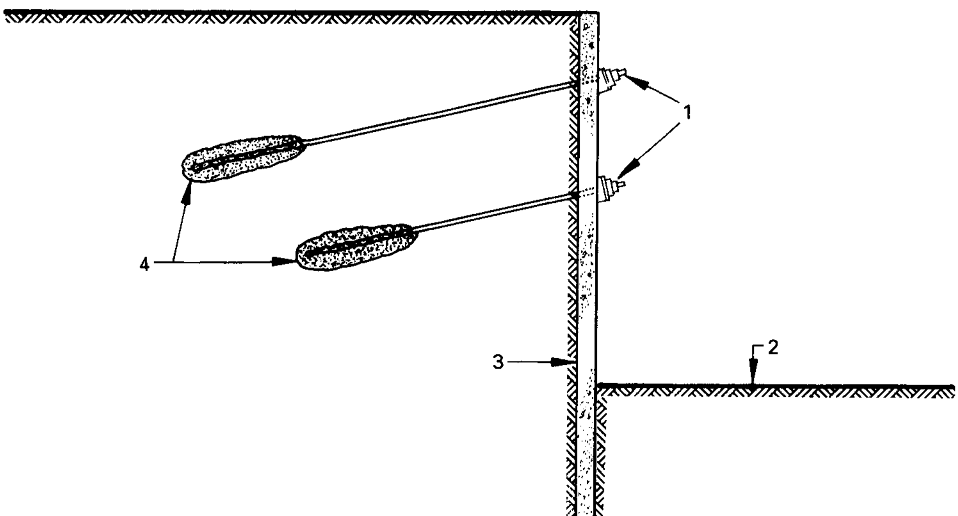

Figure 6.14 1 Walings may be used for continuity of support; 2 formation level; 3 temporary (or permanent) embedded wall; 4 grouted ground anchorages

providing stability to the sides of the excavation, ground anchors are generally more effective in limiting maximum bending moments in the peripheral walls they support rather than global deflections. Propped systems are inherently more robust. They can be made more rigid by pre-loading before excavation and subsequent adjustments of the jacking system as necessary. Failure of ground anchors, resulting in either excessive movements or complete loss of stability, tends to be more difficult to predict than that associated with a propped system. It is hard to ascertain what working load a given anchor is subjected to or its likely capacity in relation to wall movement and the overall factor of safety (although they can be prestressed and load cells installed). These aspects, although difficult to quantify, are somewhat easier to assess for walings and struts, where the variations in applied load transmitted by them across the excavation may be monitored (with strain gauges or load cells) along with the associated deflections. The correlation of ground movements, wall deflections and imposed loads is more complex for anchored walls.

The design and installation of ground anchors is a specialist activity. The design is therefore not normally undertaken by the main design team. A performance specification should be supplied and the loading requirements for support of the wall clearly defined[6].

6.4.6 Circular construction (see Chapter 9)

Circular construction in plan is typically used for deep access and ventilation shafts. Its application to create large clear spans for permanent works is less usual. Constructional requirements for clear space to install or remove tunnelling shields or boring machines may be satisfied by using ground anchors to support the peripheral walls. The chambers can then be maintained rectangular in plan. Where the use of ground anchors is not appropriate because of ground conditions or other constraints adjacent to the excavation, circular chambers may offer a viable alternative. The removal of tunnelling machines may require unobstructed vertical access for lengths in plan in excess of 10 m. For rectangular chambers the waling and prop sizes can become uneconomic or counterproductive by encroaching into the required space. Circular construction can satisfy the space requirements and in fact may be cheaper than rectangular chambers, even though the space created is larger.

The design of circular chambers or shafts needs careful consideration, especially with regard to load distribution, induced stresses and ground movements. Added complexities may arise from construction requirements. These include the sequences adopted, including that of adjacent works. In particular, the need to create large openings, thus disrupting the circumferential action in the circular walls, can completely alter structural action. The distribution of ground loading and movements will also be affected. An interesting example, requiring a range of design and construction considerations, was developed for the cut and cover works for the shield chambers at Sugarloaf Hill, Channel Tunnel, UK. Here access for removal of three tunnel-boring machines in close proximity was required. This resulted in the further complexity of creating a double chamber for reception of the service and southern running tunnel machines. The effects and requirements of adjacent construction were major considerations and the details of the methods and sequences adopted are illustrated in Figure 6.15. The construction stage is shown in Figure 6.16.

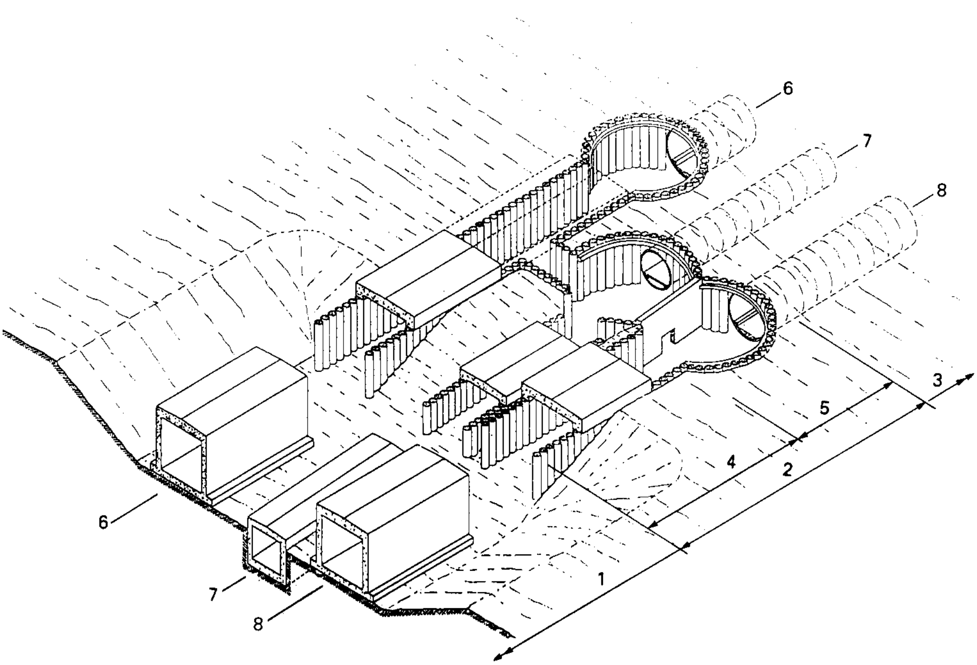

Figure 6.15 Cut and cover construction methods at Sugarloaf Hill, Channel Tunnel. 1 Temporary sloped excavation; 2 embedded contiguous piled walls; 3 bored tunnels; 4 top-down construction; 5 circular chambers for TBM reception/bottom-up construction; 6 running tunnel north; 7 service tunnel; 8 running tunnel south

6.5 Foundations

6.5.1 General

This section is primarily concerned with vertical loads and their transmission between the structure and the subsoil. The capacity of the soil to accommodate vertical loads is dependent on allowable bearing capacity and settlement. For underground railways in cut and cover the actual bearing capacity of the soil is not usually critical. The quality of the strata tends to increase with depth, and bearing capacity is also related to the net change in overburden pressure. For covered box structures this change is often small – and may well be negative. Where there is minimal cover, however, stability against uplift will then be potentially critical. It is the effects of settlement and heave that are often a major design consideration. The latter criteria tend to dominate foundation design and govern what range of bearing pressures (as opposed to actual capacities) are acceptable in practice and their application to structural design. Three main types of foundation are utilized: strip footings, rafts and piled foundations. Their selection is determined by the usual considerations of soil conditions, construction methods and cost, as discussed below.

(a)

(b)

Figure 6.16 (a) Cut and cover construction, looking west from Sugarloaf Hill, Channel Tunnel, UK; (b) contiguous pile construction adjacent to open-cut section, Sugarloaf Hill, Channel Tunnel, UK

6.5.2 Strip footings

Strip footings are built in open cut and offer the cheapest form of construction. They are typically employed for traditional retaining walls of moderate retained height (up to 8 m) and portal frame structures. This is possible where the substrata provide adequate bearing capacity for both the footings and the trackbed formation between them. Particular attention must be paid to the drainage requirements for this form of construction (Figure 6.3).

The potential effects of settlement arising from long-term softening of clay or loss of fines in granular subsoils will need to be addressed. In overconsolidated clays with a high plasticity index another consideration is the possibility of significant swelling (long-term heave) if there is sufficient reduction in net overburden pressure. The trackbed will invariably be subject to some swelling with such clays since there will always be permanent unloading of the ground between the edges of the strip footings. Whether this is critical or not will depend on the magnitude and rate of swelling. With ballasted track it should be possible to accommodate such heave effects progressively during routine maintenance. It is important to note that this convenience does not readily apply to concrete trackbed, particularly since it is the predominant form used for underground railways. Here the associated track-support systems do not afford much allowance for vertical adjustment in rail level. Consequently, limitation of heave (and settlement) is far more critical and usually requires the incorporation of a structural base slab to support the trackbed. The effects of heave are not limited to vertical movement (Figure 6.17). The full evaluation of such factors involves stability considerations and the control of the ground movements (horizontal and vertical) generated adjacent to the railway construction.

Portal frames with strip footings can be most economic for large spans and they may be constructed quickly by traditional methods requiring no specialist plant. Large spans are required for running tunnels housing two or more tracks especially where crossovers impede the location of central walls. Box structures are generally adopted for single-track cut and cover tunnels since the combined width of the two footings tends to become comparable with that of the base slab. For small material cost differences, a base slab is then usually preferable to isolated footings. The slab provides a more competent support to the trackbed and a box is more robust structurally. Box construction also allows earlier backfilling to the sides (i.e. before construction of the roof slab), since the walls can generally accommodate greater cantilever loads than the partially completed portal frame without the roof in place.

The structural form of portal frames and retaining walls may be influenced by the proximity of adjacent structures. If the spatial constraints or founding conditions of these are critical, battered excavation may not be feasible. Retained excavation with embedded walls or L-shaped footings may thus be necessary. In any case, the requirements to evaluate ground movements and the possible need for ground treatment or underpinning of adjacent structures should always be observed. L-shaped footings are not so structurally efficient as the more conventional inverted T-shape construction. Structural form is considered further in Section 6.6.5.

6.5.3 Base slabs

Slab foundations are commonly used, creating the typical box form of many cut and cover structures. They are the dominant form for station construction. Even when

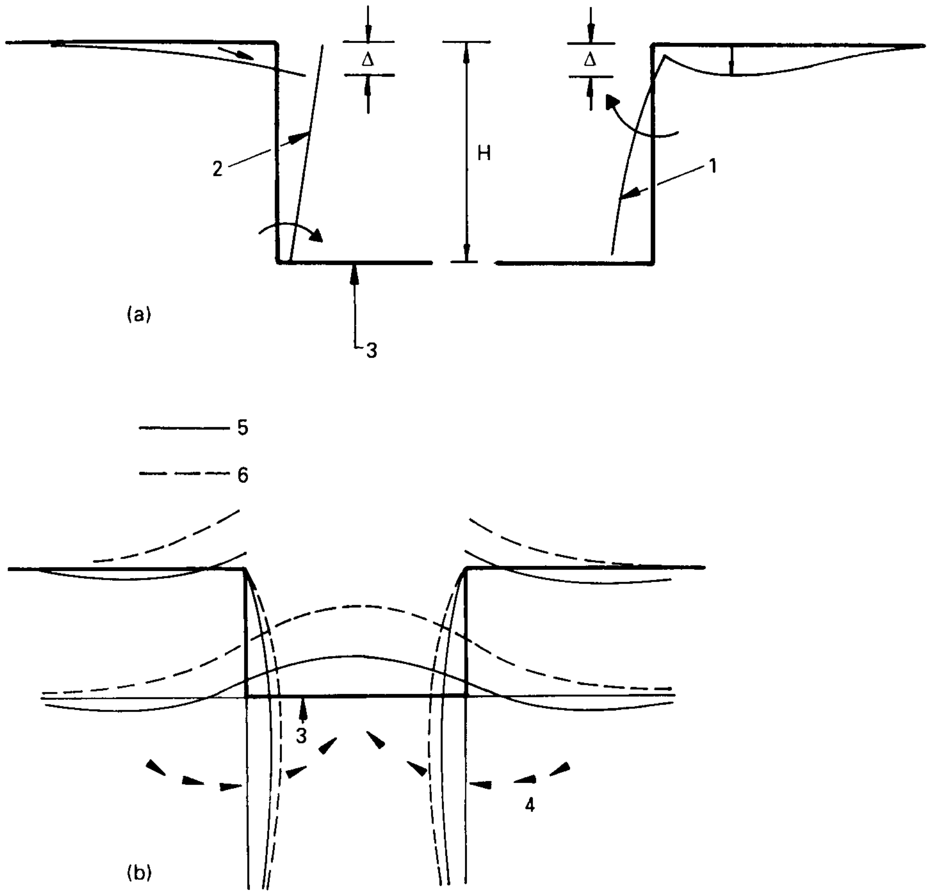

Figure 6.17 Idealized ground movements (adapted from reference 6). (a) Idealized movements from release of horizontal pressure. 1 Cantilever wall; 2 strutted wall; 3 excavated level.

Soft clay	$\Delta/_H$	Up to 2% or greater, depending on depth of soft strata below excavated level
Stiff clay	$\Delta/_H$	< 0.15%
Loose sands and gravels	$\Delta/_H$	Up to 0.5% – but groundwater and vibrations must be adequately controlled
Interbedded sands and stiff clay	$\Delta/_H$	Up to 0.15% – but case histories limited

(b) Idealized movements associated with heave. 4 Deep-seated movements; 5 immediate movement; 6 long-term movement. Note: the effects of (a) and (b) will be additive

the founding stratum is competent rock, a nominal base slab is provided to withstand hydrostatic pressure and seal the structure against groundwater penetration.

There are two main types of base slab (Figure 6.5). (The advantages of a hybrid variety are discussed below with reference to settlement-reducing piles[29].)

Ground-bearing slabs

These slabs transmit the full vertical loading to the soil. This loading comprises the dead weight of the structure, backfill and imposed loads, including those from pedestrians, vehicular traffic, installed equipment and finishes. It is important to realize that the sum of this loading will not be the same as that used to determine the design pressures for such foundation slabs (Figure 6.18). The upward loading on a ground-bearing base slab is modified not only by its self-weight but also by hydrostatic and swelling pressures where relevant. These various effects alter the distribution as well as the intensity of design pressures. This variation, combined with that from the lateral loads on external walls, requires analysis of several loading conditions.

The distribution of upward pressure on the base slab, generated from downward vertical load in the side walls, is naturally concave, as illustrated in Figure 6.16. A rectangular distribution arising from such loading could only occur if the slab was infinitely stiff compared to the soil. The degree of concavity depends on the stiffness of the soil relative to that of the structure. The greater the concavity, the lesser the maximum bending moments. This benefit of soil/structure interaction is reduced by hydrostatic and swelling pressures, and these agencies are generally treated in design to produce rectangular, or at least linear, pressure distributions on the base slab. When the effects so produced are dominant, consideration should be given to the use of suspended base slabs.

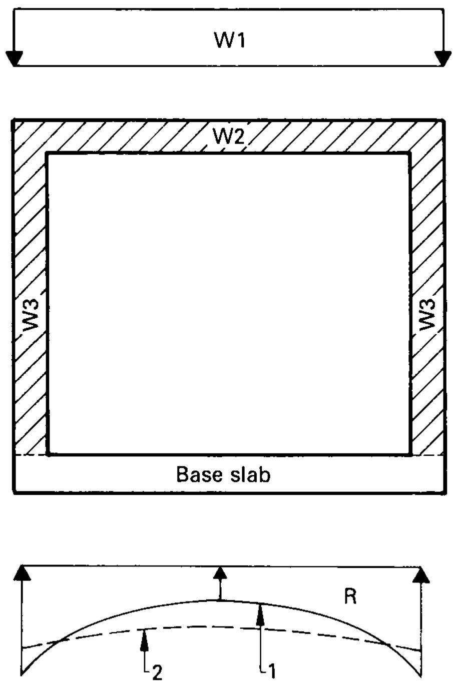

Figure 6.18 1 Indicative distribution for R: no water; 2 distribution modified by hydrostatic pressure: total design pressure still = R. Key: $W1$ = soil + imposed loads on roof slab; $W2$ = self-weight top slab; $W3$ = self-weight side walls; R = total design-reactive upward pressure on base slab

Suspended base slabs

These slabs span between embedded side walls (Figure 6.5). For wide structures or those with high internal vertical loads, such slabs may be also supported by bearing piles between the walls. The void beneath the slab may be created in a number of ways, suitable low-modulus polystyrene or certain forms of shuttering being typical. For the void formers, ease of installation combined with robustness are important considerations. Much expense and delay will result if the void former collapses during the pouring of the base slab or soon afterwards during the critical period before the concrete has gained sufficient strength to fully support itself (a minimum cube strength of $10\,N/mm^2$ is typically specified for this requirement)[30].

As noted above, suspended slabs are often adopted when high hydrostatic or swelling pressures are likely. Hydrostatic pressures can simply be resisted by the base slab which will have, at least, its self-weight to counteract the upward load. The inclusion of additional loads (such as the trackbed) in this compensation requires careful assessment, and the timing of trackbed installation and its future relaying may be critical in this context. Alternatively, the void beneath the slab may be drained to relieve the pressure. This could also be necessary for reasons of stability such as flotation. It is not attractive to incorporate drainage in this way as a basic structural requirement (as noted in Section 6.1.3).

In the case of swelling clays it is usual to avoid the full imposition of the normally high pressures that develop when long-term heave is restrained. Actual values are difficult to predict, but maximum potential swelling pressures can be of the order of the existing overburden before excavation. These pressures are strain dependent, and it is therefore usual to relieve their effect by providing a large enough void for the heave to occur largely unrestrained. However, for large spans where heave strain and, moreover, induced curvature of the slab may be accommodated and acceptable then only a partial void or no void at all may be required.

Suspending the slab usually requires much or all of the downward vertical loading to be transmitted to the subsoil through the embedded side walls. This does not normally present major problems for narrow structures with moderate depth of cover (say, 2 m or less). This configuration is typical for single-track running tunnels. As indicated, wider structures tend to incorporate intermediate support between the side walls. The vertical load carried by such support is normally transmitted through rather than onto the suspended base slab – for example, by using piles. If the side walls are required to carry substantial vertical loads, the depth of embedment may have to be increased disproportionately, potentially negating the advantages of suspended slabs. It should be observed, however, that suspended slabs are usually adopted as a secondary benefit or consequence of the need for embedded walls rather than the primary factor of the design concept.

6.5.4 Piled foundations

Apart from their use in peripheral walls, piles are not commonly required in cut and cover work. As mentioned above, they may be used to provide intermediate support for suspended slabs. A still less frequent example, although one worth greater application, is the use of settlement-reducing piles[29] or compensated foundations[31]. In these, the ground-bearing slab or raft transmits the majority of vertical load to the subsoil and piles are only placed locally where the settlement or bearing pressures would otherwise be unacceptable. Traditional practice, however, has been to design either the slab or the piles to carry all the vertical load.

Piling may be required with very high vertical loading or to reach competent strata below the structure where ground treatment or replacement is not practicable. High loads can arise from deep backfill or development above the railway. Development loading will depend on the size and nature of the proposed facility and its integration with the railway structure. If the development is not built concurrently with the railway, potential differential effects for the separate phases of construction will have to be considered, especially those relating to heave or settlement and stability.

It will be necessary to decided which type of the many forms of pile are appropriate for a given site. In particular, their load-carrying characteristics in relation to the ground conditions will help to determine whether they should be end bearing (straight shafted or with enlarged bases) or should principally derive their capacity through shaft friction. From the constructional viewpoint, availability of plant and the methods required for installation of the piles may be critical. The timing of this within the overall sequence of construction is also relevant. Limitations to plant access or mobilization periods may require early installation of the piles. If the same subcontractor, for example, is providing the piling to peripheral walls, it is usually expedient to install any vertical loadbearing piles during the same period. Early installation of piles is also a feature of the top-down construction method (see Section 6.2).

6.5.5 Heave-induced tension

Heave-induced tension in piles presents another interesting aspect of soil/structure interaction. Piles which nominally only provide vertical load-carrying capacity may be subjected to tensile forces when the overburden is reduced[29,32]. This tension is generated by shaft friction caused by the resulting heave. In such circumstances substantial amounts of reinforcement above the usual minimum requirements may be necessary. This applies particularly to clay soils, Wall piles with deep embedment may also suffer heave-induced effects. However, they normally have enough reinforcement, arising from flexural requirements, to cater for the additional tension. It is usual to reinforce such piles uniformly throughout their length due to the short- and long-term loading envelopes. Consequently the effects of bending and heave do not tend to be significantly additive.

Heave-induced tension is caused by differential movement with depth. This effect may pertain in the long term from swelling of clay soils if there is a net reduction in overburden after construction. Short-term heave may also be relevant where low cut-off piles are installed from a platform prior to main excavation. Long piles are potentially the most affected.

It may be attractive to limit pile length under heave conditions (either by adopting larger diameters or enlarged bases) since the longer the pile, the greater the proportion of tension reinforcement will be required. Where control of heave is not critical, the effects of the induced tension may be substantially avoided by the use of very shallow under-reamed piles.

There is no definitive answer regarding what amount of heave-induced tension is correct for design. However, higher values for the alpha coefficient for shaft friction are usually assumed than those for externally applied, vertical loads. For example, in London Clay, a value around 0.7 is considered appropriate instead of the 0.5 or less usually adopted for the latter situation[29]. Alternatively, effective stress design for shaft friction may be adopted.

Piles within the excavation may also be used to deliberately limit heave when control of movements adjacent to the excavation is critical [29]. This beneficial effect is difficult to quantify, but the cost of such heave-reducing piles may be reasonably offset if they are used to support the suspended base slab by the resulting reduction in slab thickness and reinforcement.

The soil/structure interaction is complex, particularly in the overall context of the complete structure, construction sequences and other time-dependent effects. More research is also necessary to understand more specifically the distribution and intensity of the shaft friction mobilized on piles. Even with simplified assumptions, evaluation of the design tensile load presents conceptual challenges.

The distribution of shaft friction and the associated induced tension will be time dependent and modified by the imposition of external vertical loads on the piles. These loads may be downward from columns or upward from hydrostatic or swelling pressure on the base slab. For certain load combinations some column loads may also be upward. The heave-induced tension, on a simplified basis, will be changed by half the amount of the externally applied load (Figure 6.19) [33].

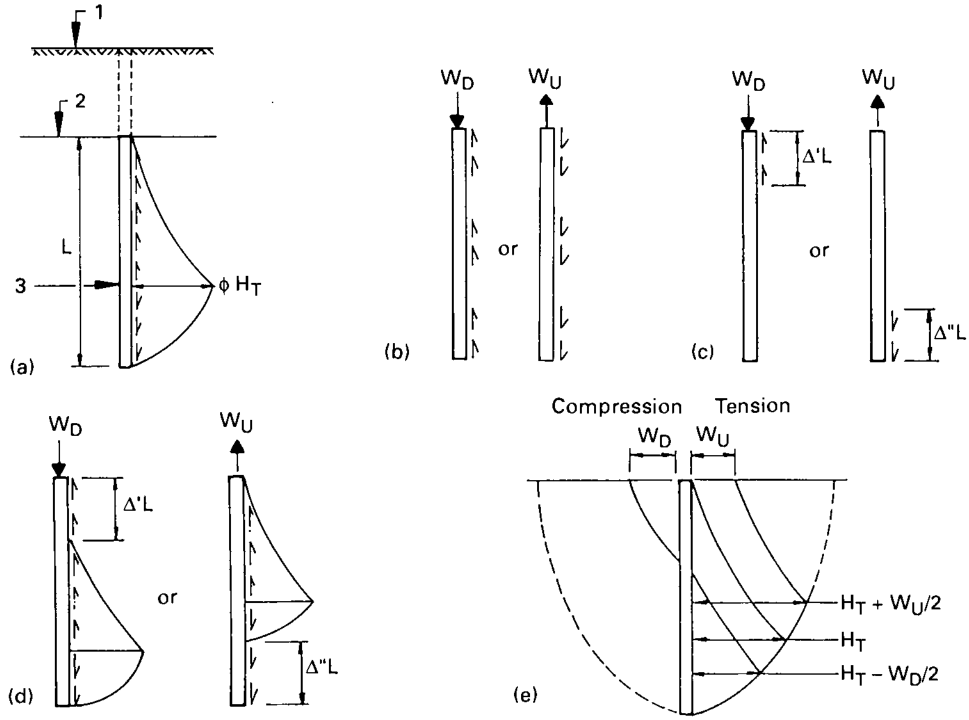

Figure 6.19 Simplified assumptions for maximum pile tension. (a) Maximum heave-induced tension = Q. Assume whole pile affected. 1 Original ground level; 2 formation level; 3 neutral point. (b) Shaft friction to resist imposed load. Could be mobilized in complex distribution – but assume case (c). (c) Imposed loads only. W_D: downward; W_U: upward. (d) Combined case assumption heave + imposed loads. (e) Pile tension/compression. (f) Maximum tension in pile (for given load case),

$T = (L - \Delta L/_2 \pm \Delta L)$

$= (L \pm \Delta L/_2)$

$= Q - W_{D/2}$

or $Q + W_{U/2}$ (absolute max. $T = 2Q$ – i.e. pull out)

6.6 Reinforced concrete design

6.6.1 General

The design of reinforced concrete structures is undertaken in accordance with the relevant codes of practice. However, the structural codes concentrate on the ultimate and serviceability limit states of the structure in its permanent condition. Moreover, such codes tend to be directed at above-ground structures while those for foundations concentrate on geotechnical aspects and refer structural design back to the structural codes. This situation might be satisfactory if it were not for significant differences between the design and construction of above-ground structures and those pertaining to foundation engineering. Apart from the predominant need in the latter to integrate structural design with soil/structure interaction and construction methods, particular differences arise concerning exposure conditions, durability, waterproofing, crack control and movement joints. For instance, emphasis on grade of concrete (compressive strength) in relation to durability requirements (based on generalized exposure conditions) is not adequate. Concrete permeability and crack control are more important parameters than compressive strength but are less rigorously assessed or checked. Compliance with specified compressive strengths is conveniently easier to achieve in practice but does not provide an intrinsic measure of durability. Moreover, increasing cement content (for higher concrete grades) to improve durability can be counterproductive. Higher cement contents can actually make concrete more permeable and may increase the risk of alkali aggregate reaction and early-age thermal cracking by raising hydration temperatures. For cast *in-situ* piles, unduly high-strength concrete also makes trimming back undesirably arduous – particularly if low cut-offs are involved.

A more appropriate approach towards durability for buried structures is offered with the use of plasticizers (thus providing the necessary workability with low water/cement ratios) and composite cements using ground granulated blast-furnace slag (ggbfs) or pulverized fuel ash (pfa). The incorporation of movement joints should also be avoided wherever possible. The global effects of temperature and long-term drying shrinkage in buried structures are not usually critical [34,35], their primary effects tending to be differential through thick reinforced concrete sections. The overall global movements are thus minimal and the associated strains adequately controlled by the longitudinal reinforcement initially required for early-age thermal crack control. Movement joints are always potential weak points and vulnerable to seepage and durability problems – particularly if the shear forces to be accommodated are high, since this will significantly concentrate stresses and complicate the associated structural detailing. The structural codes also do not adequately cover the design of struts which have lateral restraint and/or axial loads which are deflection dependent – a common situation in soil/structure interaction. Nor is shear in circular sections comprehensively addressed, as is relevant to bored pile design and particularly piled walls – see Chapter 8. (Useful empirical information on this subject is given by Faradji [36].) Other aspects on the shortfalls in the structural codes are discussed below, particularly with regard to the implications of chlorides on the durability of cut and cover tunnels.

The current codes therefore present immediate problems to the designer in producing safe, appropriate and economic designs of foundation engineering structures. There would seem to be a need for either the existing codes to be made more specific and comprehensive or to draft a separate foundation engineering code.

6.6.2 Structural analysis

Structural analysis, however important and interesting, is but a part of design. Undue emphasis may be attached to analysis because it appears scientific and attractively provides numerical answers. However, achieving overall validity in the design concept, particularly in foundation engineering, is an art dependent on engineering judgement and experience [4]. Analysis must be viewed in the context of its limitations. The mathematical model used and the numerical input to it will never fully represent reality.

Design must address the complexities of soil/structure interaction but, however comprehensive, it will never provide a complete evaluation of this mechanism. Design itself is but a part – the other is construction [37]. The construction methods, quality of workmanship and the actual ground conditions encountered all bear on the validity of design assumptions.

It is important to get the basic concepts right in the early stages of design. The soundness of these concepts must then be reviewed as the detailed information of the site, ground conditions and construction methods becomes available. The methods of analysis may range from simple hand calculations to complex computer-based techniques using finite elements or finite difference methods [19,29,38].

Computers are very useful in undertaking comparative evaluations and testing the sensitivity of the soil/structure interaction to variations in the assumed soil parameters. For example, small changes, particularly in cohesion values, can easily have very large effects on factors of safety for stability. This pronounced sensitivity is not the norm in analysis of engineering structures generally, where small changes to strength factors are often secondary or even insignificant. It highlights the importance of conservatism, qualified by specialist advice, in the selection and application of geotechnical parameters. Caution should also be exercised in exploiting the analytical power of computers to refine designs [25,39]. Failure is a notably difficult condition to model – but soil in a state of at least partial failure is the typical condition in soil/structure interaction [40]. It is appropriate, therefore, that the design overall and, in particular, stability should be broadly verifiable by basic calculations using simplified assumptions. Terzaghi has often said that if a theory was not simple it was of little use in soil mechanics [4].

6.6.3 Durability

It is essential that underground structures are durable since the facility for inspection, maintenance and repair is very limited. However, the exposure conditions with uncontaminated groundwater are moderate and typically much less severe than those to which surface structures are subjected. An important exception to this occurs when tunnels are exposed to the combination of significant external water pressures with the presence of chlorides or sulphates – and so groundwater composition must always be checked. This situation is discussed in more detail below.

The exposure conditions in cut and cover tunnels adjacent to portals with open sections of railway may also create more critical criteria through higher temperature variation and freeze/thaw cycles. Cut and cover structures away from portals, however, with a metre or more of groundcover, do not usually suffer significantly from the deleterious effects of freeze/thaw action and wide variations of temperature in the permanent condition – as would apply to an above-surface structure – but significant exceptions are possible and the overall environment needs careful appraisal [4].

The basic approach to durability is to relate concrete grade (comprehensive strength), cement content (and in the case of sulphates, cement type or cement replacement), water/cement ratio, crack width and depth of cover to the reinforcement to the conditions of exposure. A detailed review of the durability of reinforced concrete structures in the context of current design practice is given to Somerville [5]. This emphasizes the importance of the four 'Cs' – Constituents of the mix, Cover, Compaction and Curing.

Notwithstanding the above considerations, the presence of aggressive agents such as chlorides or sulphates should always be treated with caution. The potential levels of concentration of such chemicals and the measures needed to resist their effects must be considered in the design of each structure. As previously emphasized, tunnels subjected to external water pressure merit particular consideration. The permeation and diffusion conditions involved here with the wet outside/dry inside situation of such tunnels (similar to the so called 'hollow leg' problem associated with offshore structures) can lead to the transmission and eventual harmful concentration of chlorides or sulphates within the structure. Without appropriate measures of protection, serious corrosion of the reinforcement and spalling of the concrete may occur over a relatively short period and well within the design life of the structure – even with low levels of such chemicals in the groundwater. Wick action, the supply of oxygen internal to the tunnel and the electrical continuity of the reinforcement [16] tend to exacerbate the situation (Figure 6.20) [46]. Under these situations not only is the ingress of chlorides faster, but corrosion, once initiated, will be more rapid due to the 'hollow leg' phenomenon – the drier interior acting as a cathode and potentially severe localized pitting corrosion developing at the outer face from macro cell action under low oxygen conditions. For such exposure conditions with chlorides or sulphates, low concrete permeability, crack control and watertightness assume special importance to achieve durability. Extra measures of protection may also be required – robust and reliable impermeable membranes (such as steel sheeting), cathodic protection or the use of epoxy coating of the reinforcement are relevant considerations (the latter two measures relate specifically to protection against chlorides).

Stray current protection in a d.c. current environment may also be critical [16].

6.6.4 Waterproofing

Waterproofing can be a controversial subject – not least because interpretations differ on the meaning conveyed by the terms 'waterproof' or 'watertight'. In reality, it is impossible to guarantee absolute watertightness and in general this is not necessary. It is helpful to assess the requirements for watertightness in the context of grades of performance as described, for example, in CIRIA Guide 5 [41]. The basis of watertight reinforced concrete is good-quality design and construction, giving particular attention to the four 'Cs' (see the reference to Somerville [5] in Section 6.6.3) to achieve dense, low-permeability concrete with well-controlled cracks. (The combination of internal and external restraint to early-age thermal contraction inherent with *in-situ* construction of the thick reinforced concrete sections associated with cut and cover tunnels tends to make cracking inevitable – see Section 6.6.5.)

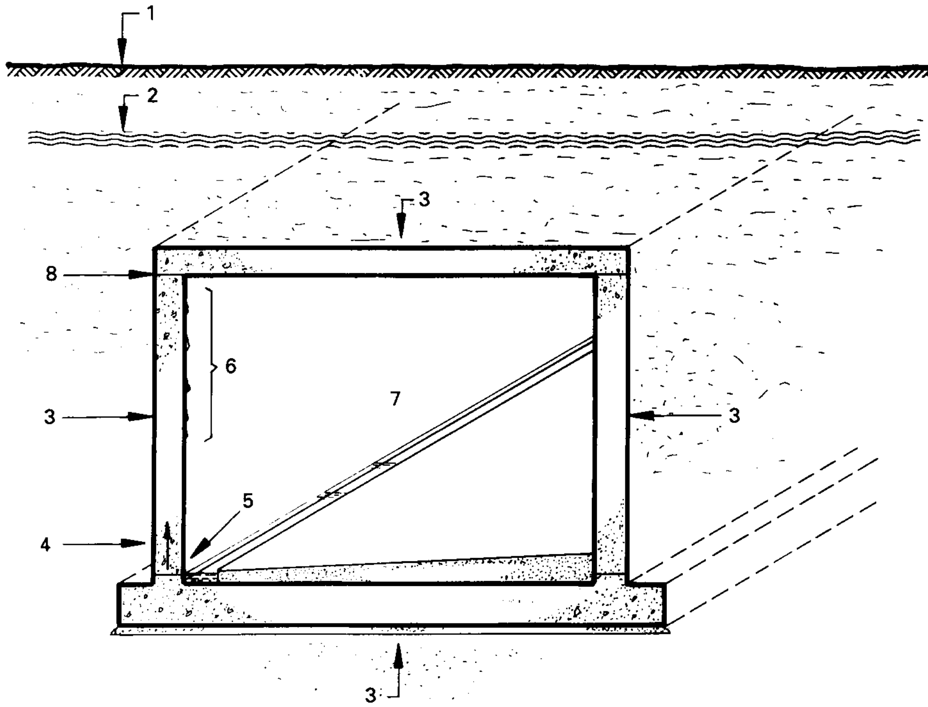

Figure 6.20 Schematic representation of the various chloride, sulphate and alkali ingress mechanisms operating on a buried box type structure. 1 Ground level; 2 water table; 3 permeation, diffusion and wick action; 4 wick action; 5 wick action and wetting and drying effects (concentration of salts, etc. in drainage water due to evaporation); 6 wetting and drying effects; 7 50% relative humidity, 20°C; 8 seepage through joint or crack [46]

In general (i.e. where conditions such as described in the last paragraph of Section 6.6.3 do not pertain) the requirements for waterproofing cut and cover railway construction may be broadly placed into two performance grades – utility and habitable, related to running tunnels and stations, respectively. Both grades should be based on watertight reinforced concrete construction, as noted above. For running tunnels (appropriately designed, detailed and constructed), reinforced concrete should be adequately watertight. There should be no need, in the absence of aggressive chemicals such as chlorides or sulphates, to provide additional protective measures such as impermeable membranes. Most specifications permit a limited amount of controlled seepage in running tunnels. Undue seepage should not be tolerated, however, and indeed seepage should not occur in good-quality construction. Seepage may arise through early-age thermal cracks or those induced by flexure in the longitudinal direction, such cracks tending to pass right through the section. The design should limit the widths of such cracks to a maximum of 0.2 mm [42,43]. At this width (or less) the cracks tend to seal autogenously [43,45]. This self-sealing process is usually effective within a few months after the initiation of seepage (and well before commission of new works), and can be encouraged by the controlled application of fresh water to the concrete surface after the initial curing period.

For stations, seepage or damp patches (i.e. visible penetration of water) are not acceptable – at least in the areas used by passengers or staff or rooms housing sensitive equipment. The habitable grade of internal environment is thus applicable here. This is frequently achieved by the same reinforced concrete design as for running tunnels but with the provision of internal drained cavity walls. The use of externally applied impermeable membranes may also be considered. It is important to note, however, that apart from the significant cost of membrane systems of waterproofing, their success depends upon the highest quality of workmanship and materials. Particular attention must be paid to simplicity of detailing (avoiding complex geometry) and to good supervision. The quality of the concrete surface, especially in the case of bonded membranes such as bitumen sheeting, is critical and is notably not easy to achieve in the heavy civil engineering conditions of cut and cover construction. There is usually significant pressure on the construction programme to backfill completed sections as soon as possible and often while the concrete is still in its early phase of hydration. Such conditions are hardly conducive to the proper application of most bonded membrane systems. Leaks caused by defects in external membranes are practically impossible to locate and repair – the water invariably enters internally through cracks or other vulnerable points in the concrete (such as movement joints if present) at some distance from the external defect.

A further critical consideration is that many such defects are latent in nature and tend to appear after the railway is in operation. The disruptive costs of the associated remedial works can consequently be onerous. A contributory factor to the incidence of leaks from such latent defects can arise from the fact that the early application of the membrane may prevent or retard autogenous healing of the cracked concrete. This healing process involves the transport of the soluble calcium hydroxide within the crack, forming the insoluble compound calcium carbonate on contact with air. Calcium hydroxide is more freely available in early-age concrete and, moreover, long-term cracks in mature concrete will have suffered some degree of carbonization from the air inside the tunnel. Such carbonization of the crack surfaces will potentially inhibit the self-sealing process. In short, there is much to be said for the simplicity of straightforward, robust, watertight concrete construction which, in the absence of an external impermeable membrane, can be fully tested and proved prior to the commissioning of the railway. As stated, such quality of reinforced concrete should form the basis in cut and cover construction – this approach, in any case, must be followed with diaphragm or piled walls where the use of external waterproofing is not possible. CIRIA Guide 5[41] provides a useful assessment of the supplementary measures related to drained cavities and external waterproof membranes.

Attention to detailing and geometry is necessary to minimize potentially vulnerable points such as stress concentrations or reinforcement congestion (which may lead to poor concrete compaction). Wherever possible, water should be directed to flow away from or past the structure – top slabs, for example, should be detailed with falls (preferably not less than 1 in 50).

Finally, as emphasized in Section 6.6.3, the presence of aggressive agents in the groundwater always needs careful consideration.

6.6.5 Structural details

Emphasis in the structural detailing, as in overall layout, should be placed on simplicity, buildability and durability. The location of construction joints and the associated reinforcement details may be critical and can clash with construction methods. The use of couplers for rebars can eliminate congestion and ease construction operations. For example, reinforcement couplers at the tops of walls for the wall/top slab continuity facilitates the use of travelling shutters for both the wall and top slab pours by eliminating the horizontal projection of starter bars beyond the walls.

Crack control

It is a fundamental fact that reinforced concrete cracks. This is inevitable in its hardened state when tensile strains arise from imposed structural loads. Cracks also occur in the plastic and hardened states from plastic settlement and thermal and shrinkage effects. The various types of cracks and their causes are categorized in Figure 6.21. This list does not include micro-cracking, those arising from incorrect practice (such as using calcium chloride as an accelerator) or those from inadequacies in constituents. It is important to control potential cracking from all these causes and many of them can be eliminated by good-quality design and construction. The importance of a robust approach with fully controlled crack widths has been emphasized in Sections 6.6.3 and 6.6.4.

Cracking from long-term drying shrinkage is often raised as a concern but generally its effect is secondary. This particularly applies to the thick sections and

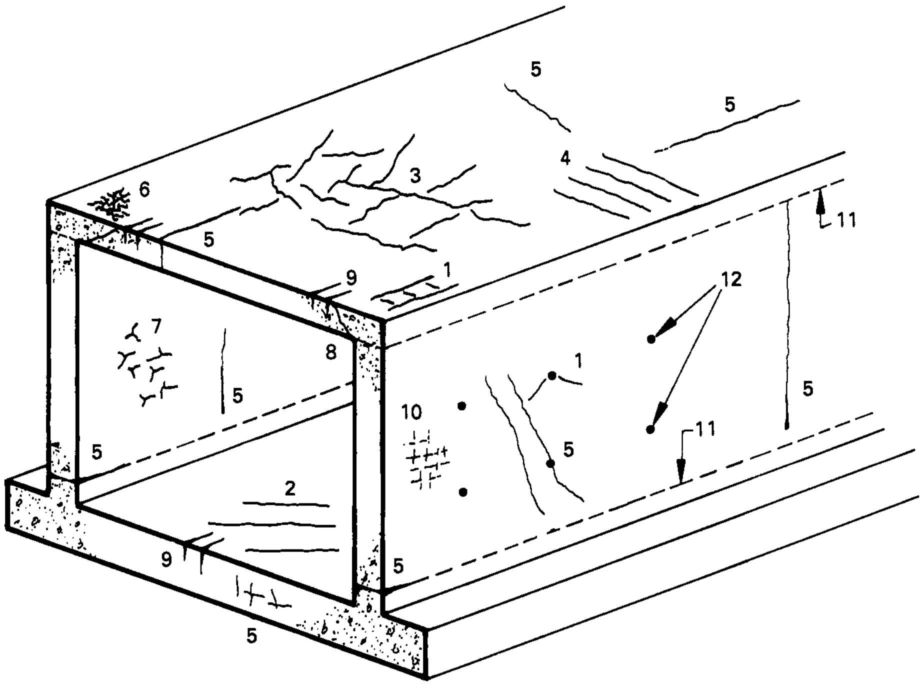

Figure 6.21 Some examples of intrinsic cracks in reinforced concrete box structure (adapted for thick sections from Figure 2 of reference 34). 1 Plastic settlement (over reinforcement and shutter ties); 2 plastic shrinkage (diagonal); 3 plastic shrinkage (random); 4 plastic shrinkage (over reinforcement); 5 early-age thermal contraction (thick sections); 6 crazing; 7 alkali aggregate reaction; 8 shear; 9 tension bending; 10 thermal shock; 11 kicker; 12 shutter tie holes

conditions associated with buried structures. The effects of long-term drying shrinkage are mitigated by creep and the fact that reinforced concrete in contact with the ground is unlikely to dry out fully. It has been demonstrated by Hughes[35] that early-age thermal strains are far greater than those of long-term shrinkage and are the prime reason for cracking in retaining walls and similar reinforced concrete structures. Two principal causes of cracking, thermal effects and flexure, are considered in more detail below.

Thermal effects These effects derive from two main factors – seasonal temperature variations and early-age thermal contraction. For buried structures the latter is dominant. In railway tunnels under normal operating conditions the annual temperature variation is low and would not typically exceed 10°C. Below a depth of 2 m the seasonal variation in ground temperature is generally only a few degrees Centigrade, although this may be increased in the ground adjacent to the tunnels. With adequate reinforcement present to control early-age thermal contraction, these effects should not be critical globally. In any case, such temperature variations tend to be differential through the thick reinforced concrete sections rather than global, so the need to accommodate any overall movement is usually negligible. Temperature variations at portals with open sections are higher and should be evaluated separately.

Early-age thermal effects are critical. Hydration temperatures generated in thick sections are high and the contraction is restrained both internally (by the core) and externally (by previously cast connecting sections)[42]. The design approach for reinforcement to control the resultant cracking must take into account the significant differences between thin and thick sections and the associated influence of the maximum and minimum cracking mechanisms[44, 50]. The maximum crack width for early-age thermal effects should be limited to 0.2 mm[11]. Control of early-age thermal cracks is critical because:

1. They pass right through the structural section, thus allowing seepage and groundwater to reach the reinforcement at both faces.
2. The cracks tend to run parallel to rather than at right angles across the main flexural reinforcement. Site surveys have shown that the latter cracks are unlikely to create significant corrosion problems but those parallel to the reinforcement are frequent contributors to serious corrosion. (As noted in Section 6.6.3, the presence of chlorides always needs particular consideration and the incidence of *any* cracking can then have a critical influence on corrosion and durability.)

Control of thermal cracking is enhanced if the distribution (or longitudinal) reinforcement is placed outside the main flexural steel. This not only makes steel fixing easier but the main reinforcement will have extra cover.

Flexural cracking Flexural strains from imposed loads are usually dominant in the transverse direction of tunnels. Deflections in the longitudinal direction (i.e. along the line of the tunnel) are generally not critical and the quantity (and spacing) of longitudinal reinforcement is then governed by the requirements for early-age thermal crack control.

Flexural cracks taper towards the compressive zone and do not pass through the section. Their influence on durability is also not generally so critical as early-age thermal cracks (see above). Consequently, limiting flexural crack widths at the surface of the concrete to the same maximum of 0.2 mm may not be necessary. The British Structural Code [30], for example, sets this limit at 0.3 mm. For a given bar cover and spacing the area of reinforcement is inversely proportional to crack width, therefore changing the maximum width from 0.3 mm to 0.2 mm can increase the reinforcement requirement in the order of 50%. The cost is therefore very significant and the benefits of reducing the flexural crack widths in this range are questionable. Another major consideration is the potential steel congestion arising from stringent limits on crack widths. Poor compaction of the concrete and the plane of weakness associated with layers of closely spaced rebars may in fact lead to serious durability problems.

6.7 Design development

6.7.1 Introduction

The range of design development involves the initiation of preliminary appraisals to generate the conceptual basis through to the production of the detail drawings for final construction. The principal aim of the design must be to enable a safe construction which is fit for its intended purpose. In addition, emphasis must be placed on achieving a cost-effective yet robust structure which is durable with low maintenance requirements. Structural design must be comprehensive in addressing the geotechnical aspects of soil/structure interaction and the implications of construction methods.

6.7.2 Design interfaces

To produce an adequate design, the designer will need to consider a multiplicity of interfaces which need rigorous coordination. This applies particularly to congested urban sites and major projects involving many disciplines. Comprehensive and verifiable controls of design information and communication are vital. This is essentially the quality assurance of the design process and is best achieved by the implementation of an appropriate Quality System. This should balance effectiveness with the need to limit undue paperwork and administration. It should be based on common sense and no system will compensate for lack of quality in the ability and attitude of design team members.

While not comprehensive, the following may serve as a basic checklist of design interfaces:

1. *Geotechnical.* Particularly soil parameters, slope stability, ground movement and groundwater control.
2. *Construction methods.* These involve not only those employed for the cut and cover structures but also those associated with adjacent or connecting construction such as bored tunnels. Chambers or portals for the installation or removal of tunnelling machines are typical examples.
3. *Environmental.* For urban conditions control of noise and dust pollution may be critical. In open or greenfield sites the protection of aspects of special scientific interest or areas of outstanding natural beauty needs evaluation. Hazardous sites and ground conditions can be major considerations.

   Adjacent structures and services. Directly relevant to (1) and (2) above and the associated needs for structural appraisal, evaluation, instrumentation, monitoring and remedial measures.

Existing roads and services. Early identification of the need to avoid, incorporate or divert such facilities is vital. Long lead times for planning and execution and high or prohibitive costs[10] are often involved which can critically affect the design approach and construction methods.

Architectural and landscaping. External appearance needs coordination with the environmental issues and will affect structural design particularly with regard to treatment of exposed portals, depth of backfill and tree planting. Architectural requirements may fundamentally affect the structural form and layout of stations. Portals are often major architectural features.

Future development. Integration with proposed development above or adjacent to the railway may be required. Effects of high vertical loading are a major factor.

4. *Operation and function.* Major facilities such as ventilation shafts and crossovers are frequently involved. Indeed for the latter it is often only feasible to construct these in cut and cover where lack of depth and quality of the subsoil obviates the possibility of a crossover housed in bored tunnel.

   Mechanical and electrical. Provision for services, pressure-relief ducts, ventilation shafts, substations, equipment rooms and drainage. Internal environment – temperature and humidity ranges, air pressures from piston effect.

   Track alignment, layout and spatial requirements.

   Trackwork. Types of trackbed, form of support and sensitivity to deflections.
5. *Safety and security.* This includes not only those aspects pertaining to construction but also those relating to the operation of the railway system.
6. *Programme.* Design production must be coordinated with the construction programme which, in turn, may dictate the adoption of particular construction methods with a consequential impact on the design approach.

6.7.3 Conceptual design

The conceptual design phase for the structure will normally commence once the right-of-way corridors are established and the preliminary railway alignments are available. These will indicate the likely extent and nature of the cut and cover works and their environmental context. The depth of construction together with the location of major facilities such as stations, crossovers and ventilation shafts will then provide the basis for establishing the broad design concepts. This will initiate an interactive process, maintained throughout the whole design development, which addresses the various interfaces with other disciplines. The requirements for site-specific information should now be identified, particularly the early evaluation of that critical to design concept validity. Initial site investigation should address spatial constraints, environmental impact, probable ground strata and conditions. Preliminary consideration of possible construction methods for each site is also important at this stage. The design development must be kept compatible with them.

6.7.4 Outline design

Outline designs are developed from the conceptual evaluation. This stage should not be design intensive but produce basic small-scale general arrangements of layout. Typically, two principal options for each site may be developed unless a clear solution is evident. Specific adjustments to the railway alignments within a given corridor will enable preferred locations for the cut and cover construction to be established. All appropriate forms of construction should be addressed within the context of cost, buildability, plant and programme. Detailed information and requirements for each site will allow refinement of each option.

While it is crucial to establish an appropriate basis to proceed with the Definitive Design, progress of the structural Outline Design tends to be limited by the lag of development in other disciplines. This results from the highly iterative and interactive nature of the overall design process for these projects. The need to produce appropriate designs to programme often requires the designer to initiate and lead design coordination with other disciplines. It is important to realize this. It will be necessary to identify the content and timing of all design inputs required in order to progress the structural design. Consequently, an Outline Design report should list all outstanding aspects of input that need to be resolved before or addressed in the Definitive Design stage. As such, an Outline Design report is a key document and an essential part of a Quality Plan. Irrespective of whether it is issued formally to the client, it must form the basis for subsequent design development.

6.7.5 Definitive design

The Definitive Design can be progressed after selection of the preferred Outline Design alternative. The principal objective is the production of general-arrangement drawings which accurately define the layout and dimensions of the structure. Reinforcing details will also need to be considered and it is advisable to produce typical layouts which show the range of these requirements. The Definitive Design drawings, together with the specification, form the technical basis for tender information. The bulk of structural analysis should occur during this phase. The establishment of member sizes and percentage of reinforcement will enable accurate estimates of the direct cost of the structure. The drawings should show proposed construction sequences including requirements such as temporary support positions and loadings.

6.7.6 Detail design

This phase concentrates on the production of full reinforcement detail drawings. Ideally, they should relate directly to the design produced for tender. However, there are invariably changes to incorporate. As noted, it is likely that changes generated from the requirements of other disciplines will occur through all stages of design development. Added to this, the full design impact of construction methods inevitably occurs after selection of tenders. This underlines the need for early evaluation of the design with construction methods. Major design changes at this stage, resulting from inadequacies in concept in the widest sense, are clearly undesirable.

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Chapter 7

Diaphragm walls

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Contents

7.1 General description of the technique

A diaphragm wall is an *in-situ* concrete membrane of finite thickness which is constructed in the ground. The trench into which the *in-situ* concrete is poured is kept open during excavation by a stabilizing fluid, usually a suspension of bentonite. Bentonite is produced from montmorillonite clays, which are converted to sodium montmorillonite, and form a thixotropic suspension. Other materials have been used as stabilizing fluids and a number of specialized drilling fluids which have been developed for the oil industry are suitable.

The stabilizing fluid or slurry fills the trench and is finally displaced by concrete. This is poured by tremie pipe progressively from the bottom of the excavation upwards to form a rigid membrane. In certain cases a plastic concrete wall is required, and this can be achieved by mixing bentonite as partial cement replacement in the tremie concrete mix or by mixing aggregates and cement with the bentonite in the trench. Prefabricated reinforcement cages, ducts and other inserts can be placed in the slurry before concreting.

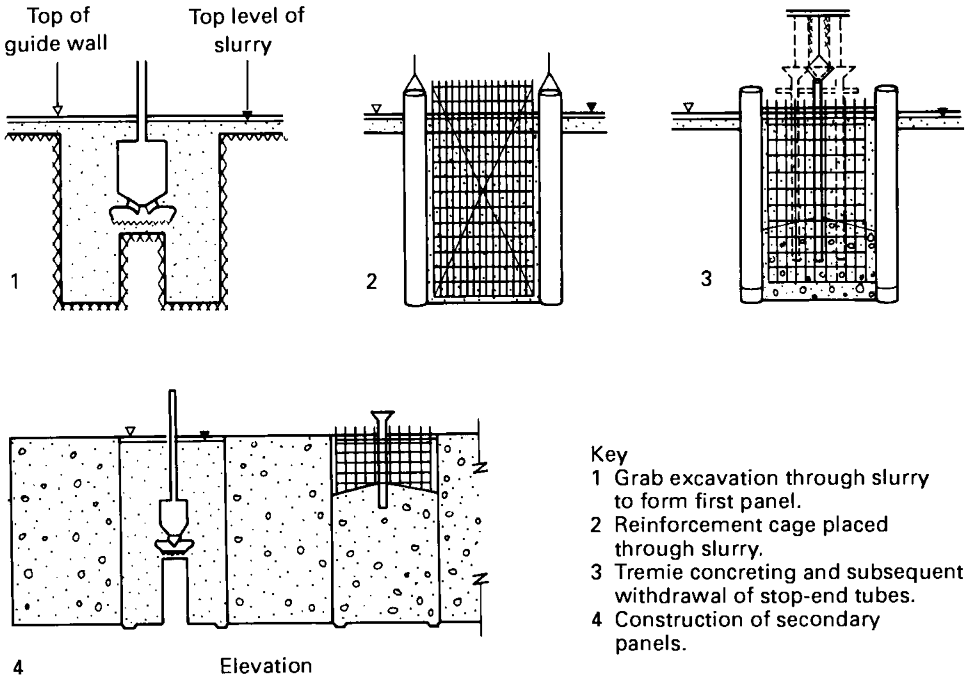
The diaphragm method is distinguished from that of the contiguous bored pile system by the method of excavation. The diaphragm wall trench excavator is guided by a shallow concrete-lined pre-trench on the line of the intended wall. The excavator typically cuts a slot 0.6–1.5 m wide by 2.4 m long in a single pass and excavates a typical diaphragm wall panel, approximately 6 m long, by multiple passes.

The panels should be as long as possible within stability limits to reduce the number of joints. Alternate panels are normally completed as primary panels between two stop ends, usually in the form of steel tubes. After the primary panels have cured sufficiently the stop ends are removed and the intermediate or secondary panels are constructed to form a continuous wall (Figure 7.1).

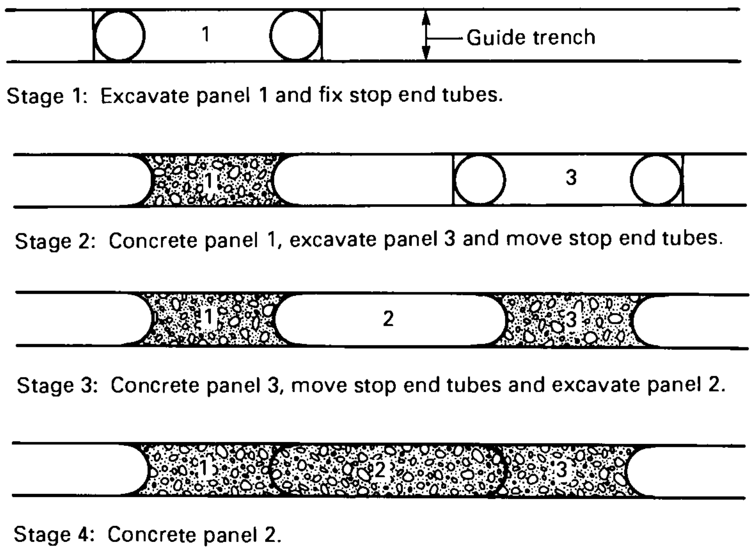
7.2 History of development

The system of slurry-stabilized trench walls was first used in the construction of cutoff walls for dams in Italy in the early 1950s. The technique was developed by Dr Christian Veder in association with Dr Ing Francesco Brunner, Chief Executive of Impresa di Construzione di Opera Specializzata (ICOS). The first large-scale structural use of the method was in the construction of the Milan metro, where it was used to construct running tunnels, following routes of existing roads, using cut and cover techniques. The first use of the system in the United Kingdom was in 1960/1961 during construction of the Hyde Park underpass in London, which incorporated the existing underground station. Since that time the method has developed rapidly and there are examples of its application in all areas of civil engineering.

Diaphragm walling is ideally suited to construction in congested city-centre sites where restrictions are imposed on noise or vibration and where limited working space is allowed. By constructing the retaining walls before commencing excavation, ground movements are reduced and settlement of surrounding buildings can be minimized. Cast *in-situ*, diaphragm walls have been used on many metros in recent years, including Hong Kong, Singapore, Caracas, Cairo, Rio de Janeiro, Kyoto and Fukuoka.



(a)



Plan
(reinforcement cage omitted)

(b)

Figure 7.1 Construction of diaphragm walls; (a) elevation; (b) plan

There are a number of well-known examples where diaphragm walling has been used purely as a groundwater cutoff. Perhaps the most notable, although not associated with rail transport, was at Chernobyl (USSR), where a 4 km long impermeable diaphragm wall underground barrier was constructed around the damaged nuclear power plant. The barrier extended into the underlying chalk at a depth of 85–100 m to prevent radioactive contamination of groundwater.

Circular cofferdams or shafts can be constructed from diaphragm walling without reinforcement between panels if compression can be maintained across the joints. Notable examples of this form of construction are the upper section of the main access shaft at the French terminal of the Channel Tunnel at Sangatte and the ventilation and access shafts for the first metro tunnel across Hong Kong Harbour.

Being a form of *in-situ* concrete piling, diaphragm walls can also be used as high-capacity vertical loadbearing members in suitable ground conditions. It is a particularly economic alternative to a raking pile group where a combination of vertical and horizontal loads are to be carried. At the new nuclear submarine construction facility at Barrow (UK) 42 m deep T-shaped diaphragm wall sections have been used both as a cantilever quay wall and to carry large vertical reactions from the ship lift. Similarly, diaphragm retaining walls for underground railway stations have been used, in conjunction with ground beams, to underpin existing buildings, such as surface railway stations with which passenger interchange is required.

Recent developments have related to the type of excavation equipment being used and a continuous refinement of the technique. A family of specialized reversed-circulation trench cutters has been developed for larger projects and it is now possible to achieve a consistent quality of diaphragm walling in most ground conditions by suitable choice of stabilizing medium.

7.3 Applications – advantages and limitations

Diaphragm walls offer a means of construction for elements of underground railways which in many cases has been found to be the most attractive and effective. Circumstances and related aspects of the construction method which lead to this are as follows.

7.3.1 Environment

In city areas it is often necessry to limit environmental nuisance arising from construction or modification of an underground system. Noise and vibration of a diaphragm wall operation compare very favourably with any driven piling alternative. Modern rotary cutting heads have resulted in even further improvement in this respect when compared with older chiselling methods for gaining penetration into rock.

There remains the problem of disposing of spent bentonite. This probably requires tanker transport in city areas and development work is taking place on alternative means of disposal.

7.3.2 Subsidence and deformation limitation

Diaphragm walling permits construction and excavation very close to existing buildings or other movement-sensitive installations such as foundations or piped services. During the forming of the wall vibration is limited as described in Section 7.3.1. Assuming that main excavation is to proceed up to one face of the wall, significant movement can be prevented by ties or struts installed progressively as excavation proceeds downwards. Waling beams to take the tie or strut loads can be built effectively into the diaphragm wall itself by beam reinforcement within the main panel cage. Alternatively (or additionally) a beam can be constructed or inserted as excavation reaches the relevant level.

7.3.3 Limited working space

Diaphragm walling is possible to within very close proximity of the edge of the working site. With normal working space available on one side, the only space requirement on the other is for the width of the guide trench wall. In extreme cases walls have been constructed within a corridor as narrow as 3 m. The advantages are substantial in congested city centres.

For a cut and cover operation along a city street, a diaphragm wall can be constructed on one side, while keeping traffic running. The same operation can be performed on the other side. A roof slab can be constructed rapidly between the walls during the only required complete street occupancy, although even then the slab can be completed in alternate panels. Starter bars for slab connections can be bent into the main reinforcement cage. Boxes to help their later exposure and other box-outs can also be connected to the main reinforcement cage. The general excavation can be carried out under the completed roof slab with city traffic again flowing above.

Most diaphragm wall equipment requires substantial vertical clearance. However, plant designers can respond to vertical clearance limitations if necessary. In the case of Argyle Street, Glasgow, walls were constructed from within an existing tunnel with horizontal clearances of 7 m and vertical of only 4.5 m. In Tokyo, diaphragm walls were carried out from temporary pits underneath road level.

7.3.4 Cutoffs

Diaphragm walls have been used extensively as cutoffs against water flow. They provide this facility in underground railway construction although perhaps not generally as a main purpose. For cut and cover construction in permeable ground below the water table a complete or partial solution to the groundwater problem for the excavation may be obtained by taking the side walls down to an impermeable stratum or suitable cutoff level, using diaphragm walling techniques. Socketing into hard rock strata can be achieved if necessary, although a change of excavating equipment to chisels or special cutters is likely to be required.

If used solely for a water cutoff, the diaphragm wall can be constructed of unreinforced plastic concrete. In this, part of the cement in the mix is replaced by bentonite, and the main specialist contractors also have a range of additives to give additional flexibility and impermeability. Alternatively, thin slot walls can be formed and backfilled with cement, clay or chemicals or a plastic sheet can be inserted. However, in underground railway construction the permanent walls are also likely to be loadbearing and therefore of normal concrete and conventional thickness.

Transverse cutoff can be achieved by two temporary transverse walls, perhaps of plastic concrete to complete a protected box. In less permeable ground an extended length of the structural parallel walls may be sufficient to protect short advances of the contained excavation, with or without some associated groundwater lowering. The achievement of groundwater cutoff by diaphragm walling may permit atmospheric working where otherwise compressed air working or sophisticated pre-treatment grouting would be necessary.

Diaphragm walls may also be used to protect both temporary and permanent works from aggressive chemicals in polluted groundwaters. To act as a protective barrier in this way, the walls themselves must be resistant to the chemicals, and this needs to be effective at two stages. First, the slurry for construction must not be reactive with the polluting chemicals, and flocculation must not take place. The specialist contractors have slurry additives or bentonite replacement materials to deal with the majority of pollutants. Second, the permanent wall concrete must be resistant to the polluting chemicals, for example by the use of sulphate resistant cement in the case of sulphate-rich groundwaters. Another possibility is the use of the diaphragm wall sacrificially, either by thickening it above structural requirements or by building a second permanent structural wall within.

For purely cutoff purposes, slurry excavated trenches have been backfilled by tremie-placed hot bitumastic, the slurry pressure preventing boiling of the suspension water until very near the surface. Such methods must be considered unusual and warranting field trials. Heat dissipation must be considered, as must long-term flow of the viscoelastic filling medium.

7.3.5 The ground as a shutter

With diaphragm wall construction, the ground and its supporting mud cake act as a shutter for the tremie-placed concrete. This is inherently cheaper than formwork with supports, but the finish is not architecturally attractive. The diaphragm walls can, however, be finished satisfactorily by secondary concrete or architectural applications after excavation. The wall itself will accept fixings to support either system.

There has been use of precast panels to form diaphragm walls but sufficiently accurate placement within the slurry before wall concreting or the setting of grouts and without damaging the mud cake is not easy. Such panels give an additional possibility of creating structural continuity across panel joints and of locating built-in items more accurately. The ground also acts as a soffit shutter in the cut and cover and top-down construction sequence described above. Slabs, strut beams and waling beams can all be constructed with support from suitably prepared ground as excavation proceeds downwards. Heavy steel or precast concrete beams have also been used as struts, minimizing delays in excavation.

An interesting possibility with loadbearing diaphragm walls is that of constructing both upwards and downwards concurrently. Property development is increasingly common over railways, be they part of an underground or overground rail system. Structural work above ground is perfectly feasible with the diaphragm walls accepting the required foundation load, while cut and cover excavation proceeds below as described previously. Particular care must be taken against deformation of the diaphragm walls before lower-level strutting or tying is completed. However, overall there could well be earlier use and income from the above-ground development, thus improving project cash flow.

7.3.6 Structural modes

Diaphragm walls can act in a great variety of structural modes. The most common in underground railway work will be as a propped cantilever, where permanent floors are used as props, excavation of each level being carried out from underneath the upper floor slabs. This is termed top-down construction (Figure 7.2(a)). With greater resistance to ground movement required, ties or further levels of propping may be added.

For approach cuttings, the wall may act as a free-standing cantilever, with or without invert level propping. Fills of 20m have been retained by such walls forming quays at Bristol, Liverpool and Redcar (Figure 7.2(b)).

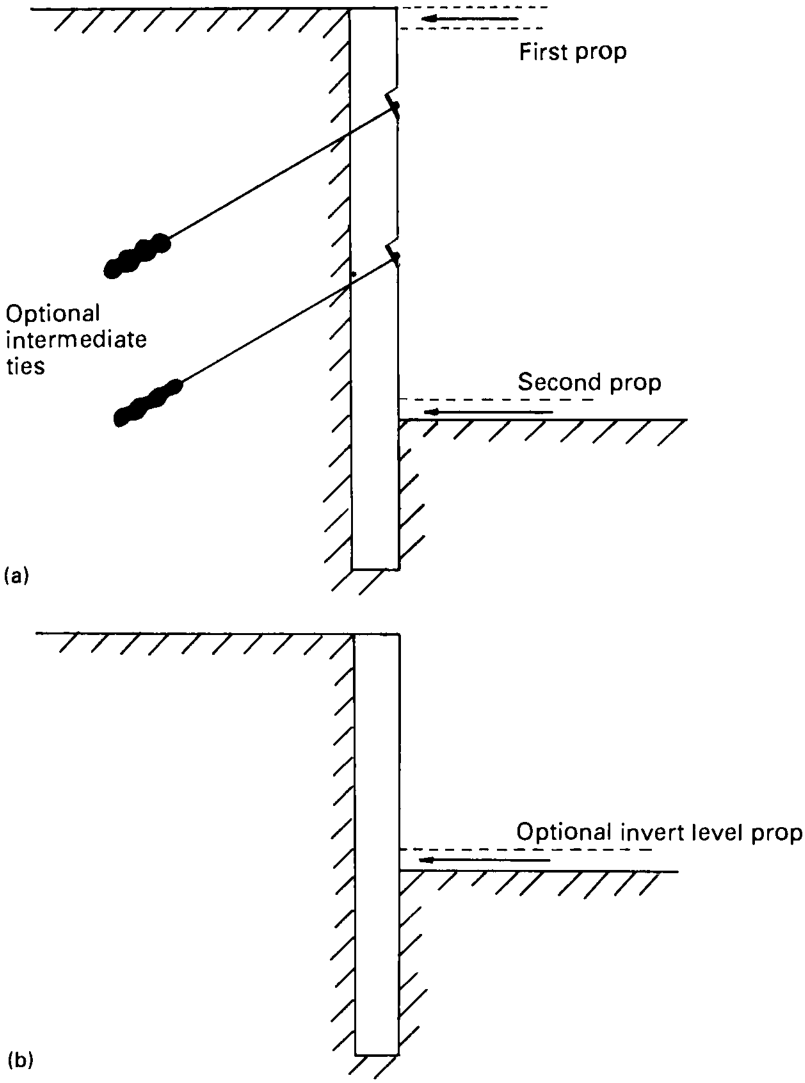
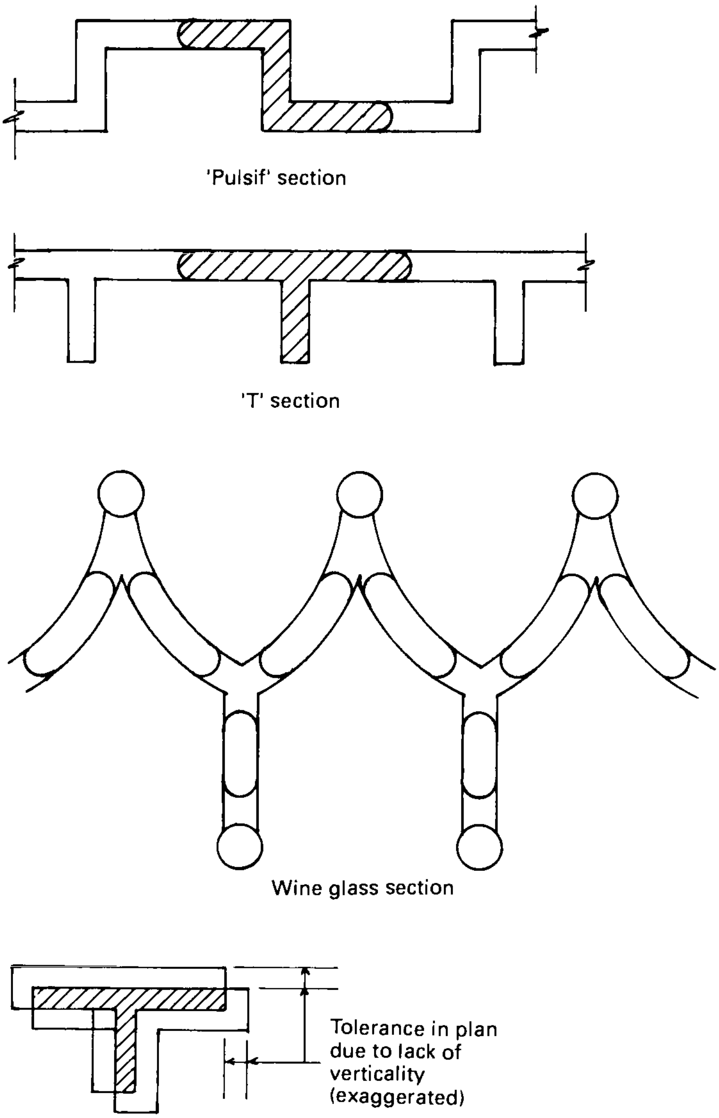


Figure 7.2 Typical structural modes. (a) Top-down construction; (b) cantilever construction

Stiffening or additional three-dimensional action can be obtained by constructing structural shapes in plan. For this purpose, connections between panels capable of taking shear and/or tension are available, but are not often necessary. Some of these shapes greatly enhance soil-arching effects and may be considered significantly self-stabilizing (Figure 7.3).

For large-diameter shafts, loads may be taken mainly by hoop action, but the effect of vertical tolerance on the interface bearing pressures and on keeping the line of thrust within the middle third must be checked. Barrel action at the top ring



'T' section showing effect of verticality tolerance

Figure 7.3 Some structural shapes formed by diaphragm walls beam, any other waling beams and invert slab level must also be taken into account in design of the reinforcement cage (Figure 7.4.). Such shafts can be used as tunnel adits. Similarly, tunnels can be driven between cut and cover station boxes constructed by diaphragm wall methods.

Diaphragm walls can be given increased resistance to bending induced by ground pressures by either pre- or post-stressing (Figure 7.5). For major isolated loads to be taken past and below underground railways, barrettes or custom-designed piles, constructed of diaphragm walls, can be adopted. This is particularly useful where

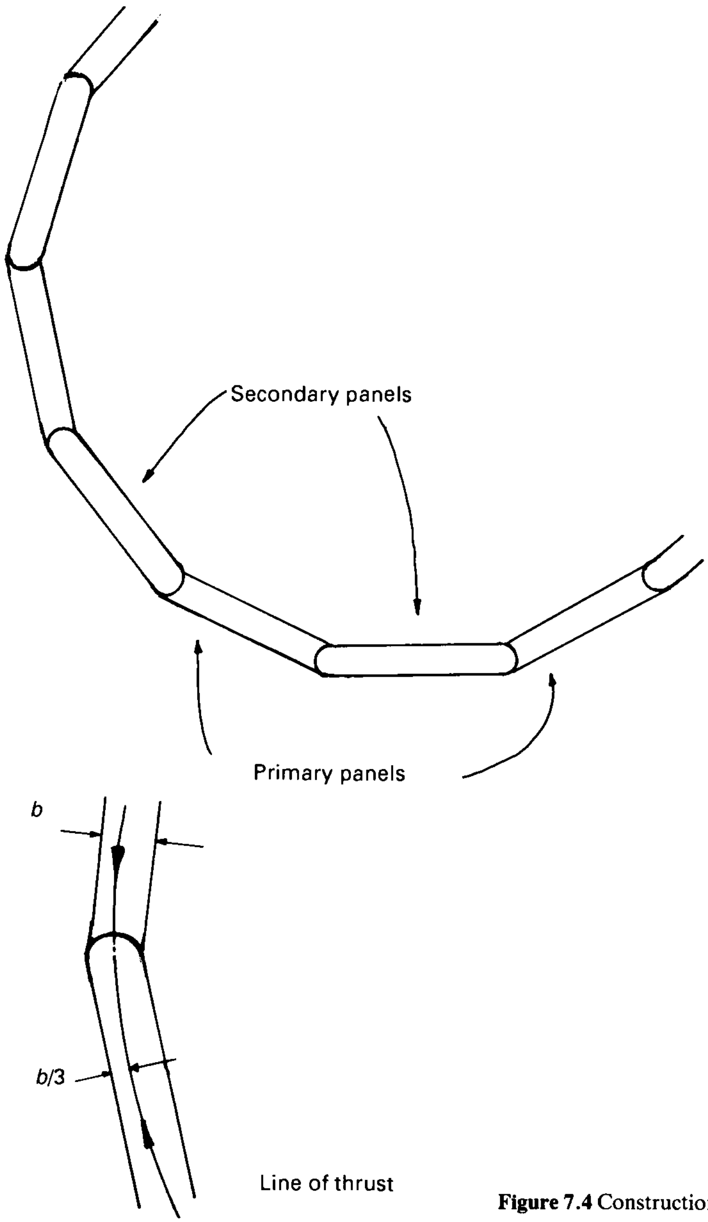


Figure 7.4 Construction of circular shafts

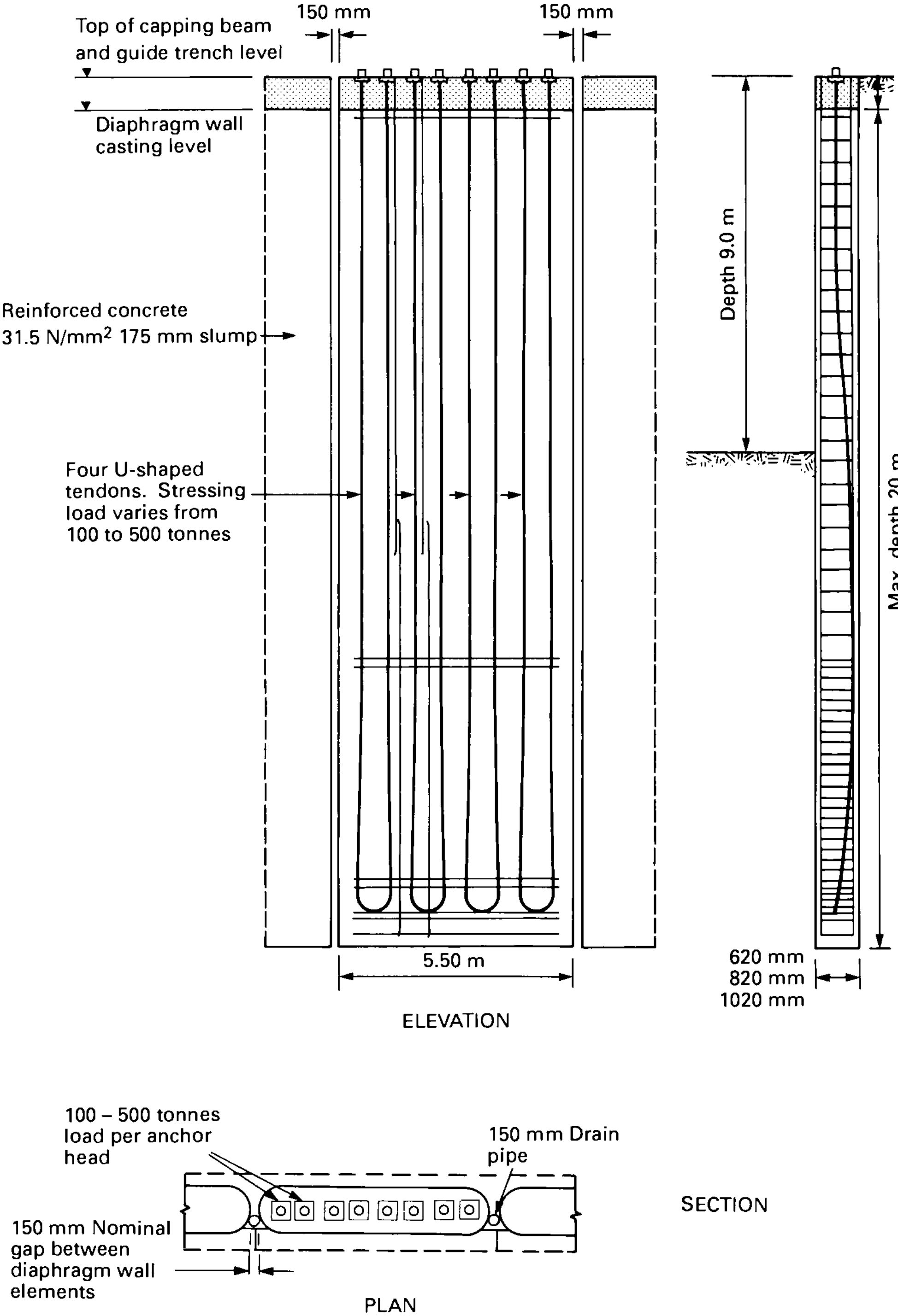


Figure 7.5 Typical post-stressed retaining wall element

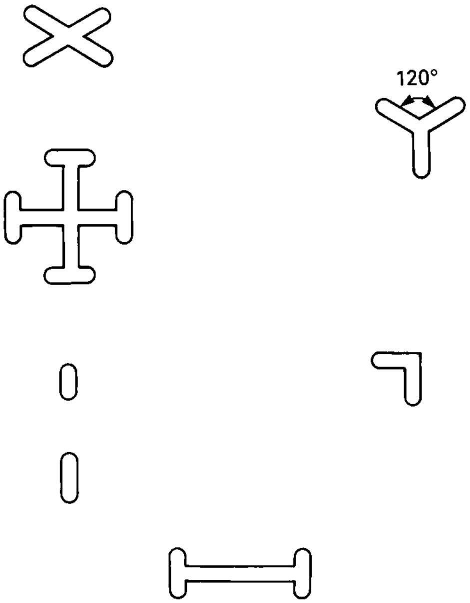


Figure 7.6 Some shapes used as vertical loadbearing elements (barrettes) (wall thicknesses typically 0.6–1.0 m)

existing buildings or permanent way are to be underpinned or where the site above the station or tunnel is to be developed subsequently (Figure 7.6).

7.3.7 Labour and operations

Diaphragm walling is plant intensive rather than labour intensive. Plant is generally to the design of the specialist contractors, though often fitted on items of standard equipment as described in Section 7.6. Basic labour, unskilled and semi-skilled, is required in comparatively small numbers for such activities as pre-trench construction, slurry batching and disposal, cage fabrication, tremie concreting and banksman duties. This modest requirement is advantageous in areas of scarce or expensive labour.

However, the technique is dependent on the skills of a few specialists, prime among which is the excavation machine operator. Modern excavation equipment is instrumented for continuous monitoring of, for example, verticality in two directions and cutting-head pressures. While watching such indications operatives are also performing the normal complex machine handling and observing with great concentration any variations in slurry levels and/or constitution. The contractual and technical success of the operation is substantially dependent on them. Specialist contractors value and reward such operatives appropriately and they move around the world as permanent staff. Other key roles include slurry control where sampling, testing, responding to ground conditions, deciding on recycling or disposal and clean-up prior to concreting all require further considerable skills. Thus the top diaphragm wall operatives may be considered among the élite on site.

7.4 Trench stability

7.4.1 Factors

There are a number of factors which operate to permit the excavation of trenches supported by slurries, generally of bentonite, in a wide range of soils. Hydrostatic pressure, soil arching and electro-osmotic forces have each been suggested as playing a part in the more difficult case of cohesionless soils. Various authors give dominance to one or another.

7.4.2 Hydrostatic pressure

The clay slurry forms an impermeable layer, or cake, at the excavated face of the trench. Thereafter, the fluid mud may no longer penetrate into the natural ground and hydrostatic pressure from the slurry on the impermeable cake face gives support to the trench sides.

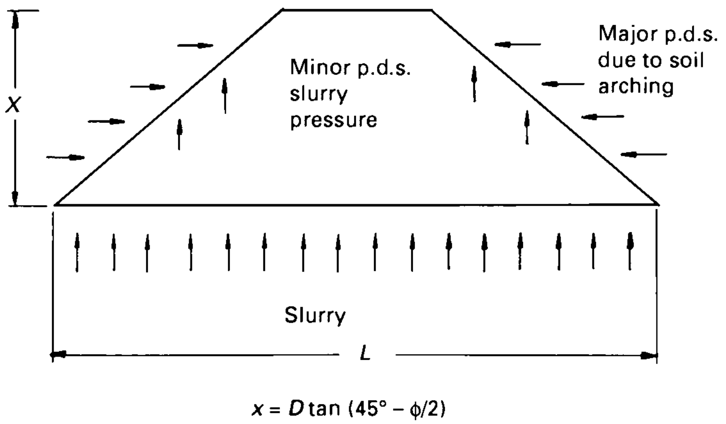
For the general case of a slurry-filled trench in cohesionless soils with continuous impermeable mud cake, Morganstern and Amir-Tahmasseb[1] have carried out a two-dimensional wedge analysis. They show that stability demands higher density for the slurry than that of normal specification mixes, and conclude that suspended excavation cuttings give the slurry the density needed for stability. They quote observations at Wanapum, for example, of slurry densities at trench backfilling of 1.28 g/ml compared with an original mix figure of 1.08 g/ml. In the case of Pierre Bénite the equivalent figures were 1.20 g/ml and 1.025 g/ml. Morganstern and Amir-Tahmasseb back-analysed cases of trench failure under high water-table conditions at Pierre-Bénite and found support for their theory, so long as the higher mud densities were assumed to operate on the impermeable mud cake. They found also that required slurry density for stability is very sensitive to groundwater level.

The theory of trench stability depending on the presence of cuttings to enhance the density of the supporting fluid is not satisfactory. Specifications for such work normally include mud cleansing, or rejection of the mud if contamination by cuttings gives a density in excess of a certain limit, which is typically 1.15 g/ml or by more than 5% of cuttings. Cleansing is always required under a normal specification clause before concrete backfilling of the trench. Such a specification clause which, on the above theory would jeopardize stability, could not have persisted as it has if excess density were a dominant factor in trench stability.

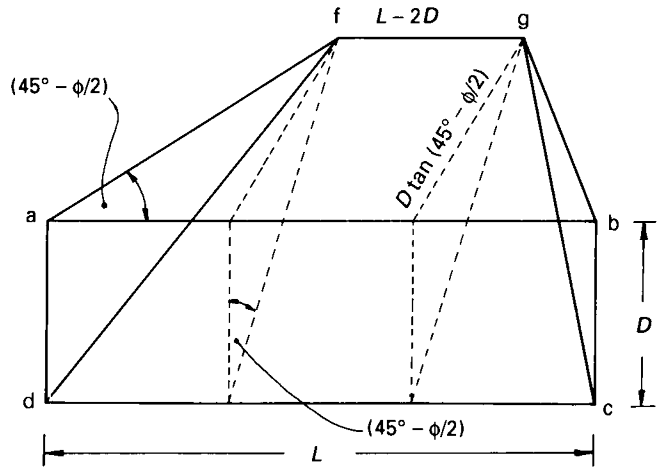
7.4.3 Arching

Washbourne[2] has presented a three-dimensional analysis for the stability of the sidewalls of a bentonite slurry trench which overcomes some of the difficulties implied in the approach of Morganstern and Amir-Tahmasseb. He deals with two categories of incipiently unstable material, one shallow and one deep. A shallow potential failure wedge is one where the depth D of the wedge is less than half the length L of the excavation. (Figures 7.7–7.9). Results of such an analysis are shown in Figure 7.10, giving, for the case of a 15 m slurry trench, factors of safety varying with depth for a typical cohesionless soil and two separate densities of slurry. Figure 7.11 shows a similar presentation for a silty soil with some cohesion.

Two comments related to Figures 7.10 and 7.11 are worth noting. First, there is the extreme importance of maintaining a head of supporting slurry above the surrounding groundwater level. Practice shows that a minimum of 1 m is required.



(a)



abcdfg is the three-dimensional active wedge
Wedge volume = $(D^2 L/2 - D^3/3) \tan (45° - \phi/2)$
Area cdfg = $D(L-D) \sec (45° - \phi/2)$
Where D is the depth of soil above the plane and ϕ is the angle of shearing resistance of the soil. Part of the wedge is above the water table and part is below

(b)

Figure 7.7 (a) Plan of simple idealized three-dimensional wedge with L greater than $2D$; (b) oblique view of simple idealized three-dimensional wedge with L greater than $2D$: abcd is the slurry soil interface

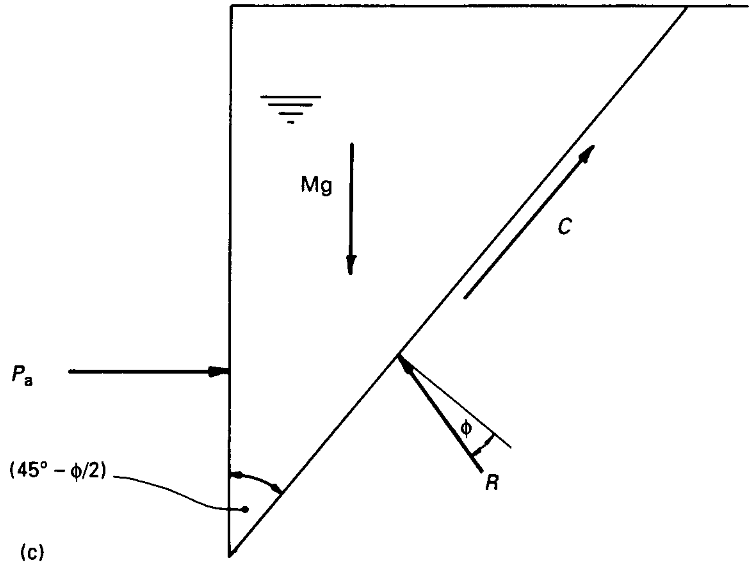
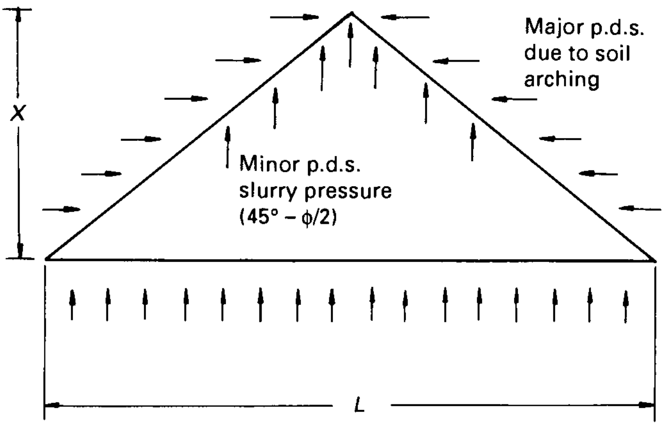


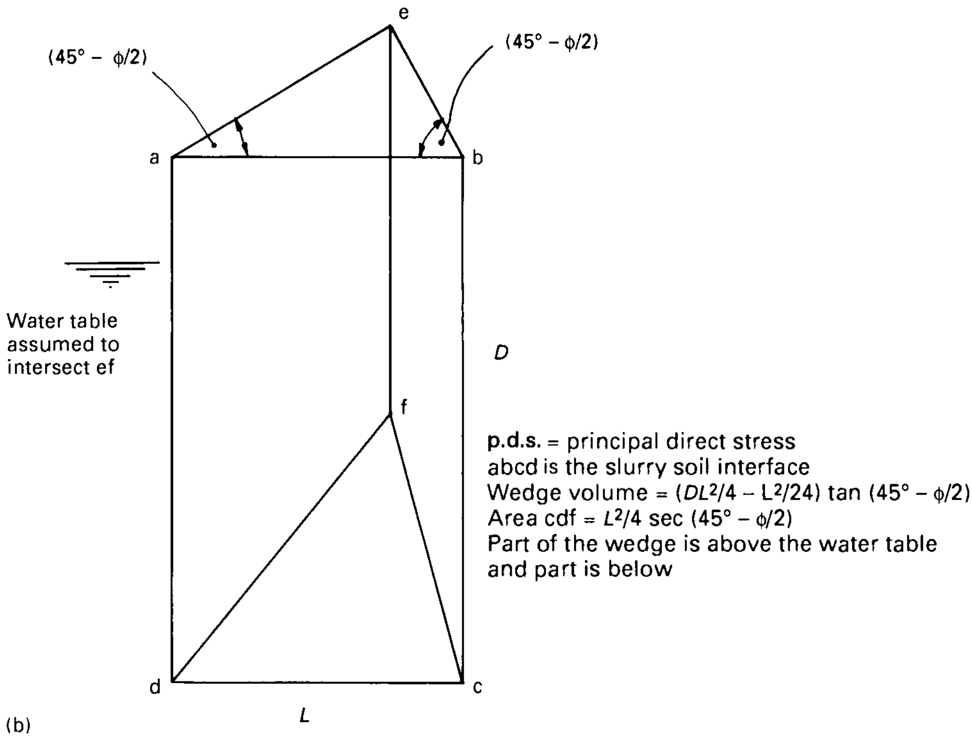
Figure 7.7 (c) Cross section through wedges perpendicular to trench for L greater than $2D$



$x = L/2 \tan(45° - \phi/2)$
Plan area $= L^2/4 \tan(45° - \phi/2)$

(a)

Figure 7.8 (a) Plan of simple idealized three-dimensional wedge with L less than $2D$



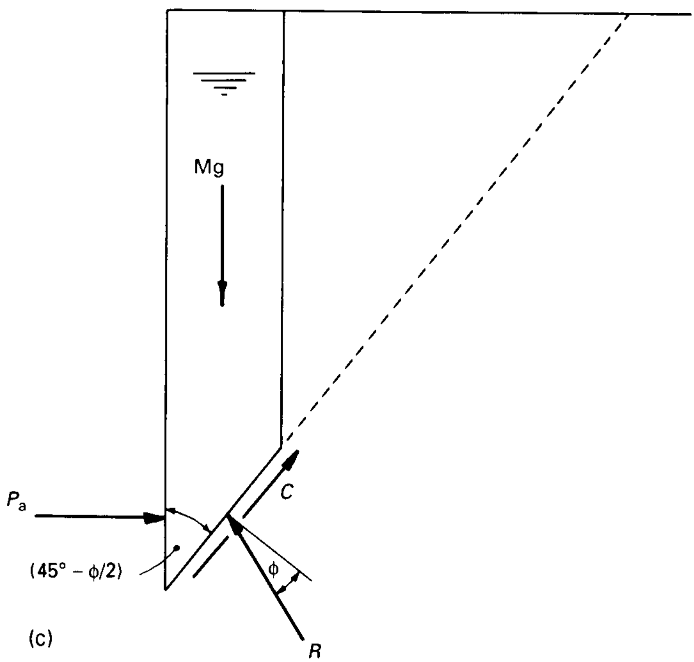


Figure 7.8 (b) Oblique view of simple idealized wedge with L less than $2D$; (c) cross section through wedges perpendicular to trench for L less than $2D$

Shallow: $P_a = [\rho g\,(D^2L/2 - D^3/3 - m^2D^2L/2 + m^3D^3/3) + \rho' g\,(m^2D^2L/2 - m^3D^3/3)]\tan^2(45° - \phi/2) - 2cD\,(L-D)\tan(45° - \phi/2)$

Deep: $P_a = [\rho g\,(DL^2/4 - mDL^2/4) + \rho' g\,(mDL^2/4 - L^3/24)]\tan^2(45° - \phi/2) - 2cL^2/4\,\tan(45° - \phi/2)$

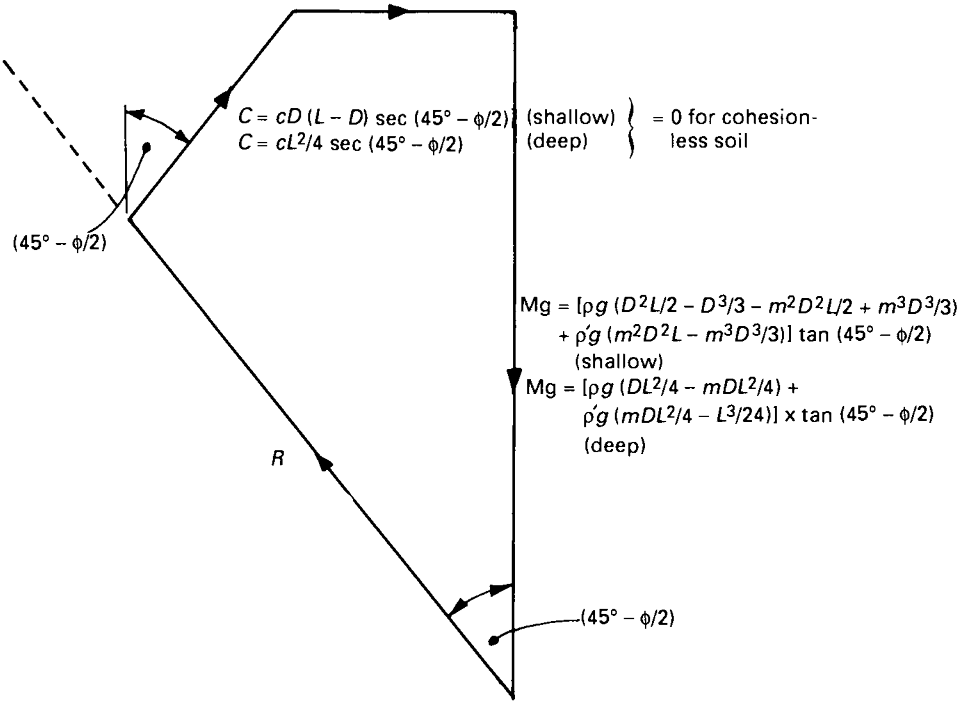


Figure 7.9 Force diagram for wedges. Symbols used in the analysis are as follows: ρ Dominant mass density of soil; ρ' submerged mass density of soil; ρ_w mass density of water; ρ_s mass density of slurry; g acceleration due to gravity; D depth of wedge; L length of wall excavation or trench; m distance of water table above bottom of wedge divided by depth of wedge; M mass of wedge; Ø dominant angle of shearing resistance of soil (undrained test); c dominant apparent cohesion (undrained test); R resultant reaction on the lower face of the wedge; C the cohesive force on the lower face of the wedge; P_a active thrust of the wedge; P_w thrust on slurry/soil interface of groundwater; P_s slurry support thrust. The factor of safety of any particular wedge F is defined as $F = (P_s - P_w)/P_a$

Second, the predicted critical zone for stability is near the ground surface. Both these phenomena have been borne out by failures at Pierre-Bénite. These were induced by the water table rising to the surface during flooding and occurred within a short distance of the surface, although the excavated trenches were, at the time, as much as 20 m deep. General observation also shows a tendency for any widened and misshapen areas of exposed completed diaphragm wall to occur near the surface, just below the guide walls, where the lowest trench wall stability has been predicted.

It is not suggested that the reader should use Figures 7.10 and 7.11 alone for deciding the relationships of panel length and mud slurry density for any particular combination of ground type and groundwater level. The lessons learned from previous experience and available through competent contractors will always be vital elements in the decision making. The relationship of the factors is, however,

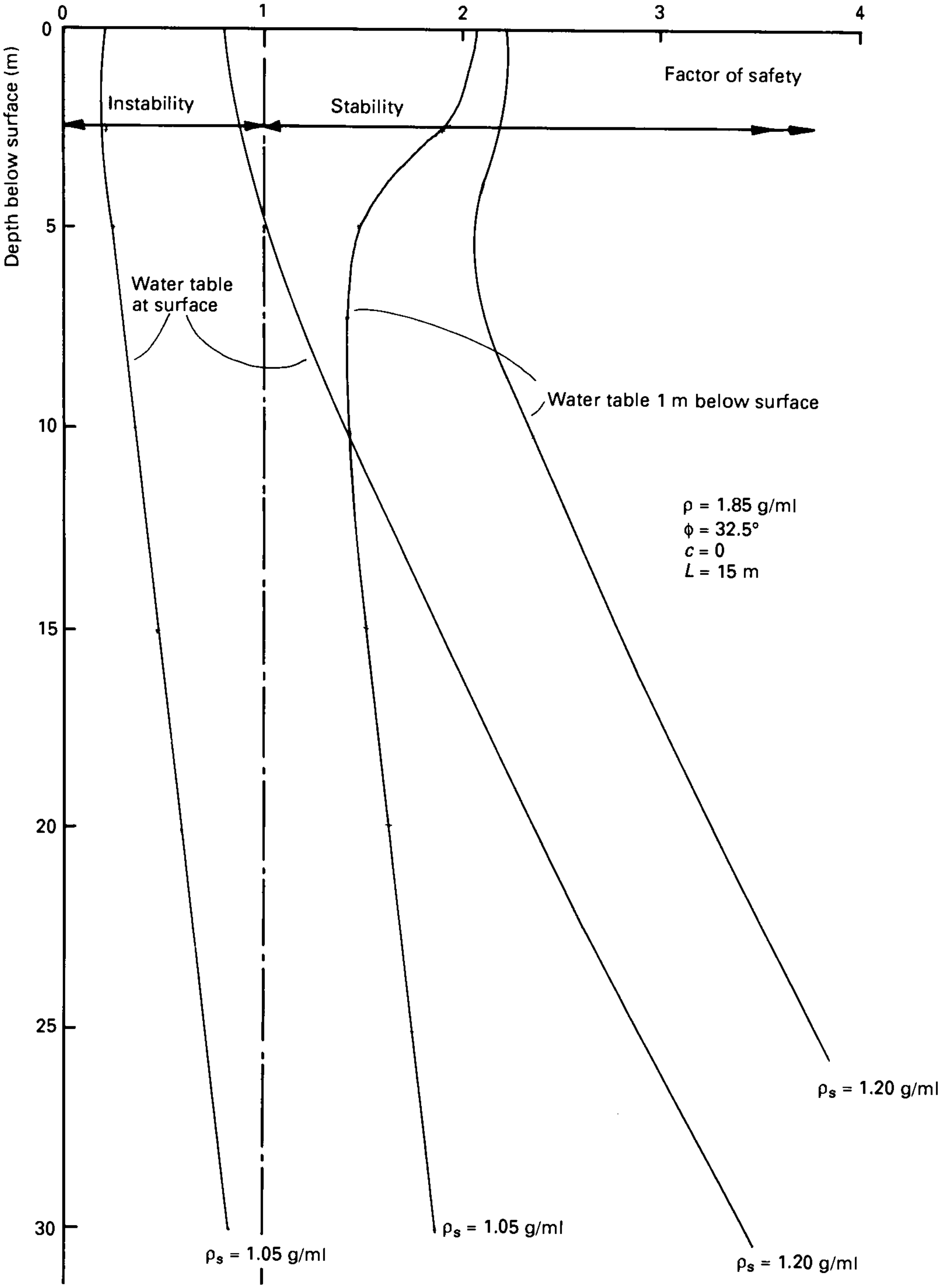


Figure 7.10 Graph of factor of safety against depth for slurry support trench length $L = 15$ m in soil: $c = 0\,kN/m^2$, $Ø = 32.5°$, $\rho = 1.85$ g/ml with slurry, $\rho_s = 1.20$ g/ml and 1.05 g/ml

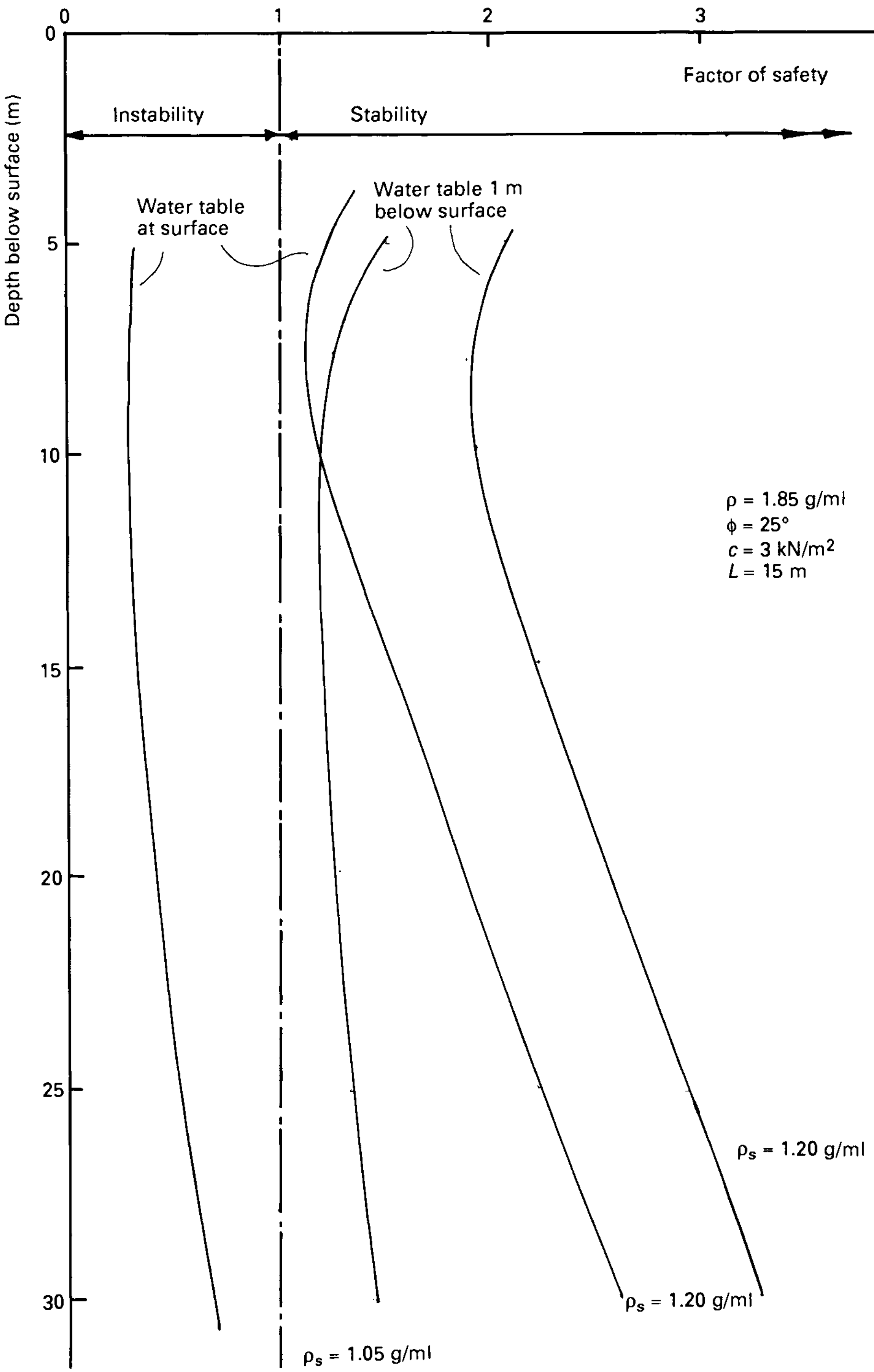


Figure 7.11 Graph of factor of safety against depth for slurry support trench length L = 15 m in soil, c = 3 kN/m^2, Ø = 25°. ρ = 1.85 g/ml with slurry, ρ_s = 1.2 g/ml and 1.05 g/ml

clearly indicated. It may be expected to have to respond to poor ground conditions by increasing suspension concentration, particularly at upper levels. At all times there must be immediate response to any loss of slurry with every effort being made to maintain the level in the trench. In the event of severe loss, immediate backfilling will be required.

7.4.4 New excavation, free-standing height and pretreatment

The analyses presented above assume an effective impermeable membrane between slurry in the trench and the adjacent ground, as provided by a developed mud cake. However, a special, but vital, case exists at the first opening up of a new excavation depth [3]. The upper trench faces are supported either by the concrete guide walls or by the bentonite cake, as described above. A certain amount of time is required for cake to deposit on the newly excavated lower faces, and for that period, stability of a free-standing height of the excavated faces is required. In cohesive soils this is normally achieved by the available cohesion according to the standard relationship for free-standing height, depending on whether or not there is water in adjacent tension cracks, of up to $(2.5/\gamma)\,C_u$, where C_u is the undrained shear strength and γ the bulk density. However, for cohesionless soils there is dependence on particle rugosity, osmosis or, above the water table, on meniscus effects in the partially saturated zone.

For dry and saturated fine cohesionless soils of low rugosity dependent on meniscus action for free-standing height, slow rates of excavation are recommended. In the dry case, menisci form from water drawn from the slurry. Slow excavation is also recommended in coarse-grained moist or saturated soils of low rugosity, where free-standing height is dependent on early formation of some mud cake.

The case for which none of these stabilizing effects is present is that of dry, coarse-grained, smooth-particled sands. In this type of material pre-treatment by densification or grouting has been used succesfully. Densification by vibroflotation or surcharging achieves additional particle interlocking and grouting may be with cement or gel. Alternatively, such difficult materials can be dealt with by using one of the modern, but more expensive, drilling media instead of the normal bentonite mix.

Other natural deposits requiring special measures for satisfactory diaphragm wall work are estuarine clays of very low strength, and very permeable ground. Weak estuarine clays can be preconsolidated by surcharging and/or draining. Very permeable coarse materials can be pregrouted. Any significant artesian pressures must be relieved prior to the trench reaching the aquifer involved. Good site investigation ahead of diaphragm wall work is of great importance.

7.4.5 Guide walls

The dimensions of these depend on the particular plant adopted by the specialist contractor. The concrete strip lining each side of the intended trench may take the form of a simple vertical rectangle, or it may be in the form of an inverted L with the lip outwards at the top. Construction in cohesive soils is simple, with vertical or near-vertical excavation and no bracing required. Design is as for a strip footing with the construction plant loading likely to be the most severe condition. The faces of the guide walls towards the intended trench should be vertical to within ± 1:200

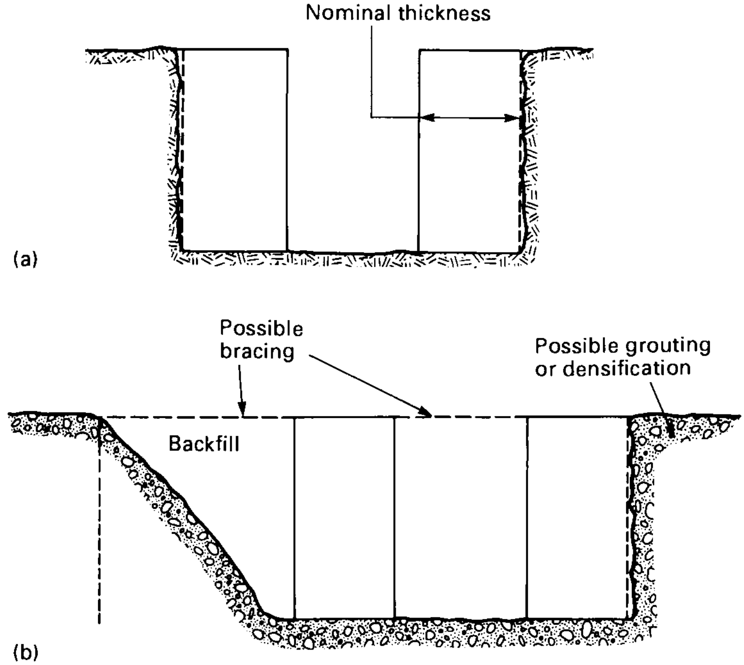


Figure 7.12 Construction of guide wall related to soil properties. (a) Cohesive soils; (b) cohesionless soils

and set apart by the intended diaphragm wall thickness plus 25 to 50 mm (Figure 7.12(a)). Additional clearance is required for walls curved in plan.

For the soils which are deficient in maintaining free-standing height as described in Section 7.4.4, bracing of the guide walls by sheet piles or similar components may be required. Alternatively, pretreatment of the ground by grouting or consolidation may be the more expedient option (Figure 7.12(b)). In such cases great care must be taken with the main trench excavation proper just below the guide walls. The methods of Section 7.4.4 should be adopted and, if necessary, slower rates of progress accepted until more stable conditions are reached at greater depth or in more favourable soils. Excessive loss of ground immediately below the guide walls will lead to their instability and movement, with potential risk to plant and operators.

7.4.6 Bentonite mixes and admixtures

Typical specification limits for standard bentonite are shown in Table 7.1 and the relationship between the concentration, expressed as a percentage by weight, and density is given in Table 7.2.

Sections 7.4.4 and 7.4.5 have described certain soil types where significant difficulties have to be overcome in order to achieve safe excavation of a slurry-supported trench. After listing conventional treatments used successfully in the past to remedy such problems brief mention was made of the alternative of new drilling media. These are dealt with in Section 7.10.

Table 7.1 Typical bentonite specification limits

Item to be measured	*Range of results at 20°C*	*Test method*
Density	Less than 1.10 g/ml	Mud density balance
Viscosity	30–50 s or less than 20 cP	Marsh Cone Fann Viscometer[a]
Shear strength (10 min gel strength)	1.4–10 N/m^2 or 4–40 N/m^2	Shearometer Fann Viscometer
pH	9.5–12	pH indicator paper strips or electrical pH meter

[a] Where the Fann Viscometer is specified, the fluid sample should be screened by a No. 52 sieve (300 μm) prior to testing.

Table 7.2 Relationship between benonite concentration and density

Concentration (%)	*Density* (g/ml)
4	1.023
5	1.028
6	1.034
7	1.039
8	1.045

In addition to dealing with adverse soil conditions, variations in slurry properties are sometimes required to deal with contaminated or saline groundwater. Flocculation of the suspended bentonite must be avoided. Solutions for typical problems are shown in Table 7.3. For comparatively simple and low-concentration sodium chloride contamination, the rigorous completion of hydration in fresh water may be sufficient to maintain a satisfactory bentonite slurry.

7.5 Design considerations

7.5.1 General

The design of a diaphragm wall structure is carried out in the same way as any *in-situ* concrete structure with the following restrictions. First, the walls must generally take the form of vertical prisms, although in at least one case a slight batter has been achieved in order to avoid an existing foundation. Second, walls have to be constructed in a series of panels commonly of maximum length approximately 6 m, and it is generally uneconomic to provide continuity of reinforcement between panels. Continuity can be provided using special techniques, but it is generally better to carry vertical shear stresses between panels through a substantial capping beam. It is possible to design structures to span horizontally using arch action. Reinforcement continuity is only needed in the rare

Table 7.3 Some solutions to adverse soil and groundwater conditions

Problem	*Solution*			
Contamination	>0.15 mm 0.01–0.15 mm <0.01 mm	Remove by vibrating screens Remove by hydrocylones Use additives to prevent dispersion		
Slurry too thick due to increase in calcium ions or cement contamination		Sodium carbonate Sodium bicarbonate Sodium phosphate } (soda ash) additives (get pH in range 9.5–12)		
Slurry too thick due to excess calcium (e.g. chalk excavations) or sea water		Lignosulphate additives Use of polymer slurries		
Excessive loss of bentonite	Add intermediate size particles to suspension	Clay Silt Sand	Particles carried into pores or fissures causing blockage	Not suitable if pores are very large
	Chemical additive treatment	Potassium aluminate Aluminium chloride Calcium	Flocculates bentonite gel, flocs block pores	Once seal is made the fluid may have to be renewed
	Cement/bentonite mixture	Cement added to fluid at mixing	Allowed to penetrate areas of loss and then set	Mixture usually supplied only to area of loss, and area subsequently re-excavated under normal fluid
	Fibrous materials (used in oil-drilling work)	Graded fibrous or flake materials, e.g. plant fibres, glass, rayon, cellophane flakes, mica, ground rubber tyres, ground plastic, nutshells, etc.	A suitable mixture will bridge fissures and fill a wide range of pore sizes effectively	Trial-and-error method with wide selection range; many materials available under trade names from oil-drilling sources
Only sea water available for mixing	Use Attapulgite (health factor to be considered)			

case when tension stresses are to be carried between panels or when bending stresses at joints are unavoidable.

The design of a diaphragm wall structure must take account of the construction method. Wall ends are cast as primary or secondary. A primary end is excavated and a stop-end section, usually in the form of a circular tube of the same diameter as the diaphragm wall thickness, is placed through the bentonite. After concreting, the stop-end is withdrawn. The stop-end tube forms a neat end against which the secondary end can be formed once the next panel is excavated. Complete panels are usually constructed alternately on a primary and secondary stop and start basis, but can, with care and additional programme time, proceed progressively along the trench. The sequence and timing must avoid damage to completed work.

Straight walls between 600 mm and 1500 mm thick are generally used but a number of alternative plan shapes can be formed as shown in Figure 7.3. Each pattern is formed by repeating a panel of a simple plan shape containing a single reinforcement cage. Particular care has to be taken when detailing reinforcement cages of complex shapes to allow for the tolerance on verticality, which can significantly modify the plan shape of the wall at depth, and mean that the core area of a wall, in which the minimum cover can be obtained, is reduced.

7.5.2 Continuity

If reinforcement continuity between panels is essential, it is necessary to cover one end of the connecting reinforcement or mechanical coupler while concreting the other end into the primary panel. This can be achieved in a number of ways. One technique uses a stop-end plate through which reinforcement projects. The plate is sealed against the side of the excavation using a plastic membrane folded back against the side of the primary panel excavation, protecting the projecting reinforcement. The stop-end plate must be cast into the concrete wall and cannot be re-used.

A slightly different technique has been used to carry direct tension, where straight-webbed steel sheet piles are inserted into the primary panel before concreting. The last clutch is protected by a specially designed steel diaphragm welded across the pile pan. Generally, the pressure of concrete is sufficient to provide a seal between the stop-end tube and the diaphragm. After concreting the primary panel the stop-end tube is withdrawn leaving an exposed pile clutch ready to be connected to further sheet piling in the secondary panel [4].

These are only two of the methods used to provide structural continuity. A considerable amount of ingenuity has been expended in developing different techniques, but all are relatively complex and expensive and are best avoided if possible. However, a modified version of the latter technique has proved successful for inserting water bars at joints.

7.5.3 Deformations

Deformations of retained ground can be calculated in the same way as for a sheet pile wall if the ground parameters are known with sufficient accuracy. However, the advantage of the diaphragm wall system is that the much higher section modulus available using large concrete sections leads to much smaller movements than with steel sheet piles. Ground deformations must also be considered when using tie-backs or struts in order to determine the stresses in individual anchorages or props, particularly where several rows are installed as excavation progresses. Again, this is a similar calculation to a tied-back or strutted sheet pile wall and expert geotechnical advice is essential. Finite element analytical methods have been used successfully.

7.5.4 Bearing capacity

Comparative pile tests have been carried out and analysed to determine whether there is any loss in bearing capacity for those cast under bentonite suspension [5]. The results may be assumed to apply to diaphragm walls constructed in a similar way. At normal working displacements, no loss of wall friction is indicated so long as excessive thickness of filter cake is avoided. However, friction may not be assumed on any part of a wall above main excavation level nor in any other area where deformation may cause loss of contact between soil and wall.

For end bearing, working practices are of great significance, particularly excavation, clean-up before concreting and concrete placing. For cohesive soils and rocks, good practice will permit use of full end-bearing values. With cohesionless soils the situation is also satisfactory if there is a low water table and dense sands. However, with looser materials and/or pore water pressures it is unlikely that high end-bearing pressures will be developed.

7.6 Construction considerations

7.6.1 Excavation equipment

The primary consideration as to the plant to be used in metro construction will often be site-access restrictions. Provision must be made for the handling of bentonite slurry, excavated spoil, concrete and reinforcement cages.

Two basic methods of excavation are used, with equipment usually mounted on normal or modified crawler cranes. The first method employs specially designed grabs which, in their simplest form, can be rope operated but are more commonly rope suspended with hydraulic operation of the jaws. Where ground conditions are liable to lead to deviation of the trench a kelly bar grab is often used to assist in maintaining verticality over the top few metres. In difficult conditions a number of different chisels can be used to break up rock or large boulders to prevent the grab being forced off-line.

This method is suitable for small sites since it requires the minimum in specialist plant, can be used with batch supply of bentonite slurry to a header tank and permits the spoil to be loaded by the excavator directly onto tipper trucks. However, as the excavation depth and/or rock quality increase the method becomes less efficient, since the hoisting time gets longer and tools have to be changed to pre-break the rock. A maximum tolerance on trench verticality of 1 in 80 should be specified for this type of excavation.

The more sophisticated reversed-circulation trench cutter avoids these problems. The machine has a powered cutting head which can have horizontal axis cutters, as in most European machines, or a rectangular array of vertical axis drilling bits, as in one Japanese machine. Cuttings are removed by a continuous circulation of bentonite from the cutting head to a special bentonite mixing and treatment plant where the bentonite is screened to remove the spoil, tested and, if necessary, partially replaced by fresh slurry before being returned to the top of the trench. The cutting head is advanced either by gravity or, as in a full-face tunnelling machine, by clamping an anchor frame to the sides of the trench and jacking the head forward. It is possible, by adjusting the head angle, to steer the trench cutter to achieve a much tighter tolerance on verticality than with a grab excavator. Various cutting tools can be attached such as clay spades and rock-cutting heads. It is claimed that, because of the additional power of these machines, rock with an unconfined compressive strength of up to 100 MPa can be cut. Where several different strata are to be penetrated the cutter may have to be removed from the trench to change tools. Unlike the grab method, this is the only time during excavation that the machine has to be removed from the trench.

Although the sophisticated trench cutters have a number of technical advantages over the more conventional grab devices they are expensive pieces of equipment. Also, since they need an associated slurry treatment plant and, in the case of the European machines at least, a crane of approximately 200-tonne capacity, they are only suited to large projects. Figure 7.13 shows the difference in scale of the two types of equipment, a cable grab on the left and a trench cutter on the right.

The choice of excavating machine should be left to the specialist contractor, who will have a knowledge of the relative performance of the various machines. However, it must be remembered when planning a project that it would be uneconomic to specify a tight tolerance on verticality, which can only be achieved by the trench cutters, if the site cannot be handed over so that the work can be completed in one occupation.

Figure 7.13 Typical diaphragm wall excavation equipment; clamshell grab on the left and trench cutter on the right

7.6.2 Alignment

Irrespective of the type of excavator the plan alignment of the trench is ensured by means of *in-situ* concrete guide walls formed either side of the proposed diaphragm wall in a shallow trench. The design of the guide walls depends on a number of factors:

1. The weight of the excavation plant;
2. Ground stability;
3. Location of adjacent structure or road;
4. Groundwater level.

Using a grab excavator the bentonite slurry is agitated as it is displaced each time the grab is raised or lowered. This action can wash out the trench sides, particularly towards the top of the trench, where there is lower basic stability. For this reason, the grab itself should have a streamline profile when closed and the guide walls should extend to a sufficient depth to prevent washout.

Verticality can be controlled in reversed-circulation trench cutters by steering the head or by altering the torque applied to each cutter. When using grabs, the only way to adjust the verticality is to use chisels or hammers either to remove the obstruction causing the grab to deflect or to widen the trench to allow the grab to return to its true line. If consistent problems are encountered with verticality the type of excavation equipment can be changed and better results may be obtained using a kelly bar grab or a rope grab with a short kelly bar to improve verticality over the most important top section.

7.6.3 Material properties

Control of bentonite quality and level (1 m minimum above groundwater) is most important for maintaining both stability of the trench walls and concrete quality. Bentonite slurry from the bottom of the trench should be sampled and tested before concreting to ensure that the material is not contaminated. Thus the free flow of concrete from the tremie pipe is not impaired and the bentonite is fully displaced by the rising concrete.

Concrete for tremie placement should have high workability, with slumps in the range of 150–200 mm and cement contents in excess of 400 kg/m^3. Plasticizers are often used. If the concrete is designed on the principles of BS 8110, bending moments induced by earth pressures should be factored by the order of 1.5. Tests on core samples of set concrete from various levels in diaphragm walls have indicated enhanced strengths at lower levels, presumed due to larger confining pressures applying at depth before setting of the concrete.

Reinforcement cages can be inserted before the concrete is placed and these should be of rigid design. Main vertical reinforcement should, by the use of large-diameter bars, be well distributed to allow free flow of concrete around the bars and sufficient links and diagonal bars should be provided to prevent distortion of the cage. Sufficient cover should be provided to allow for tolerances in the excavation plan shape particularly where the shape is formed from intersecting panels. Experience and tests have shown that bond is not significantly impaired by slurry.

7.7 Special details

Construction of individual diaphragm wall panels is straightforward but special details are required to provide continuity. Methods of providing continuity between panels have been described in Section 7.5.2. Continuity with structural floors is obtained by covering bent-up bars, or bar couplers, with polystyrene foam pads during the tremie concreting. After main excavation, any thin concrete cover can be broken away and the bars bent out for connection into the reinforced concrete slab.

A similar technique should be used for tie-back, and a typical detail is given in Figure 7.14. A foam pad is attached to the reinforcement at the location of the proposed tie-back and a sealed duct is placed through the wall. After excavating,

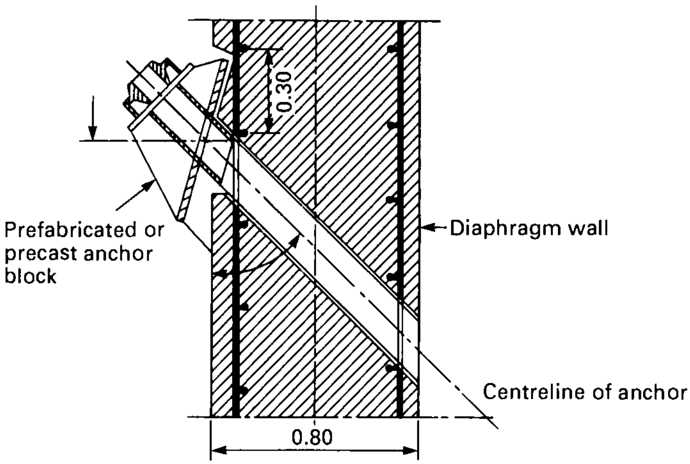


Figure 7.14 Typical tie-back anchor detail

the foam can be removed and an anchor block grouted in after which the duct can be drilled out and the anchor installed through the duct. Larger box-outs can be formed in the same way, by attaching slabs of polystyrene within the reinforcement cage. This method is useful at details where access passages through the diaphragm wall will be required later.

Diaphragm walls can be supported by individual props bearing through grouted sole plates or by waling beams supported by rows of props. In each case an *in-situ* mortar or concrete packing is required to take up the tolerance on the wall verticality. The method of support depends largely on the type of structure to be constructed between the walls.

The natural cast finish of diaphragm walling is usually of rough texture and any exposed surface normally has an applied finish. This can be a simple sprayed or cast *in-situ* concrete finish or, more usually, where visual appearance is important, a precast concrete cladding system is used. Other cladding systems such as aluminium panels or g.r.p. are equally suitable, depending on the architectural requirements. In all cases where the wall is below the surrounding water table a small amount of seepage through the wall should be expected and the back of the cladding should be drained.

7.8 Typical costs and progress rates

Diaphragm walling is normally undertaken by one of a limited number of specialist subcontractors who have available the specialized equipment and experience of trained staff. Each contract is different and only indicative costs can be given. Costs of large areas of plastic concrete cutoff walls in sand would be expected to be at the low end of the cost range at about £200/m^3 at 1989 prices. Those of small areas of heavily reinforced diaphragm wall in difficult conditions (for example, where boulders are expected, or in hard rock) could easily be double this. In addition, there must be added the cost of providing cladding panels where the appearance is important and any special jointing detail or complex fabricated inserts.

Similarly, progress rates can also have large variations. Grab excavators and conventional construction methods can be expected to permit completion of a maximum of about 500 m^2 of diaphragm walling per week per rig.

However, after setting up a reversed-circulation system, an unreinforced cutoff wall can be completed at a much greater rate. At Sizewell B in optimum conditions, the 70 000 m^2 plastic concrete cutoff wall was completed at an average rate of 18 m^2/h/machine or about 2000 m^2/week/rig with 24-hour working. At Sangatte, in more usual ground conditions, rates of 10 m^2/h/rig were achieved for the 3800 m^2 circular shaft and 29 000 m^2 cutoff with Hydrofraise milling machines. In competent sandstone, rates of excavation could reduce to 3 m^2/h or lower.

7.9 Case histories

Examples of running tunnels constructed by cut and cover techniques can be found throughout the world. For example, nearly 300 000 m^2 of diaphragm walling was used on the Cairo Metro and 100 000 m^2 in constructing the Caracas Metro. The four selected examples show different aspects of the method in various situations.

The first example (Figure 7.15) shows the construction of a section of running tunnel in a very confined site beneath an existing three-level elevated expressway interchange on the Keio New Line in Tokyo. This shows the possibility of constructing cut and cover tunnels in confined areas adjacent to sensitive structures. The excavations were undertaken from the top down using seven levels of temporary struts. The running tunnels were then constructed inside the excavation, one on top of the other, as-cast *in-situ* boxes providing the permanent support to the diaphragm walls and the upper section was backfilled to prevent uplift.

In the second example a new station was constructed on the line of an existing running tunnel just below street level at Argyle Street in Glasgow. The stages of the construction can be seen in Figure 7.16. First, the services were diverted and the guide walls constructed below the pavements. Second, guide trenches were built within the existing tunnel, then the invert was broken out and four diaphragm walls were constructed. The outer walls form the external walls of the station while the inner walls support the rail slabs. In stage three the outer walls were anchored, the original tunnel was removed and the roof on which the traffic on Argyle Street is carried was constructed. During stage four the concourse was excavated and the rail slabs, platforms and access were all constructed. This shows the possibility of working beneath an existing right of way with minimum interruption of traffic using special low-headroom rigs.

A number of similar examples can be found in Japan, notably where underground railways have been constructed under existing stations in Tokyo, Fukuoka and Osaka. Again, in Japan it is common, where space is limited, to construct the mud treatment and desanding plant under the street, thus avoiding the severe congestion above.

The third example is of Admiralty Station in Hong Kong, which was also constructed in a densely built-up area. In this case an open excavation with its sides supported by anchored sheet piles and diaphragm walling was used. Figure 7.17

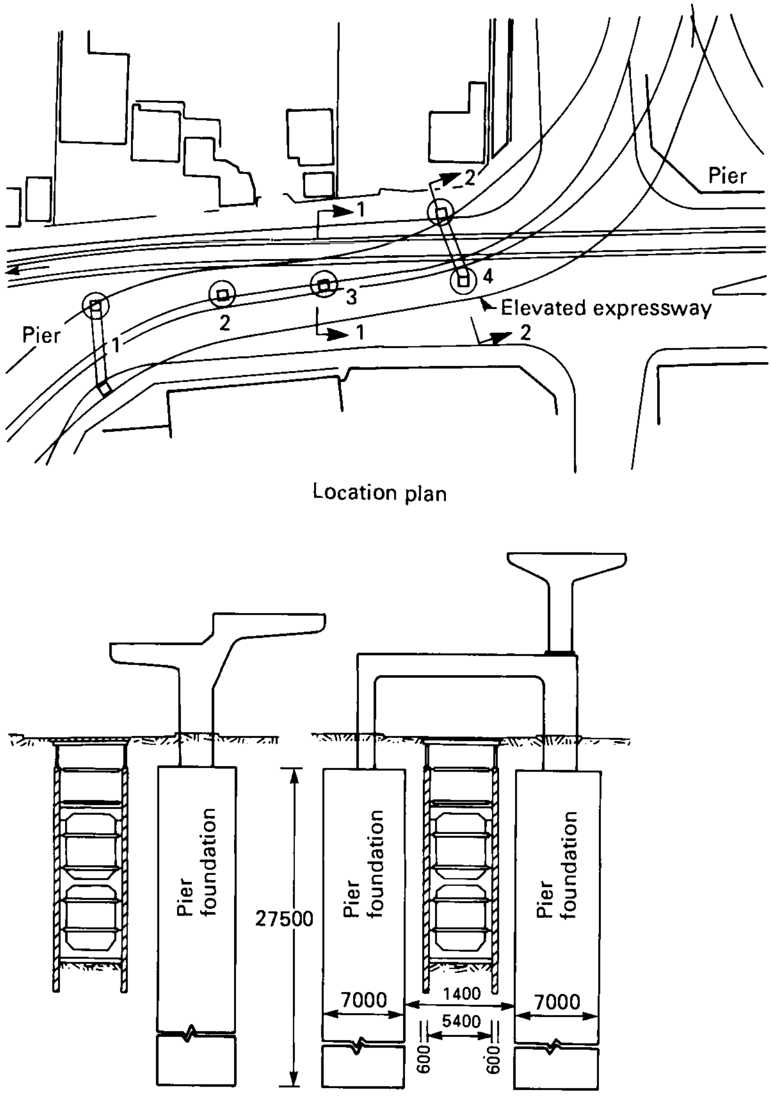


Figure 7.15 Keio New Line, Tokyo, crossing an elevated expressway showing proximity to existing foundations and restricted working headroom

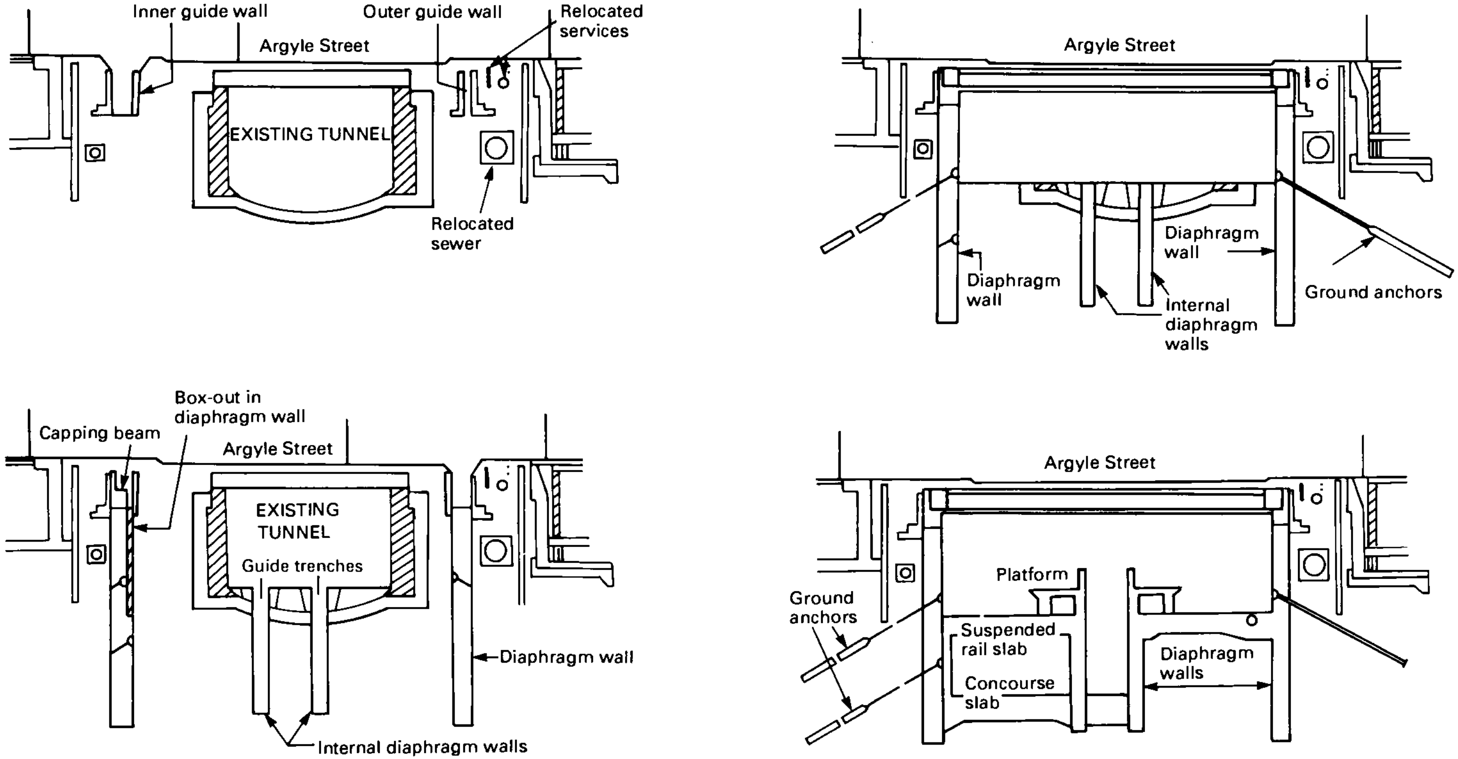


Figure 7.16 Argyle Street underground station, Glasgow, showing stages of construction within an existing tunnel

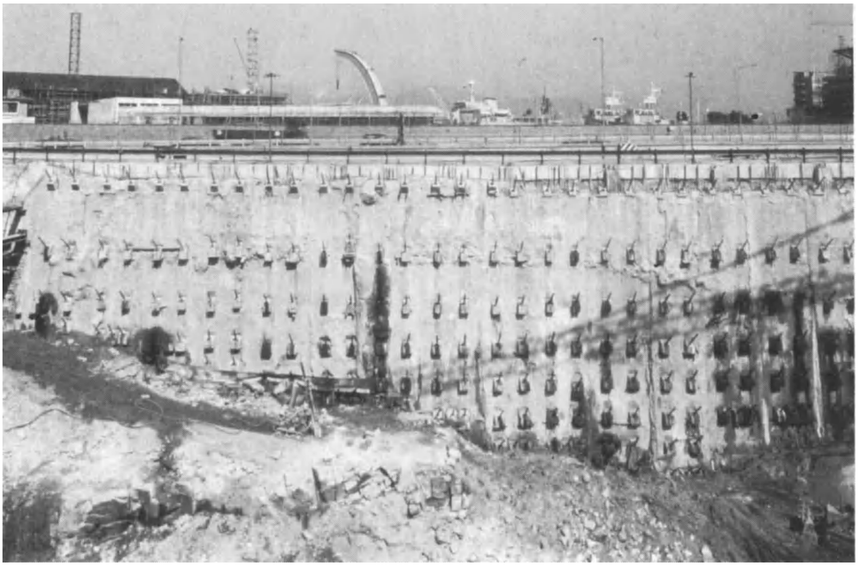


Figure 7.17 View of tied back diaphragm wall at Admiralty Station, Hong Kong

shows the large open excavation and the six rows of anchors needed to support one of the sections of diaphragm walling. In all, 4500 m^2 of diaphragm walling was used in this project. After the excavation was completed the station was constructed in the conventional way from the bottom up.

The fourth example is slightly different in that it is for the Channel Tunnel access shaft at Sangatte. This example shows features used in metro construction projects but on a larger scale. Two diaphragm walls were used as shown in Figure 7.18. The inner wall forms the upper section of a 58 m diameter circular access shaft. The diaphragm wall was 21 m deep and 1 m thick and was designed to support the upper strata of sandy silt by hoop compression. The shaft was subsequently excavated below the diaphragm wall to a depth of 60 m, through chalk, and lined with *in-situ* concrete. This technique was also used for the access shaft for the first metro tunnel across Hong Kong Harbour.

Within the Sangatte shaft the tunnel-boring machines (TBM) were to be assembled. Initially, the shields were in an aquifer and driving cannot start in these conditions. Consequently the area had to be dewatered, and to reduce the pumping requirement a second diaphragm wall cutoff was constructed to a depth of 60 m around the TBM assembly area. This was a 'plastic concrete' wall made of an unreinforced cement/bentonite mixture which provides a low-strength, deformable barrier with a low permeability. Similar techniques have been used elsewhere in many instances to reduce the radius of influence when dewatering large excavations and to prevent ground-movement problems in built-up areas.

7.10 Future developments and prospects

The leading diaphragm wall specialist contractors, their materials and equipment suppliers, and their own subcontractors are all active in research and development.

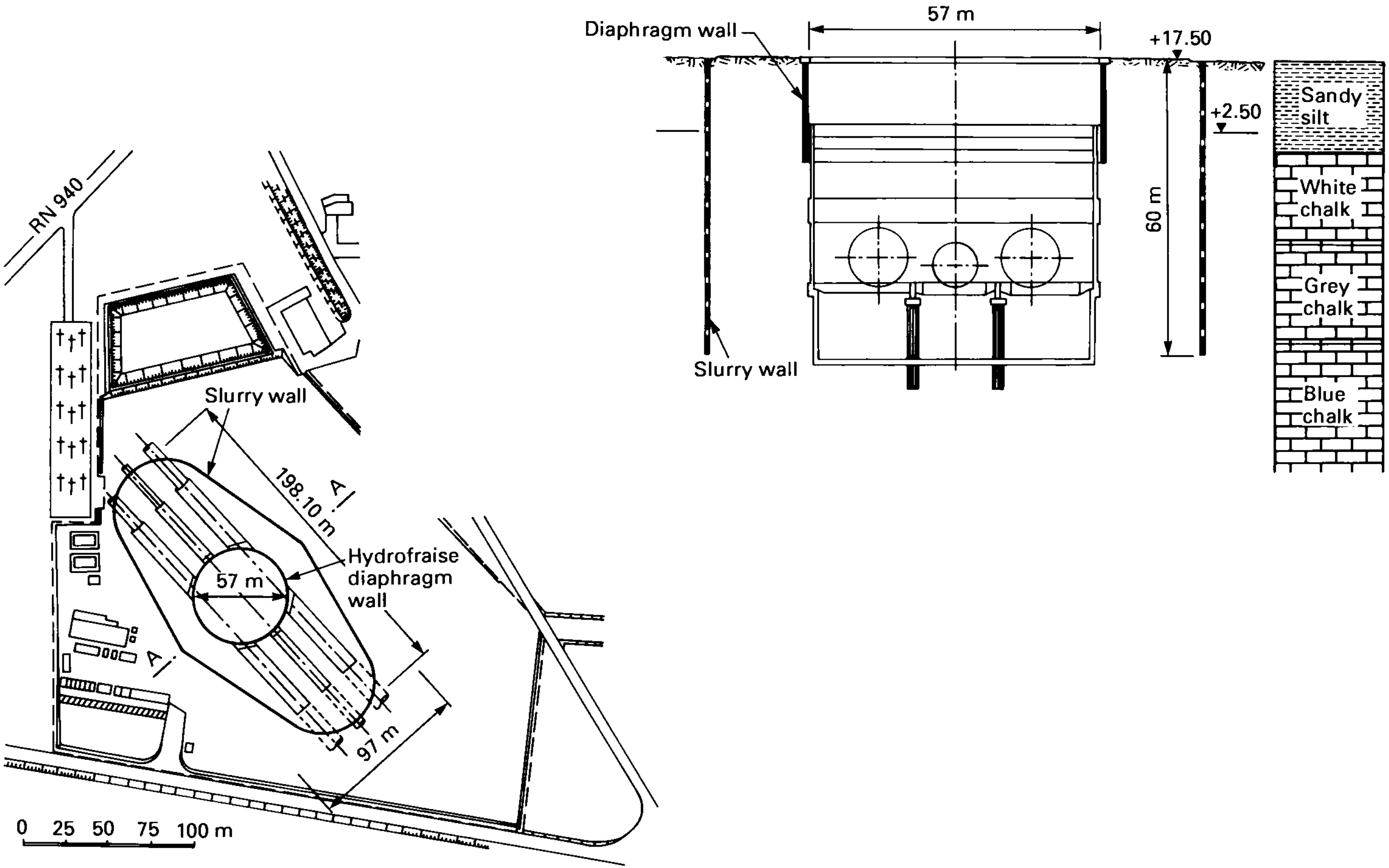


Figure 7.18 Layout of diaphragm walls at the Sangatte Channel Tunnel access shaft

Table 7.4 Research and development projects

Development	*Purpose and use*	*Problems to be overcome*	*Source contractor*
Use of long-chain polymers in slurry formulations	For environmental benefits. (Safe disposal to sewers and rivers; selected polymers acceptable for water supply aquifers: smaller volumes to handle.) To vary viscosity, density and caking properties for different ground and groundwater conditions, e.g. openwork gravels	Extra difficulties of stirring, pumping and cleaning	Bachy Cementation Trigon Geochemical
Use of Revert biodegradable suspension (natural polymer, hydrated guar gum)		Expense (normally limited to thin slit cutoffs) (health aspects to be considered in water supply aquifer situations because of potential growth of bacteria)	Cementation
Use of precast panels	For structural continuity and/or architectural finish	Formulation of a self-hardening slurry (bentonite-cement with lignosulphate retarders and silicate regulators is referred to)	Soletanche Bachy
Mix-in-place deep (up to 15 m) soil stabilization by use of interrupted flight augers up to 1 m dia. and grouting through central tube	Protection of shaft, trench and tunnel excavations in unstable silts and sands		Cementation
Diamond wire cutting	Cutting openings in retaining walls from excavated side using cast-in draw tubes with diamond wire sawing techniques	Accuracy in placing draw tubes. Expense	Trigon Geochemical
Palsif (high-modulus) walls			Bachy

Precast columns inserted into barrettes	Support for top and bottom slabs in top-down construction		Bachy
Insertion of high-density polyethylene sheets in vertical cutoffs with continuity at joints	Permanent enclosure of polluted soils with predicted life of up to 100 years and good resistance to the following: Aromatic compounds: Ethylene benzene, Xylene, Phenol Inorganic compounds: NH4, Fluorine, CN, Sulphides, PO4 Polycyclic hydrocarbons: Naphthalene, Anthracene, Phenanthrene, Fluoranthene, Pyrene, Benzopyrene Other sources of contamination: Pyrides, Tetrahydrothiophene, Cyclohexanone, Styrene, Petrol, Mineral oil Chlorinated hydrocarbons: Aliphatic chlorinated hydrocarbons, Chlorophenol, PCBs Pesticides: Organic chlorine compounds, Pesticides	Seal at panel joints, obtained by an expanding neoprene rubber insert. Insertion beyond 15 m depth requires further development	Bachy

The methods and materials involved are often commercially confidential, and some will remain the exclusive property of the developer for a significant period. Others are more generic and may be adopted quickly throughout the industry. They are all aimed at improving diaphragm walling as a technique and widening its applicability. The developments shown in Table 7.4 are reported as being in progress by the contractors to whom reference is made.

Acknowledgement

The writer wishes to thank Mr M. W. Pinkney, MA, MSc, DIC, MIStruct E, for his assistance in preparing this chapter.

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Chapter 8

Concrete piling walls

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Contents

Choice in the provision of underground walling is, as in all branches of engineering, a compromise, since no ideal solution exists. It is dependent on many factors, including ground and groundwater conditions; local expertise and economics; available plant and technology; consequence and risk balance; and required design life of the system. Bored-pile walling produces the minimum of noise, vibration and disturbance of all the intruded walling systems and, within the general constraints listed above, often offers an effective, cost-competitive solution.

8.1 Criteria and wall types

8.1.1 Criteria

Underground walls have to satisfy four main criteria, They must:

1. Offer temporary and permanent ground support;
2. Achieve or conform to specified geometrical requirements;
3. Conform to some standard of water exclusion; and
4. Be inexpensively and quickly constructed.

8.1.2 Wall types

The walling types described in this chapter are distinguished as:

1. Contiguous bored-pile walls;
2. True secant bored-pile walls; and
3. Pseudo-secant bored-pile walls (a hybrid of the above).

Contiguous bored-pile walls are composed of piles constructed in a row along the length of the wall with a distance between their centres comprising a pile diameter plus a few millimetres to allow construction. Piles are usually constructed on an alternate (or greater) basis and the distance x illustrated in Figure 8.1 is dependent on construction requirements (e.g. width of collar casing or length of pile). Typically, x varies:

Pile length (a)	<15 m	<20 m	<25 m	<30 m
x (mm)	150 mm	200 mm	250 mm	300 mm

Pile diameters can range from the minipile at 150 mm to the caisson-sized 2000+ mm.

True secant bored-pile walls comprise a row of piles intersecting one another. Spacing of the piles is fixed by achievable construction tolerances and by the need to fit reinforcement in alternate or every pile(s). The initially placed (female) piles do not necessarily extend to the depths of the intermediate (male) piles (rather to one which allows water retention to be effected). Since the intermediate or intersecting pile is formed in a temporary casing usually rotated by an oscillating platform and guided by the initial piles, it is normal to specify a slower rate of gain of strength of the initial pile concrete to facilitate construction. The overlap of such piles is such that the dimension h is seldom less than 300 mm. The range of diameter of such piling is more limited than the contiguous wall because of the need for heavy job-specific plant.

Pseudo-secant wall is composed of either a contiguous bored-pile wall with the immediate area behind the wall jet grouted to effect a good waterflow seal (in

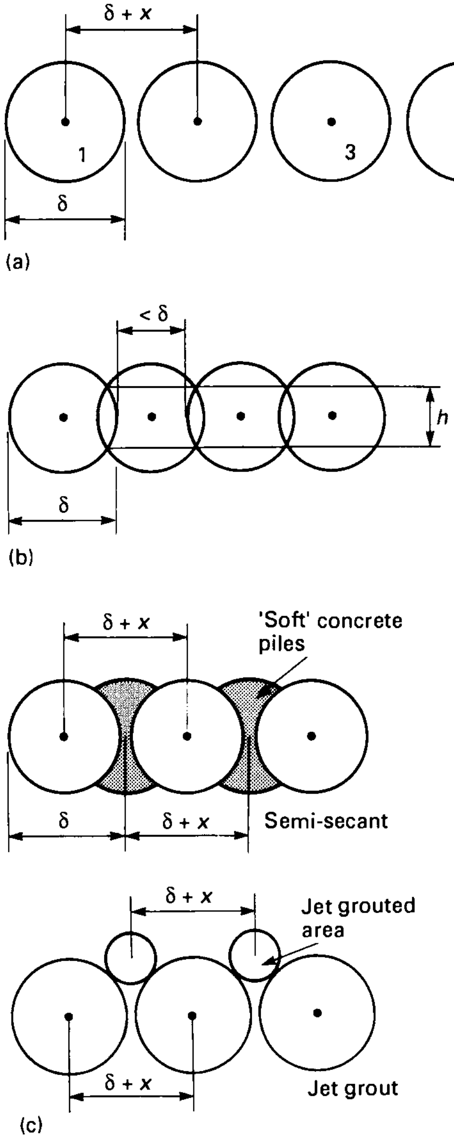

Figure 8.1 Bored-pile walling types.
(*a*) Contiguous piled wall;
(*b*) true secant piled wall;
(*c*) pseudo-secant walling

non-cohesive deposits) or a secant wall at close-pitched centres where the initial piles are not reinforced and are soft enough to allow drilling through them using bits or augers rather than heavy oscillating casing.

Augering through female piles has been developed particularly for use with continuous-flight auger-injected piling. The soft pile constituents are still being investigated but combinations have included a bentonite/cement mix, a PFA/ cement mix and a silica fume mix (a very fine waste product of steel production). The requirement is the need for low initial strength (for, say, 7–14 days) then a sufficient gain to satisfy engineers that a reasonably durable 'watertight' wall has been achieved.

8.2 Ground support

8.2.1 Design to limit and serviceability states

Good design must consider limit and serviceability states of ground support at all stages during construction. Often, serviceability limits are the more cost critical and, certainly in the case of ground and surrounding structures, movement can be the most publicly emotive feature. It must be realized that in building walls (and tunnels) ground will inevitably move[17]. This movement must be contained so that it least affects adjacent and surrounding 'structures', where drains, services, pavements and buildings are included. Permanent walling (or substantial temporary walling) in an urban environment should not be commenced until an agreed detailed schedule of conditions exist for surrounding structures. Such a schedule offers an important basis for subsequent negotiation and is very valuable to all parties, not least the scheme promoter.

It is unlikely in any urban environment that general lateral movement at the head of a wall of much greater than 40–50 mm will be acceptable, although there may be exceptions. Given such a figure and recognizing that bending moments increase with the square or cube of the effective span, wall types and spans are then limited to:

1. A cantilever not much greater than 5 m; and
2. A single-propped (top or bottom) not much greater than 10 m.

8.2.2 Temporary conditions

In ground engineering most 'wished-in-place' complete structures are easily demonstrated to be stable and structurally competent. It is more often the temporary condition which requires attention and dictates design. For example, the temporary case, illustrated in Figure 8.2, will give rise to larger bending moments, greater top-prop forces and the most sudden (if not the greatest) change in ground movement.

8.2.3 Ground factors influencing design

These factors include whether the ground is cohesive or non-cohesive soil or soft rock (rock SPT > 200 is unlikely to need piled support); whether it is loose, dense,

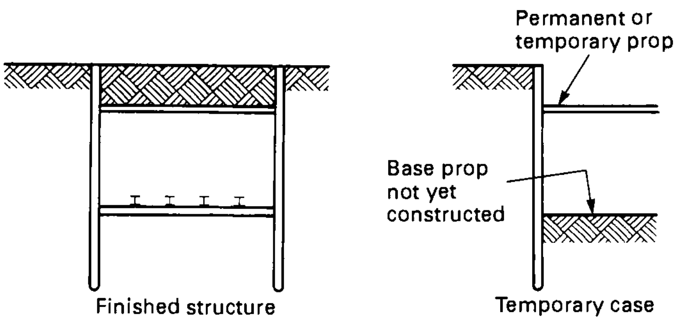

Figure 8.2 Temporary case conditions critical for design

sensitive, normally or overconsolidated; whether the groundwater situation is advancing, retreating, hydrostatic or part artesian; or whether it is likely to be influenced by the wall position.

The combination of these factors will influence wall type and are more fully explored in Chapters 4 and 6. However, this is an extremely complex matter, principally because of the highly interactive soil structure condition and its relationship to active, at-rest and passive earth pressure states (backfill pressures are unlikely to be relevant to bored-pile walls but, where applicable, the details in Chapter 6 may be relevant). Fundamentals of ground pressure may be appreciated but, other than where specific experience exists, engineers still have difficulty in relating ground knowledge to ground/wall behaviour. In some places metros are being developed in soils not directly comparable with past experience, where ground is not particularly well researched and features such as partial saturation not fully understood. The scales of underground rail transport schemes are such that a detailed design investigation should be undertaken. Using the best up-to-date practice, including finite element modelling, centrifugal modelling and well-researched local knowledge and experience, major design economies can often be made. Only in a very variable material could such an approach be questioned, and UK experience (in, for example, Keuper Marl soft rocks) has shown that simple research can permit generalizations and economies based on a rational rather than on an inspired guess. The alternative to such an approach may be to employ high factors of safety to the overall design, relying on these to ensure behaviour which satisfies the limit states, Although this may not be logical to the purist there may be cases where it should not be ignored by the pragmatists. Where prediction in design terms is difficult, then changing the solution may solve the problem. For example, adding a top or bottom prop may significantly reduce movement and bending moments without significantly changing the work cost.

8.2.4 Design literature

There is much 'wise' literature on wall design and ground support. The following list of authors, although not exhaustive, may prove useful should a more detailed appreciation be required.

Bolton [2]
Burland *et al.* [3]
CIRIA [6–8]
Fleming [11]
Goldberg *et al.* [12]
Golders Assoc. [13]
Huntington [14]
Peck [17]
Symons and Murray [19]
Teng [20]

Despite the reservations held regarding design understanding, it must be borne in mind that if one follows past practice as detailed in the literature no major wall is known to have failed in shear; no well-controlled excavation has been known to cause movement which has threatened adjacent stability; and no prop forces have been recorded which exceed the 'Peck's rules' distribution by a significant amount (say, 30%).

8.2.5 Ground conditions and design

None of the classic soil (rock) mechanics criteria can be ignored in piled walling, although some features may be exacerbated and some mitigated, dependent on wall type. The following text describes some of the ground/wall interactions under the headings of:

1. Earth pressure;
2. Groundwater;
3. Stability;
4. Base integrity; and
5. Miscellaneous problems.

Earth pressures

Prior to wall installation a lateral stress exists in the ground which is commonly known as the K_0 (at-rest) pressure and is given a value 1-sinθ in cohesionless normally consolidated soils. Overconsolidated soils have a higher K_0, dependent on their overburden pressure and stress history. If a wall could be 'wished in place' and no wall movement permitted at excavation stages, then any wall would have to carry fully developed K_0 pressures.

In practice, movements cannot be avoided (indeed the act of placing the wall itself, be it a diaphragm or a bored-pile one, allows considerable ground relaxation) even in the stiffest of structural configurations, and pressures fall towards the active pressure state K_a). This is the minimum earth pressure the wall could experience. K_0 is given the value of 1-sinθ/1 + sinθ in non-cohesive soils.

Resultant pressures are often intuitively visualized as a sliding wedge of material behind the wall, as shown in Figure 8.3. When the wall is being pushed into the soil then pressures generally greater than at rest can be generated, and at the limit passive pressures can be developed where K_p is given the value 1 + sinθ/1 − sinθ in a cohesionless soil.

The simple concepts using linear boundaries for the active and passive zones do not truly reflect the plastic failure mechanisms behind and in front of the wall. More detailed analysis including the effects of cohesion (c') and soil weight on the shape of the plastic zones will be found, together with appropriate earth pressure coefficients, in the works of Sokolovski and Caquot and Kerisel (Fleming[11]).

For a cantilever wall at failure, active and passive pressures are those that are relevant[7]. For a propped wall, particularly a stiff-propped one, despite relaxation when the wall is placed, it appears to be current practice to design the serviceability limit pressures somewhere between the active and at-rest state on the uphill side of

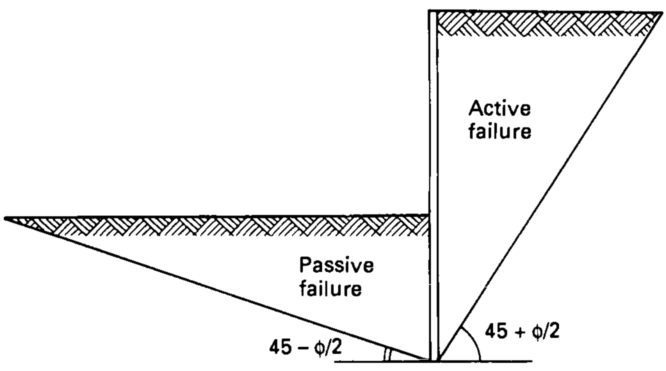

Figure 8.3 A simple conceptual failure state

the wall and passive states on the downhill. Such practice appears to correspond with typical Finite Element Method (FEM) analysis predictions, and although perhaps somewhat conservative (it will give higher bending moments than Teng's sheet-piling method, for example), seems a reasonable design route to follow for a structure with a life requirement of greater than a hundred years. This design feature is one which causes disagreement among engineers and needs research input.

The other feature which generates debate in design is the use of c' and its value. It is recommended that practice should follow Fleming's[11] advice, with c' approaching zero in high strain areas (i.e. to excavation level and above) and increasing with depth as soil stiffens, and strain and effective stress changes are small. High c' designs can only be tolerated if high safety factors are introduced to stability and moment predictions. Lateral pressures on walls because of surcharges should follow CIRIA 104[7] and DM7[15]. Similar ground pressures apply to all three bored-pile wall categories.

Groundwater pressures

True and 'psuedo'-secant walls should, for the purposes of design pressures, be considered as watertight structures to the base of their highest founded pile. A contiguous bored-pile wall, however, has virtually a drained face, and unless a direct 'facing' wall is stitched onto the bored piles with no drainage backing, groundwater pressures can be assumed to be less than those generated for the secant walls.

Provision of walls themselves can cause changes in the local groundwater regimen which must be incorporated into the design. It is essential that short- and long-term global field hydrology be understood, controlled and made to work with, rather than against, the walling system. Interestingly, in many developed countries, cities with existing metros are suffering from water tables which were depleted by pumping demands for industry and which are now being recharged as heavy industrial requirements are being replaced by the needs of lighter, modern industries. Renewed corrosion attack, loss of watertightness and increase in design pressures will result from such a fundamental change, which was unforeseen and unquantifiable one or two decades earlier.

To analyse differential wall pressures and any upward seepage forces on base slabs it is necessary to produce a full seepage analysis based on flow nets (or electric potential models). Simplified water-pressure methods are discussed in, for example, Burland *et al.*[4], Fleming[11] and CIRIA 104[7], which are generally accurate enough for preliminary design and indeed for non-sensitive full design. It is suggested that the scale of a metro is such that further analyses based on seepage flow net drawings are justified.

Contiguous bored-pile walls (and many secant walls below excavated level) do not act as a barrier to horizontal flow; this may be significant in relation to piping and stability (see below). Where the contiguous wall is permitted to drain and high active sidewater pressures do not exist close to the wall, engineers tend to opt for a nominal water difference to cater for local burst drains and or 'far-field' high water levels; typically, the depth quoted is in the range of

$$\frac{\text{Depth to excavation}}{5} \text{ to } \frac{\text{Depth to excavation}}{3}$$

dependent on the total dig.

Finally, as a prudent check a minimum design pressure for any wall equal to a full hydrostatic pressure would appear to be a necessary engineering measure (after Bolton [2]).

Stability
All single propped bored-pile walls share the same stability requirements; failure modes are illustrated in CIRIA 104 [7], factors of safety and design criteria being thoroughly discussed for stiff overconsolidated clays: the philosophies are equally relevant to other soils. Multi-propped stability is seldom a problem, but, as frequently emphasized by Ostermayer [16] and Fleming [11], the prop forces themselves and the adequacy of the prop must be fully defined and designed.

Changes in water table, particularly those which lead to equilibrium of pore pressures which are high at the passive excavation level, are unlikely to cause serious stability problems if the track slab is encouraged to act as a prop. Such a feature can be made even more effective if the prop is dished and an arch theory utilized in gauging the prop effect. In such a situation translational or slip stability is seldom a demonstrable problem. Structural connection details need careful consideration (Figure 8.4).

Wall translational problems could occur in cantilever and top-propped walls only where penetration is insufficient, perhaps because of a rock ledge or boulders (Figure 8.5). If this happens over only a metre or so (e.g. boulders) then the capping beam arrangement can be such that local spanning occurs. If the problem occurs over several metres, it is best avoided by ensuring anchorage back into the rock or introducing a bottom prop slab.

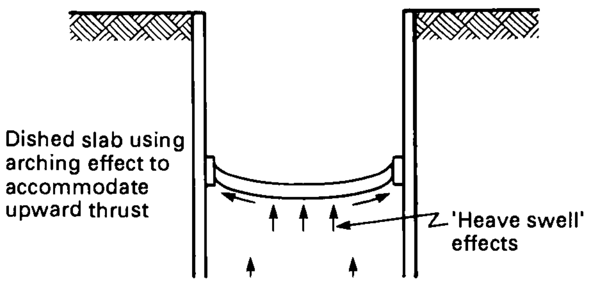

Figure 8.4 Slab design to cater for ground and groundwater pressures

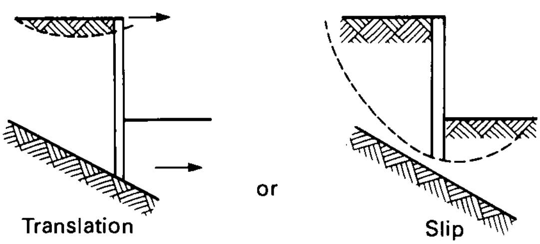

Figure 8.5 Disadvantage of 'hard' ground too close to surface

When underground works are being provided in earthquake areas there seems little guidance on (or internationally accepted manner of) design against translational or structural failure. Intuition suggests that wall pressures could be factored up by some functions of *g* (the gravity force) and that good jointing details (props to wall) are necessary to ensure that mechanisms are not encouraged. Presumably, any further actions are too costly in relation to the benefit the metro brings. It is ingenuous to expect, because metros tend to be in conurbations, that such regions are not susceptible to earth tremor effects.

Base integrity

Base stability was early established as an important feature in any excavation in loose sands or soft clays when below the water table.

Bjerrum's [1] requirement that the stability number (P_z/S_u) is below 6 for safe construction probably needs modifying to aim for a number closer to 2 to ensure that soil is not too plastic and that ground movement does not become excessive. Obviously, if piping in looser granular deposits (reclaimed land, loess, wind sands) appears to be a problem, the piles must be deepened (after methods stated in DM7 [15]) with some form of secant wall used to cut off the direct water head.

Where circumstances are such that neither of these options can be followed, extensive and time-consuming temporary works become involved, usually requiring local dewatering (and possibly local recharging). Again, the use of a curved bottom prop may well aid the permanent design case.

Miscellaneous problems

In any extended route there will be local variations which considerably inhibit perceived notions of how construction will progress. Examples may be local soft spots in soft rock; fissure, pipe or sand lens water flow; rapid changes in geology; presence of obstructions, boulders, impenetrable rock, organic material, or contaminated material. Piled walling shares this with other wall forms, but is sometimes disadvantaged in that the wall has no shear interlock (in a structural sense) in its longitudinal direction. To compensate for these possibilities, and in some ways to be prepared for them, no bored-pile wall should be contemplated without provision of a substantial, robust and adaptable capping beam. Generally, the beam should be able to distribute two or three suspect pile loads into their immediate pile neighbours with a possible option of even more distribution if necessary.

8.2.6 Props

Propping of piled walls follows conventional procedures, although some care must be taken in detailing prop connections. Ground anchors can be used to provide permanent or temporary prop support for walls. The design life requirement of underground railway works is, however, long, and many authorities still question the longevity of anchorages, particularly stressed anchors. Many underground railways will be in an urban environment. In hostile climatic conditions and in case of severe corrosivity if anchors have to be provided in such areas, then retesting and replacement facilities must be part of the wall detailing. Much good practice in anchor assessment and provision is provided in the UK DD 81 [9].

It is worth reiterating that walls seldom fail structurally, and that where props are involved it is their adequacy and importance which must not be overlooked. In building, the responsibility for these two items can often be divided, yet the loading from wall to prop and vice versa is very interactive. It appears sensible in underground rail works to ensure that complete responsibility (i.e. design as well as construction) for wall and prop or drainage provision is in the hands of one party, with the responsibilities of specialist subcontractors clearly defined.

8.2.7 Capping beams

Despite the lack of shear connection in contiguous bored-pile walling and the pseudo-secant wall, this is seldom inhibiting in the use of the systems compared with, say, diaphragm walls, provided the capping beam or some load spreading arrangement can be utilized.

For example, Figure 8.6 shows a building excavation of 35 m diameter which could be an access shaft for a tunnel or station. Here a continuous-flight auger (CFA) contiguous wall was used considering the wall as top-propped with the capping beam acting as a circular beam in hoop stress (as illustrated, the ground is of a dense, dry, granular material). Such a provision compares very economically with shaft sinking or a diaphragm wall.

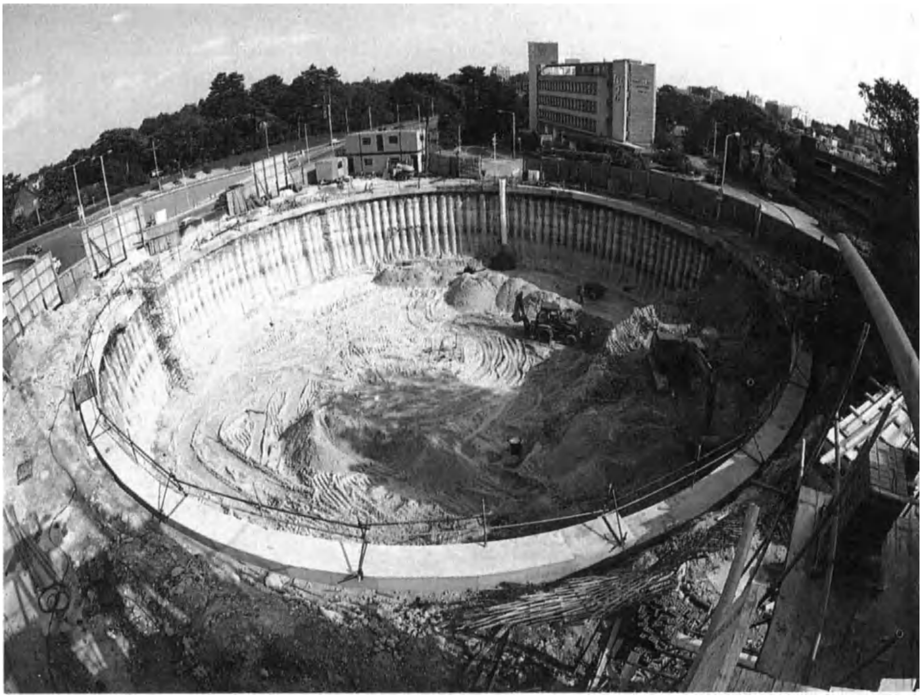

Figure 8.6 Use of capping beam and slender CFA piles to construct a circular access area in sands (courtesy Cementation Piling & Foundations Ltd)

8.2.8 Walls carrying vertical loads

Piled walls may act as barrettes carrying significant vertical loads (Figure 8.7). Walls are designed using classical soil mechanics approaches for deep strip footings. For example, if the wall is in a clay:

Q_{ult} total/m $= 2 \times A$ side $\times \bar{c}_u \times \alpha + 7.5 \times$ pile dia. $\times$ 1 m $\times c_u$

where 7.5 is substituted for 9 in the classical pile formula since the wall is a strip rather than individual circular footings.

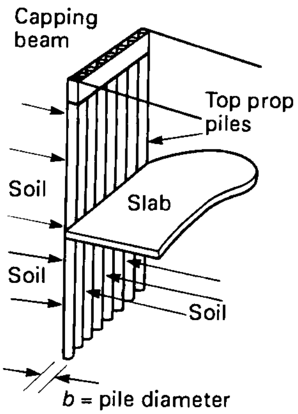

Figure 8.7 Use of a piled wall as a barrette to carry vertical loading

Even where loads are not applied uniformly at the head of the piles, the capping beam, waling, props and the ground reaction itself (the ground is a continuum – not discrete blocks) will generally spread the load over a sufficient number of piles to ensure a reasonably uniform load condition. Deepening individual piles in such a wall may permit them to carry more load, but it is generally considered better to maintain a greater uniformity in wall design, and this practice is not ususally followed.

8.3 Geometry and restrictions

8.3.1 Piling adaptability

The scale range of pile units (from 150 to 3000 mm) means that it is comparatively easy to negotiate sharp corners and bends compared with diaphragm walling. This feature is perhaps of more use in building applications (stations, drainage structures, inspection pits) than in long runs of gently curving permanent way.

8.3.2 Depth restrictions

There are certain depth restrictions to the various forms of piling which may affect local choices. Few underground walling schemes will be very deep, since tunnelling will then become more economic. However, the following approximate depth cutoffs may be usefully borne in mind at the planning stage:

1. Mini-augered or drilled piles are seldom provided at more than 30 m deep but more typically at 20 m deep.
2. Tripod-excavated piles in clay (limited casing of pile) can extend for 35 m but more typically about 25–30 m. They are easily reinforced throughout their length.
3. Continuous-flight augered (CFA) piles can be augered to greater depths but again typically extend 25–30 m. Where reinforcement (bars or stanchions) is pushed into the grout- or concrete-filled hole, its depth limit is between 12 to 20 m, depending on cage stiffness, and a typical 15 m limit can currently be assumed.

   Even with a highly plasticized (fluidized) concrete mix, difficulties in cage placement in dry sands/gravels have been encountered. The current explanation for this is that the surrounding sand soaks up water from the mix, stiffening the pile's periphery area and limiting steel penetration. One successful way of remedying this effect is to provide a given pile diameter with its corresponding steel helix for the pile size below (e.g. a 750φ pile with the helix for a 600φ – say, 420 mm). Although this may appear uneconomic, the time saved in cage placement can soon recover extra steel requirements. Cage placing in CFA piles must always be treated carefully, any overtly heavy handling should be treated with suspicion and cages must not be driven into the adjacent ground.
4. Rotary augered piles can be advanced to more than 50 m and are unlikely to be required deeper than this in underground railway works. Where ground is particularly water bearing or granular and collapsible the economics of using deep casing or introducing bentonite to the works will dictate whether the system should be used.
5. Large Benoto and Wirth rigs were used to oscillate true secant walls to depths of more than 30 m in Hong Kong in 'decomposed granites' with SPTs >150 at depth. The limit of such machines, as for rotary auger piling, probably far exceeds the economic limit between changing from walling to tunnels.

8.3.3 Tolerances

Piling specifications produced by trade associations such as the FPS of the UK[10] quote pile location geometrically to a tolerance of ±75 mm. Although, by use of guidewalls and tight control, such a figure can be and very often is bettered by field performance, it seems prudent that designers assume that such tolerance variations can occur in walling. All three of the piled wall types discussed require this tolerance allowance.

There is a suspicion that the provision of guidewalls, as well as aiding control plan variation (to, say, +25 mm), can help CFA piles in their alignment control. Such a view is not particularly objective, and (rather like water exclusion) many practitioners feel that much money is spent tackling a problem that is better accommodated within the design width of the permanent works.

Piling specifications, like those for diaphragm walls, show a vertical alignment tolerance of 1:75 in any direction. Again, although much better alignment can be achieved with careful workmanship, this will not be guaranteed by practising piling companies.

When positioning the permanent way on a tight curve this alignment tolerance, in addition to the location tolerance, can be critical, particularly at depth. For example, a piled wall with a top prop 5 m below the pile platform level (ppl) and the running track 9 m below the ppl could lose:

At the prop 75 + 5000/75 = 145 mm
At the track 75 + 9000/75 = 195 mm

Since this can happen on both sides, the trench width for the rolling stock could be reduced by 300–400 mm. If not perceived as a possibility at the planning stage and the wall positions suitably adjusted, such a loss could seriously affect service location ducts, air flow calculations, etc.

8.3.4 Unexpected deviations

More significant than controlled deviation is unexpected deviation resulting from pile deflection on, for example, boulders, claystones, adjacent piles (or adjacent piles concrete overbreak) or indeed any obstructions. Deviation problems and their correction appear to be exacerbated as excavation size decreases, i.e. the smaller the diameter of piles, the greater the possible deviation. For all pile sizes the following good practice recommendations need close attention:

1. Reasonable, firm piling platforms. There is an understandable reluctance to deliver pristine ground conditions to heavy civil engineering plant which almost immediately deposits clays, silts and bentonite over the platform. This reluctance must be put aside, a competent piling mat comprising a geotextile fabric and a significant amount (say, +300 mm) of well-graded granular material should be considered a minimum provision.

   Most piling rigs comprise a power pack with long auger/casing leaders, and if the piling mat is so soft that the rig tilts, then, with the best will available, the piles formed are likely to be affected.
2. The wrong system in a poor environment will always be problematical. Although, for example, CFA piling may be correct for most of a contract, it should not be attempted in a boulder medium (or one very full of residual corestones). Equally, rotary piling is seldom used in predominantly non-cohesive ground where deep casing and large overbreaks cause expense, possible pile deviation or even damage to adjacent structures.
3. Avoid solutions which require significant penetration into hard strata. This may only be possible by the introduction of extra props or an anchorage system.
4. Avoid areas of steeply sloping hard strata. Where such areas must be traversed consider use of solutions other than piling.
5. Ensure that the local technology is capable of supplying any special requirements necessary for the piling system. Most concrete supply is based on the philosophy that high strength is desirable. It should be recognized that low-strength mixes (for secant and pseudo-secant piles) require as much (if not more) care and control in their regular production than high-strength special mixes; one badly batched load in a 'soft' pile can cause major problems.

8.3.5 Contract control

Alignment (and deviation) variations occur outside as well as inside wall boundaries. Although not as visible (nor as contractually difficult), there may be legal considerations which must be known and addressed at planning stages.

Similarly, contract deocumentation and quantities must recognize the possibility of alignment changes causing variation. Figure 8.8 indicates typical worst-case deviations which may need consideration.

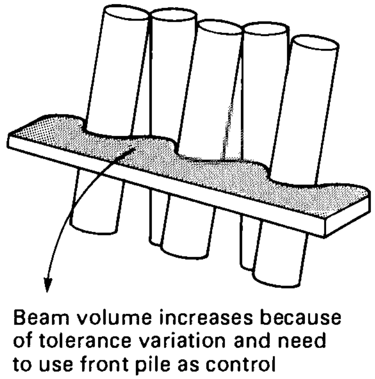

Figure 8.8 Alignment control means more excavation and concreting

Capping beams must be supplied to bored-pile walls not only as a cosmetic feature but also to act as a spreading beam to account for pile behaviour variations. Where piled walls are employed, different diameters will often be necessary, as shown in Figure 8.9. In such situations the control for trench geometry will be associated with the front face of the piles (or a set distance from this so that facing and alignment variation can be defined).

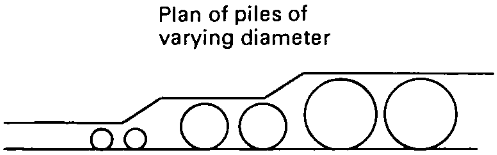

Figure 8.9 Front face used as alignment control for capping beams

The capping beam in this case can either always extend to the front face of the facing or, more generally, it may be a standard width above the pile diameter. Some practitioners would suggest pile diameter + 150 mm tolerance + 150 mm overhang either side = pile diameter + approx. 500 mm.

8.3.6 Example of good control

Many of the geometry problems in walling can be mitigated by simple but good practice. The rise and application of quality assurance schemes may help in this area. An example of cooperation in a very satisfactory piece of walling is that of Sceptre Court in the UK, where building piling straddled a possible tunnel route and was in effect contiguous walling on a curve. Without the use of guidewalls and with close control of the supervisory engineers and piling contractors (Pell Frischmann & Partners, Mott Hay & Anderson and Cementation Piling & Foundations Ltd) the worst alignment results were 1:500.

8.4 Water exclusion

8.4.1 Amount of water exclusion

The degree of water exclusion achieveable in underground works is proportional to the amount of money the specifier is prepared to spend, with the added proviso that total water exclusion is impracticable. This feature is worse on bored-pile walling since there are many more joint interfaces (at every pile) than with other walling systems, and joints are the most likely area for water seepage or issue.

In such situations engineers tend to follow one of two courses. Either water penetration is accepted as inevitable, encouraged down controlled paths and eventually piped away, or maximum exclusion is encouraged and clients live with the residual water penetration. This is the essential difference in the use of the contiguous and the true or pseudo-secant bored-pile wall, the former allowing near-field drainage, the latter attempting to exclude water.

8.4.2 Problematical ground

Water problems tend to be associated with non-cohesive materials or soft rocks containing areas of preferred channel flow. Recent investigations show that, overall, wall concrete permeability is very similar to that of overconsolidated clays with a high plasticity index.

8.4.3 Application to contiguous walling

Contiguous piled walling tends to be adopted in subsoils that are predominantly clayey, since granular materials require deep and expensive temporary casing. Flow in such clays tends not to be large to relieve local excess pore pressures. The soil face theoretically could be left exposed but practically this is seldom sensible.

In a predominantly stiff clay it may be sufficient to infill the pile cusps with a 'no-fines' concrete and to introduce a positive drain at the excavated level of the wall. Where there is a possibility of flow and fines being drawn through a wall, geotextile fabric with protective plastic facing can be introduced and connected to a positive drain. Such a system is illustrated in Figure 8.10 for a contiguous 750 mm piled wall in soft clays and coal measures of the English Midlands. The thin facing to the wall is purely cosmetic and can be replaced with bricks, cladding panels or concrete blockwork. Although it is possible to design an internal skin wall which is watertight to abut a contiguous bored-pile wall, this is seldom economic, and attempts to combine the internal wall with the piles lead to difficult and expensive local wall/pile connections.

8.4.4 Application to secant walling

Secant walls attempt to obviate the need for 'structural' internal walls by intermarrying piles with a good structural bond. In the scale of railway walling it is not usual for the intermediate pile to be reinforced.

True secant walling (oscillator-formed piles) has been accepted as a reasonable alternative to diaphragm walling in the provision of 'watertight' walling but not, so far, pseudo-secant walls. Typically, cement/bentonite mixes or PFA/cement mixes have been explored for the soft intermediate pile. Insufficient long-term evidence of water exclusion is available to satisfy many authorities that the design life will be

Figure 8.10 Contiguous wall 'drained' with a geotextile fabric (courtesy BTR Landscaper and Cementation Piling & Foundations Ltd)

sufficient for railway walling, particularly with the use of the more friable bentonite mixes. Attempts to use stronger mixes increase deviation problems with augers unless the strength growth rate is delayed. Some experimentation in the use of silica fume concrete is currently underway.

Practitioners advocating 'hybrid walling' often prefer the use of jet grouting in granular materials behind walls, arguing that where grouting is not appropriate then permeability will not be a problem (plastic, overconsolidated clays). Use of secant or pseudo-secant walls does not, of course, preclude a facing wall and drainage. Pseudo-secant bentonite walls were in fact first developed as efficient temporary wall solutions.

8.4.5 What is watertight?

Definitions of watertightness (accepting that no structure is 100% watertight) are always subjective. The planner/specifier is referred to the guidance given in CIRIA Report No. 81 [6], where a control framework is laid out. General adherence to this type of classification will facilitate exchange of experience and control measures in a reasonably objective way.

8.4.6 Total exclusion impossible

It is important to recognize the difficulties in excluding groundwater from bored-pile walling structures and the high cost penalties involved in trying to do so. Wherever possible, it seems more pragmatic to accept some water penetration and to design for its positive removal from the wall face. In carrying out such a practice a sympathetic overview of the effect of allowing for groundwater penetration or groundwater damming on the surrounding environment and the effect on global hydrogeology is required.

8.5 Construction and economy

8.5.1 Parameters

Good design of any product requires fitness for purpose, ease of safe construction, economy and some attention to aesthetic quality. Piled walling tends to be stronger in the first three areas. Fitness for purpose has been described: construction ease, use of proper materials, safety and economy are no less important.

8.5.2 Materials

The materials in use for bored-pile walling are those common to the construction and piling industry. As in most construction and material control, the simpler the specification, the better the construction. Material specification is well covered in FPS [10] and Goldberg *et al.* [12].

1. Concrete mixes for piling are required to be workable, not fast setting, cohesive under tremie or dropping situations, and made from sufficiently small aggregate to allow flow around steel (sometimes closely spaced, although never less than 90–100 mm) and self-compacting. In addition, they must be sufficiently strong to satisfy bending and bearing requirements. Mixes may also be required to be corrosion resistant and various additive/cement changes or additions are employed to meet these requirements. Local conditions and supply will dictate the type of concrete available.

   One disturbing feature of many modern specifications is the increasing tendency to insist on 'strong' mixes. Because higher-strength mixes contain more cement (therefore fines) and because many initial mix requirements are met with high-fines concrete, engineers have slipped into the habit of equating high strength with suitability. This need not be so. For example, piles ideally should be ductile units and should not suffer excessively from shrinkage cracking. Higher-strength concrete (i.e. higher cement) produces quite the opposite effect, encouraging brittleness and shrinkage. Underground walling is likely to be of a scale where particular materials specification design and control can be introduced and where application of inappropriate specification can be avoided.
2. Use of steel in pile walling will be dependent on local supply capacities. Steel can be supplied in the form of stanchions, channels or, more likely (and more efficiently, in a structural capacity), as bars. Steel can be mild (R) (fst $\approx$ 250 N/mm^2) but more typically is high yield (T) (fst $\approx$ 460 N/mm^2).

   Welding control and care is necessary in steel fabrication, particularly when steels are mixed. A cover of not less than 50 mm and often 75 mm is required in piled walling. Where bars are used as a helix (for hoop steel) cover also requires consideration (piling specifications tend to quote internal diameters of such hoops/links/helix). Where stanchions are employed then the 75 mm cover must be added to half the stanchion diagonal to define the minimum pile radius. Some practitioners advocate even greater cover to ensure stanchion fit (Figure 8.11). Bar steel will normally be inside the helix to ensure against bars bursting out.

Occasionally, bars are placed on the outside for economy. Where this happens, provided good welding detailing is followed, this practice can be adopted. However, it should not be encouraged.

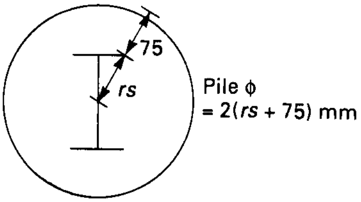

Figure 8.11 Cover to pile with stanchion

3. Facings to piled walls can be directly connected (as in a thin concrete facing) or totally divorced (as in a blockwork cavity skin wall). The whole gamut of facings are available to the scheme coordinator, from simple spray-on concrete through blockwork, brickwork and metal cladding panels. Figure 8.12 shows a true secant wall (totally unfaced) placed in the completely decomposed granites of Hong Kong. The finish standard appears to be good enough for station approaches and is certainly adequate for normal track areas.

Figure 8.12 Use of true secant piling to form a reasonable quality finish requiring no facing – Hong Kong Metro (courtesy Lillbore – F. C. Lilley, UK)

8.5.3 Structural integrity

The structural integrity of each unit or the piled wall itself must be assessed and ensured. Such a requirement will be mandatory in the specification to local ordinances and local codes of practice.

Typically, bending steel design is likely to be required to practices similar to ACI or BS. Some authorities may adopt higher ultimate load factors because of the expected longevity of the structure (e.g. BS 5400 required LF = 1.5: cf. BS 8110 (LF = 1.4)). Where bars are being used it should be remembered that two may lie on the neutral axis of the pile and contribute little to bending strength. Stanchions must be carefully aligned to ensure maximum economical use of their geometry. Where many hundreds of piles are being placed, as in railway systems, it is naïve to expect all reinforcement to be perfectly aligned, and, where possible, bars are favoured rather than stanchions and symmetrical steel arrangements preferred to (the perfectly possible) assymetric ones.

Shear in circular sections is an area that needs research to clarify engineers' understanding of its mechanics. Shear strength in piles and circular members derives as much from dowelling and aggregate interlock[18] as it does from the truss effect of shear links. Specifiers continue to insist on shear link design to codes such as BS 5400, which often involves costly steel and fabrication provision and the need to adopt special mixes for concrete penetrations.

Similarly, there are occasions where crack steel requirements are too onerous. Perhaps if significant crack widths can be developed, it may be more sensible to use epoxy-coated steel (BCA are doing much useful research in this field).

Bond, curtailment and lap specifications for piles follow normal reinforced member requirements. Where steel is needed for fixing into capping beams, bends are seldom sensible on-site on steel much thicker than T20 diameter bars.

8.5.4 Connections

Connections into a row of circular sections are not easy and consequently can be expensive. It is reasonably simple to provide boxouts and coupler connections in diaphragm walling, but this tends not to be done with circular piles. Practitioners agree that drilling connections after excavation appears the most pragmatic method of dealing with this problem; they appear less reluctant to provide boxouts in front of main steel (for location purposes rather than the full boxouts). If boxouts and couplers are required, fixity and tolerances must be very carefully detailed.

Connections are usually designed not to transmit moment but rather thrust and possibly (unless the ground slab is fully suspended) vertical upthrust. Such loadings are dealt with by using a form of knuckle joint incorporated into the slab on one side and the waling beam connected into the pile on the other. Such a system was successfully used on the A406 Chingford to Hale End roadworks in the UK (Figure 8.13).

Any connection involving attempts to create a pin within a monolithic concrete connection must involve concrete cracking, which may not be acceptable in such an important area. As has been stated, prop integrity and connections are critical to all underground wall stability.

Connections to capping beams can be easily effected and capping beam connections to the piles are widely practised techniques, perhaps explaining the tendency to utilize top props in temporary and permanent works. The comments on connections apply to all types of bored-pile walling.

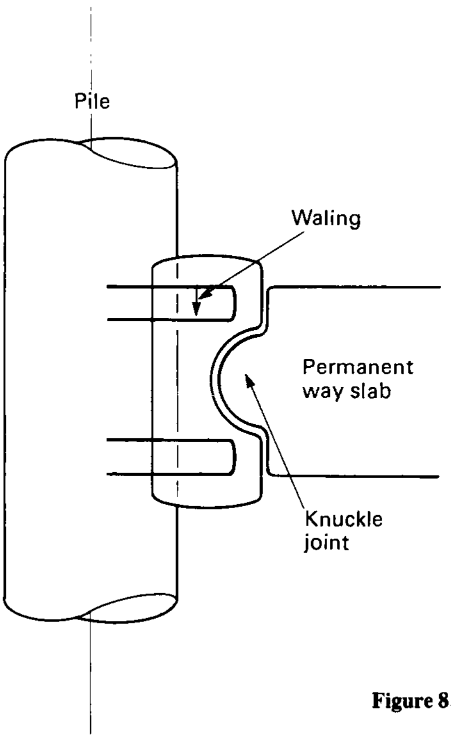

Figure 8.13 Slab/pile connection via waling beam

8.5.5 Plant

Guidance on pile type and construction is clearly explained in the Pile Guide series[6]. Tripod-excavated piles are formed using a labour-intensive shell and auger technique, winch driven and supported on a three-legged frame. Although few main wall runs would be constructed using this technique, it is useful where access is difficult, time being saved by employing several rigs at once. The system is at its most effective in clay strata, where casing requirements are at a minimum.

Rotary piles use powered cutting augers to advance piles to depth. They are used particularly in clays and soft rocks. Although they can be employed in cohensionless materials (with support from casing or bentonite) they soon become economically uncompetitive if excess granular material requires to be piled.

CFA piles are ideal for mixed ground conditions (but are not useful in ground with significant obstructions). In this system a hollow-stemmed continuous auger advances a hole to the required depth, the powered auger and flighted casing acting as ground support. At the required depth the end 'bung' is blown and grout or concrete is pumped into the hole as the auger is withdrawn. In some systems steel cages can be placed inside the auger's hollow stem but more usually are driven into the wet concrete at the end of the operation. Controllable steel depth penetration is limited, which limits the effective length of the CFA pile as a bending member.

Secant piled walling carries its own casing as part of its installation procedure making it suitable in most ground conditions. It is the popularity of the continuous-flight auger pile that is encouraging the growth of the pseudo-secant wall.

It can be seen from the above that most piled walling tends to be built using proprietary specialist mounted leaders and augering/drilling/oscillating tables on lorry-, track- or crane-mounted bases. Typical power pack requirements, leader heights length and access width are shown in Table 8.1. In urban areas simple geometry logistics will dictate the type of plant and solution that can be employed. Piling type mixtures are likely to be obligatory.

Table 8.1

Pile type	*Plant dimension* H(m) L(m) W(m)	*Power unit*	*Leader Height* (m)	*Closest*[a] *distance of centre to nearest building* (mm)	*Typical pile* φ *range* (mm)	*BM range* (kNm)
Tripod	2.5–4 × 6.5 × 2.5	Winch (air diesel electric to 14t pull)	2.5–5	600	450–600	100–350
'Grout' and mini-pile	1.5–2 × 2.5 × 0.75	2t to 6t feed	2.1–6	300–450	150–300	50–100
Rotary	3–4 × 9 × 4 3–4 × 11 × 4	30/60t crane base	25/26	900–1000	450–1500	100–4000
Continuous-flight auger	3–4 × 11.5 × 6.5	30/60t crane base	21/29	900–1000	300–900	50–1000
Secant	3–4.5 × 9 × 4.5	30/60t crane base	22	1000–1500	1000	Up to 3000

[a] Greater allowance necessary at corners.

8.5.6 Factors affecting economy

Piling contractors are motivated to keep their production fast on the grounds of economy. Fortunately, this coincides with most design requirements in that faster construction involves less disturbance, less ground relaxation and less opportunity for groundwater seepage to establish itself as a problem to good integral pile formation.

Major plant requires shallow-sloped ramps between changes in elevation (usually not greater than 1 vertical to 10 horizontal) and a competent piling platform formation. Provision of a well-placed, well-maintained mat will, in any case, enhance overall project efficiency, safety and economy, permitting rapid and effective removal of pile 'risings' (often a considerable quantity of earthworks) and delivery of reinforcement cages and concrete to the pile position.

Speed of construction is seldom limited by machine capacity; rather it is affected by logistics of access, materials supply, cage fabrication, erection and concreting (whether dropped, tremied or pumped). Also there are limitations on working hours, on noise and by site-delivery constraints, particularly in urban environments. With these difficulties in mind, Table 8.2 indicates typical productions (for types and typical pile depths) for piled walling. It must be remembered that walling is not done sequentially: at best, alternative piles are constructed. A planner must not expect the production rates quoted to correlate with rates of complete run of lineal wall.

Table 8.2. Number of piles/week[a]/rig

Pile diameter range	*Cohesive ground with no water*	*Cohesive ground with water*	*Cohesionless ground with no water*	*Cohesionless ground with water*	*Pile type*
450–750	Not less than 50/week all soils (depedent on concrete supply)				CFA
900–1200	25	20	5–10	5–10	Rotary
1350–1500	20	15	3–5	3–5	Rotary

[a] Assumes no serious obstructions including rock: all piles approx. 20 m deep.

8.5.7 Comparative costs

Given the range of ground, environments and general problems, it is not possible to predict costs of walling in any general application. Some indication may be gained by indexing bored-pile walling to diaphragm walling and assessing approximate relative costs. This has been done in Table 8.3 from local UK information supplemented by some US and Asian data. These figures are necessarily approximate: they are for guidance only, and must not be considered absolute.

Table 8.3

Wall type	*Diaphragm wall*	*True secant*	*Soft pseudo-secant*	*Jet grout pseudo-secant*	*Contiguous piled*
Cost/m² (%)	100	120–90	85–65	80–60	65–55

8.6 Future use

Over the past decade there has been a large increase in buildings with deep basements and a worldwide requirement for such structures. The need to find rapid, economic solutions has encouraged much research and understanding of wall behaviour. Within this period piled walling has been introduced as a real alternative to traditional deep-walling solutions, primarily because temporary support features can be incorporated into a permanent solution relatively cheaply.

Cost comparisons show how competitive bored-pile walling can be. With continuing research into innovative techniques and plant currently spearheaded by Japanese, French, Italian, West German and UK practitioners, the use of bored-pile walling is likely to increase for the provision of underground walling for transportation purposes.

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Chapter 9

Hand-dug caissons or wells

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Contents

9.1 Introduction

The British Standard Code of Practice for Foundations, BS 8004: 1986 [1], defines a caisson as 'a structure that is sunk through ground or water, for the purpose of excavating and placing work at the prescribed depth, and which subsequently becomes an integral part of the permanent work'. Consequently the method of construction described as hand-dug caissons, commonly abbreviated to HDC in Hong Kong and elsewhere, should really be classified as shaft sinking (also known as Californian wells or Chicago caissons). The definition given above is partially met since the temporary support for the ground is usually in the form of cast-*in-situ* concrete rings, which subsequently become part of the permanent works. In reality, however, such construction is best described as a bored cast-in-place pile.

The advantages of hand-dug caissons in the construction of underground railways are those of an excavation technique which is able to overcome obstructions and services while at the same time requiring little working space with minimal headroom. The system also provides a relatively quiet means of constructing temporary support to excavations which can be incorporated into the permanent works. It facilitates control of settlement of surrounding ground and buildings. Both features are of great benefit in the urban situation where the need for mass transit systems arises.

The technique can be used with equal facility for both cut and cover tunnels and station concourses utilizing either bottom-up or top-down construction, with the knowledge that connection details between slabs and walls can be readily detailed and precisely located. The method is particularly suitable for the construction of permanent columns and piles in top-down construction, allowing commencement of superstructure construction while bulk excavation is still continuing below.

9.2 Means of construction

Caissons are normally hand excavated by a miner and winch operator team in stages about 1 m deep. The minimum diameter is about 800 mm, but 1200 mm provides better working space allowing the miner to operate more conveniently. The usual method of spoil removal uses a bucket and winch, supported by an A-frame over the excavation. Each stage of excavation is successively lined with a minimum 100 mm thickness of 20 MPa strength concrete using a tapered steel shutter suitably braced and designed for ease of striking. Larger-diameter caissons require greater thicknesses of concrete lining. The shutter remains in place, providing support for the fresh lining and surrounding ground, as the next stage of excavation proceeds (Figure 9.1). As with all mining excavation techniques, it is important to relate the timing of the installation of support to the stand-up time of the excavated ground. Submersible electric pumps are commonly used for dewatering within caissons, and in large groups of caissons, those in which excavation has been completed are frequently used as wells to dewater the adjacent caissons. Rarely, specific external dewatering has proved to be necessary to prevent bottom failures. Some cities, such as Chicago, place limits on pumping rates. Baker *et al.* [2] state that the maximum allowable pumping rate from a caisson is 60 l/min.

Excavation is undertaken using pneumatic spades or pick and shovel and loading into muck skips, while that through corestones and other obstructions is invariably

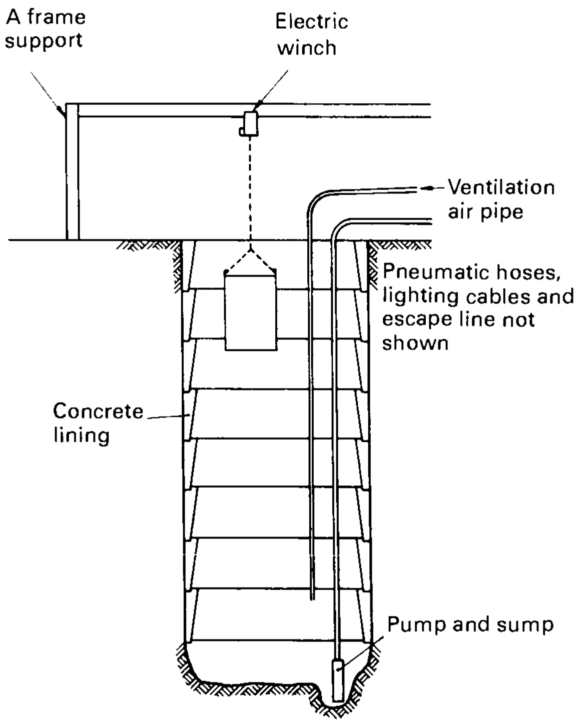

Figure 9.1 Section through hand-dug caisson

carried out by pneumatic drilling. Hand-dug caissons encounter difficulties in soft clays and coarse sands which are present in material such as colluvium, alluvium or marine deposits. The difficulties may include collapse of the caisson side, heavy flow of water, piping and heaving in the base, and leaching of fines. If ignored, these difficulties often lead to settlement of the surrounding area.

These problems cannot only be overcome by dewatering. Grouting or techniques such as forepoling with timber boards or reduction in the depth of excavation stages may be used. Groundwater recharge is sometimes used to overcome settlement problems due to groundwater lowering.

Contiguous hand-dug caisson walls have been used to support deep basement excavations in areas of high groundwater. A cheaper system of hand-dug caissons and intermediate concrete lagging has been adopted where groundwater is low. Although hand-dug caissons to support deep excavations may initially be more expensive than a steel pile wall, their use is quieter, allows the works to proceed without delay, is often suitable for confined sites and offers the advantage of incorporation into the permanent works. Comparison of costs is obviously heavily dependent on the ground conditions encountered. Hand-dug caisson methods have the considerable advantage of affording better access to effect removal of obstructions which would sometimes prove impossible by other techniques (Figure 9.2).

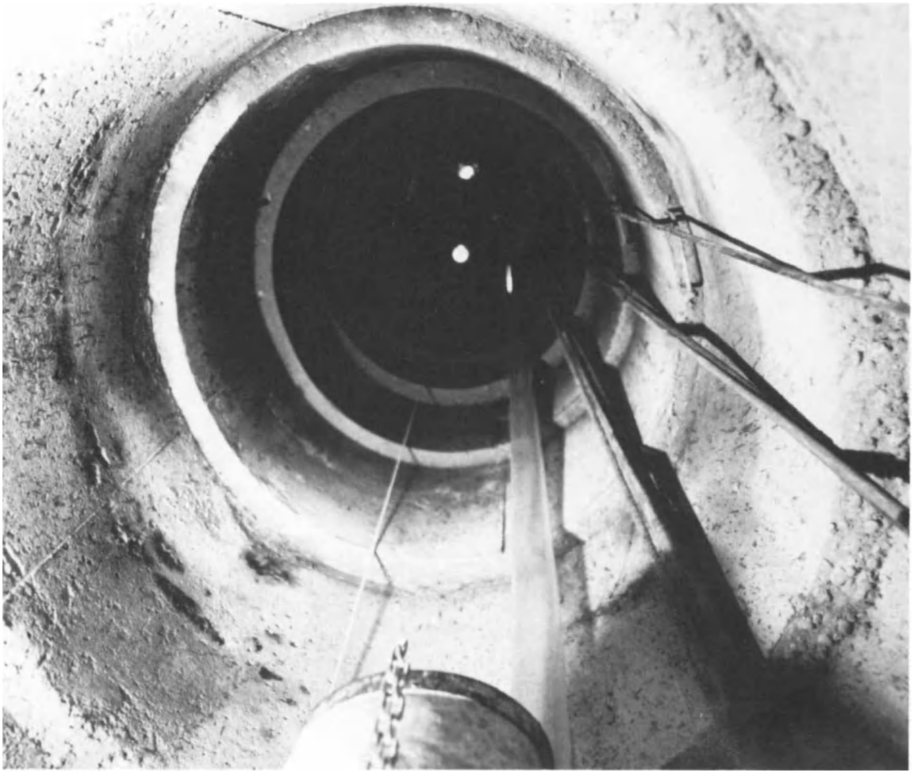

Figure 9.2 *In-situ* lining, air-ventilation hose, pneumatic hoses and lighting cables

9.3 Safety precautions

Safety precautions are paramount for the construction of hand-dug caissons, particularly with regard to falling objects, caisson collapse and adequate ventilation to prevent accumulation of noxious gases. BS 5573: 1978[3] describes safety precautions in detail.

Prior to work commencing, all equipment should be checked to ensure compliance with safety regulations. Regular checks should also be undertaken during operation. Important aspects are the provision of:

1. Adequate site investigation;
2. Experienced supervision;
3. Surface observation to ensure assistance is made available in the case of need;
4. Adequate cleared and fenced surface-access platforms;
5. Ventilation and gas-detection equipment;
6. Lining, to be placed as soon as possible and within reasonable stand-up time;
7. Harnesses, helmets, breathing apparatus and escape ropes.

The procedures used in Hong Kong[4] are described below.

A pair of timber frames supporting a steel runway beam is set up immediately above the centre of the caisson position. The support frames especially should be checked regularly to ensure that they are stable and that the connections between the runway beam and frames are adequate. A drum skip of $0.2\,m^3$ approximate capacity is attached to an electrically operated chain block supported by the runway beam. The chain block can be moved along the beam by means of a hand-chain operated gearing system. The drum skip is of steel construction and fitted within the frame to allow tilting for discharge. The catch controlling the skip-tilting operation should be fitted with a safety device to prevent release through inadvertent contact with the lining or other obstruction when lifting or lowering. Where the skip is used for access into the caisson a safety harness should be worn with the line securely clipped to the hoist.

Air should be supplied to the caisson using electric fan blowers with very light plastic tubing ducting the air supply directly to the bottom of the caisson. Such equipment can deliver air at the rate of $40\,m^3/min$ and should ensure that the ventilation within the caisson will be adequate to ensure a non-toxic working environment. Particular care is necessary when excavating through grouted material or where the presence of methane or other gases is suspected.

To protect the open excavation, guard rails and toe boards should be positioned around the caisson. During periods when the caisson is not being worked a substantial wooden cover should be provided to cover the open bore.

All electrically driven submersible pumps should be regularly checked to ensure proper insulation of the outer casing and all lighting should be of the waterproof type, fitted with heavy-duty cable and fully sealed connections.

Excavation should be undertaken by hand in stages not exceeding 1 m. At each stage of excavation the soil should be examined to establish its solidity and to assess stand-up time and the likelihood of settlement occurring as a result of the excavation continuing.

The water flow into the caisson should be carefully monitored, and if sudden increases become apparent the supervising engineer must be advised. Monitoring using piezometers and survey points on adjacent structures may be necessary depending on the particular circumstances prevailing.

If difficult ground is encountered, digging depths may be reduced, thus enabling the lining to be applied more rapidly. Prior to each successive excavation operation timber poling boards, sloping downwards and outwards, can be driven into the surrounding ground providing support for the segment of lining previously cast. The size and length of timber needles required depend on the condition and depth of each dig.

Each stage of excavation should be lined prior to cessation of work for the day with a minimum of 100 mm thick 20 MPa grade compacted concrete using a tapered steel form suitably braced and designed for ease of striking. Where poor or disturbed ground is encountered consideration must be given to using increased ring thickness and/or a concrete of higher strength. The shutter should remain in place to provide support for the fresh lining and surrounding ground while the following stage of excavation proceeds. Each ring should abut the preceding one.

Concrete lining hoop reinforcement is not generally used, but in poor or disturbed ground, vertical reinforcement giving a mechanical tie between rings should be employed. The lap is achieved by driving the reinforcement into the ground and into the space to be occupied by the next lower ring. The size and quantity of vertical reinforcement should be calculated taking account of the weight of lining which would hang in the absence of skin friction.

Prior to placing the hearting concrete the linings should be inspected for water leaks which should be sealed with a quick-setting cement mortar. When the caisson hearting is reinforced, the cage is often assembled within the empty bore. Concrete spacers are attached to the cage to enable accurate locating to be achieved.

All concrete placement shall be undertaken in accordance with normal good practice. For concrete placed in the dry, high workability concrete, 100 mm slump, is desirable. Hoppers and elephant trunking should be used to deposit concrete in the caisson avoiding segregation. Vibration is normally employed.

Care should be taken to ensure that adjacent caissons are not being excavated during the concreting operation to avoid the risk of breakthrough caused by the rising head of concrete.

9.4 Advantages and disadvantages of the technique

9.4.1 Advantages

The advantages of hand-dug caissons as a construction technique are briefly listed below:

1. *Quality control.* Concrete can often be placed in the dry. The quality of the shaft surface and shaft concrete is ensured to a high degree compared with machine-bored piling utilizing tremie concrete placed under water or bentonite.
2. *Reinforcement steel and steel column members may be accurately positioned.* Cages and columns sections can be easily and accurately aligned and fixed in position when compared with cages or assemblies, which must be suspended from the surface during concreting or machine-bored piling. Connections of slabs to walls can be easily detailed with 'bent out' bars. In many instances, because of space limitations, the reinforcement cage is actually built up within the caisson.
3. *Founding-level inspection.* The base of the caisson foundation can be inspected, thus eliminating the difficulties experienced in machine-bored piles. Quality of the founding-level conditions is thus guaranteed. An additional advantage is where the founding level is in rock, as it can be further assessed by probes drilled from within the caisson.
4. *Bell enlargement.* Bellouts can be employed to overcome problems with unexpected ground-bearing conditions. This enables concrete in the pile shaft to be stressed to the maximum allowable stress and stresses to be reduced at founding level. As a commentary on inspection requirements, in Chicago, Detroit and elsewhere, when bored piles are used, casings are sealed to bedrock and a caisson crew is often used to excavate and clean rock sockets (Baker and Khan [5]).
5. *Lateral continuity between contiguous wall elements.* Bent-out bars or bars threaded through inserts can be used to make longitudinal connections otherwise impossible in machine-bored piles or diaphragm walls.
6. *Environmental.* The noise level at the surface is greatly reduced in comparison with that generated in machine-bored piling. In addition, the technique is vibration free.
7. *Limited capital investment.* The limited capital investment required means that mobilization can be rapid and that many caissons can be worked on simultaneously.

8. *Confined spaces.* The space required is not much greater than the diameter of the caisson, although room for a spoil heap is needed. In a line of caissons this can be between the caissons, with temporary access required to remove the spoil heap. The headroom required is only 2 m. Reinforcement cages do not need to be prefabricated on site and can be built up within a caisson.
9. *Obstructions.* The technique is often used to resolve unforeseen construction problems which cannot readily be surmounted by other methods (e.g. as described by Gale and Baker [6] and Archer and Knight [7]).
10. *Depth.* Caisson depths can exceed the maximum depths of conventional driven and bored piling systems.
11. *Pile caps eliminated.* The economy of eliminating pile caps when hand-dug caissons are employed is often overlooked by designers. If the caissons are hand dug the diameters can be varied to allow a single caisson under each column, thus eliminating pile caps. Lateral stability can be achieved with simple structural solutions utilizing bracing members and bending capacity.
12. *Variable sizes.* The opportunity to use varied-diameter caissons has the advantage of minimizing costs by matching caisson capacity to the required design load. Differential settlement can also be controlled by varying caisson stiffness if required. Much larger cross sections can be utilized than are available in machine-bored piling.
13. In-situ *scale testing.* Hand-dug caissons offer the opportunity to carry out *in-situ* testing for more rational design (e.g. Sweeney and Ho [8]).
14. *Separation or 'slip' linings.* Hand-dug caisson linings can be used as sleeves around a shaft constructed within independent formwork to allow relative movement between the soil and the shaft. Oweis and Muller [9] describe the Exchange Place Centre in Jersey City, where some 53 caissons of 1.12 m diameter were socketed into rock and sleeved to eliminate loading on the PATH transportation tunnels.
15. *Steeply dipping strata.* Steeply dipping bedrock strata can be inspected and a horizontal founding level prepared to avoid subsequent movement in the direction of dip.

9.4.2 Disadvantages

1. *Potential settlement problems.* When hand excavation is used below the water table in fine-grained soils the dangers of piping failure and loss of ground affecting workers and adjacent structures are very real. Loss of ground can occur as base heave or washing out of fines due to inflow of water. Lowering of the water table can also cause ground compaction and settlement.
2. *Specialist labour force.* The necessary pool of experienced labour may not be available.

9.5 Building foundations

Major multi-storey developments are frequently sited over underground stations and hand-dug caissons provide a ready and suitable means of providing foundations within complex underground station structures. Typical examples are Hong Kong Mass Transit underground railway stations at Causeway Bay and Sheung Wan East, which employed hand-dug caissons in very different site conditions. General building experience utilizing hand-dug caissons is relevant and applicable to underground railway construction and a wide range of examples is available for reference. Some examples are given in Table 9.1.

Table 9.1 Typical HDC building foundations

Site	*Principal dimension*	*Design load* (MN)	*Reference*
Asia Insurance Building, Singapore	4.6 m dia. shaft, bases belled out to 7.6–7.9 m dia.		Nowson [11]
Treasury Building (52 storeys), Singapore	Central core – 8 m dia.	240	Davies [12]
Development Bank, Singapore	7.3 m dia.		Ramaswamy [13]
Union Bank (62 storeys), Singapore	5.0–6.0 m dia. belled out to 6.0 m and 9.0 m, depth to interbedded siltstone and sandstone 100 m	Up to 380	Kurzem and Rush [14]
Bank of China Building (70 storeys), Hong Kong	9 m dia., maximum bell size 12 m dia, 26 m to bedrock	1000	
Sun Hang Kai Centre, Hong Kong	5–7 m dia.		Cheng and Cheng [17]
China Resources Building, Hong Kong	5 m dia., 40 m deep		Morton and Tsui [18]
Carlton Centre (50 storeys over 30 m deep basement), Johannesburg	2.5 m dia. belled out to 5 m dia.	80	Heydenrych and DeBeer [19]
Standard Bank Centre (150 m high on a 20 m deep basement), Hong Kong	Raft foundations incorporating caissons 5 m dia. belled out to 6.0 m	60	

The Chicago method of hand-dug circular holes, excavated to a hard-bearing stratum, often with bellouts and with the excavation wall braced with semicircular steel rings and wooden lagging, is described by Baker and Khan [5]. These caissons were often more than 20 m long and were extended to bedrock if required. The technique was used from about 1900 to 1950, but machine boring after the Second World War led to quicker construction and cheaper foundations. Gill [10] states that one auger rig can typically install three 30 m deep 1.5–2 m diameter caissons in a week, while hand excavation is still used for restricted areas inaccessible to rigs or for underpinning.

A major benefit recommending the use of large-diameter caissons for building foundations is their high load-carrying capacity. Further, the method does not require good access and the dimension of caisson can be readily varied to optimize economy and efficiency. Where a high concentration of load occurs in buildings which support their load on a central core or several towers any attempt to spread building load would require an exceptionally thick and highly reinforced raft. Hand-dug caissons allow a direct column-to-pile connection to be made with an appropriately sized pile at each column location, thus eliminating pile caps completely. Changes in caisson diameter can be effected to some degree with mechanical boring, but with an impact on cost and operational activity which is not reflected in hand-dug caisson techniques.

The use of hand-dug caissons in Asia is common, perhaps because of the low capital requirements and traditional labour skills, organization and regulations. Table 9.1 illustrates the range of the projects where hand-dug caissons have been used; principal features and further references are quoted.

The 5–6 m diameter caissons constructed for the foundations of the Union Bank building in Singapore were excavated in 2 m lifts, at 0.7 m per day simultaneously over the site, and lined with 300 mm thick unreinforced cast-*in-situ* lining. When rock was reached, geological mapping indicated that the design pressure should be reduced to 4600 kPa maximum and the bell sizes were increased accordingly. The bell was carefully constructed, with four mined buttresses supporting the shaft concrete and curved steel supports spanning between the buttresses supporting the rock (Kurzeme and Rush [14]).

Mackey and Yamashita [15] describe the first attempt at caisson sinking in Hong Kong, which commenced in 1958 and which used techniques significantly different from those currently employed by HDC contractors. The method employed was the Shinso Well system, a Japanese version of the Chicago method, which involved sinking shafts of 2.6 m diameter by hand excavation. The shaft lining consisted of corrugated iron sheets about 0.9 m long by 0.6 m wide supported by 150 mm T-section waling rings at approximately 0.9 m centres vertically down the shaft. The excavated hole was larger than the diameter of the lining and the 150 mm annular space so formed was filled with gravel to form a continuous vertical filter down to a sump at the base of the excavation. In order to minimize the difficulties envisaged, a system of groundwater lowering over the whole site area was installed. However, adjacent buildings experienced settlement and this, combined with the presence of boulders and ground heave in the caissons, led to cessation of the excavation.

The versatility offered by hand-dug caissons in dealing with obstructions and allowing variations in the foundation system is demonstrated by their use to maintain the building programme of the new Hongkong & Shanghai Bank headquarters building in Hong Kong. Originally, a diaphragm wall was designed to support the basement excavation. Although every panel position was explored by boring, all failed to detect a wall of steel sheet piles which proved to be part of the temporary works associated with the previous building. Conventional methods of removal failed; following chemical grouting down to a depth of 10 m, a series of contiguous split hand-dug caissons were built to surround the sheet pile iron which was then removed. The resulting trench was filled with lean concrete through which the diaphragm wall was subsequently excavated.

This work seriously affected the overall construction programme. To overcome the delay, large-diameter hand-dug caissons were constructed in advance of main bulk excavations. The caissons, having an internal diameter of 9.9 m and a wall thickness of 500 mm, were dug using both machine and hand techniques using lining rings 500 mm deep. These caissons were sunk to the lowest basement level. Within each caisson, four further caissons with internal diameter of 3.5 m and a wall thickness of 200 mm were hand dug into the bearing strata some 30 m below (Archer and Knight [7]).

The main depot building for the Hong Kong Mass Transit Railway Corporation at Tsuen Wan is some 400 m long and 160 m wide overall and covers some 5.2 ha.

Superimposed over the depot building are 17 residential tower block buildings of between 28 and 30 storeys. The columns which carry the surcharged depot building are supported on single caissons sunk to bedrock (Burrows and Croft[16]).

During construction of the China Resources Building significant ground settlements were recorded. A reduction in the number of caissons and groundwater recharge reduced the magnitude of the settlement later in the construction period, despite large piezometer headloss (Morton and Tsui[18]).

9.6 Caisson walls

The main advantages of caisson walls over conventional retaining walls are similar to those of caisson foundations: they can be constructed in areas of limited room; they act as both temporary and permanent works; and their use solves the problem of obstructions (Figure 9.3).

There is also scope for innovation. For example, a most unusual buttressed retaining wall was constructed by mining a series of interconnected hand-dug caissons. Restrictions on the use of permanent anchors precluded the use of a tied-back caisson wall, so a buttressed wall was used, incorporating vertical anchors into the buttress to carry the loads. Beattie and Mak[20] describe the work.

An anchored retaining wall formed using 1.8 m diameter hand-dug caissons at 3.0 m centre-to-centre spacing was constructed to support Ching Chueng Road in Hong Kong. The site area was some 50 m by 90 m. This was reduced to about 40 m width and the wall formed using 30 primary caissons about 12 m high with a 45° slope above. The caissons were anchored with steeply sloping anchors to ensure that all support works remained within the site. Arched lagging with a drainage layer behind was constructed between the primary caissons. The anchors were permanent and instruments were incorporated into the wall to monitor long-term performance (Moh *et al.*[21]).

Examples from some of the Hong Kong Mass Transit Railway stations are described below.

9.6.1 Argyle Station

Hand-dug caissons of 2.2 m diameter were used at Argyle Station to form secant walls where lack of headroom prevented the use of secant Benoto piling and to form the internal columns which also carried a temporary traffic deck. Dewatering in the hand-dug caissons caused building settlements, which reached 25 mm/month and in one instance exceeded 100 mm. Work ceased, and eventually all hand-dug caissons were pregrouted, with a single line curtain from 2 m below ground to bedrock found to be adequate. Hole spacing was about 1.0 m circumferentially around a 4 m-diameter circle encompassing the caisson (Morton and Leonard[22]).

9.6.2 Causeway Bay Station – Island Line

Hand-dug caissons provided an effective means of excavating through 5–10 m of heavily water-bearing ground consisting of marine clays and completely decomposed granite and some 10 m into the underlying bedrock. The HDC provided working space for the construction of cast-*in-situ* reinforced columns

Figure 9.3 Contiguous HDC wall

effectively socketed into rock to support the interim load conditions imposed by top-down construction and the intended simultaneous construction of the two multi-storey buildings superimposed over the underground railway station. The socketed column foundations were subsequently supplemented by a base slab cast on the granite floor of the excavation and designed to achieve effective transfer of load between the slab and column.

The columns were very heavily reinforced with High Yield Steel reinforcement coupled with mechanical connectors. Shear details and bar couplers were accurately installed to form slab and beam connections at each successive floor level.

To protect the columns during excavation in the softer overlying materials and during blasting operations in the rock, the annular space surrounding the columns was backfilled with sand after completion of column construction at each level, with the added benefit of improving safety conditions in the shaft.

9.6.3 Choi Hung and Diamond Hill Stations

For these underground stations, built in the first phase of the Hong Kong Mass Transit Railway, hand-dug caissons were used to construct the permanent walls of the station box and to form shafts within which the internal columns were constructed. This construction technique was selected as being capable of dealing with substantial boulders which were present on the site and to permit top-down construction of the stations (Benjamin *et al.* [23]).

At Choi Hung Station 1.5 m diameter shafts were sunk at 2.8 m centres to a depth of embedment below track slab level and these were then reinforced and concreted. The depth of embedment of these king piles was chosen to avoid the need for any temporary strutting during the subsequent excavation. The intervening infill shafts between the king piles were then excavated to a cut-off level below the final excavation level to permit the satisfactory dewatering of the excavation; these were also reinforced and concreted (Figures 9.4 and 9.5).

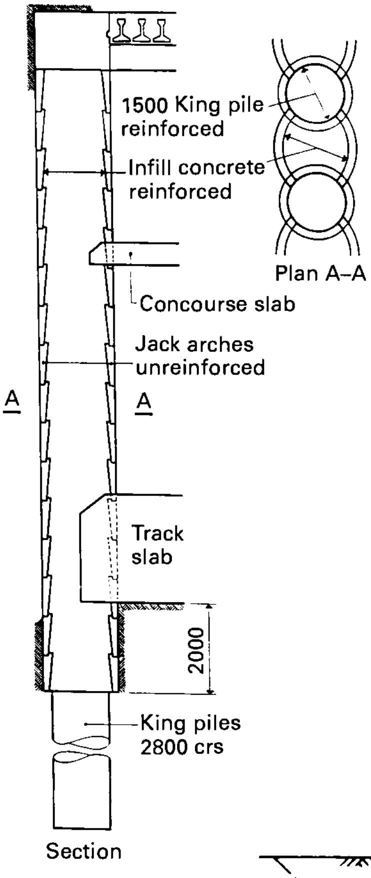

Figure 9.4 Choi Hung caisson wall arrangement

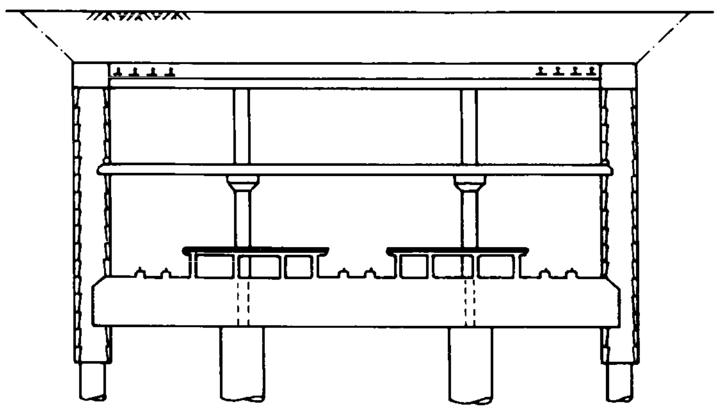

Figure 9.5 Cross section of Choi Hung Station

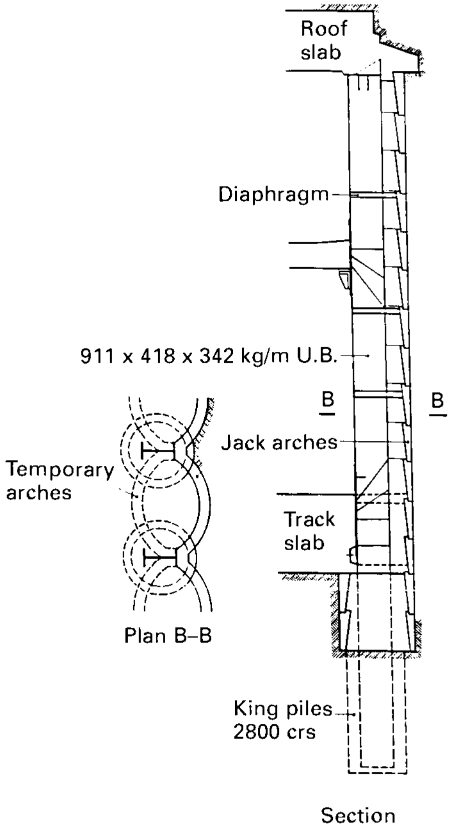

Figure 9.6 Diamond Hill caisson wall arrangement

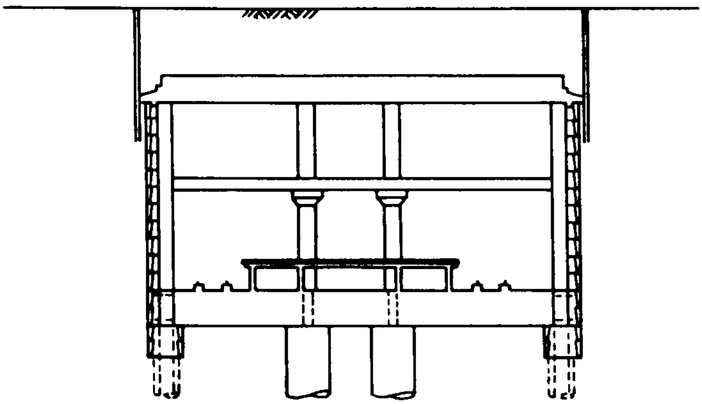

Figure 9.7 Cross section of Diamond Hill Station

At Diamond Hill Station, however, a 911 × 418 × 342 kg/m Universal Beam Grade 43A was set in each of the 1.5 m diameter shafts to form the main structural element spanning between the slabs. The soil between these king piles was subsequently excavated within infill hand-dug caissons. The outer 300 mm thick lining of these was retained to form jack arches, thus completing the perimeter wall, while the inner lining provided temporary support to the soil until completion of the main excavation. Steel king piles were used to permit the future widening of the station box, the steel beams being removed once new supports to the widened slabs would have been installed (Figures 9.6 and 9.7).

The station columns were constructed within 2 m diameter caissons (Figure 9.7). The initial intention was to precast these in reinforced concrete or to use circular cast-steel columns which had been specially developed in Japan for carrying heavy loading. The final decision on economic and practical grounds favoured *in-situ* construction, and it was found possible to erect the necessary shuttering for 1300 × 1900 columns inside the 2 m bore caissons, the reinforcing cage generally being fabricated at ground level and lowered into position in one unit. In certain locations where ground-level obstructions precluded this, the reinforcement was fixed *in situ*.

Figure 9.8 Caisson wall exposed in completed station structure. The HDC linings have been removed

9.7 Design of caissons

Where piles are founded on bedrock, allowable bearing pressures on the rock can be met by using appropriate bellouts in the hand-dug caisson. The concrete shaft strength then dominates the design and rarely does the strength of the rock become critical. Since load tests on caissons are very difficult and expensive to carry out, a proper site investigation is essential for such foundations with borings penetrating at least 5 m into bedrock from the base of the caisson.

There is very little information available on load testing of single caissons. The scarcity of caisson test results is understandable, as caissons have high load capacity and testing at high loads is an expensive undertaking. It is routine for highly loaded caissons to be founded on bedrock which can be rigorously inspected, and usually the only tests required are to verify the structural integrity of the caissons (commonly, core testing and, more recently, sonic testing of the shaft concrete).

For hand-dug caissons founded in dense soils the question of allowable skin friction and bearing pressure becomes more important. Even with the greatest care exercised in sinking shafts through dense soils, the soil removed will be in excess of the theoretical shaft volume because of heave at the base of the excavated hole following removal of the overburden pressure. The phenomenon is more pronounced in wet conditions, and this is evidenced by comparisons of SPT-N values taken before and during excavation.

The effect of heave is to cause horizontal and vertical soil movement which has been found to cause loss of soil strength as much as 10 m or more away from the hole. Bores taken through completed caissons have shown softening of the underlying soil to depths of up to 1 m immediately under the shaft concrete as a result of heave.

Work by Evans *et al.* [24] confirmed softening of the material at the base of caissons during construction and subsequent recovery of strength with time beneath unloaded caissons, similar to that observed at Choi Hung and Diamond Hill Stations. The design techniques which are outlined below take this into consideration.

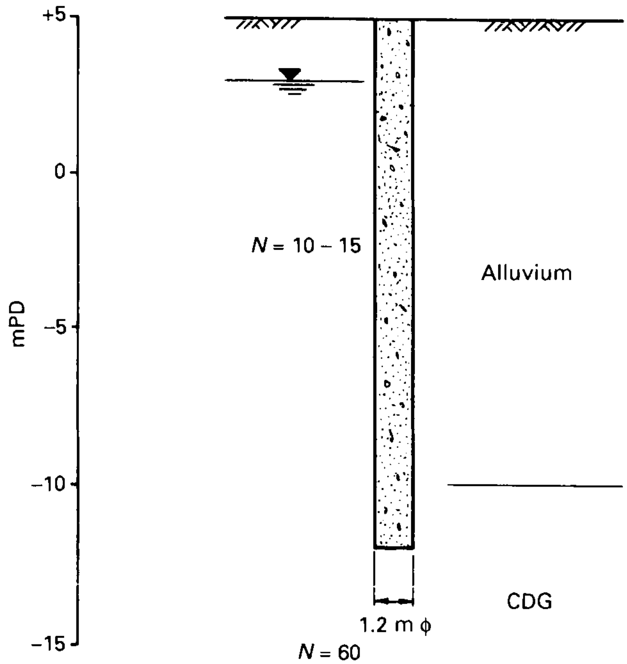

Figure 9.9 Soil conditions for pile tested at Shatin Tau

Some of the earliest load testing on hand-dug caissons not founded on rock in Hong Kong was reported in Jagger[25] and Fraser and Lai[26]. The four caissons reported by Fraser and Lai were all installed in Kowloon and serve as foundations for highway structures. These caissons were not founded on sound rock and derived a substantial portion of their resistance from shaft friction. Settlements of less than 6 mm were recorded under a test load equal to the working load, and the integral equation method (Poulos[27]) gave an acceptable estimate of settlements up to working load.

The integral equation method of settlement prediction can account for shaft slip, and this technique was used to make a prediction of load-settlement performance for the Hong Kong Housing Authority for pile P45 at Shatin Tau prior to the test loading being carried out (Fraser[28]; Evans *et al.*[24]). The soil conditions, prediction and the test results are shown in Figures 9.9 and 9.10.

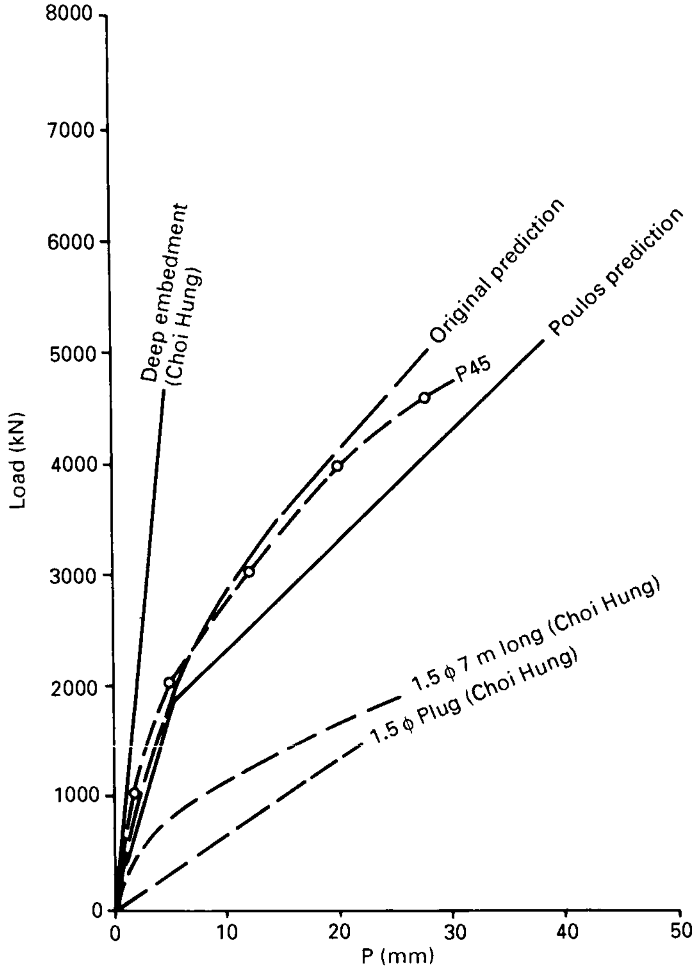

Figure 9.10 Overall load settlement curve for pile P45 (Shatin Tau) predicted and test results

It is usually assumed that skin friction consists of adhesion, which is independent of the normal stress acting on the pile, and friction, which is a proportion of the normal stress, i.e.

$$f_s = C_a + \sigma_h' \tan \delta \tag{9.1}$$

where C_a is the adhesion and σ_h' is the horizontal effective stress. The coefficient of friction ($\tan \delta$) between the soil and the shaft is usually taken as $\tan \phi'$, the coefficient of friction of the disturbed soil in terms of effective stresses. C_a is usually neglected in granular soils such as decomposed granite.

The normal stress on the shaft (σ'_h) is conventionally related to the effective vertical stress at the corresponding level prior to placement of the pile by a coefficient of skin pressure (K_s), defined as σ'_h/σ_v' so that equation (9.1) can be rewritten as

$$f_s = K_s \tan \phi' \sigma_v' = F'_{\sigma_v} \tag{9.2}$$

The value of F (i.e. $K_s \tan \phi'$) is highly dependent on pile construction methods. Empirical correlations for hand-dug caissons were suggested by Fraser[28].

Classical design methods generally assume the ultimate static capacity of piles (Q_u) to be represented in the form:

$$Q_u = q_o A_b + f_s A_s \tag{9.3}$$

where q_o is the ultimate base resistance,
f_s is the ultimate shaft friction,
A_s is the area of the pile shaft, and
A_b is the area of the pile base.

These equations ignore the actual distribution of load or rate of mobilization of friction down a pile. Although some design methods consider the distribution of load or rate of mobilization of skin friction, it is generally assumed that if Q_u can be calculated, an allowable working load of half this value is acceptable, i.e.

$$Q_a = Q_u/2 = (q_o A_b + f_s A_s)/2) \tag{9.4}$$

From testing of hand-dug caissons in Hong Kong (and elsewhere) the above type of analysis has been shown to be inadequate, as in many instances little or no base resistance is mobilized at loads of up to twice or even three times working load.

Base resistance is generally mobilized at greater displacements than skin friction, and many caissons in Hong Kong carry their working load by skin friction only. For caissons, a great proportion of ultimate skin friction is mobilized at displacements of below 10–15 mm or approximately 1–1.5% of diameter, whereas ultimate base resistance is mobilized at movements of the order of 5–10% of diameter. A more reasonable equation for a working load design approach is:

$$Q_a = q_o A_b/3.0 + f_s A_s/1.2 \tag{9.5}$$

In this formula, the base resistance mobilized represents a large component of the factor of safety, as reliance is placed on skin friction to carry loads up to the allowable load.

Methods of load-displacement analysis, including load transfer, present techniques of analysis which allow for movement-related resistance. Curves of rate of mobilization of skin friction from load tests may be used to estimate load transfer. A relationship between skin friction and shaft displacement is assumed and the pile to be analysed divided into a number of segments. Displacement of the toe of the pile is assumed and the point resistance caused by this movement calculated. The displacement of the midpoint of the lowest segment is then assumed, allowing a value of skin friction to be estimated from the design load transfer or *t–z* curve. The compression of the segment may then be calculated and the displacement at the midpoint compared with the assumed value. The procedure for the lowest segment is repeated until convergence is achieved. The next lowest segment is then considered and so on, until a value of load and displacement at the top of the pile is obtained. The method is tedious when carried out by hand (especially where the load transfer relationship varies with depth), but is relatively simple to program for computer application.

Since testing of caissons has rarely resulted in failure, ultimate skin friction and end-bearing values are unknown. Various methods of predicting ultimate load from tests not carried out to failure exist, and the results of such analyses may be used to make a reasonable estimate of ultimate loads, although it is difficult to distinguish between shaft friction and end bearing, as most of the applied loads are taken by skin friction.

Specific testing to derive load-displacement relationships has been carried out in Hong Kong by Sweeney and Ho[8] and Sayer and Leung[29], where a concrete caisson ring has been isolated from the remaining caissons rings and jacked through the soil using the other rings for reaction. Similarly, testing of end bearing of base resistance of hand-dug caissons has been carried out utilizing concrete bearing pads cast at the base of the excavation and a steel shaft to transmit load from a jack reacted by kentledge or ground anchors at the surface (Sayer and Leung[29]; Benjamin *et al.*[23]).

From the results of this limited testing, tentative design curves to estimate mobilized skin friction (*t*) and base resistance for *t–z* curve analysis of hand-dug caissons in decomposed granite soils are given in Figure 9.11. Figure 9.12 summarizes a number of the end-bearing test results.

9.8 Grouting

Fundamental to the successful use of hand-dug caissons is the use of grouting in particular situations (see also Chapter 4). The construction of the Mass Transit Railway in Hong Kong resulted in the extensive use of grouting, primarily to allow free-air tunnelling in soft ground, but also to facilitate the use of hand-dug caissons as ground support. The transported derivatives of the decomposed granite – alluvium and colluvium – generally pose most problems in caisson construction. The important requirements for a grout under these conditions are a viscosity close to that of water and a distinct and predictable gel time.

Some true solution grouts, such as acrylamides, approach this ideal. However, the high cost and toxicity associated with most of these grouts restricts their use, although they have been used for grouting of caissons in Chicago (Baker *et al.*[2]; Gale and Baker[6]).

Colloidal grouts, such as those based on sodium silicate, are not ideal, as their viscosity increases steadily before gelation (Baker[30]). Some very small particles are also present, unless these are removed by centrifuge. Nevertheless, the low toxicity and relatively low cost of silicate grouts makes them the most widely used class of chemical grouts. The choice of the gel reagent and the proportion in which

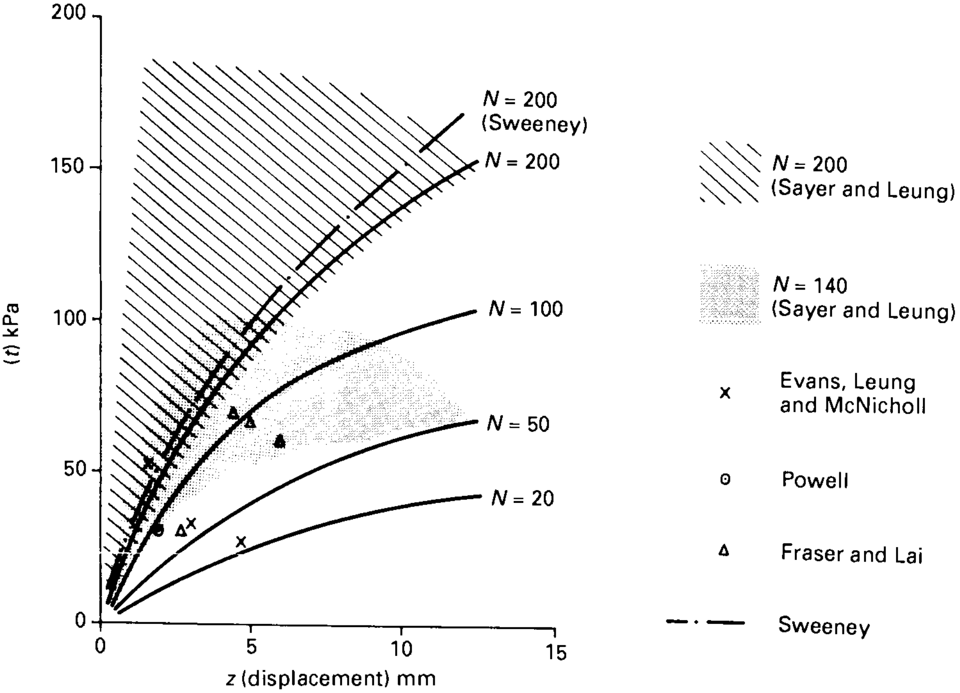

Figure 9.11 Tentative t–z design curves

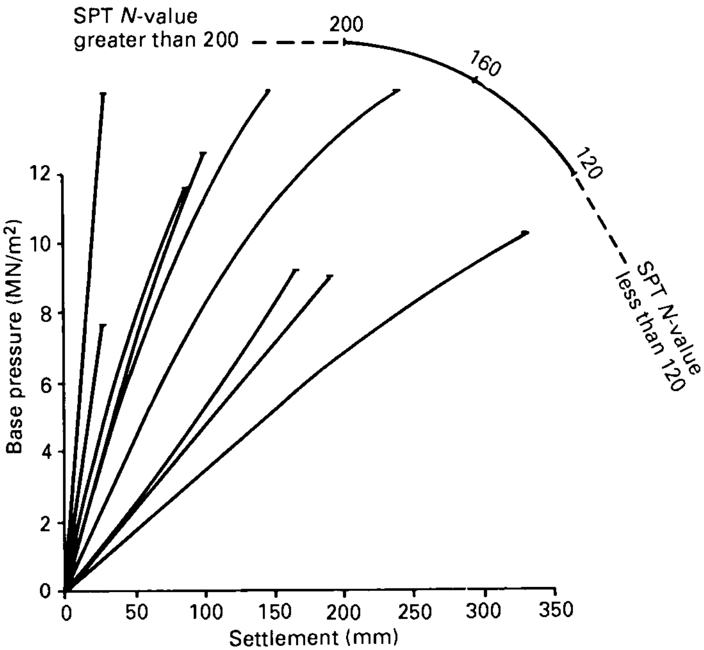

Figure 9.12 End-bearing test results (after Sayer and Leung[29])

it is mixed with the sodium silicate have a significant effect on the viscosity evolution and the strength and permanence of the resulting grout.

One gel reagent often used with sodium silicate is cement, usually gelling the silicate in 30 s to 2 min. However, when used in sand this grout suffers the same problems as pure cement grouts, in that the cement particles are filtered out and any silicate which permeates does so without the required gel reagent, resulting in either no gelation or a weak unstable gel. This method is usually called LW grouting. The material appears to behave largely as a quick-setting cement grout. It generally fissures the ground, but in smaller, more frequent, fissures than cement/bentonite. Other more common reagents are organic diacid esters.

A high proportion of the grouting work for caissons follows the procedures developed during trial tunnelling for the Hong Kong Mass Transit Railway (Morton and Leonard [22]). 'Tubes-à-manchette' installed from ground surface were used to inject a two-phase sequence of cement/bentonite followed by silicate/reagent grouts, with the initial quantities as shown in Table 9.2. Where grouting pressures during the injection of this quantity are low (typically below 5 bar at 20 m depth), then the low pressure sleeves are re-injected. Grouting from the base of a caisson is sometimes employed (Benjamin *et al.* [23]).

Table 9.2

Soil type	*Cement/bentonite*[a]	*Silicate/reagent*[a]
Marine deposit	10	35
Alluvium/colluvium	10	40
Completely decomposed granite	5	25

[a] Expressed as a percentage of the volume of the ground to be treated.

Assessment of the effectiveness of grouting for hand-dug caissons is difficult, because in many situations, construction may well have been successful without grouting. One means of assessment is the consideration of the reduction in soil permeability, as described by Morton and Leonard [22].

Tunnelling for the Island Line encountered little trouble with soils that could not be effectively treated, but there were problems with treated caissons in coarse sands. At various points along the Island Line some 15 caissons (out of several hundred constructed) encountered the problem of blow-in in coarse sand (usually beach or alluvial deposits). It was first thought that the problem was one of an inappropriate grout choice (LW grout) or the effects of using only single-line grouting in a layered soil, but these caissons were successfully recovered using LW grout. For instance, at Causeway Bay East concourse a Nitto organic reagent was used and one corner caisson of the many constructed on site collapsed. It was recovered with LW grouting in the original 'tubes-à-manchettes'.

However, the construction of 22 hand-dug caissons for a shaft in Sheung Wan demonstrated a fundamental problem in the choice of grouts. On average, the alluvial layer was injected with nearly 25% by volume cement/bentonite grout and 50% by volume of low-viscosity silicate grout. After treatment was complete, a pumping test indicated a post-treatment permeability of about 10^{-6} m/s, considered to be an acceptable figure. It was noted, however, that the water being pumped out of the centre of the treated ring was discoloured and appeared to have been contaminated with the silicate grout.

When the caissons were excavated, major inflows of soil and water were experienced in the alluvial layer in several of these. To recover these caissons, re-injection was carried out using LW grout through the 'tubes-à-manchettes'. Refusal pressures of 30–50 bar were obtained before excavation recommenced. Nevertheless, some of the caissons again experienced major inflows of soil and water. It was concluded that the LW grout had been permeating into the loosened soil, but with its short gel time only limited permeation was possible. The remaining caissons were therefore injected with neat sodium silicate followed by neat cement grout. The sodium silicate alone could permeate the ground and the subsequent cement injection then caused it to gel.

Subsequent laboratory testing indicated that not only does a grouted coarse sand have a lower strength than a fine sand grouted with the same formulation but, when subjected to flow, the grouted coarse sand rapidly loses strength, whereas the grouted fine sand maintains its original strength for a longer period. It was concluded that coarse sands required the use of high-strength silicate grouts. As a general rule, the strength of silicate grouts increases with the concentration of silicate, although care must be taken in the volume of reagent used to obtain both an appropriate gel time and a suitably high neutralization of the silicate (Shirlaw [31]).

This conclusion was tested but not proven when boreholes at two caissons in the Shau Ki Wan area revealed a coarse sand layer. After inspecting the borehole records it was decided to use a high-concentration (50% silicate by volume) silicate grout in the coarse sand layer. A sequence of cement/bentonite and LW grouting was carried out in the more silty marine deposits. The grouts were injected through a single line of holes round each caisson and a single tube in the centre of each. The first caisson was constructed without difficulty, but in the second a blow-in occurred just as the excavation reached the marine/coarse sand interface. It appeared that the coarse sand had been higher than expected on one side of the caisson, and this section had not been effectively grouted as only 0.5 m overlap had been provided for the high-concentration silicate grout. Once the caisson had been recovered using LW grout in the loosened coarse sand, the rest of the excavation proceeded smoothly (Shirlaw [31]).

The grouting procedures described by Baker *et al.* [2] also indicate that grouts, techniques and procedures do need to be carefully considered and modified for each situation. A major advantage of the 'tube-à-manchette' method is the ability to regrout when required.

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Chapter 10

Large reinforced concrete caissons

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Contents

10.1 General description of construction technique

A well-known technique for constructions which must be built beneath the (ground)water level in soft ground is the pneumatic caisson method (Figure 10.1). The construction is built entirely on ground level and is lowered to its final position by excavating the underlying soil. The underside of the construction is therefore shaped as a diving bell. The incoming water is controlled by applying compressed air in this workchamber and the air pressure is balanced by the water pressure beneath the construction.

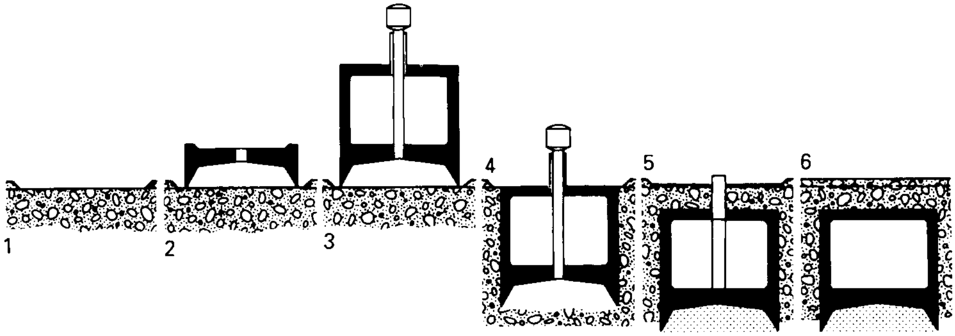


Figure 10.1 Pneumatic caisson method. 1 Levelling; 2 construction of caisson; 3 fitting equipment; 4 sinking procedure; 5 stripping equipment; 6 finishing work

The workchamber is accessible through an airlock, and the excavated material is transported through a material lock or mixed with water and pumped out through pipes. When the caisson has reached its final position the workchamber is filled with concrete.

Pneumatic caissons are used as foundation elements, quay walls, cellars, etc. In some cities railway tunnels have been constructed in this way.

A large-scale project has been the Metro Eastline in Amsterdam, which was built in the 1970s. The tunnel on this line largely consists of 80 pneumatic caissons at a total length of approximately 3 km. As the author has taken part in the design of the tunnel this chapter is based mainly on this project.

10.2 Conditions

Only in specific conditions will the pneumatic caisson method be chosen for an underground railway tunnel. The most important of these are as follows.

10.2.1 Soil conditions

1. The groundwater level is high. Lowering it is not (or not sufficiently) possible because of the high permeability of the soil or the vulnerability of the foundations of adjacent buildings.
2. A foundation level for the tunnel is available at a level according to the design depth of the railway line and the construction method.
3. The soil can be excavated easily by water jets and pumped through pipes when mixed with water.

4. The bearing capacity of the ground at the surface is such that a caisson of sufficient quality (strength and watertightness) can be constructed. This means that uneven settlements may not occur before the construction has achieved sufficient strength. The sand layer applied for the mould of the workchamber also improves the bearing capacity of the upper soil layer.

10.2.2 Building conditions

1. A sufficient and easily accessible building site for the caissons should be chosen with a width of at least 3 m on one side and 7 m on the other.
2. A central building yard should be located in the immediate vicinity of a 1000 m stretch of tunnel.
3. The tunnel must be situated at least 5 m from existing buildings to prevent settlements while building and sinking the caissons.

10.3 Design

10.3.1 Shape and dimensions

In principle, a caisson can be adapted to almost every shape of underground construction. However, forces which act upon the construction during building on a possibly unstable ground surface, during the sinking and when it is in its final position require a simple box-like shape with straight walls. This also applies to curved sections of a tunnel. A polygonal tunnel layout is then chosen. Building efficiency requires repetition of form and dimensions (see Figure 10.2).

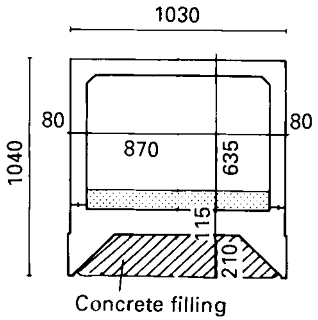


Figure 10.2 Cross section of a normal caisson for a straight, double-track running tunnel in Amsterdam

In large caissons which, for instance, may be part of an underground station it could be necessary to insert temporary or permanent stiffening walls to accommodate the forces during the building and sinking process. As the joints are complicated parts of construction the number of these should be limited. Consequently the caissons should be as long as possible. The above considerations indicate that a careful study of the location of the joints between the caissons is necessary.

In Amsterdam a standard length of approximately 40 m was chosen. For one station a caisson with a length of 70 m has been built. For another, a future crossing line required a caisson with an area of approximately 35 × 45 m and a height of 20 m. The cross section of the running tunnels is approximately 10 × 10 m including the workchamber. For stations it varies from 17 × 10 m to 20 × 10 m. If a ticket hall is situated above the tunnel the height increases to 14 m (see Figures 10.3–10.6).

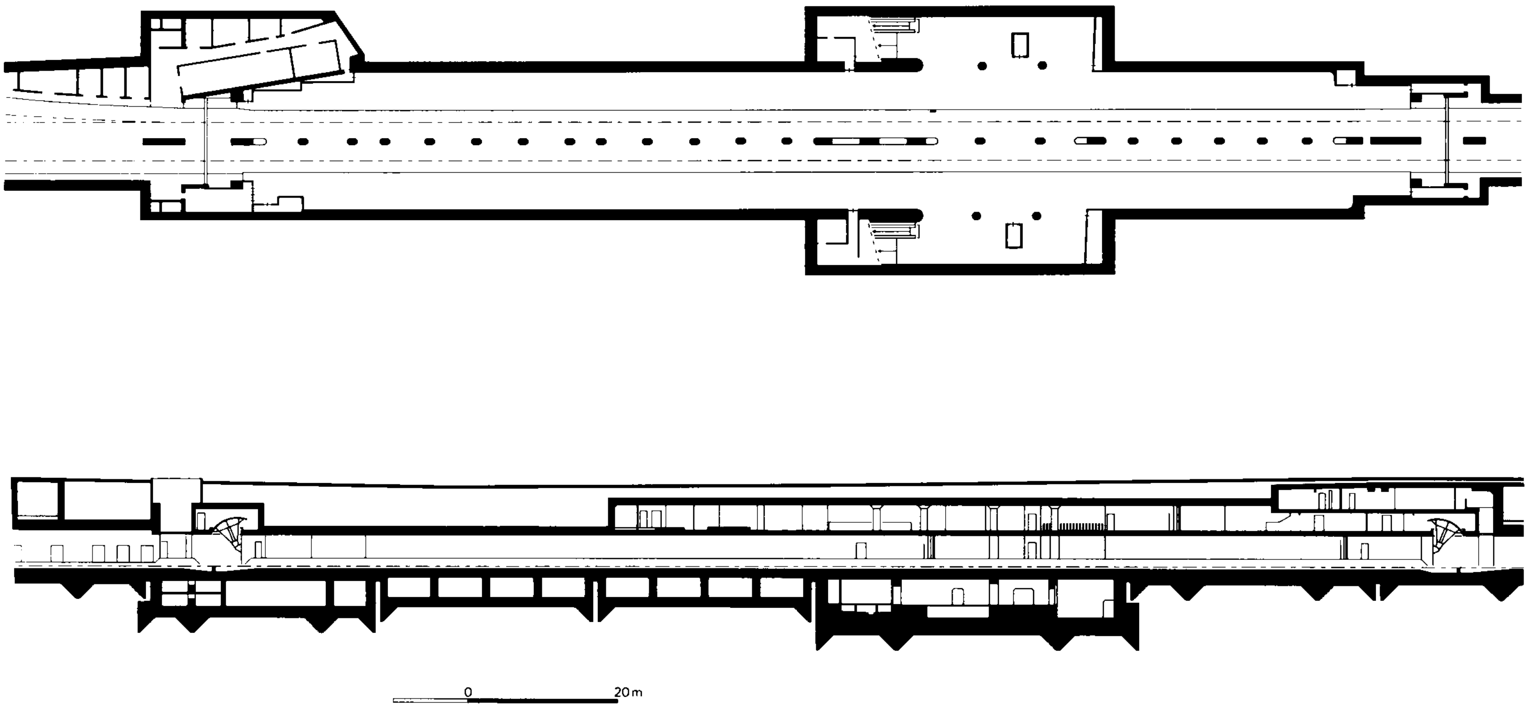


Figure 10.3 Weesperplein Station in Amsterdam: general view

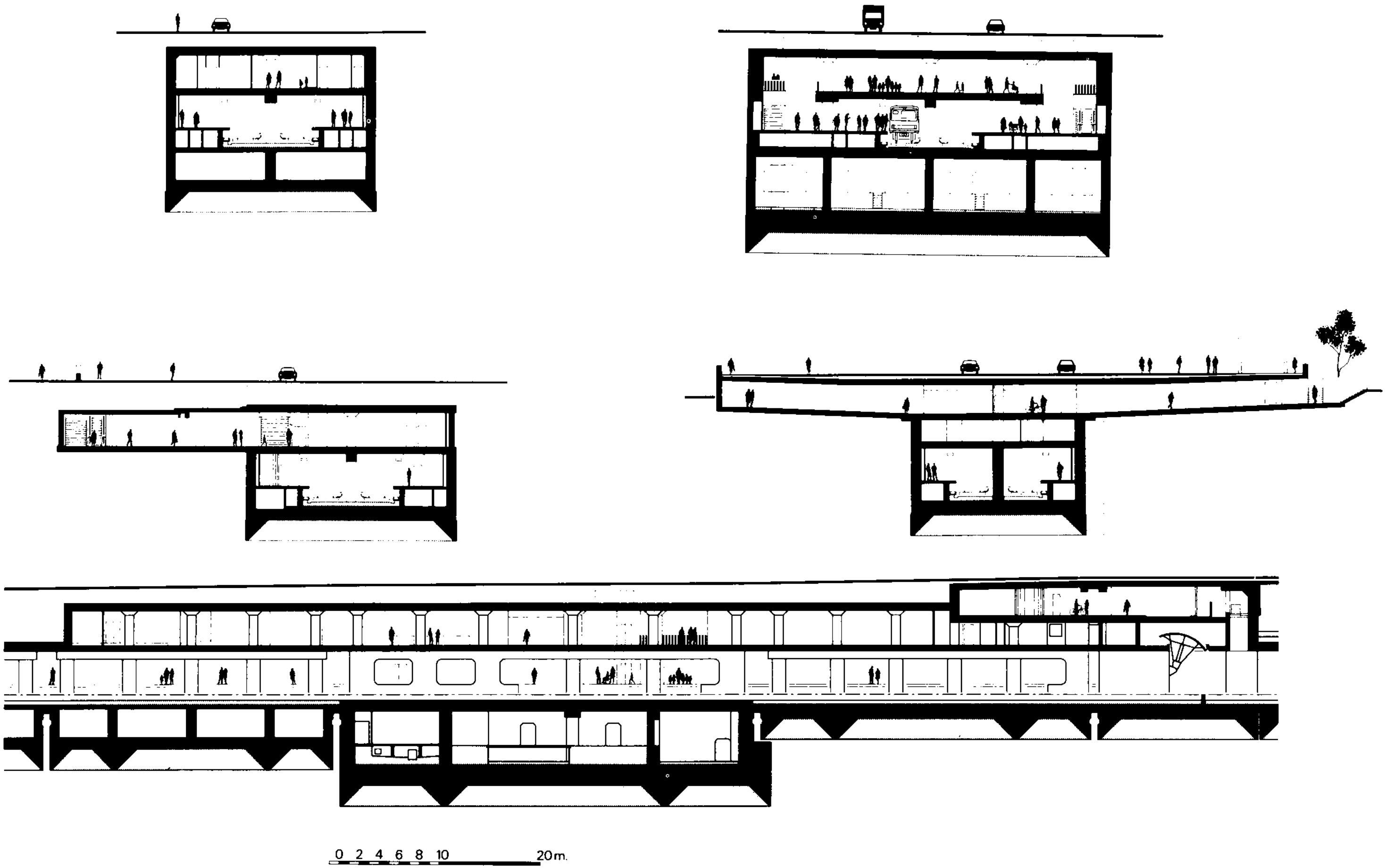


Figure 10.4 Weesperplein Station in Amsterdam: cross- and longitudinal sections

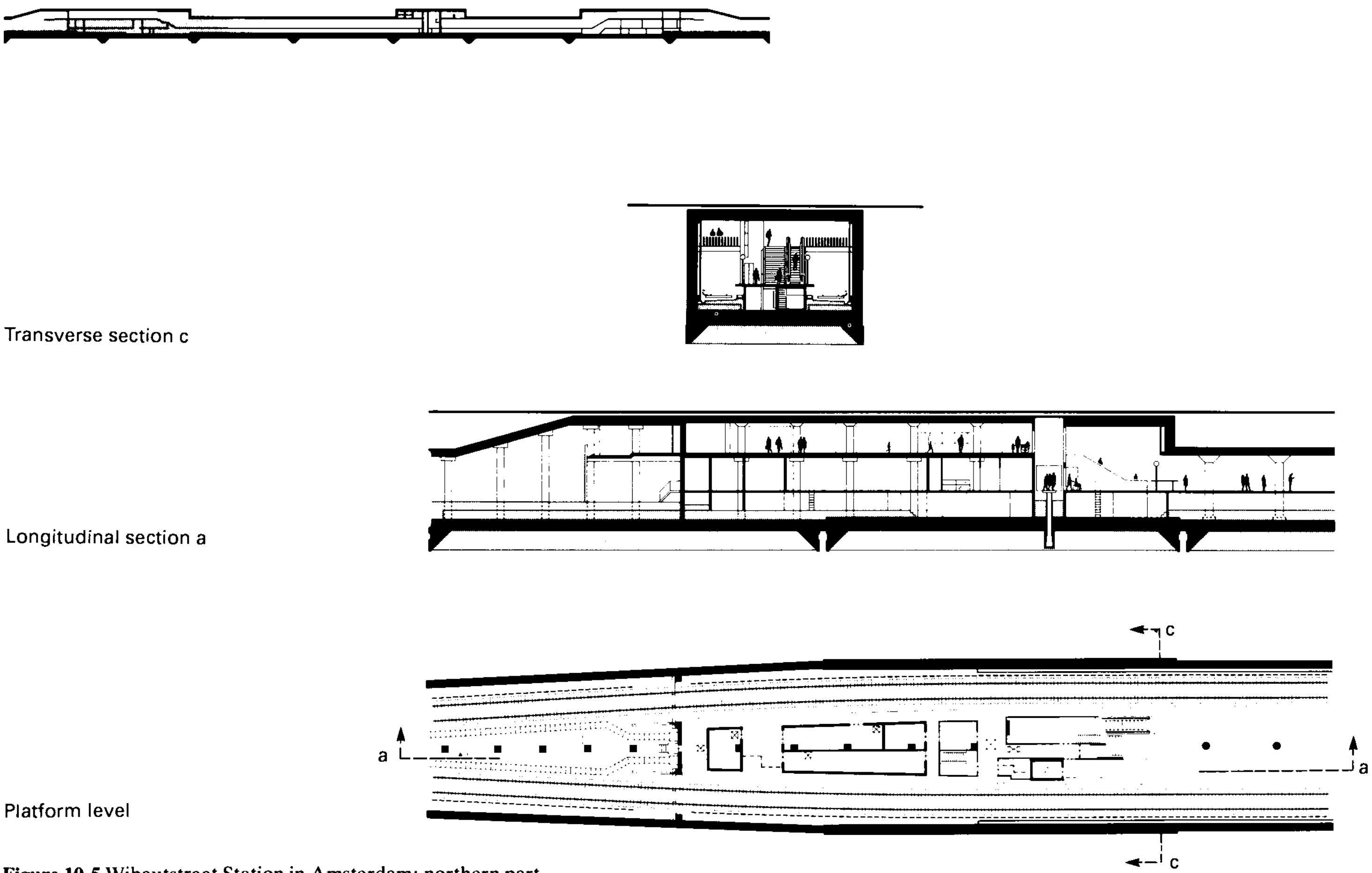
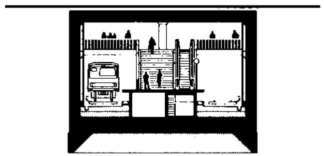
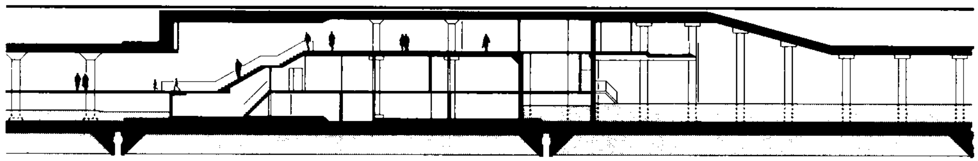


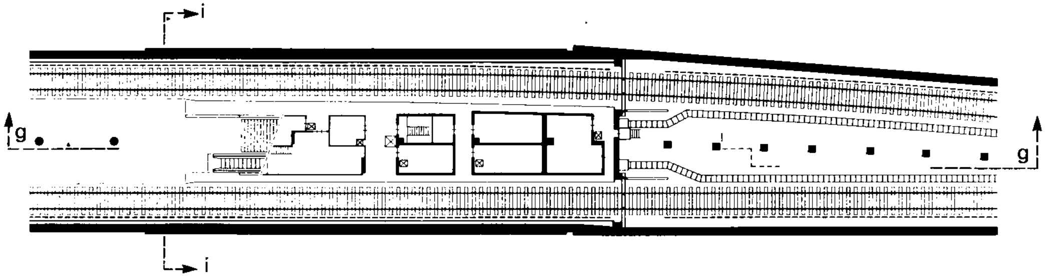
Figure 10.5 Wibautstraat Station in Amsterdam: northern part



Transverse section i



Longitudinal section g



Platform level

Figure 10.6 Wibautstraat Station in Amsterdam: southern part

The large dimensions of the caissons require special provisions for the quality of the construction mainly with respect to watertightness.

The construction dimensions of a caisson are mainly determined by providing equilibrium against uplift, as in its final position the caisson will be entirely under the (ground)water level. The weight of soil on top of the tunnel is not taken into account as it should always be possible to remove the upper load from a shallow tunnel in an urban environment. Generally, the thickness of walls and roof of a running tunnel is at least 0.80 m, the floor at least 1 m.

The workchamber is formed by the tunnel floor and the cutting edges, and after the caisson has been sunk the workchamber is filled with concrete. This concrete can be connected to the tunnel floor by means of reinforcement bars to provide additional weight against uplift. For very large caissons or in more solid soil it may be necessary to add two bearing edges in the workchamber to accommodate the forces in the construction while it is being sunk (see Figure 10.4).

Normally, the weight of the caisson is only slightly higher than the uplift. To reduce friction, the outside walls of the caissons, from 2 m above the cutting edge, have a 30 mm deep recess. As an additional measure the gap can be filled with bentonite.

Computation of caisson construction is based on the forces which act upon it in the stages of building, sinking and final position. These forces are caused by the super load on top of the caisson, ground and water pressure, the construction's own weight and, to a lesser extent, trains, installations and passengers. For distribution of the reaction forces of the ground on the cutting and the bearing edges, if these are part of the construction, a number of models are assumed (see Figure 10.7). Computations should be made for all these assumptions. In the final position after filling the workchamber with concrete the foundation force is distributed equally.

10.3.2 Tolerances

As the caisson may be built in an unstable position because of uneven settlements (caused by the increasing weight of the construction), when it is sunk to its final position various tolerances must be taken into account:

1. Building tolerances, due to the inevitable inaccuracy of building a large unit on the surface;
2. Sinking tolerances, related to the controllability of the sinking process;
3. Measurement tolerances, related to the difficulties of measuring on and in the construction at its different stages.

Clearly, deviations which inevitably occur should be kept as small as possible. In Amsterdam the values in Table 10.1 were adopted and were found to be workable.

Table 10.1

	Transverse (mm)	*Axial* (mm)	*Vertical* (mm)
Building tolerances	±50	±50	±50
Sinking tolerances	±125	±100	±50
Measurement tolerances	±25	±25	–
Total	200	175	100

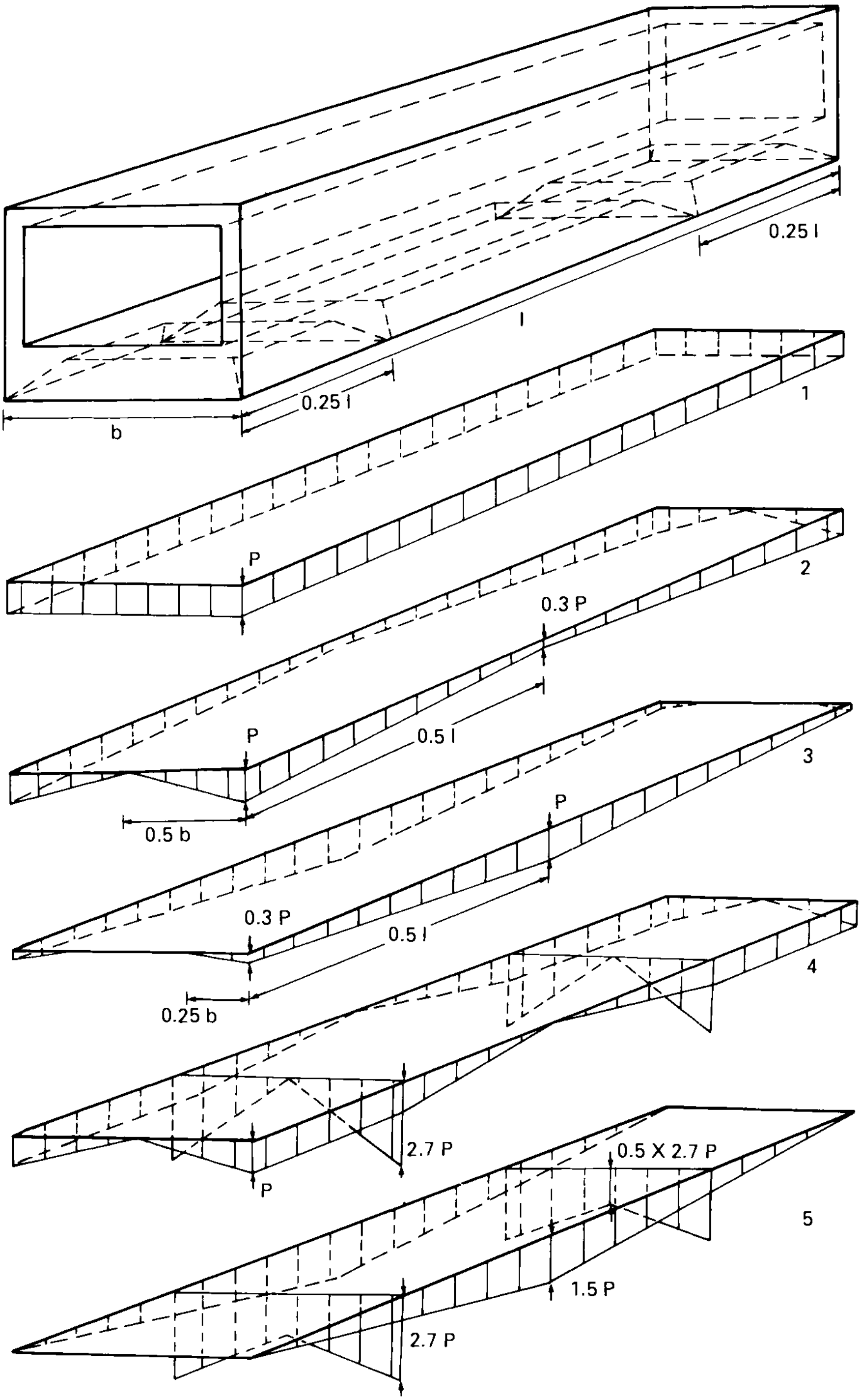


Figure 10.7 Distribution of the reaction forces of the ground on the cutting edges (1, 2, 3) and on the cutting edges and bearing edges (4, 5)

10.3.3 Joints

Joints are a very essential part of a caisson tunnel and should be designed very carefully. Starting points for the design are:

1. Simple design to minimize building risks;
2. Minimal dimensions;
3. Adaptability to all possible combinations of tolerances;
4. Watertight construction;
5. Flat surfaces of both caisson front faces while sinking;
6. Limited mutual movement of adjoining caissons (in Amsterdam this was set at 10 mm in all directions).

The design which was applied to the metro tunnel in Amsterdam (Figure 10.8) was based on the principle of lengthening the caisson construction on one side. The expansion joint is placed on the other side of the caisson. The provisions on the

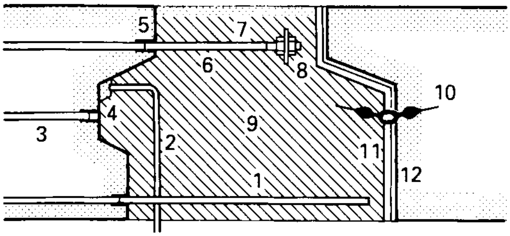


Figure 10.8 Design of connection – joint between two caissons. 1 Screw bar 14 mm Ø; 2 grouting tube; 3 reserve connection; 4 grouting duct filled with styropor; 5 screwed fitting; 6 screw bar 24 mm Ø; 7 insulation; 8 end anchorage; 9 joint concrete; 10 joint dealing with joint sheet; 11 lining with foam rubber 2 cm; 12 triplex $d = 5$ mm

caisson front faces were limited to three rows of screwed fittings connected to reinforcement bars on one side, the middle one being a spare to be used for large tolerances, and a joint seal on the other side. In addition, the front faces were shaped to accommodate the temporary bulkheads and to provide for a minimal distance between the caissons (an average of 0.65 m). The main construction of the joint consisted of reinforced concrete and was connected to one caisson by screw bars, to the other by the joint sealing. To improve watertightness of the connection at the anchoring side it can be injected. A provision for this can be built into the concrete joint. The front face of the caisson on the side of the expansion joint was lined with foam rubber.

10.3.4 Watertightness

Economic considerations rule out an external waterproof lining for a caisson. The watertightness of the tunnel has therefore to be achieved mainly by means of impervious concrete, i.e. by fissure-free concrete and a compact particle structure. The measures to be applied are:

1. Formwork without rods passing through the concrete;
2. Limiting the number of casting joints;
3. Adapting the composition of the concrete;
4. Crack-distributing reinforcement;

5. Reduction and control of setting temperature;
6. Subsequent treatment of the concrete.

Anchoring of the formwork
The outside formwork for the two opposite walls is anchored in the floor construction and connected to each other by a long rod across the roof of the tunnel.

Casting joints
In the running tunnel only one casting joint is inserted at 200 mm above the floor of the tunnel. For watertightness, a steel plate is inserted into this joint.

Composition of concrete
It is necessary to aim in particular at two properties of the concrete when deciding on the composition of the concrete mixture: compact texture of the concrete and low heat evolution during setting. After tests the concrete mixture in Amsterdam was so adjusted that with a cement content of 275 kg/m^3 (blast furnace cement with about 80% slag) the setting temperature of the concrete was reduced to the lowest possible level and the anti-corrosive effect on the steel inserts was retained. At the same time, the desired strength value of the test cubes was reached. Special attention must, of course, be paid to the graduation of particle sizes in the concrete: the proportion of very fine particles, which determine the watertightness, is adjusted by the addition of 50 kg/m^3 of glacial fine sand. In this case the reserves of strength of the concrete have been reduced in favour of a smaller evolution of heat during setting.

Crack-distributing reinforcement
Reinforcement in the vulnerable section of the caisson walls above the casting joint was determined to prevent any continuous cleavage cracks which despite all precautions may still occur as a result of exceptional stresses during the sinking of the caisson. The maximum crack width was set at 0.2 mm, and as reinforcement, a total of 0.5% of the cross section of the wall in the lower third of the wall. This reinforcement (distributed in the form of the thinnest rods) lies horizontally on the outer surface of the walls. In the upper section of the wall it is reduced to the normal figure of 0.3% of the cross section of the wall.

Reduction and control of setting temperature
Experience with building operations has shown that heating of the concrete in the walls during setting and consequent shrinkage in relation to the previously concreted floor of the tunnel during cooling are the main causes of continuous cracks in the walls and that even with wall thicknesses of less than 1 m special measures are required. Temperature changes in a 1 m thick section of wall during setting are shown in Figure 10.9. This shows that the highest temperature is reached some 36 h after setting begins. There are two ways of levelling out this peak: reduction of the initial temperature of the concrete and cooling of the concrete while it is setting.

In Amsterdam the latter method was chosen because cooling during the setting period by means of pipes embedded in the concrete involves relatively little expense and is very effective. The arrangement of the cooling plant and pipes is shown in Figure 10.10. The walls were cooled above the casting joint, in the lower third of their height. The aim was to equalize the temperature differential between

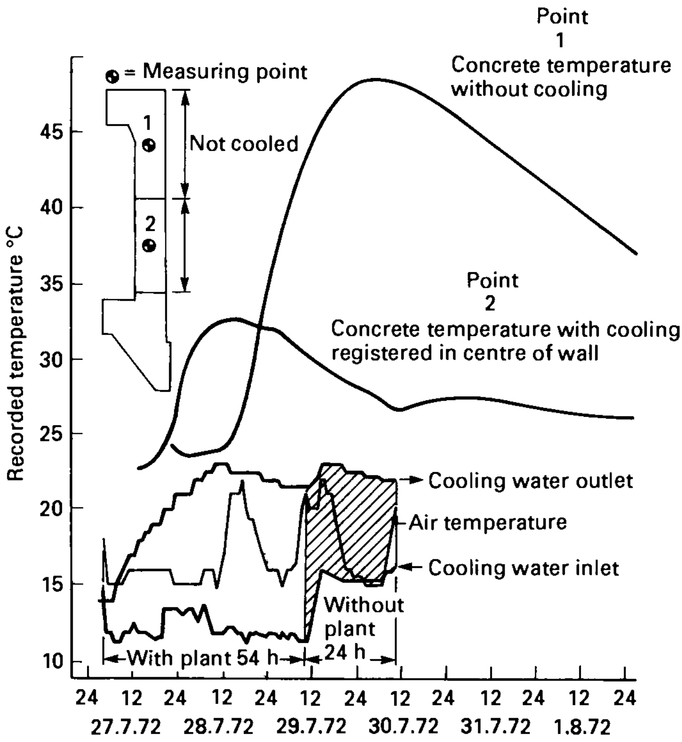


Figure 10.9 Temperature measurements in caisson wall

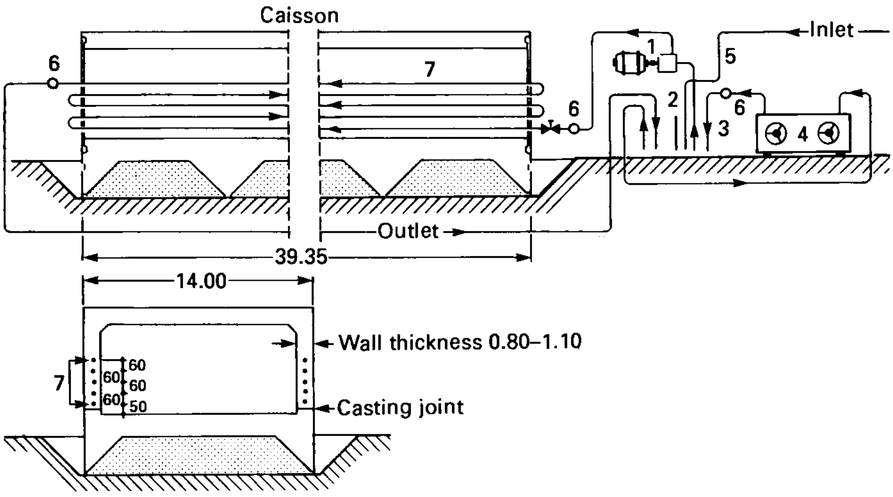


Figure 10.10 Cooling pipes in the walls of a caisson for reducing concrete temperature during setting; arrangement of cooling plant and pipes. 1 Electric pump; 2 tank partition; 3 reserve tank (1 m^3); 4 cooling plant capacity (12 600 kcal/h); 5 water supply; 6 thermometer; 7 cooling pipes (1¼ in)

the roof of the tunnel and the upper, uncooled parts of the wall, on the one hand, and the floor of the tunnel, on the other. A separate cooling system, was installed for each wall so that in the event of breakdown it was possible to switch over.

Standard 1¼in (30mm) gas pipes were used for cooling and were inserted between the reinforcing rods. The cooling water (2 m^3/h for the whole caisson) was mains water. The cooling units had a maximum capacity of 52000kJ each, were designed for continuous operation and required practically no maintenance. In summer, the water flows from the Amsterdam mains at 12–17°C; it was cooled to 7–8°C and warmed up again to between 13°C and 17°C during its passage through the cooling pipes.

With an average heating of the cooling water by 10°C, about 2900000kJ could be abstracted during a cooling period of 70h. The amount of heat arising during this time in a wall 1 m thick and 40m long is 6.7 million kJ, so that about 45% could be eliminated by cooling pipes. It can be seen from Figure 10.9 that the temperature of the concrete in the zones without cooling rose from 23°C to 48°C. In the zone equipped with cooling pipes, however, it rose only from 23°C to 33°C. The cooling effect here is 15°C, with a possible increase of 25°C, or about 60%. The cooling period is about 70h from the commencement of concreting. After this, the cement in the vicinity of the cooling pipes has given off most of its setting heat. To continue cooling any longer in the caisson walls could result in undesirably low temperatures.

Subsequent treatment of the concrete

The differential in shrinkage of the concrete components (walls and tunnel floor) can also be influenced by suitable aftertreatment. The best effect is due to the thickness of the constructional elements, which cause the structure to contract very little and also slowly. In Amsterdam the prevailing climate (moderate air temperatures and high air humidity) has played a beneficial role here.

After the floor of the tunnel has been concreted it is kept damp by spraying. This method cannot be used on the walls, as undesirable cooling may result after removal of the formwork (at the earliest, 7 days after concreting). For this reason, in summer the walls were protected after removal of the formwork against drying out with a liquid aftertreatment preparation (curing compound), while in winter the caisson was covered with tarpaulin as a protection against overcooling.

The measures described above have been successful in Amsterdam. In some cases, however, leaking cracks appeared in the caissons after the sinking. All these were dealt with by injection.

Present studies to obtain crack-free concrete are based on the application of cement with a low heat production. Also being studied is whether good results can be obtained by heating the first cast instead of cooling the adjoining part of the second.

10.4 Building and sinking

The construction stages are shown in Figure 10.11 and 10.12.

10.4.1 Preparing the site

In an urban area the route of the tunnel must be cleared by the demolition of existing buildings and rerouting the public supply mains and roads. Thereafter the

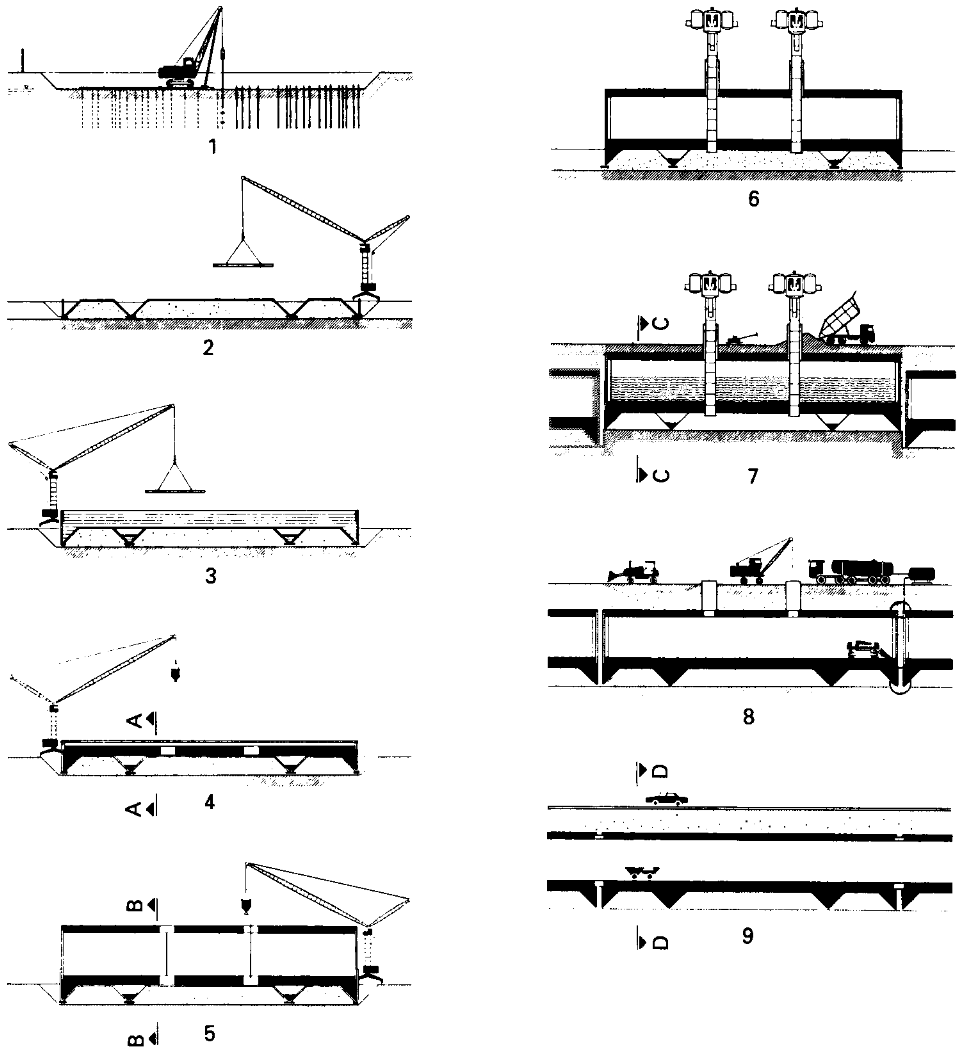


Figure 10.11 Construction stages of a caisson. 1 Extraction of piles; 2 earth mould; 3, 4 formwork and casting of first section; 5 completion of second casting section; 6 installing locks; 7 sinking caisson; 8 freezing joint; 9 completed tunnel

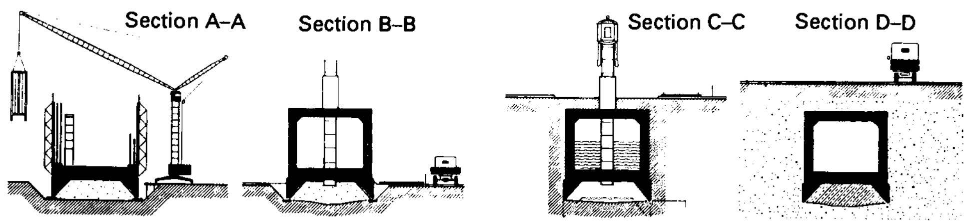


Figure 10.12 Construction stages of a caisson: cross sections A-A to D-D

soil must be removed to a depth of 2–3 m in order to eliminate old foundations, building rubble, road pavements, etc. Old foundation piles can then be pulled out if present. In general, this will be the case in soil conditions which require the caisson method for building a tunnel. Filling the pile holes with sand should be attempted. After this the trench is filled with sand to a height of 2–3 m which is then compacted. In the case of a compressible subsoil this sand layer should be allowed to settle for a few months to provide a more stable base for the construction of the caisson.

10.4.2 Manufacture of caissons

Due to manufacturing techniques, the caissons which, when sunk, are located about 65 cm apart, are not concreted in the order of their location in the tunnel but in such a way that between two units on which work is in progress there is at least one caisson which either has already been sunk or is being lowered. The actual construction work begins with the production of the earth mould for the workchamber. The cutting edges are dug out of the compacted sand and the surfaces, which are inclined at 45°, finished and profiled by hand. The inclined surfaces are covered with prefabricated sheets and the horizontal ones are given 10 cm thick reinforced subconcrete. The steel cutting edges, formed of bisected wide-flange or channel sections, are laid on prefabricated auxiliary foundations which also serve as a base for the wall formwork. These are welded together and secured in the concrete with clamps.

This, and the high-grade quality of the steel, make it possible to use 75% of the cross-sectional area of the steel as reinforcement for the constructional elements. The caisson is manufactured in two concreted sections and the casting joint is situated 20 cm above the roof of the workchamber.

The formwork must not be secured by rods passing through the concrete, to avoid leakage points and to prevent the entry of stray electric currents. For this reason, when concreting the cutting edges and the floor of the tunnel the formwork is secured, first, to auxiliary foundations at the depth of the cutting edges and again above the casting joint, by means of tension members with an adjustable centre piece.

The external formwork of the tunnel walls consists of large panels and sectional steel lattice girders fixed in the tunnel floor by anchorages and above its roof by the tension members already inserted into the first section. Formwork tables, with extendable inner wall formwork, are used for the tunnel tubes, each with four securely fixed screw spindles for assembly and dismantling which, after removal of the formwork, are removed on hydraulic trolleys. The casting joints are sealed with joint sheets. The front bulkheads for temporary and provisional closure of the front faces of the caissons are made of masonry provided with a 3 cm thick layer of waterproof plaster, or prefabricated concrete plates, resting on vertical steel girders.

10.4.3 Sinking the caissons

The caissons are sunk by a flushing system, as shown in Figure 10.13. First, the compressed air and flushing equipment is installed in the readymade caisson; this comprises two locks for entry and exit of staff and material and the sand pumps, including the necessary control equipment and pipelines. Fresh air (at least

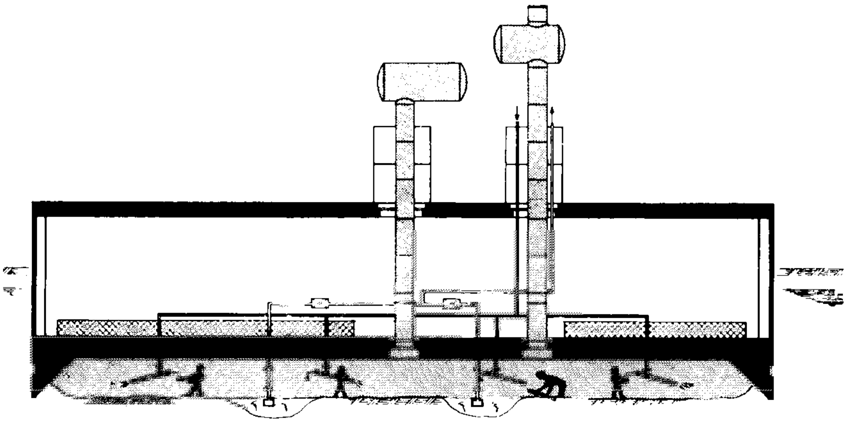


Figure 10.13 Sinking a caisson

40 m^3/h) is produced in a compressor station and fed to the caisson via expansion chambers, regulators and pipelines. The air pressure in the workchamber is adjusted to the water pressure of the groundwater table. In the event of current failure, diesel compressors can be used to produce compressed air.

The working hours of the operatives in the chamber during sinking are largely dependent on the excess pressure in the chamber. At up to 1 bar, the time taken to pass through the lock is 5 min, after which it increases rapidly: e.g. at 1.3 bar to 20 min, at 2.0 bar to 80 min. Only up to 1.3 bar may the compressed air working period be 8 h. A 30-min break is taken outside the caisson. Application of remote-controlled waterjets can diminish the number of operatives working under compressed air.

In Amsterdam flushing water for soil removal was taken from one of the neighbouring canals. The mixture of water and soil resulting from flushing in the workchamber was conveyed by pumps through pipelines to an intermediate pumping station less than 300 m away, and from there 2200 m to a flushing area in a harbour basin, where the bottom is regularly dredged. From there the soil was removed in barges. This method of carrying soil in pipelines through the city avoided inconvenience to inhabitants and disturbance to traffic. When it has sunk the caisson to its assigned level the workchamber is flushed free,

After bending the connection reinforcement for suspension of the filling concrete into position the workchamber is filled with concrete by pumps. The day after pouring air pressure can be free and dismantling of sinking equipment can proceed.

10.4.4 Construction of joints

The caissons lie at their assigned level, their faces being from 30 cm to 100 cm apart, 65 cm on average. With the freezing method, as shown in Figure 10.14, the soil in and around the joints is frozen by liquid nitrogen (boiling point 196°C). For this, a system of double-walled steel pipes is partly sunk together with the caisson (horizontal pipes), partly built in after sinking boreholes beside the joint (vertical pipes).

The soil freezes within a few days and forms a watertight joint for the adjacent caissons, progress of the freezing being traced by temperature measurements.

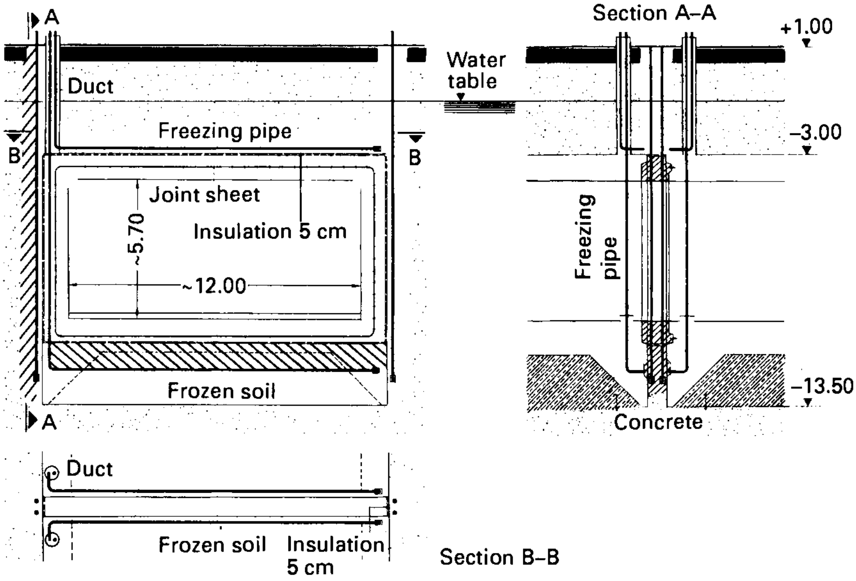


Figure 10.14 Closing of joint with frozen soil: arrangement of freezing pipes at the front faces of caissons

After the frozen soil has reached a sufficient thickness, demolition of the temporary bulkheads begins. To maintain the frozen volume of soil only about a third of the initially required nitrogen is needed. After excavation of the soil in the space between the caissons and removal of frozen soil entering the joint (the soil now having the strength of concrete), anchorages, reinforcement and an elastic joint

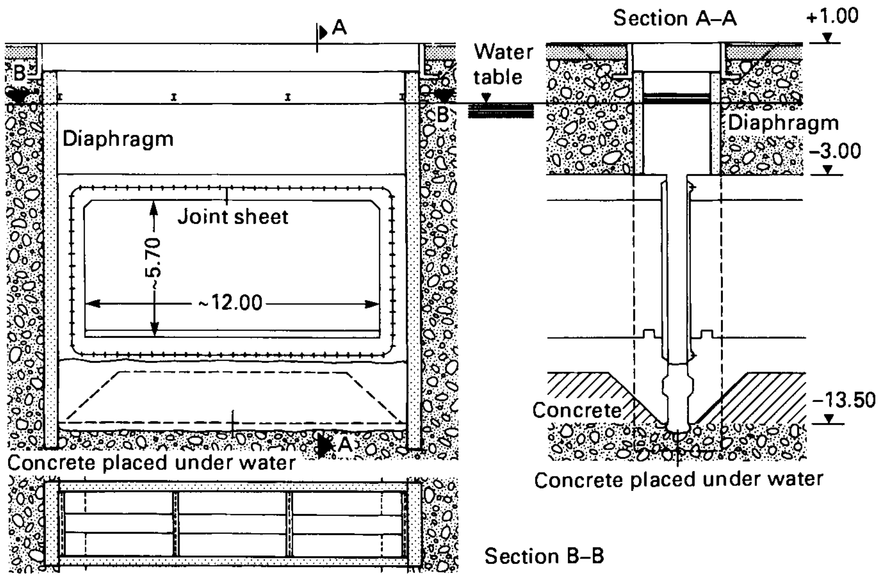


Figure 10.15 Closing of joint in open pit: enclosure of working pit with diaphragms and watertight floor

filling are installed. The joint is concreted in two stages: floor and walls with the top slab. An insulation is provided against the frozen soil.

As an alternative to the freezing method, closing of joints by means of concrete diaphragms (see Chapter 7) was practised, as shown in Figure 10.15. Diaphragms are sunk in the joint area closing tightly to the sides and the roof of the caissons. After excavation and sealing of the floor with concrete placed under water within the pit enclosure the pit is pumped dry and the actual closing of the joint is carried out by conventional methods in an open pit.

10.4.5 Finishing the construction

After the tunnel has been built work can be carried out for finishing the construction of the rail tunnel. These include profiling the tunnel floor, installing the permanent way and signalling equipment, etc. For stations, entrances must be built, as well as ventilation shafts and all the other amenities and equipment that transforms a rough concrete tunnel into a space suitable for railway users. Above the tunnel the surface can be landscaped.

Further reading

Cement, **XXV**, No. 8 (1973)
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Chapter 11

Immersed-tube tunnels

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Contents

11.1 Principles of construction

11.1.1 Selection

The choice of a tunnelling method to construct a railway under open water will depend on two major factors: first, the depth of the railway on either side of and, second, the nature of the subsoil conditions under the stretch of water to be crossed. Where the depth of the railway is to be kept relatively shallow and the subsoil conditions require extensive precautions to prevent the ingress of water into conventionally driven tunnels, then immersed-tube tunnelling techniques are applicable.

11.1.2 Outline of the technique

Definition
An immersed-tube tunnel is one in which prefabricated lengths of tunnel structure (termed 'units') are sunk into a dredged trench in the bed of the open water crossing.

Tunnel units
The units are typically constructed of either reinforced/prestressed concrete or a steel shell lined and surrounded by concrete. They are fabricated in lengths normally up to 120 m, although much longer units have been used. They may be fabricated at a site immediately adjacent to the tunnel site or, in the case of some American tunnels, towed from shipyards up to 2000 km away.

For transport to the tunnel site, the units are made watertight by closing off their open ends with temporary steel or concrete diaphragms, termed 'bulkheads'. The units can then float in water with a positive freeboard; this condition is termed 'positive buoyancy'. In this condition, the units can be towed to the tunnel site from wherever they have been fabricated. Figure 11.1 shows units for the Eastern Harbour Crossing, Hong Kong, awaiting towing out from the casting basin.

Sinking
At the tunnel site, pontoons are attached to the top of the unit and water is pumped into tanks inside the unit as ballast. The ballast is sufficient to convert the previous positive buoyancy into a negative one sufficient for the unit to sink. The downward motion of the unit in this condition is controlled by winches on the pontoons (Figure 11.2).

Foundation
The buoyant weight of the units is very low. For this reason, the foundation of the tunnel is relatively simple and typically consists only of a layer of gravel or sand about 0.8–1.0 m thick placed directly onto the as-dredged base of the trench. Piled foundations are occasionally necessary.

Jointing
The units are joined underwater either by surrounding the joint with tremie concrete or, more commonly in recent times, by use of a flexible joint incorporating a natural rubber gasket.

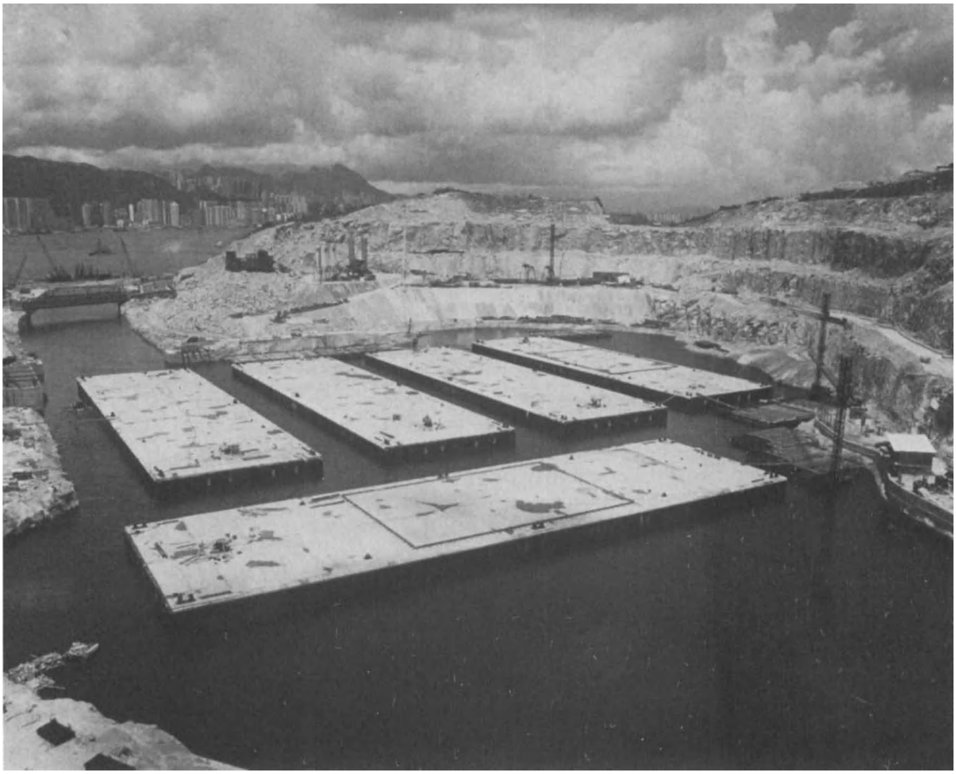


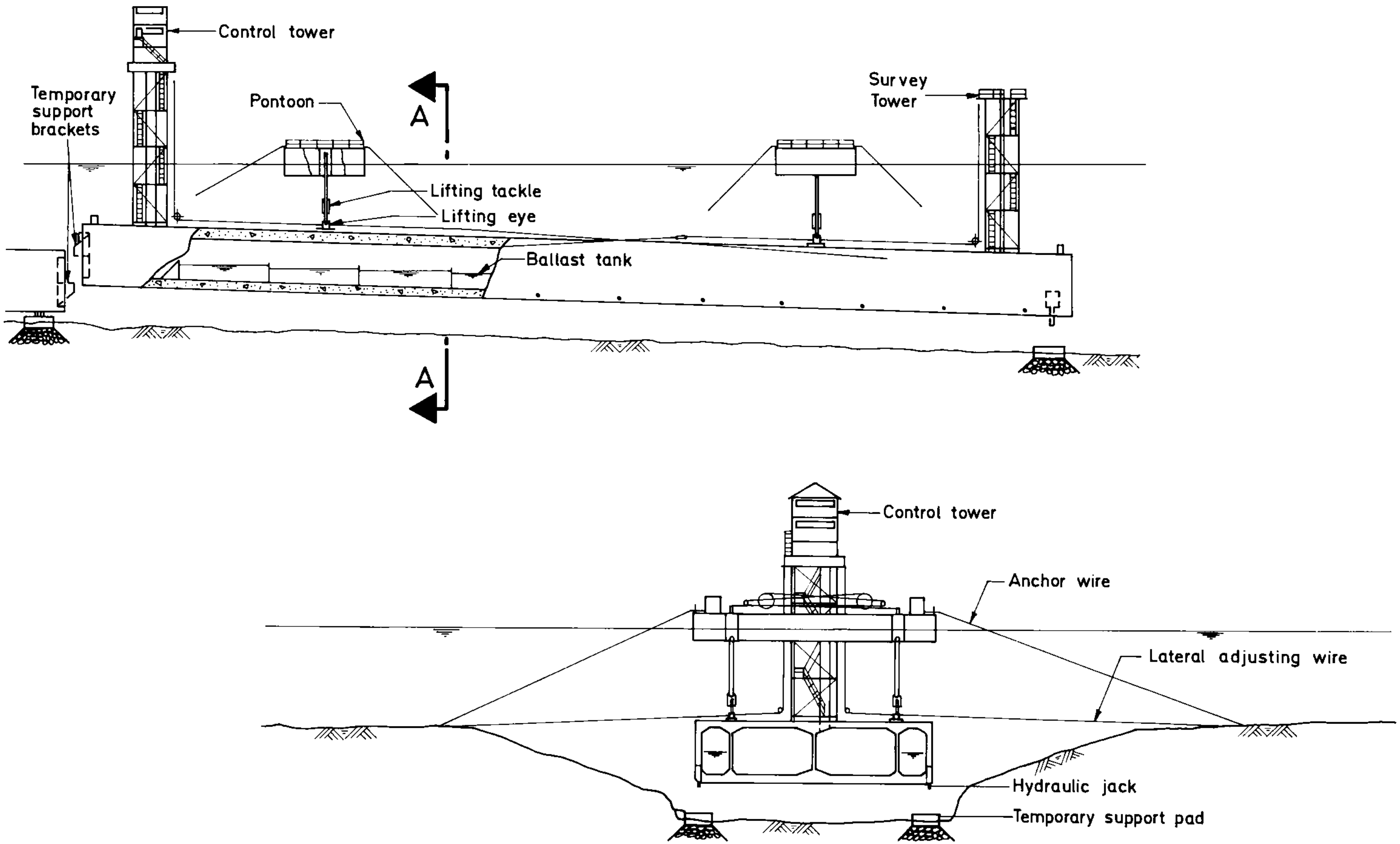
Figure 11.1 Tunnel units ready for towing (Eastern Harbour Crossing)

Backfilling
After placing the units, the trench is backfilled; at least 2.0–2.5 m of fill is placed above the tunnel unit to protect it against marine hazards such as stranding ships or falling or dragging anchors. The tunnel vertical alignment is normally chosen so that the tunnel and its backfill remains below existing or future bed level, although in areas of local deepening it may be necessary for the tunnel to project (Figure 11.3).

Finishing
When the joints have been made watertight the bulkheads can be removed and the joint completed from the inside. The water ballast used for sinking is removed and replaced with permanent concrete ballast; this may be placed externally but is more commonly placed internally and forms the trackbed. The tunnel is then ready for tracklaying and installation of equipment.

11.1.3 Structural type

History
The principles of immersed-tube tunnelling are by no means new. The first known attempt was in the early nineteenth century, when two brick tunnel units were built



SECTION A – A

Figure 11.2 Typical sinking arrangement (Sydney Harbour Tunnel)

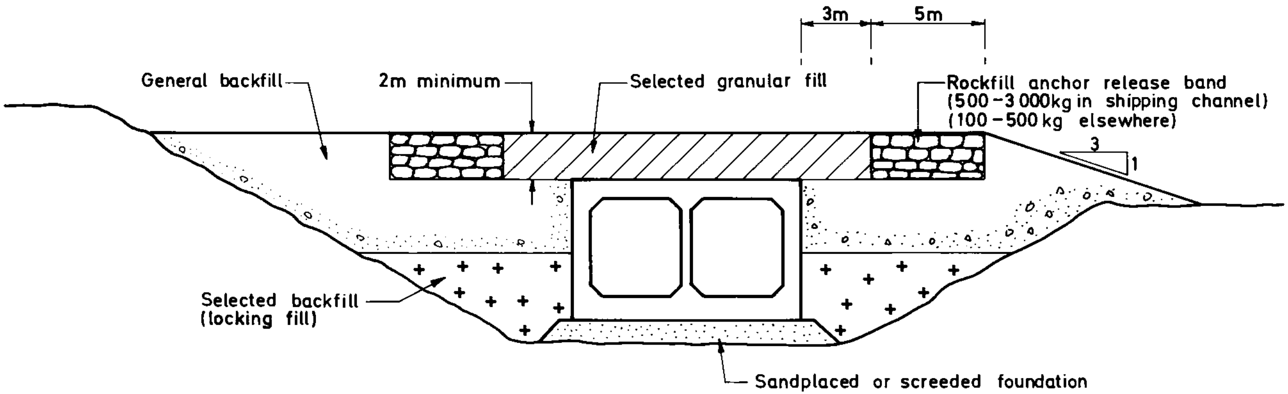


Figure 11.3 Typical backfill/anchor protection

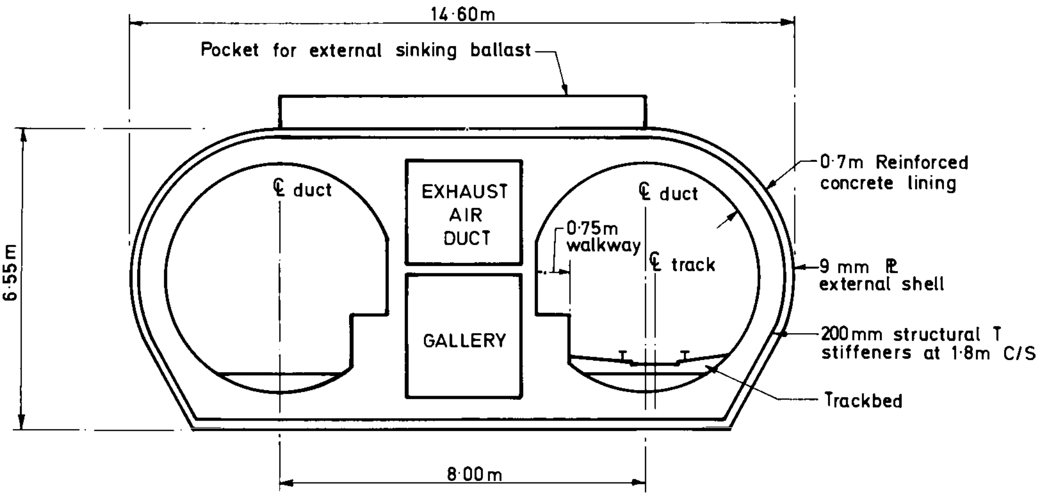


Figure 11.4 Circular steel shell tunnel (BART Tunnel)

and jointed at Rotherhithe on the River Thames[1]. Following were a number of primitive subway tunnels constructed in the early years of this century in the United States.

However, the immersed-tube tunnel is recognized in its modern form as utilizing one of the two structural types referred to above, i.e.

1. The circular steel shell tunnel; and
2. The rectangular concrete box tunnel.

Circular steel shell tunnels

In this method, circular steel 'cans' are fabricated and assembled on a slipway. The circular tube thus created is stiffened by diaphragm rings transversely and by 'T' or bulb flat stiffeners longitudinally. For road tunnels, an outer casing of steel plate is added to contain a jacket of concrete, giving the unit a characteristic octagonal cross section. Two such tubes may be combined in a single unit to form a two-track binocular section; for rail tunnel units, such as those for the Bay Area Rapid Transit (BART) in San Francisco, this shape may be modified into a more compact cross section (Figure 11.4).

After welding, ballast concrete is cast into the 'keel' of the unit and it is launched. At this stage the unit weighs 1500–2000 tonnes and floats with a draught of 2.5–3.0 m. A reinforced concrete lining ring (for the BART units 700 mm thick) is then sequentially cast inside the shell. In larger units, external concrete may be poured within the outer casing or sprayed on, providing additional ballast and protecting the steel plate. This concrete has to be placed in a strict sequence to control the forces imposed on the partially completed unit. Units of this kind are typically sunk on to a prescreeded foundation (Figure 11.5).

This method of immersed-tube tunnelling originated in the United States with the construction of the Detroit Windsor Tunnel, a two-lane road tunnel. Almost every other immersed tunnel in the United States has been of this form; they have been mostly road tunnels, but examples of rail tunnels are the Bay Area Rapid

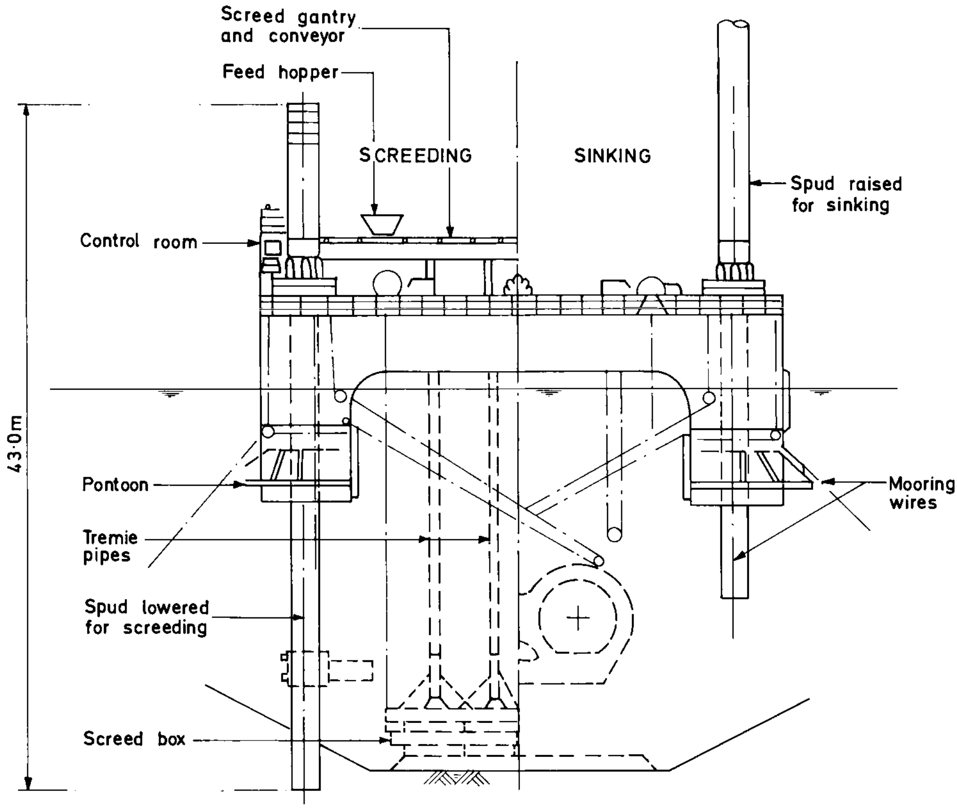


Figure 11.5 Dual-purpose jack-up barge used for prescreeding/sinking (HKMTR Tunnel)

Transit (BART) (Figure 11.6(a)) in San Francisco [2] and the Charles River Tunnel in Boston. These are both of binocular form; the East 63rd Street Tunnel in New York [3] differs in containing four ducts in 2 × 2 configuration (Figure 11.6(b)).

Rectangular concrete box tunnels

In this method the tunnel units are completely constructed 'in the dry', usually within a purpose-built drydock or casting basin, although commercial drydock/ floating dock facilities have often been considered. The basin is expensive and time consuming to construct, and in order to limit cost and make efficient use of it, units are often cast in a number of batches, the basin being flooded to release each batch and then pumped out for construction of the next.

The basin requires to be excavated sufficiently to enable the units to float when it is subsequently flooded. The units are constructed of reinforced or prestressed concrete, typically in short segments of 9–18 m long. Formwork is massive and usually of steel for multiple re-use; heavy bracing ensures rigidity and dimensional accuracy of sections without the need for through ties which might cause leakage. When the units are complete, the ends are closed off with temporary bulkheads and the basin flooded.

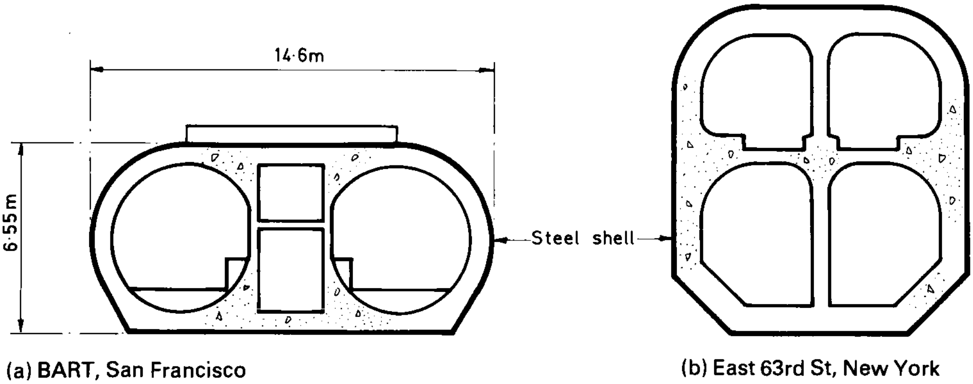


Figure 11.6 Comparative steel shell tunnel cross sections

Figure 11.7 shows units for the Hong Kong Mass Transit Railway (HKMTR) Tunnel under construction in a purpose-built casting basin. Fourteen such units were built in four batches containing four, four, four and two units, respectively. The basin is almost ready for flooding and the braced travelling formwork used to cast the units in 9 m segments is just visible on the right of the figure, hauled clear of future water level. Eleven such segments were cast sequentially to form the 100 m

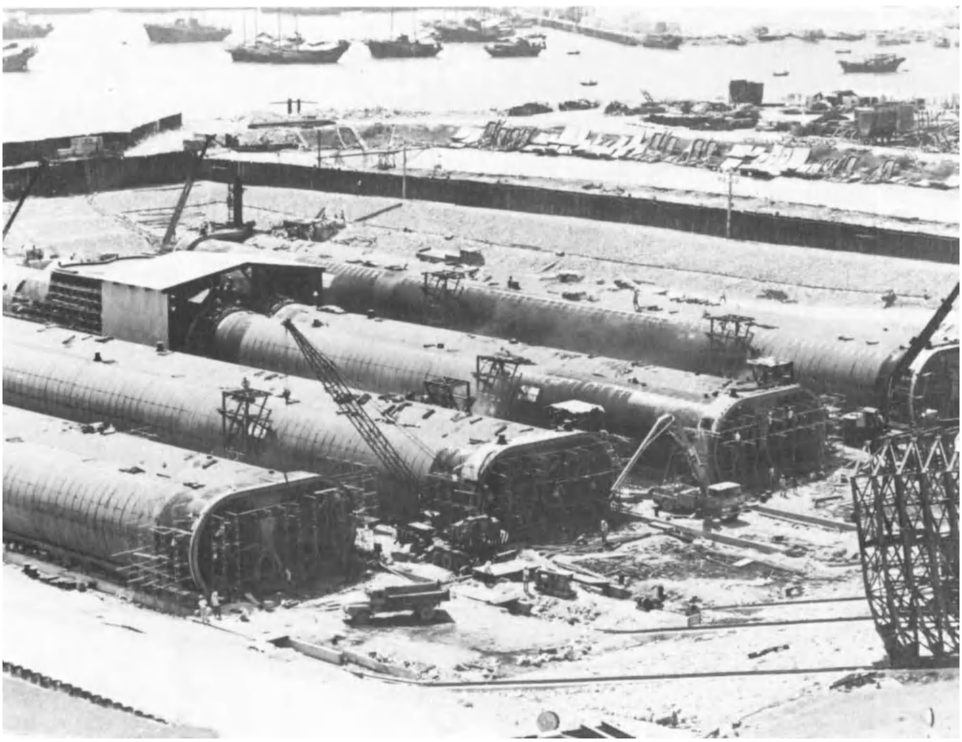


Figure 11.7 Casting basin (HKMTR Tunnel)

units. In contrast, Figure 11.8 shows the large rectangular cross section of the Eastern Harbour Crossing, also in Hong Kong, and carrying both road and rail facilities.

Concrete tunnel units are most commonly used for road tunnels which in some cases are very wide, making it difficult to screed a foundation to the required accuracy. As a result, such units utilize a sand foundation which is placed under the unit after sinking, in effect using the soffit of the unit as a form. A number of techniques have been developed, alternatively called sandjetting, sand flow or sandpacking; the technique will be generally called 'sandplacing' here. Essentially, in sandplacing, a sand/water mixture is pumped under the unit, the sand settling out of the mixture to form a pad. The process is repeated until the void under the unit is filled (Figure 11.9).

The use of this technique means that the unit must be temporarily supported above the base of the trench. This may be either on four temporary hydraulic jacks bearing onto concrete pads or, more commonly, the unit being laid is supported off the previously laid unit on brackets, with the leading end being supported on jacks (Figure 11.2). However, since railway tunnels are generally much narrower than road tunnels, a prescreeded foundation has often been used, even for concrete tunnels.

The rectangular concrete tunnel and many of the techniques associated with it, including the sandplaced foundation, originated with the Nieuwe Maas (road) Tunnel in Holland (constructed 1937–1941). An improved method of sandplacing and the use of the Gina gasket joint (see Chapter 12, Figure 12.3) was introduced with the Deas Island Tunnel in Vancouver[4]. A typical rail tunnel in this form is Hemspoor Tunnel (Figure 11.10(a)).

While the above methods can be described as typical for a rectangular concrete *road* tunnel, the generally smaller nature of a railway tunnel permits greater variation. The HKMTR Tunnel, for example, shown under construction in Figure 11.7, is a binocular section reinforced transversely but prestressed longitudinally (Figure 11.10(b)). It was placed on a screeded foundation[5]. The Rotterdam Metro tunnel, although of rectangular concrete section, was founded on piles because of very soft ground conditions (Figure 11.10(c)).

Choice

For road tunnels the advantages of the concrete box cross section are fairly clear, particularly its ability to fit the duct size precisely to requirements. The circular steel shell cross section fits less efficiently, leaving (sometimes) unwanted segmental spaces at top, bottom and sides. This lesser efficiency may discount the structural advantage of the circular shell in carrying hydrostatic pressure in membrane compressive action rather than in bending and shear of flat slabs.

For railway tunnels, the situation is less well defined. The additional vertical height of most railway structure gauges fits the circular section better. Since each cross section requires the same overall mass in order to sink, and that mass is most economically provided by concrete, the steel shell is largely an extra cost which is only partly offset by the cost of the casting basin. Circular/binocular concrete tunnels, very practical for railways, combine advantages of both types of units.

The concrete box will generally be the cheaper option, for the reasons given. However, in countries where there is no history of immersed-tube technique for guidance it may be necessary to assess alternative cross sections to determine the

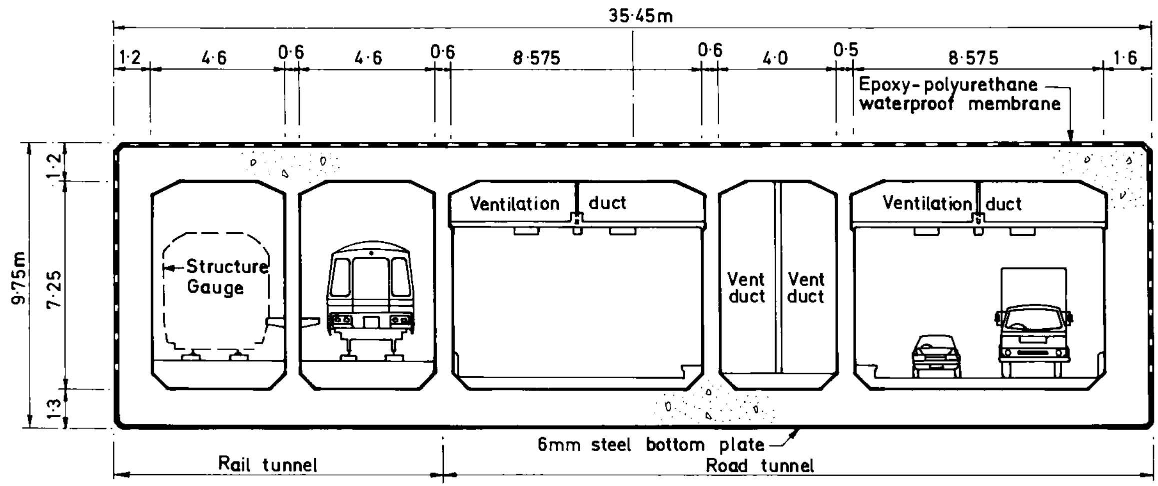


Figure 11.8 Rectangular concrete box tunnel (Eastern Harbour Crossing)

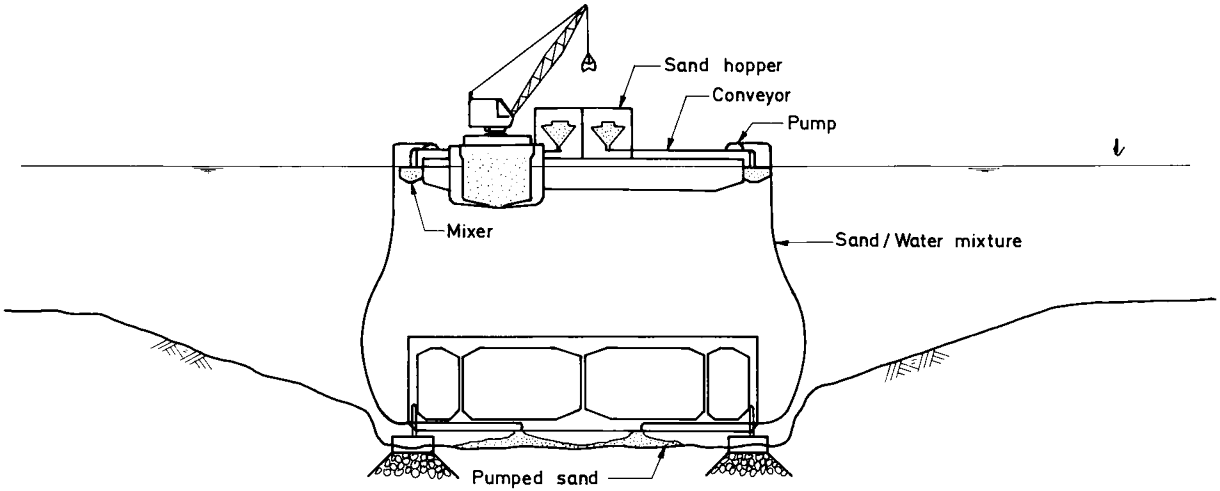


Figure 11.9 Placing the sand foundation for a rectangular concrete box tunnel

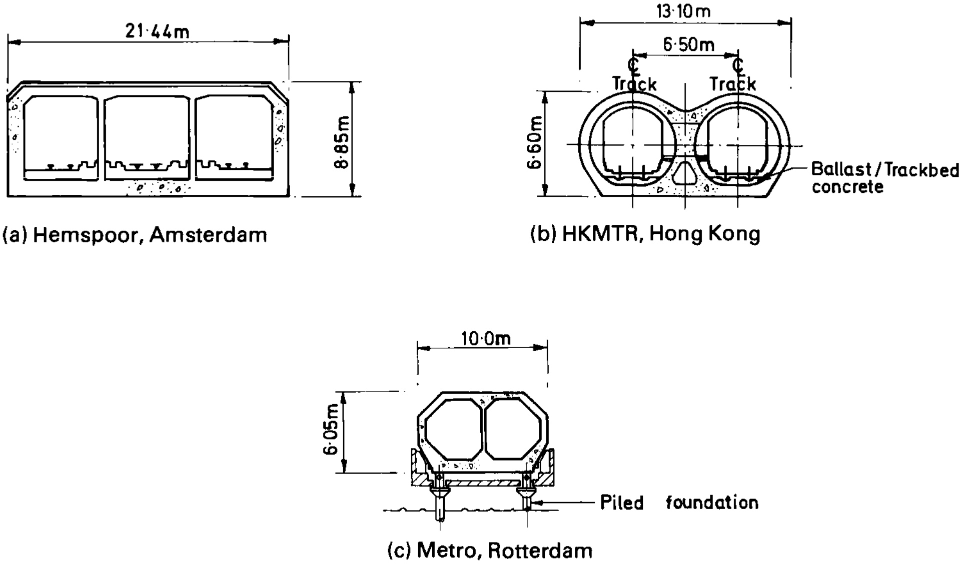


Figure 11.10 Comparative concrete box tunnel cross sections

most cost-effective solution for a specific project. A methodology for such an assessment carried out for the Sydney Harbour (road) Tunnel has been described elsewhere by the authors[6].

11.2 Vertical and horizontal alignment

11.2.1 Vertical alignment

The primary requirement of the vertical alignment is that it shall allow navigational clearances for existing and future shipping. Usually the port or river authority will set depth and width of a main navigational channel; this should include a margin for channel deepening based on a prediction of future growth. A further margin should be included to enable dredging to be carried out to industry tolerances without disturbing the tunnel backfill. Additional zones of defined depth and width flanking the main channel may be specified, together with minimum depths at moorings, wharves, etc. (Figure 11.11).

Between the fixed points on the vertical alignment so delineated, maximum gradients for most railway systems will be set only by traction requirements (typically 3.0–3.5%). A minimum gradient of 0.3–0.5% is normally provided to allow gravity drainage to a sump at the lowest point of the tunnel.

11.2.2 Horizontal alignment

Immersed-tube tunnel units are most easily constructed if the horizontal alignment is straight. This not only simplifies setting out and enables rapid repetitive construction using standard components or formwork but also avoids buoyancy and

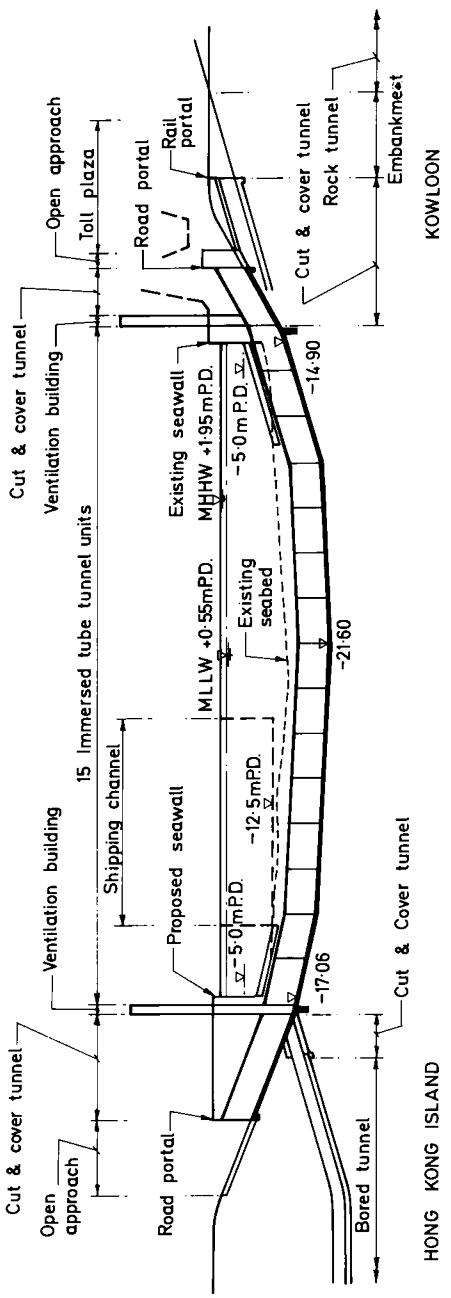


Figure 11.11 Longitudinal section (Eastern Harbour Crossing)

stability problems that may stem from a unit curved in plan. Such an ideal is not always realized due to route-planning constraints. The alignment of the HKMTR Tunnel was fixed as a 3000 m horizontal radius because of station location constraints; units to accommodate this were successfully constructed, floated and sunk [5].

11.3 Cross section and layout

11.3.1 Internal cross section

The structure gauge will be defined by the railway authority and will already allow for standard clearances to trackside structures. To determine the necessary internal cross-sectional dimensions, additional allowances must be made for:

1. The possibility of the units being sunk out of position. To avoid excessive recalculation of alignment after final survey, an additional clearance of 75–100 mm either side of the structure gauge is typically added to allow for this 'sinking error'. Vertical misalignment errors can generally be taken up in the permanent ballast thickness.
2. The incorporation of permanent internal ballast to ensure long-term negative buoyancy of the tunnel units. This ballast will normally be placed as trackbed and enables plinths and rail bases for rigid track to be installed to precise line after survey of the sunk units.

11.3.2 Provision for electrical and mechanical equipment

The prefabrication or precasting of tunnel units to a constant cross section places a restriction on the positioning of major items of electrical and mechanical plant which must be accommodated within the overall cross section rather than placed in niches or adits as in driven or cut and cover tunnelling. This restriction will generally require provision of conduits and blockouts in the structural cross section while maintaining structural integrity; in some cases additional dedicated service ducts will be required.

11.3.3 Layout

For most railway systems, incorporating an up and a down line, a side-by-side layout is a simple and obvious solution. This results in a tunnel unit wider than it is deep, which provides floating stability and minimizes dredging depth.

For multiple-track installations a two up/two down arrangement could be considered (some very early US tunnels and the more recent East 63rd Tunnel had this arrangement), but it is likely that a side-by-side arrangement would be most economical for the same reasons.

11.3.4 Cross-section design

The size of the structural cross section is, of necessity, governed by buoyancy and stability requirements rather than by structural ones. The complete unit must float with a positive freeboard; typically, a factor of safety against sinking of 1.02 is required, i.e. the total displacement must exceed the total mass by at least a factor of 1.02. In the final condition, sufficient ballast must be placed inside the unit for it to have inherent negative buoyancy. A factor of safety against flotation of about 1.05 is required. If the plan area of backfill (if any) over the unit is included, the factor of safety is normally increased to a minimum of 1.20. Using these buoyancy conditions, an approximate structural cross section can be determined by trial and error.

11.4 Structural design

11.4.1 Loading

There is no standard loading code for an immersed tube, and it will be necessary to draw up such a code for each project. This must be based on the experience and judgement of the designer, on previous practice and on site-specific factors such as marine hazards, seismic activity, etc. Typical sources of loading are shown in Table 11.1.

Table 11.1 Load sources

	Loading condition	
Source	*Construction*	*Service*
Self-weight	×	×
Hydrostatic		
based on Mean High Tide	×	
based on Highest Astronomical Tide		×
Environmental (wind, wave, current temperature)		
based on 1 in 10-year event	×	
based on 1 in 100-year event		(×)[a]
Particular construction loadings, launching etc.	×	
Backfill		×
Foundation pressure[b]		×
Marine hazards,		
stranding/sinking ships	(×)[c]	×
falling/dragging anchors		×
Seismic		×
Fire, explosion		×

[a] In service condition, environmental loads, except temperature, will be indirect, i.e. may affect stability of backfill, etc.
[b] Allowance should be made for local increases in foundation pressures due to loss of foundation support elsewhere.
[c] If extensive towing is envisaged, it may be necessary to consider effects of collision with towing vessels, other vessels or floating debris.

Ships and their anchors are capable of exerting extremely large forces, and it may be necessary to make an assessment of risk of such a hazard occurring[7]. In zones of minor seismic activity, checks for ductility only are necessary, also ensuring that joints do not open as pressure waves pass[8]. If there is potential for ground shearing or liquefaction, the immersed-tube method is probably best avoided.

11.4.2 Analysis and design

Transverse design

The transverse design of the unit is carried out in accordance with normal structural principles based on a unit length analysed as a plane frame. The principles have been well described by Bickel[9] for circular steel tunnels, and Brink[10] for rectangular concrete ones. For the latter, particular attention to the serviceability limit state of cracking is required; the CEB-FIP method of calculating flexural crack widths is recommended[11]. There is controversy over allowable flexural crack widths and their effect on corrosion[12, 13], but a maximum figure of 0.2 mm on the outside surface is recommended.

Longitudinal design

Construction Circular steel tunnel units are commonly launched sideways, which imposes little or no significant stress on the steel shell. Longitudinal launching imposes higher stresses which can be calculated by examining successive states of buoyancy as the unit enters the water. Once floating, a sequence for concreting the lining must be calculated to minimize longitudinal bending moment.

All units are subject to wind, wave and current forces while under tow. In inshore conditions simple design checks are sufficient. For offshore towing, particularly for deep-draught rectangular concrete units, numerical and/or physical modelling may be necessary to determine behavioural characteristics and towline forces.

Service The buoyant weight of the unit is small and the imposed foundation load is less than the weight of soil removed when dredging the trench. Thus, settlements are small and mostly elastic and consolidation settlements can generally be ignored.

In most soils the establishment of a reload elastic modulus and its variation with depth will be sufficient to determine the elastic behaviour of the underlying soil. The behaviour of the foundation layer is best established from tests; 3–4 MPa is a typical figure[14]. Longitudinal analysis can then be carried out as a beam on elastic foundation problem. Bending moments are reduced by the provision (or assumption) of flexible joints transferring no moment between units. These joints are typically placed at the junctions between units. However, many Dutch tunnels take an opposite approach, making the unit joints rigid but building in flexible joints at quite close centres (say, 20 m) along the length of the unit. These are held rigid by temporary prestressing during floating, towing and sinking but released after completing the foundation.

11.4.3 Durability

Introduction

Immersed-tube tunnels are water-retaining structures; hydrostatic loading forms the greatest component of loading and the tunnel is subject to this constant load throughout its design life. Under hydrostatic pressure, water (often seawater) will attempt to penetrate the concrete structure either by permeation or capillary action and, by a number of chemical or electrochemical mechanisms, cause corrosion of reinforcement or steel shell or deterioration of the cement matrix[15]. The process will be slowed by provision of dense, high-quality concrete and accelerated by the presence of cracks[12, 13].

For circular steel tunnels, great reliance is placed on the steel membrane as the primary defence, protected from corrosion by the outer concrete jacket. As a consequence, there is less concern for the durability of the inner concrete shell, which also has to be placed under less ideal conditions after the unit is afloat.

In the rectangular concrete tunnel, the external waterproof membrane (if applied at all) is seen only as a second line of defence. The reinforced or prestressed concrete shell must itself ensure durability. The primary safeguards to ensure serviceability of the concrete are dense, relatively impermeable concrete and the limitation of crack widths to acceptable values. Cracks in immersed-tube units have two components; flexural, which must be controlled by serviceability design, and direct tensile, which results from restraint of thermal cooling or drying shrinkage of the concrete mass. It is these latter cracks which are most critical, since (in theory) they penetrate the entire section. It is recommended that restraint cracks should be controlled to 0.1 mm or less by one of the methods described below.

Concrete mix design for durability
Factors to be considered in designing for durability are minimum water/cement ratio (perhaps by use of additives), minimum permeability (by use of well-graded aggregates and possibly pulverized fuel ash (pfa)), and increased chemical resistance (by use of optimum cement content, pfa, ground granulated blast furnace slag or other additives)[16].

Thermal effects are minimized in the first instance by reducing heat of hydration output, and this can be done by reducing cement content (subject to durability requirements), cement replacement (by pfa or ground granulated blast furnace slag), use of low-heat cements and controlling placing temperature. Drying shrinkage can be minimized by careful aggregate selection.

Control of cracking due to thermal and drying shrinkage
Prestress Post-tensioning is an effective method of closing up direct tensile cracking due to thermal effects and shrinkage and has been commonly used for this purpose in water-retaining structures. Applied to the thick concrete sections of an immersed tube, the method can be expensive. Longitudinal prestress was used for the HKMTR tunnel[5].

Cooling By conducting away the heat generated by hydration as it occurs the initial temperature rise can be reduced to such a level that the strain on cooling is below the tensile strain capacity of the concrete. This method, pioneered by the Dutch, depends on casting cooling pipes into the concrete and circulating brine through these. The method has been very successful, to the extent that many Dutch tunnels no longer use any kind of waterproof membrane[17].

Reinforcement Crack widths may also be controlled by providing additional reinforcement to carry the tensile stresses generated. The additional reinforcement may be considerable but can be 're-used' to carry flexural stresses once the initial cooling period is over. The method of design is described by Harrison[18], and was used in the design of the Sydney Harbour Tunnel[19].

Waterproof membranes
Circular steel tunnels The steel shell forms an effective waterproof membrane provided that welding has been carried out to the highest standards. The durability of the steel shell is often questioned because of its continuous exposure to water, particularly seawater. However, the shell is usually covered by concrete on the outside as well as on the inside and, being fully submerged, suffers much less corrosion than steel in the tidal zone. Work by Larrabee[20] suggests that high

initial rates of corrosion reduce rapidly as the shell becomes protected by an undisturbed layer of corrosion products. Average rate over 50–100 years was 0.03 mm/yr.

Rectangular concrete tunnels A steel plate is often considered the most practical form of waterproofing for the underside of the units, since it provides both a durable membrane and a good working surface; for the Conwy Tunnel currently under construction, the plate has been carried up the outside walls as well. A steel plate was used under the Nieuwe Maas Tunnel; although protected by a concrete layer, recent coring tests indicated negligible corrosion after 45 years. For the Sydney Harbour Tunnel, a continuously ribbed PVC sheet was used as an alternative.

For the walls and roof of the tunnel, a flexible membrane, post-applied to the concrete, may be more economical. Traditionally, bitumen-impregnated felt was applied in multiple layers, using molten bitumen as an adhesive.

The availability of solventless spray-on epoxy materials led to these being considered for use as a waterproof membrane, since they offered much greater speed of application (2000 m^2/day as opposed to perhaps 200 m^2/day for bitumen felt). The blending of coaltars, and then polyurethanes, enabled excellent crack-bridging properties to be developed. The use of spray-on materials demands particular attention to substrate condition, and individual manufacturers must be consulted regarding exact conditions for their product.

11.4.4 Temporary fitments

In order for the immersed-tube unit to float it must have its open ends sealed with temporary bulkheads. The change from positive to negative buoyancy is (usually) achieved using water ballast which must be contained in tanks, pumped in or out and transferred between tanks. The units must be moored, towed and sunk via bollards, towing brackets, lifting lugs, etc. All these facilities will be removed or abandoned after sinking, and are termed ‘temporary fitments’.

The planning and design of these fitments is dependent on the contractor’s proposed method of towing and sinking the units. Normally, therefore, the design and detailing of such items is carried out in conjunction with the contractor who will carry out the work.

11.5 Joints

11.5.1 Introduction

The joints between units of an immersed-tube tunnel have a dual function. They enable the connection between units to be made under water and, as described above, introduce flexibility into the structure. Two basic joint types have been used:

1. The tremie concrete joint, much used in US steel shell tunnels; and
2. The hydrostatic joint, developed by Rotterdam City Works Department for the Rotterdam Metro Tunnel[21] (see Chapter 12, Figure 12.3) and first used on Deas Island Tunnel[5] in 1959.

Of these, the latter is now achieving dominance, being used on nearly all European, Japanese and South-east Asian immersed tubes and some recent US ones as well.

11.5.2 Tremie concrete joint

This type of joint was developed for use with steel shell tunnels; it follows that they have also been used mostly when sinking units onto a prescreeded gravel bed. The arrangement is shown in Figure 11.12.

The advantage of this joint is its comparative simplicity. The disadvantages are the necessity for underwater working in fitting the closure plates and for tremie concreting. Until this concrete has gained strength, the bulkhead space cannot be dewatered and the watertightness of the joint assessed. An apparently fundamental disadvantage is the lack of flexibility in the joint to relieve bending moment. However, quite large settlements and joint rotations have been recorded in steel shell tunnels despite the lack of articulation[22]. It is assumed that the joint relies on microcracking of the concrete and the ductility of the steel shell.

Recent immersed tubes in the United States, while of steel shell construction, have adopted the European compressed gasket joint (e.g. Fort McHenry[23], 2nd Elizabeth River[24]).

11.5.3 Hydrostatic joint

This type of joint requires the use of a primary seal in the form of a heavy rubber gasket of a shape which has become known as a Gina profile or section. This consists of a fabric reinforced seating flange, a main body and a soft rubber nose.

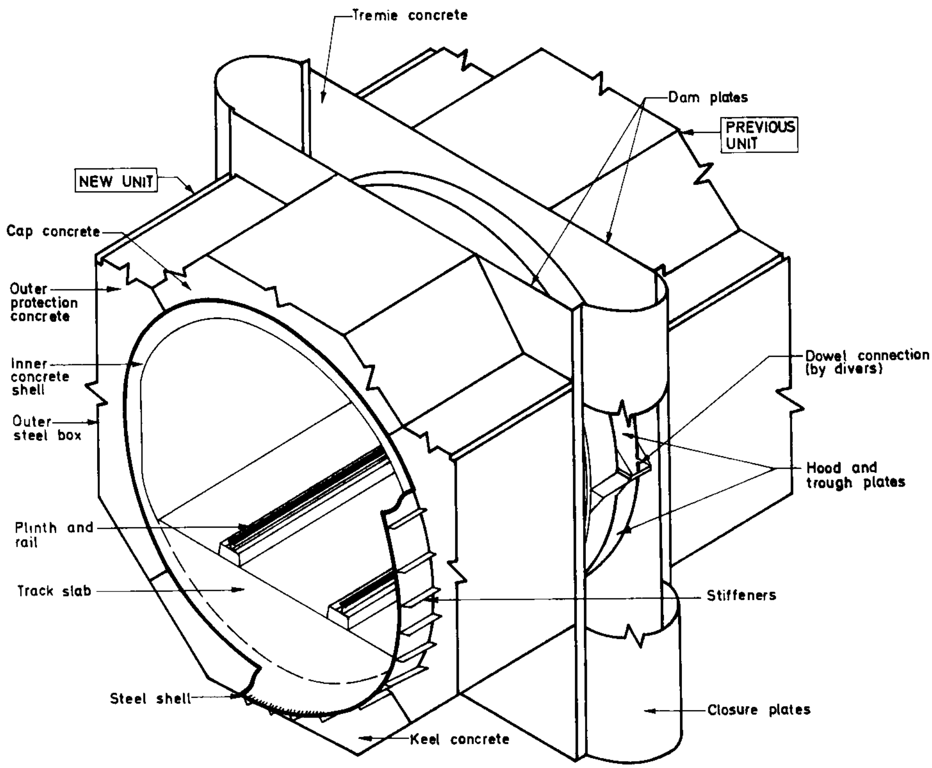


Figure 11.12 Tremie concrete joint in circular steel shell tunnel

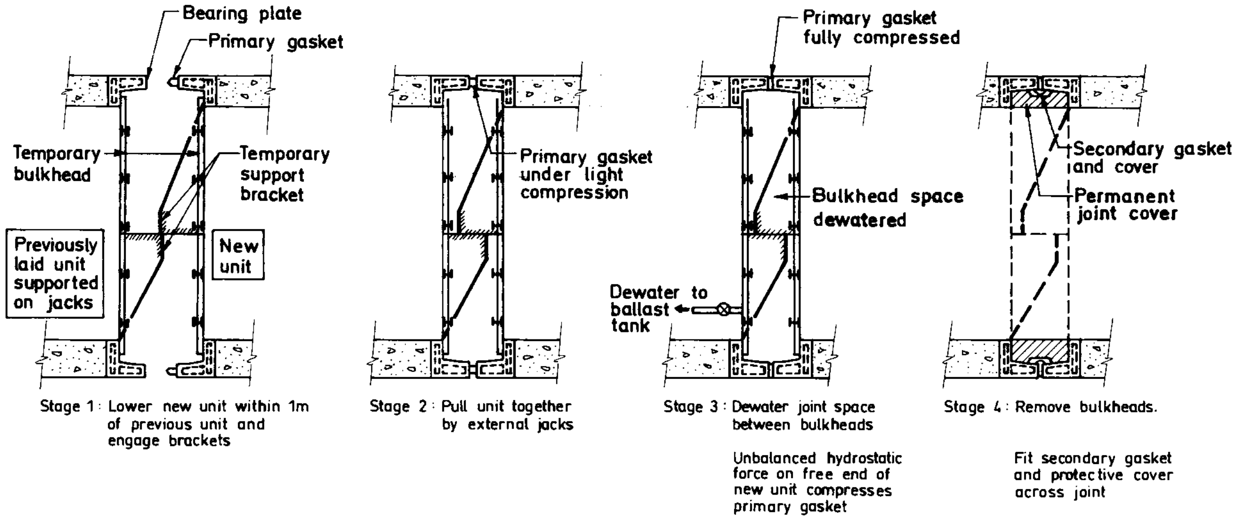


Figure 11.13 Hydrostatic joint in rectangular box section tunnel

The gasket is fitted to the perimeter of one end of each unit. Considerable accuracy is necessary in achieving planeness of the gasket; when fitted to concrete units a steel mounting plate is generally welded in place after concreting. A similar steel plate is used as a counterplate on the end of the unit forming the other half of the joint. The method of making the joint is shown in Figure 11.13.

A secondary seal, known as an omega seal because of its shape, is fitted around the inside of the joint and pressure tested. If the joint is to be made rigid, with flexibility to be provided within the length of the unit (see above), reinforcement is made continuous and the joint concreted. If the joint is to remain flexible, the seals are protected from fire by steel panels, mineral wool or both.

11.5.4 Shear transfer

In order to avoid differential settlement at the joints, some means of transferring shear force across the joint is required., In steel shell tunnels, the continuity of the shell across the joint, together with the concrete lining, creates uniform shear capacity throughout the tunnel. The same applies to a 'rigid' hydrostatic joint.

In flexible hydrostatic joints, shear capacity must be separately provided for. The HKMTR Tunnel utilized belled ends to the units, resembling the sockets on the ends of a pipe. Into each pair of sockets, after jointing, was cast a concrete 'spigot' providing shear transfer across the joint [5]. The Eastern Harbour Crossing, Hong Kong [25], a combined road/rail tunnel, is too large for this method. Instead, large steel dowels were installed across the internal walls after jointing.

11.6 Further development

11.6.1 Introduction

This chapter has briefly described the methods of immersed-tube construction commonly applied to rail tunnels. The technique is almost infinitely adaptable, being amended as required to suit site conditions, contractors' plant and equipment or construction sequence. There is a tendency for immersed-tube contractors to use the same method and temporary works details as in the immediately preceding project, for the excellent reason that they know they work. A situation with a 20 000-tonne unit suspended in 20 m of water is no place to discover that a new technique is flawed. Indeed, if a procedure works well, there is no point in changing it unless by doing so the cost can be reduced or greater safety or a better finished product achieved.

Improvements in durability, waterproofing membranes, survey and unit-positioning techniques will stem from natural development within the civil engineering industry, in some cases led by requirements of immersed-tube tunnel engineers. Other changes may require bolder steps, and some of these are briefly reviewed below.

11.6.2 Dredging

The dredging of the trench for the immersed-tube units is perhaps the only problem area in an otherwise environmentally 'clean' construction method. During the planning of the Sydney Harbour (road) Tunnel, concern was raised over the effect of silt deposition on nearby prawn beds and the dispersion of heavy metals which had accumulated in the bottom silt. Environmental concerns over dredging also contributed to the decision to make the Storebaelt Crossing a driven tunnel, rather than immersed-tube, crossing. Partial solutions to these problems are available within the dredging industry and industry sources should be consulted when planning an immersed-tube crossing.

11.6.3 Joint seals

The use of the primary or Gina seal in hydrostatic joints has become almost universal in Europe, Japan and South-east Asia. While the method works admirably, criticism is sometimes levelled at the cost of the gasket, part of which results from its large cross section and natural rubber content.

The primary gasket only performs its main function during the making of the joint and is afterwards largely replaced by the secondary or 'omega' seal (although the primary gasket remains as a substantial back-up). Hansen[26] has suggested replacing the primary seal with cheaper components which individually take over the separate functions of resisting the axial hydrostatic thrust (with timber blocks) and sealing the joint (with inflatable rubber seals).

11.6.4 Tethered buoyant tunnels

A limitation of the immersed-tube method is the need for relatively shallow water and a regular bed formation. The former is to minimize hydrostatic pressure and the latter to avoid excessive dredging or the possibility of having to construct underwater embankments on which to found sections of the tunnel. This may result in constriction of the waterway area and scour/siltation problems. Since the units are initially buoyant, this opens the possibility of crossing deep-water channels with a permanently immersed but buoyant tunnel tethered to seabed anchors, rather like an inverted suspension bridge. The vertical alignment would then be dictated only by landfall levels and shipping clearances.

Clearly, the method is not fail-safe and is vulnerable to sinking ships, anchors and, possibly, earthquakes. It has yet to be tried in practice.

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Chapter 12

Precast concrete tunnels

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Contents

12.1 Introduction

One of the methods of minimizing the disruption caused by cut and cover construction that has been developed is the use of large precast elements. A good example of this is the east–west line (Line B) in Rotterdam.

12.2 General

The Rotterdam metro dates from the end of the 1950s. In 1959 the Rotterdam Council took a decision to construct the first section of the metro under the Nieuwe Maas river. This resulted in an important north–south public transport link between the Central Station on the northern bank of the Maas and Zuidplein, a shopping centre, on the southern bank. The first section of the metro, with a length of 5.4 km, of which 3.1 km was tunnel and 2.3 km was viaduct, was in the 1960s the shortest mass rapid transport line in the world. The line was put into operation in 1968. Although it was very short, it was the beginning of a public transport system to meet the needs of the future. The length of the metro system is now about 40 km. From the beginning, prefabrication has been the keypoint of the design philosophy of the Rotterdam metro. The shell tunnel is an example of this.

The Rotterdam region has more than a million inhabitants and is the largest seaport in the world, situated at the mouth of the rivers Rhine and Meuse. The geographical position on the river mouth divides Rotterdam into two parts; a northern part with the old city centre which was destroyed in World War II and a southern part. Offices, shopping centres, theatres, cinemas, etc. are concentrated in the northern part, surrounded by residential areas. The southern part mainly consists of residential areas.

The total port area stretches from the Willemsbridge in the city centre to the North Sea over a length of 35 km along the river. Annually, some 30 000 ocean-going vessels and 170 000 inland vessels put into Rotterdam to load and unload. Rotterdam is 'the gateway to Europe'. Inland waterway vessels can sail to West Germany, Switzerland, Belgium and France along the Rhine and the Meuse.

The development of an infrastructure with a target date of 2000 is still continuing, and the construction of the metro has improved the public-transport situation in Rotterdam considerably. In order to tackle the problem of traffic congestion in densely populated regions, especially at bridges and tunnels crossing the river, an extended and integrated system of metro, trams and buses is necessary, and is an investment for the future (Figure 12.1).

12.3 Metro layout

The north–south metro line was constructed in a number of successive stages opened from 1968 (the first tunnel section) to 1985. Building the east–west metro line on the northern river bank started in 1973. There is an interchange with the north–south line in the centre of the city. The east–west line runs in a tunnel in the central area of the city and above ground at street level on the eastern side in the newly built residential areas. There the metro is constructed as a rapid tramway system with a traffic-free track except at road crossings. The first part of the east–west line is in tunnel, the Coolhaven – Capelsebrug section, and was opened

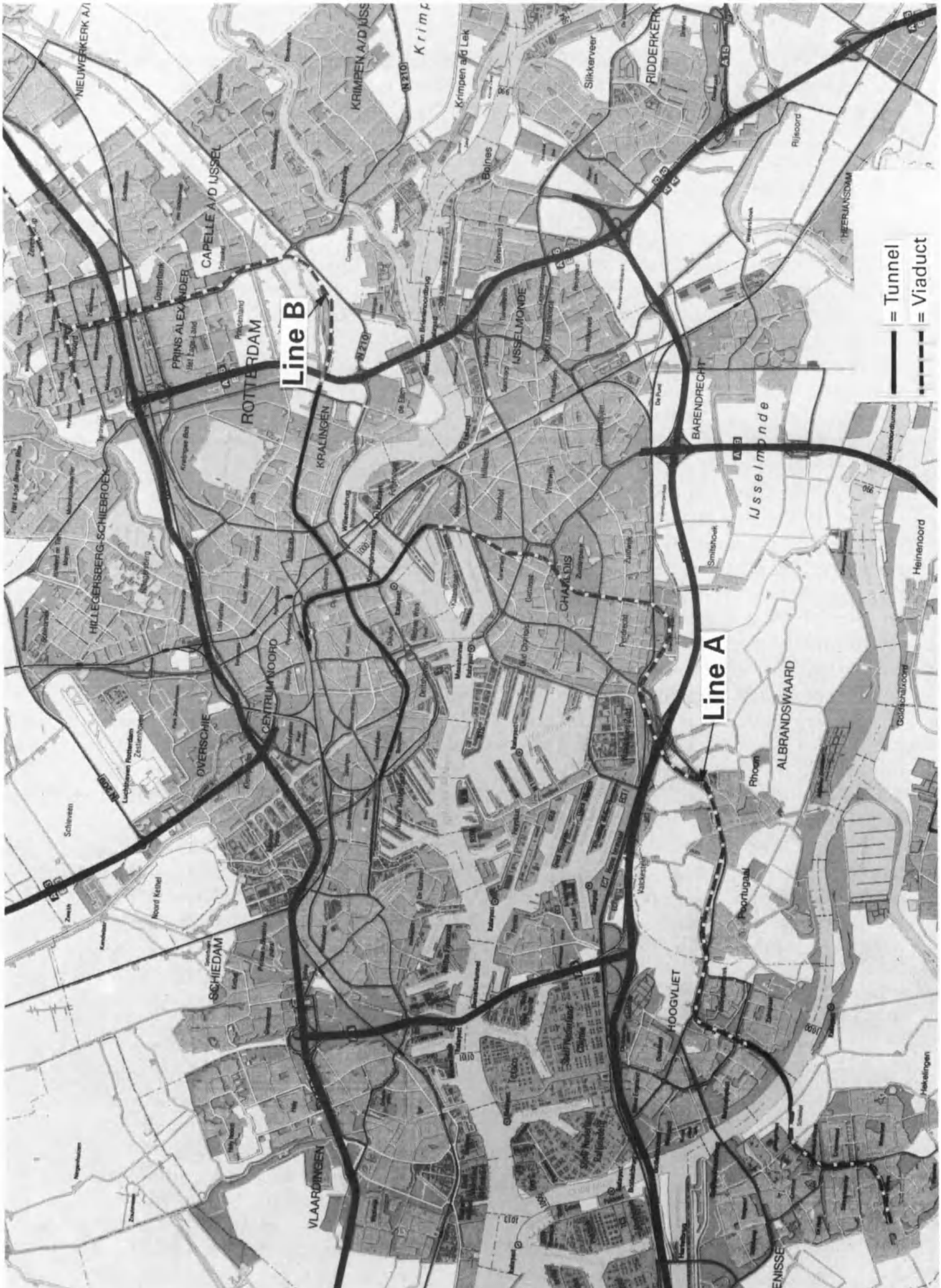

Figure 12.1 Layout of the metro system

in 1982. Extensions followed in 1983, 1984 and 1986. The metro system in Rotterdam is now 40.1 km long. The line consists of 13.8 km tunnel, 8.5 km viaduct, 8.7 km embankment and 9.2 km of railway track on concrete slabs supported on piles. The pile-supported track on the east–west line has been chosen as the foundation in the polder area in the eastern part of the city. Here we find the lowest-lying polder of the Netherlands, at a level of 6.5 m below sea level (Figure 12.2).

Rotterdam is a dynamic city with good living, working and recreation conditions. The urban area is extending systematically. The continuing expansion of the port

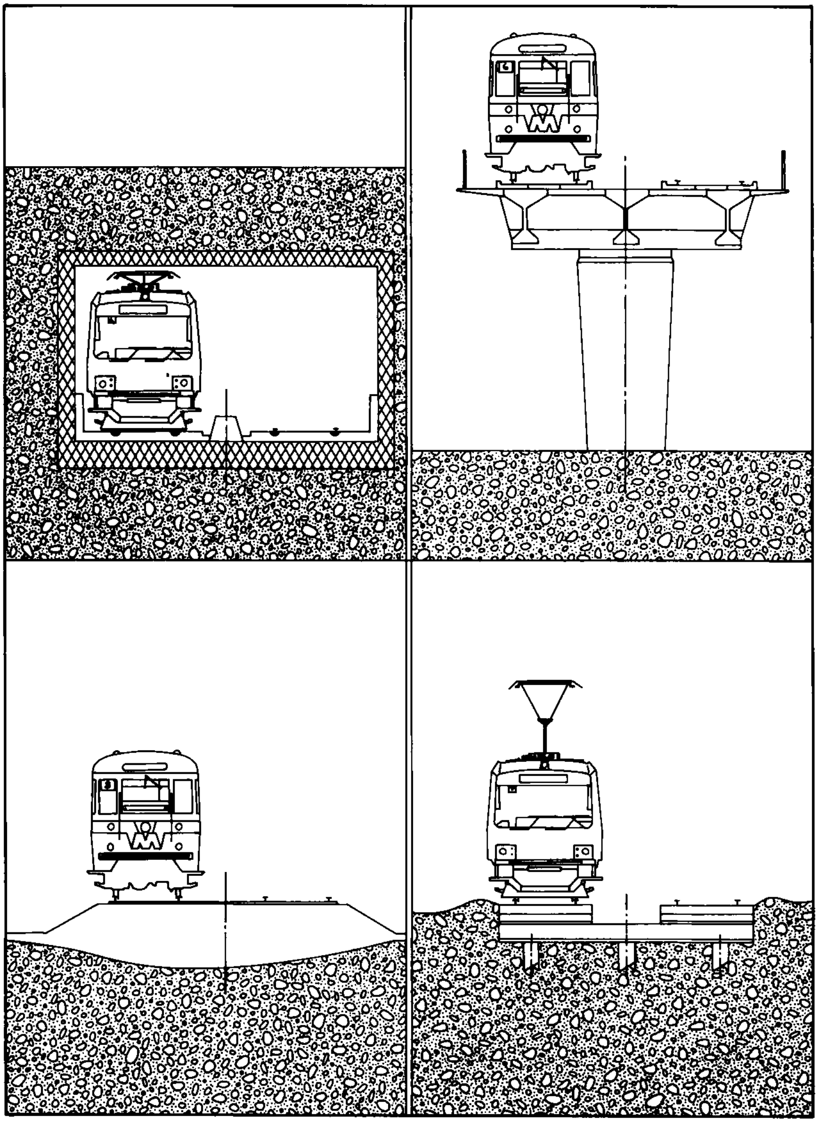

Figure 12.2 Metro structures; tunnel, viaduct, earth embankment, pile-supported track

area, new districts, renovation of old residential areas, construction of new industrial areas and business parks can be mentioned. At present there is a redevelopment of the city centre with high-rise buildings. On each working day in Rotterdam more than 600 000 people travel by public transport. The metro forms the backbone of this. On the outskirts of the city there are special interchange stations for trams and buses and on the ring roads around Rotterdam there are extensive park-and-ride facilities. Extensions to the metro are being planned.

Metro building in Rotterdam involves many specialized methods of construction. Being situated in a river delta with soft subsoil makes it an interesting working area for civil engineers.

The composition of subsoil has a great influence of construction methods and is usually critical for design. Tunnel construction in Rotterdam takes place in trenches with earth-retaining sheet pile walls. Consequently the route of the Rotterdam metro in the city area is, for the most part, under main roads. Concrete and prefabrication form an integrated combination in the Rotterdam metro, for both underground and above-ground structures. It is a suitable construction material for public transport systems and is the cheapest for tunnels and viaducts. Concrete can be a watertight material and can have a high durability. High-quality concrete requires little maintenance and prefabrication in concrete offers optimum possibilities of achieving high material and construction quality.

In the underground parts of the Rotterdam metro we find immersed concrete tunnels (see also Chapter 11, Figure 11.10(c)) not only as the building method for the river and harbour crossings but also for the land tunnels of the north–south line, which lies with a groundcover of 4–8 m deep below the groundwater table. Here parts of the tunnel were prefabricated as units in a number of construction docks in the city centre, transported floating and sunk into artificially excavated canals running across the city. Due to neighbouring buildings with vulnerable wooden pile foundations, large-scale groundwater drainage, necessary for a dry building pit, was not possible. In Rotterdam we also find underground, beside tunnels constructed *in situ*, those which are 'partially' prefabricated and laid in open, dry and shallow trenches. It is this method of tunnel construction which is described here.

The high degree of prefabrication means for all structures not only the shortest building time but also low costs. Fundamental research in Rotterdam brought new developments in tunnel construction technology. High grades of concrete have been produced in terms of watertightness and strength. For the immersed metro tunnel in Rotterdam the well-known Gina rubber profile (see Chapter 11) was developed (Figure 12.3). This has been applied in many immersed tunnels.

12.4 Design of the shell tunnel

This chapter is limited to the parts of the tunnel on the Rotterdam east–west metro line which are constructed as a so-called 'shell tunnel' (Figure 12.4). The walls and roof of the tunnel are prefabricated as composite segments which can be handled easily. The name given to the type of tunnel relates to the shape of the combined wall–roof element. In a narrow building site, after driving concrete piles and pouring the tunnel floor *in situ*, the prefabricated shell segments are placed on the tunnel floor at the bottom of the trench. By so doing, the walls and roof of the tunnel are constructed in one stage. The segments are prefabricated elsewhere and

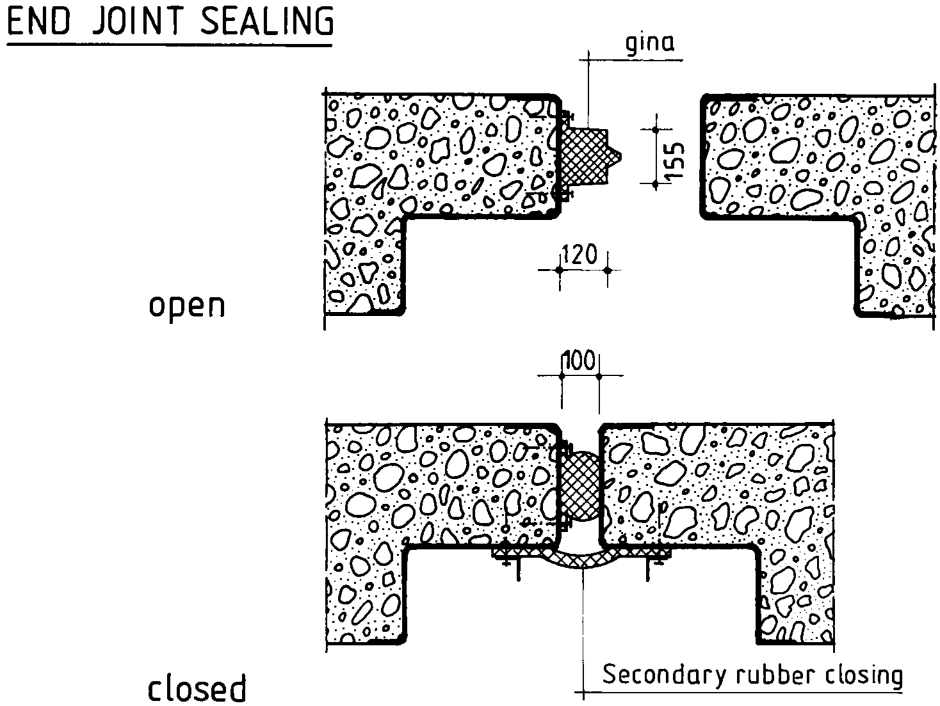

Figure 12.3 A Gina rubber profile

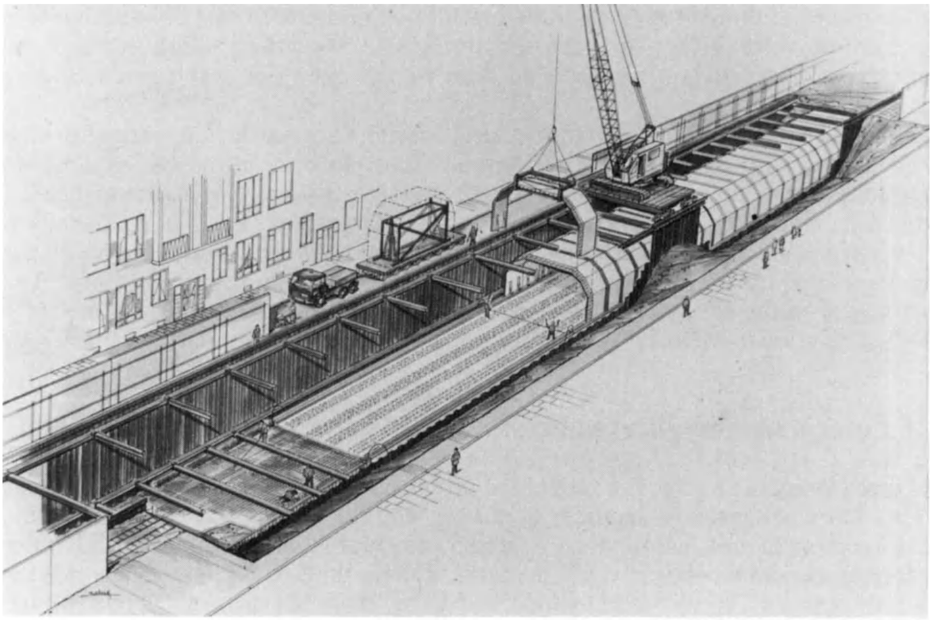

Figure 12.4 Artist's impression of a shell tunnel

transported to the building site. The shell segments are joined to each other and to the tunnel floor by a rigid joint. The result is a monolithic tunnel structure. In the longitudinal direction, like the traditional tunnels in Rotterdam completely poured *in situ*, the shell tunnel is divided into sections. Expansion joints are located at regular distances over the entire cross section of shell and floor. Each section therefore consists of one floor part with several shell segments joined to it. As a result of an optimal design, the thickness of the walls and roof of the tunnel can be limited.

The east–west metro tunnel generally has a small ground cover of 1.7–2.5 m. The tunnel has a length of 6 km. Apart from a 500 m section of immersed tunnel at the harbour crossing Coolhaven, the tunnel is mainly constructed *in situ* in the building trench. Due to local conditions, of the 6 km of tunnel on the east–west line, 1000 m in the 's-Gravenweg, the last tunnel part on the eastern side and 600 m in the Schiedamseweg, the last on the western side, are made as shell tunnels. The main building activities for the tunnel are driving the sheet piling and concrete piles from ground level with the help of an extension piece, excavation of the trench and assembling the struts, pouring the tunnel floor, placing prefabricated shell segments on the tunnel floor and joining the segments to each other and to the tunnel floor.

The construction of the shell tunnel can be characterized as environment-friendly and the degree to which disruption is caused by the tunnel construction is limited as much as possible. The space for building in the 's-Gravenweg, an historical street, and the Schiedamseweg, a busy shopping street, is very restricted. In the case of the shell tunnel, the building area is virtually limited to the area above the building site. Tunnel construction can take place in a trench with a minimum width. The building sequence length is limited (Figure 12.5).

The term 'building sequence' is understood to mean the linking together of activities, from removing the pavement to and including reconstructing the road surface after the tunnel has been completed. The rapid rate of tunnel building itself also reduces the effect on the environment. For the 's-Gravenweg and Schiedamseweg tunnels a high rate of 30 m tunnel per week was chosen (Figure 12.6).

Technical requirements of the tunnel design are sufficient strength and watertightness in particular. The tunnel cross section must be capable in longitudinal and lateral directions to withstand the load of its own weight, the metro vehicles, the soil and water and an overhead load (for example, road traffic). The watertightness of the tunnel places demands on the vertical joints between the shell segments and the horizontal joint between the segments and the floor. The shape given to the shells and the joints must be such that these are suitable for a speed of 30 m of tunnel per week.

12.5 Lateral and longitudinal cross section

Measured along the axis of the tunnel the prefabricated shell segments have a width of 3 m. The thickness of a segment is 350 mm and the weight is 37 tonnes. During prefabrication in the factory the segments are poured on their 'side' and, after hardening, tilted into the 'upright' position. The method of prefabrication of walls and roof permits a cross section which may differ from the usual rectangular cross section of a conventional tunnel in Rotterdam. The ideal form would be a parabola,

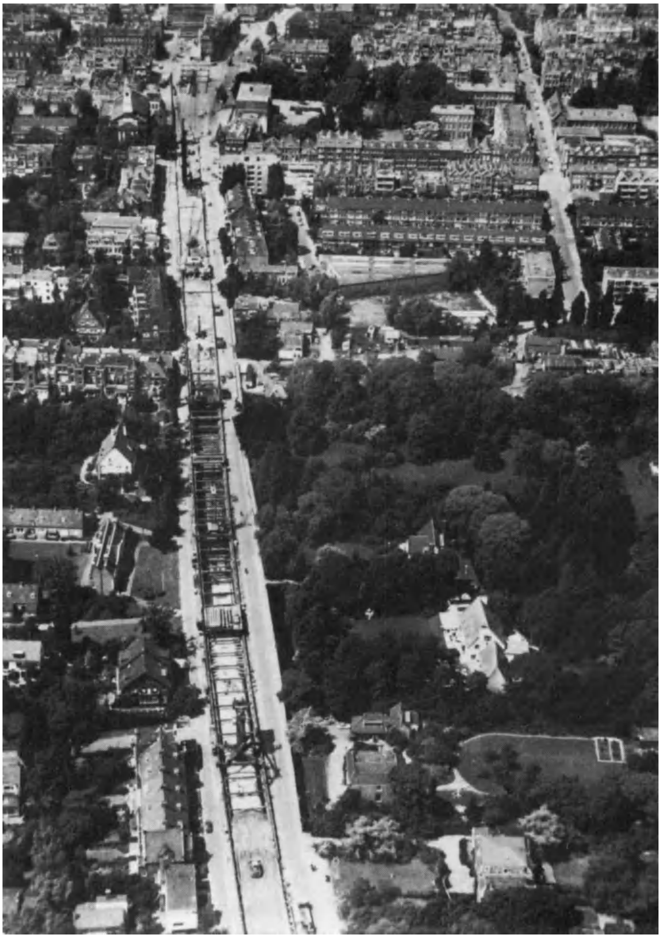

Figure 12.5 The construction of a shell tunnel in 's-Gravenweg

as a result of which, under the influence of a symmetrical load in normal circumstances, the bending moments are reduced to a minimum. Critical for the lateral cross section, in addition to the required profile of clearance for the metro vehicles, is the condition that normally there must be a groundcover above the tunnel of 1.7 m minimum for accommodating transverse cables and piping. There is also the minimum width and depth of the building trench. In the light of these conditions the choice for the shell segment was a flattened shape of walls and roof with a thickness of 350 mm. The floor thickness is 700 mm. In a part of the Schiedamseweg, the thickness of the roof of the shell segment is 450 mm. At one location the groundcover on the tunnel is 4 m and at another the roof span with normal groundcover is enlarged by approximately 0.4 m. Here the tunnel profile is widened where a switch in the rail track is mounted (Figure 12.7).

The foundation of the shell tunnel consists of prefabricated, prestressed concrete piles of 380 × 450 mm, driven into the pleistocene sand. The bearing capacity of the piles is 2100 kN. In the normal loading situation and taking into account a

(a)

(b)

Figure 12.6 (a) A pile-driving building trench; (b) entrance to Schiedamseweg

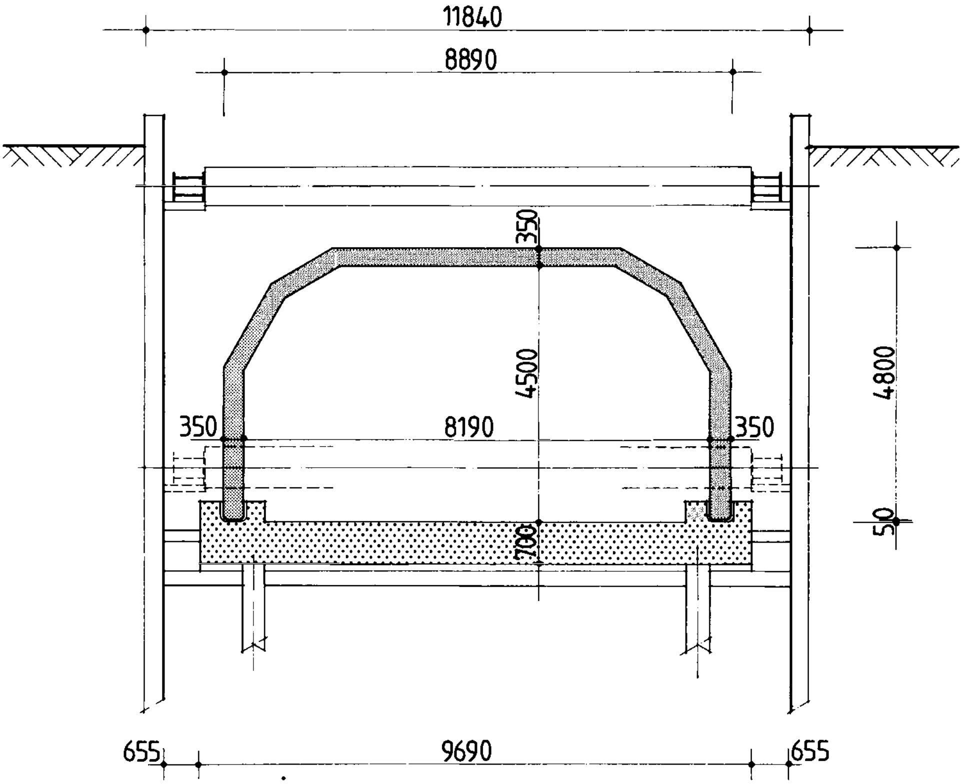

Figure 12.7 Cross section of a shell tunnel and building trench

groundwater level of 1.10 m below ground level, the pile loading is approximately 1000 kN. If the groundwater level is lowered to the underside of the tunnel floor as a result of drainage then the pile loading rises to approximately 1500 kN. This can occur during construction or later, due to some unforeseen construction activity in the neighbourhood of the tunnel, and is an extreme situation.

In order to limit the effect on the environment the piles are driven with noise-reducing equipment. The clear space necessary for this requires a minimum distance from the centre of the pile to the inside of the sheet piling of 1.5 m. Due to this limiting condition and the minimum width of the trench, the piles are not located centrally under the walls. The result is a higher bending moment in the floor (Figure 12.8). The design of the shell cross section is calculated on the basis that the shells are assumed to be both hinged and fixed at the foot. Maximum moment and shear force diagrams are determined by this. A transverse percentage reinforcement of 0.8% is required for the maximum positive moment and a reinforcement of 0.63% for the maximum negative one. Due to exceeding the permissible shear stress, transverse reinforcement is required near the transition between the horizontal roof part and the first bevelled edge. For this purpose, part of the reinforcement is bent up out of the roof. For floor construction reinforcement is 0.4% for the maximum negative moment and 0.3% for the maximum positive one.

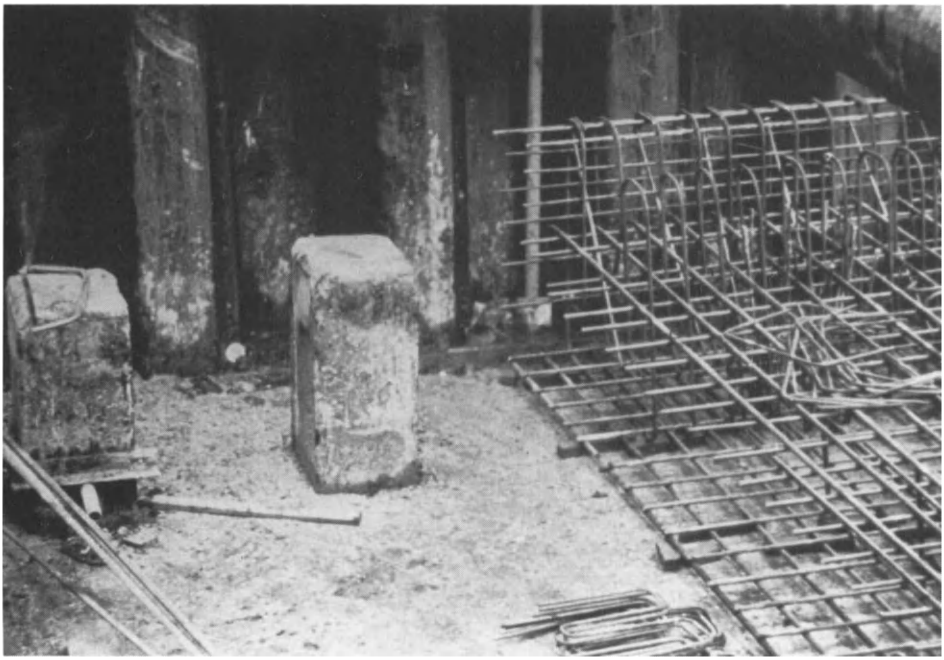

Figure 12.8 Reinforcement of a shell tunnel floor

To minimize cracking in the tunnel walls due to shrinkage and temperature changes during the setting of the concrete, in conventional tunnels poured *in situ* (first the floor, then the walls and roof) in Rotterdam, expansion joints are made approximately every 15 m. The distance between the expansion joints passing through the floor and shells in the shell tunnel is double (30 m). The precise dimension of this was also determined by the speed of advance of 30 m of tunnel per week. Because, in this case, the floor is poured *in situ* and the walls and the roof are prefabricated, cracking plays hardly any role and the distance between the expansion joints is less restricted. The ten joints between the shells within a tunnel section are made as structural joints. Consequently, the combination of floor and shells is to be regarded as a longitudinal beam on elastic supports. Statistical data from deep-drilling and cone-penetration tests show that for the loadbearing capacity and the load-deformation characteristic of the piles a variation of 10% downwards and 30% upwards must be allowed. The differences in the load-deformation character of the piles is generally corrected by an interaction between foundation and structure. For the shell tunnel it requires special attention. In order to eliminate any tensile stresses in the tunnel roof in the longitudinal direction the piles at the ends of each tunnel section are placed closer to each other. Thus there is a relatively stiffer support at the ends of the beam. When there are load variations in the longitudinal direction and variations in structural characteristics of the components of the tunnel there will always be a positive moment in the tunnel roof. This results in a continuous pressure in the roof of the shell and only tensile stresses in the tunnel floor (Figure 12.9).

For the mobile load at street level above the tunnel, the heaviest traffic load class according to Dutch regulations is adopted, namely two trucks of 600 kN.

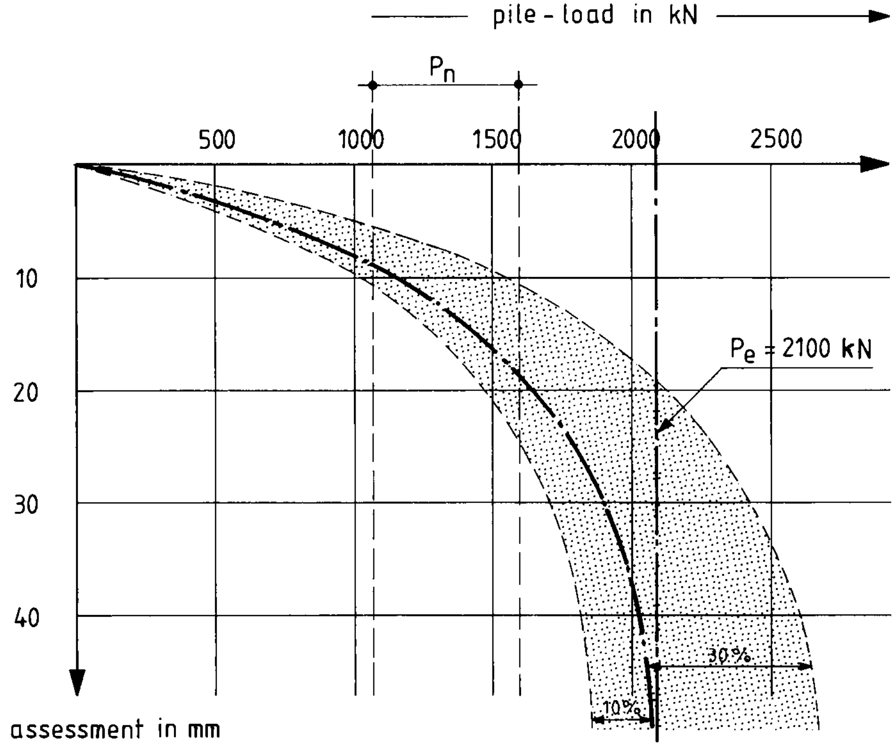

Figure 12.9 Load deformation characteristics of a pile foundation shell tunnel

12.6 Joints

The tunnel must be watertight, and this means that the joints must be resistant to a water pressure of 12 m. A 1.5 safety margin is observed. The design of the joints must be adapted to the speed of advance of tunnel building. In addition, the joints must have adequate durability and be resistant to fire and mechanical damage.

Three different types of joints can be distinguished: horizontal joints between the floor and the shell segments, vertical joints between the shell segments themselves and expansion joints which are made between the tunnel sections every 30 m. The choice of the shape of the joints and the materials used will be explained below.

12.6.1 Horizontal joint

The horizontal joint between shell and floor is formed as a rigid joint in such a way that moments and transverse and longitudinal forces can be withstood. On the floor there are two upstanding edges on each side between which the segments are placed. After all the shells for a tunnel section have been placed next to each other in the trough-shaped structure of the floor the joint must be poured (Figure 12.10). A grout or pouring mortar can be used. Research was carried out in order to test the material on water penetration, workability and watertightness of the joint.

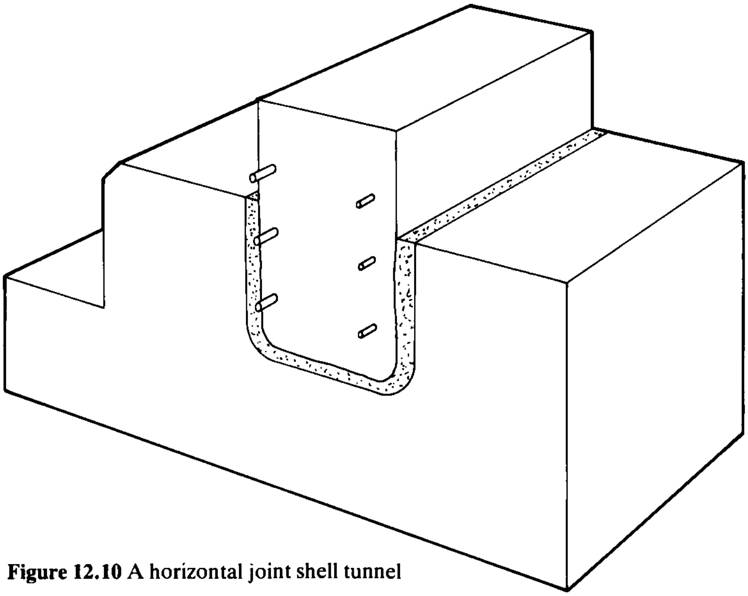

Figure 12.10 A horizontal joint shell tunnel

Some test cubes were made in moulds completely filled with the material to be tested, others were made in moulds half filled with concrete and, after hardening, filled to the top with grout or pouring mortar. By so doing, the material itself as well as the contact surfaces between the different materials could be tested for water penetration. For both materials the test gave a favourable result. After laboratory research the two materials were used in a full-scale model joint and tested for workability and watertightness of the complete joint structure.

For the grout test a material composition of 50 kg Portland cement class A, 87.5 kg sand and 1% (referred to the cement weight) plasticizer, Tricosal make, was used. The quantity of water was determined by the required workability. This must be such that the joint can be filled entirely from the inner side of the tunnel in a single pour. No air inclusions may occur. In total, five tests were carried out. In the samples the water–cement factor varied between 0.6 and 0.65. Of the five test pieces with grout filling, two appeared to be able to withstand a water pressure of 20 m. The others showed continuous leakage at 5 m water pressure. For the testing of a pouring mortar joint, the raw materials were obtained directly from the mortar manufacturer. The mortars were made up on a cement basis, a plasticizer being added in order to obtain a good degree of flow, even with a minimum quantity of mixing water.

In addition, a swelling agent was added to the mortar in order to balance the shrinkage. The swelling of the material under the influence of this was approximately 0.3%. All the full-scale joints tested appeared to be able to withstand a water pressure of 20 m.

On the basis of the test results applying the pouring mortar was chosen. In order to improve the quality of the horizontal joint the cement skin was removed from the foot of the shell and the inside of the trough structure. This took place by coating the formwork before pouring, with a retarder. After removing the

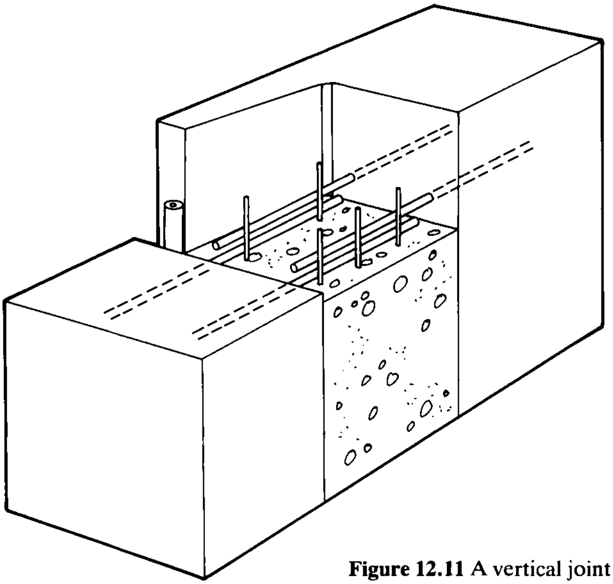

Figure 12.11 A vertical joint shell tunnel

formwork the cement skin, which was still soft, could be removed with a powerful water jet. The shell segments were placed in position with steel constructed bearings bolted to the inside of the wall of the shell and resting on the upright edges of the trough. Thus the entire filling space of the joint was kept free and could be poured easily.

12.6.2 Vertical joint

For the vertical joints between the shells, several possible solutions were tested. The criteria for the design and the assessment are, once again, watertightness at 12 m water pressure, durability, resistance to fire and mechanical damage and simplicity of construction. The entire work also had to be matched to a speed of 30 m of tunnel per week. For the design of the joint, flexible and rigid solutions were investigated. In the first, a simple joint structure was provided in which displacements and angular rotations between the segments could freely occur under the influence of loads. In rigid joints there exists a monolithic tunnel structure on which moments and perpendicular and transverse forces can be transmitted.

It is essential that the vertical joint should be watertight and also a watertight link be made with the horizontal joint. Allowance must be made for dimensional tolerances in the shells, assembly tolerances during the positioning and, although small, deformations which can still occur as a result of shrinkage and temperature and differences of settlement in the foundation. For the flexible joint, various fillings with rubber tubes and elastic plastic resin compound between the shells were considered. Tests on scale models of flexible joints and considerations of durability, protection against fire and mechanical damage and construction aspects have not led to an acceptable solution for the flexible joint. The fixed joint design between the shell segments shows a filling with concrete poured *in situ* (Figure

Figure 12.12 Full-size testing of a shell tunnel

12.11). Continuous reinforcement is provided in the joint, consisting of overlapping protruding reinforcement from both shell segments. The reinforcement is galvanized and has a diameter of 10 mm, centre-to-centre distance 100 mm, quality FeB 400. The pouring surfaces of the shells are also cleaned from their cement skin, as in the longitudinal joint.

The shells are given a 'tooth' on one side which functions as an inside formwork for the joint. The outside formwork consists of a simple wooden strip formwork which is firmly clamped to the adjacent shell segments. The rigid vertical joint was finally tested in combination with the horizontal joint at full size with good results (Figure 12.12).

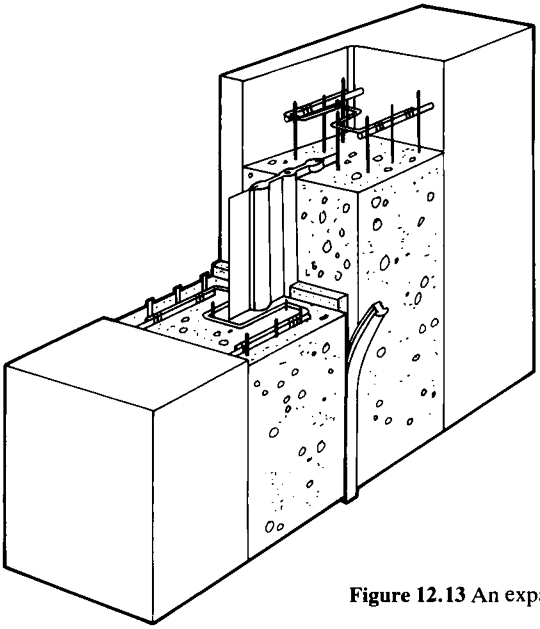

Figure 12.13 An expansion joint shell tunnel

12.6.3 Expansion joint

The combination of vertical and horizontal poured joints has determined the design principle of the expansion joint between the tunnel sections. This joint is made with a jointing seal made of rubber profile and steel strip, as is usual in tunnel building (Figure 12.13). The steel strip is cast into the floor. Before positioning the shell segments on the tunnel floor the joint strip is made continuous over the tunnel cross section by welding it in the building trench so that it can be included entirely in the end joint. As a result, the shell segments can be more or less identical.

12.7 Building trench

In the soft Rotterdam subsoil with a high groundwater table, tunnel construction in the building trench protected by sheet piles is the most suitable method. Basically, the geotechnical conditions must be known. The ground level in Rotterdam varies from approximately sea level to more than 6 m below it. The land is protected by dykes, and the upper layers consist of peat and clay to a depth of 15–17 m below ground level. The deep sand formation, the bearing layer for pile foundations, is encountered only at this level. Subsoil conditions are directly involved in the tunnel design and its feasibility. The condition of the soil does not allow tunnels to be dug freely underground, as is the case in many other cities, where there is a hard subsoil (e.g. rock and heavy clay). The method of construction results in a system of routes for tunnels which usually follow the main streets in the town. However, Rotterdam also has tunnels under existing buildings.

The road routing of the metro line influences the layout of tunnels and stations. On the deep north–south line the stations lie above the tunnel, below street level, and on the shallow east–west line the stations mostly lie beside the tunnel tube and partly above street level. For the shell tunnel in the Schiedamseweg the ground-cover amounts to 4 m at the connection with Delfshaven Station. This station is also completely underground. It is connected on the east side to the immersed Coolhaven tunnel.

Special attention must be given to the geotechnical conditions of the ground in order to prevent damage to surrounding buildings. Moreover, the subsoil along the line is very heterogeneous (Figure 12.14). At different locations varying measures

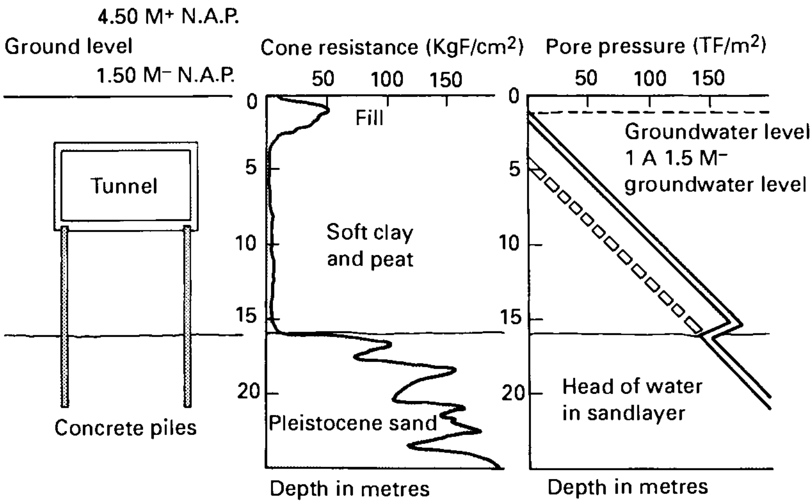

Figure 12.14 Subsoil in Rotterdam; tunneling principle, cone resistance and water pressure

are necessary to ensure vertical stability of the tunnel's building trench after excavation. The groundwater conditions are characterized by two independent water regimens: the phreatic groundwater in the upper layers is separated by impervious layers of clay and peat from the piezometric artesion groundwater. Unrestricted construction works with unlimited dewatering of the subsoil is not permitted. Sometimes lowering of the groundwater table of the upper layers and horizontal deformation of the soil next to the building trench are not acceptable.

Sometimes lowering of the piezometric level of the groundwater is still acceptable. It was possible for the east–west line. For the dry building pit of the east–west metro line different techniques were used to ensure the vertical stability of the bottom of the trench. One of these is an environment-friendly 'chemical' injection process (Figure 12.15). This involves injecting the sand layer between the sheet pile walls with a chemical compound in order to make it impermeable. The fluid compound fills the voids between the sand grains and the elevation and thickness of the injected layer is determined so that the artesian water pressure beneath is equalized by the weight of the soil strata above. The chemical compound is injected through pipes into the sand layer to the required depth.

An alternative method for ensuring the vertical stability of the bottom of the trench is the use of underwater concrete (Figure 12.16). The artesian water

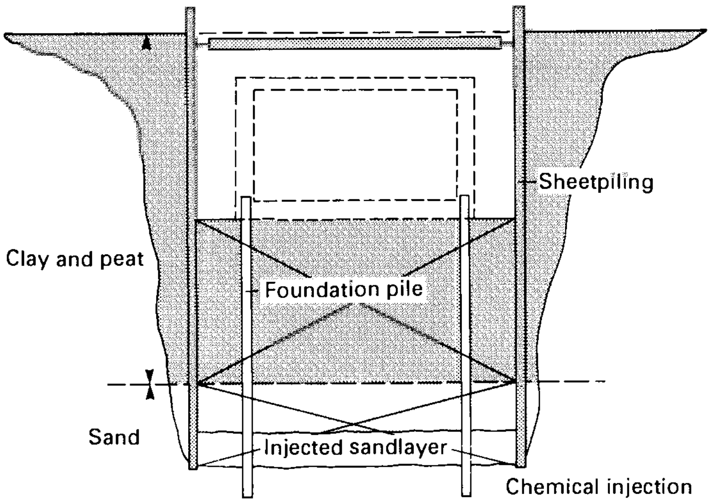

Figure 12.15 The landpart tunnel principle; chemical injection

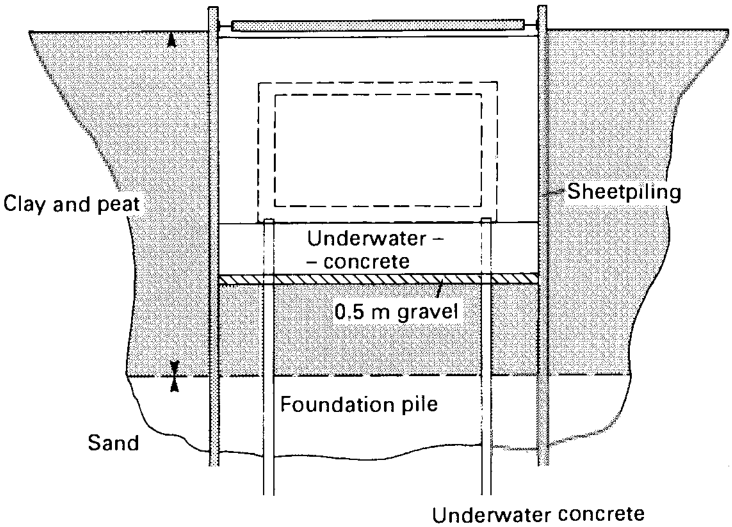

Figure 12.16 The landpart tunnel principle; underwater concrete

pressure is counteracted by the weight of the concrete and the relatively heavy concrete base is constructed after the foundation piles have been driven. If necessary, the underwater concrete can be reinforced. As soon as the concrete has hardened sufficiently, the water can be pumped out of the trench. This method was used, among others, at the above-mentioned Delfshaven station. The piezometric level of the water can also be lowered without causing damage to surrounding buildings by discharging the pumped water back into the deep sand layer immediately outside the excavation via a recharging wellpoint system. Usually, when lowering the artesian head of water is necessary a single method or a combination of different ones has been chosen to avoid damage to the surroundings. For the shell tunnel both injection of the subsoil and the recharging wellpoint system have been used (Figure 12.17). In some places it was possible to make only a small degree of drainage of the groundwater without additional measures.

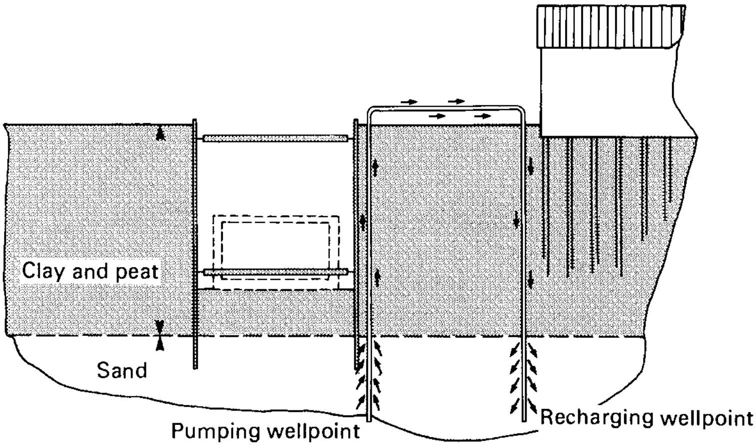

Figure 12.17 The landpart tunnel principle; a recharging wellpoint system

In the case of the shell tunnel, at a particular phase in the execution a surface load of $110\,kN/m^2$ must be assumed near the building trench. This is due to the transporting of the shells to the site immediately alongside the trench. The consequences of this relatively high surface loading are limited to stress concentration in the upper struts and girders of the sheet pile wall. Due to load spreading with depth, the extra horizontal load on the sheet piling as a result of the weight of the truck loader and shell segment may be disregarded. In addition to their resistance function, the sheet piles were also designed for a vertical load. It comes from the crane and the traversing cars used for the construction of the tunnel which were running on a rail mounted on the top of the sheet pile wall. With the underside of the sheet piles in the deep sand the vertical loadbearing capacity was adequate.

12.8 Construction

The width of the shell segments measured along the tunnel axis is fixed at 3 m on the basis of the following. Transport from the factory to the building site must take place during the day within normal working hours (Figure 12.18). This is possible with a 3 m shell mounted in an upright position on a flatbed truck and escorted by police. The height of such a shell transport (approximately 5.5 m) causes no problems along the route. Dimensional restrictions also apply to the positioning of the shells in the trench. The distance of the upper struts of the trench is 6 m. With a 3 m shell the struts have not to be removed while manoeuvring the shell segment from the truck transporter to its position in the trench (Figure 12.19).

Figure 12.18 Handling a shell segment from a truck transporter in Schiedamseweg

The actual tunnel construction is 30 m per week. A continuous building process of sequential activities with a maximum length of 500 m 'wandered' along the route. From the taking up of the road to the renewal of the road surface, 1000 m of tunnel could easily be built in a year. The 'passage time' per point along the route for all tunnel activities was four months.

The tunnels in the 's-Gravenweg and Schiedamseweg were built at different times and by different contractors. For both, the shell segments were prefabricated outside Rotterdam in specially equipped factories and transported by water to Rotterdam. The segments for the 's-Gravenweg tunnel were made in the province of Zeeland on the same site (Kats) as used for the large prefabricated segments of the Zeeland bridge and later the Eastern Seheldt stormsurge barrier (Figure 12.20). The distance to Rotterdam is approximately 90 km.

For the Schiedamseweg shell tunnel the segments were made in a factory in Utrecht. The distance to Rotterdam is approximately 50 km. The advantage of producing the segments in existing factories was experienced personnel, the

Figure 12.19 Manoeuvring a shell segment into the tunnel trench in Schiedamseweg

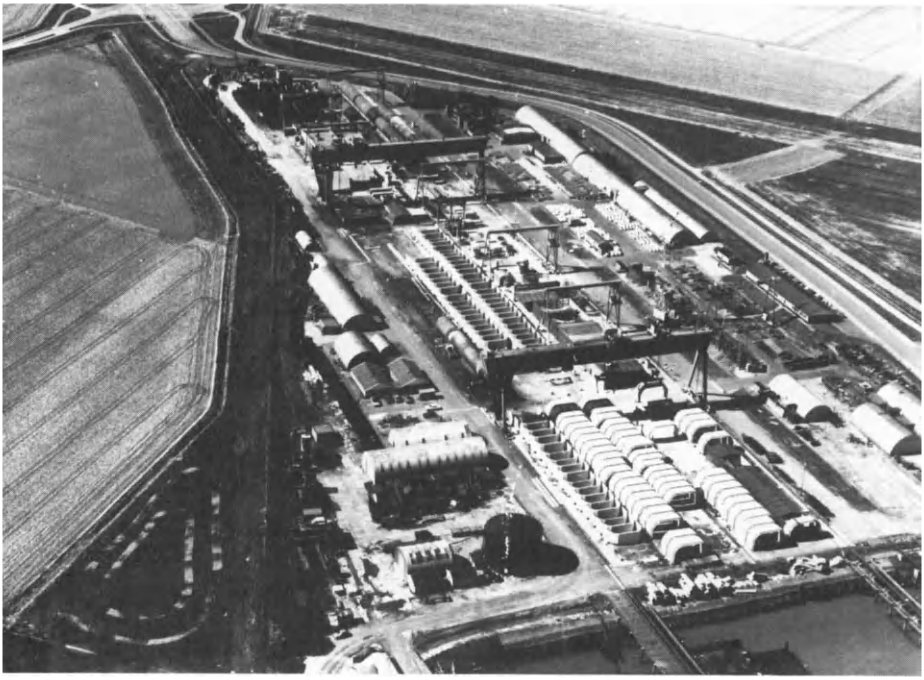

Figure 12.20 The prefabrication site at Kats

Figure 12.21 Transport of shell segments

availability of technical knowhow in the field of large concrete segments, storage capacity and adequate equipment capacity for loading the heavy segments for transport. The extra cost for the long route to Rotterdam were compensated by this. From the factories the segments were taken to Rotterdam by ship and from there by trucks to the building site (Figure 12.21).

Prefabrication of the shell segments in the factory for the 's-Gravenweg tunnel was based on production of two segments a day. These were prefabricated sheltered from the weather in a simple shed from which the roof could be removed. Six sets of formwork were made, consisting of steel frames covered with wooden formwork panels and six formwork moulds were chosen in order to achieve a cycle time of three days per unit. The segments were lifted 2.5 days after hardening. The required strength of 18 kN/mm^2 was achieved by using a concrete with a start temperature of 15°C. The temperature in the shed was also kept at 15°C. For the Schiedamseweg tunnel one segment a day was made in the factory using two steel moulds. Manufacturing time was relatively more generous for this work than for the 's-Gravenweg. In general, no additional steam heating was needed for hardening the concrete. The segments were poured on their side in the factory, the tooth structure for the vertical joints now on the underside of the mould. In this way good quality in this part of the concrete was also possible. However, the segments had to be tilted a few weeks after manufacture. With the equipment on the building site this caused no problems (Figures 12.22 and 12.23).

The production of the segment was as follows. At the production site the previous segment was removed from the mould in the morning. Then the formwork

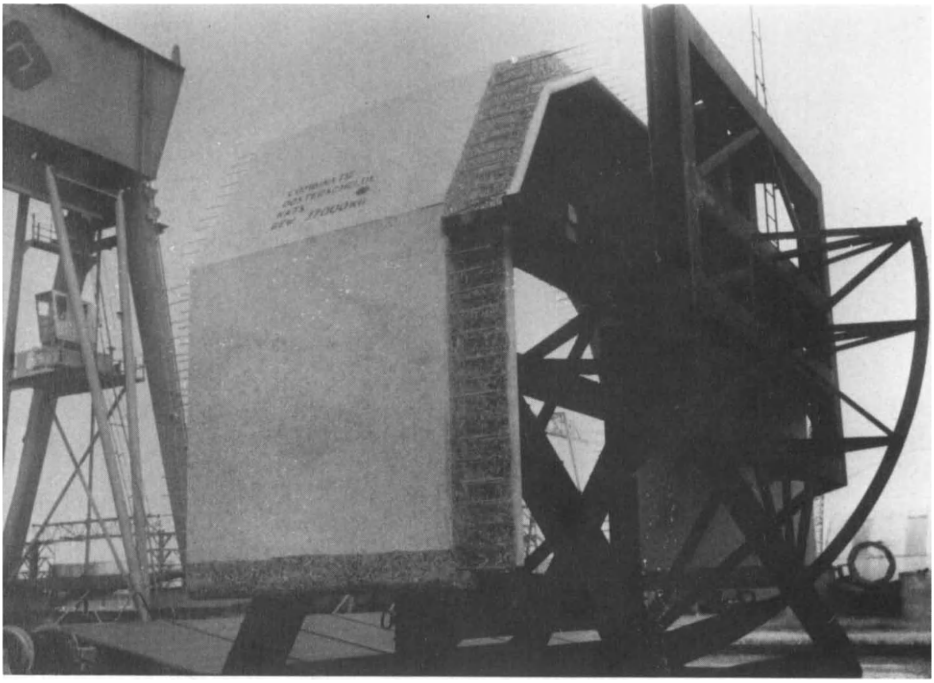

Figure 12.22 Vertical position of a shell segment at the prefabrication site

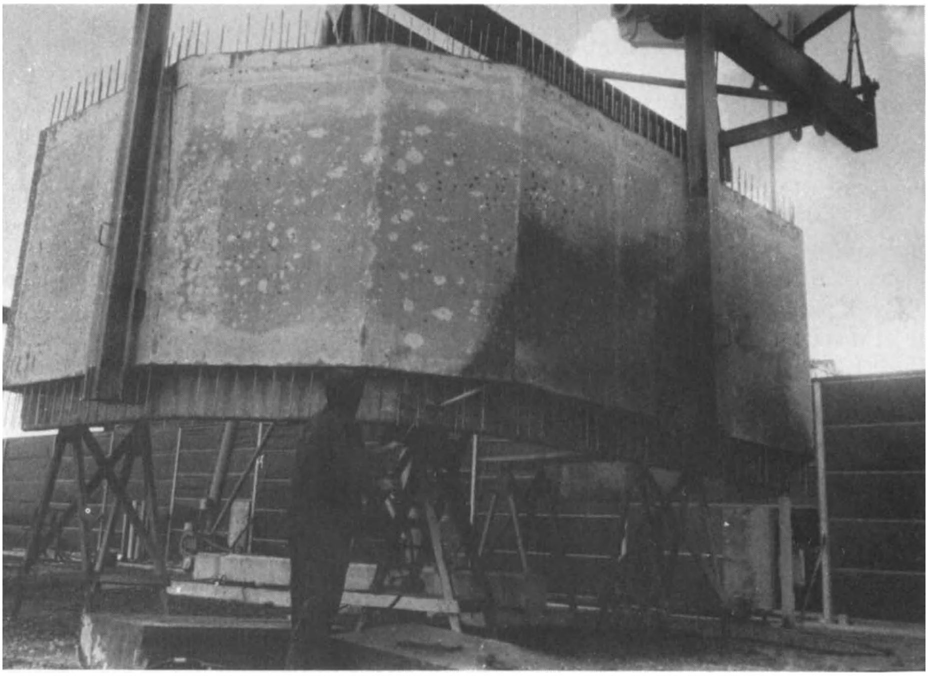

Figure 12.23 Horizontal position of a shell segment at the prefabrication site

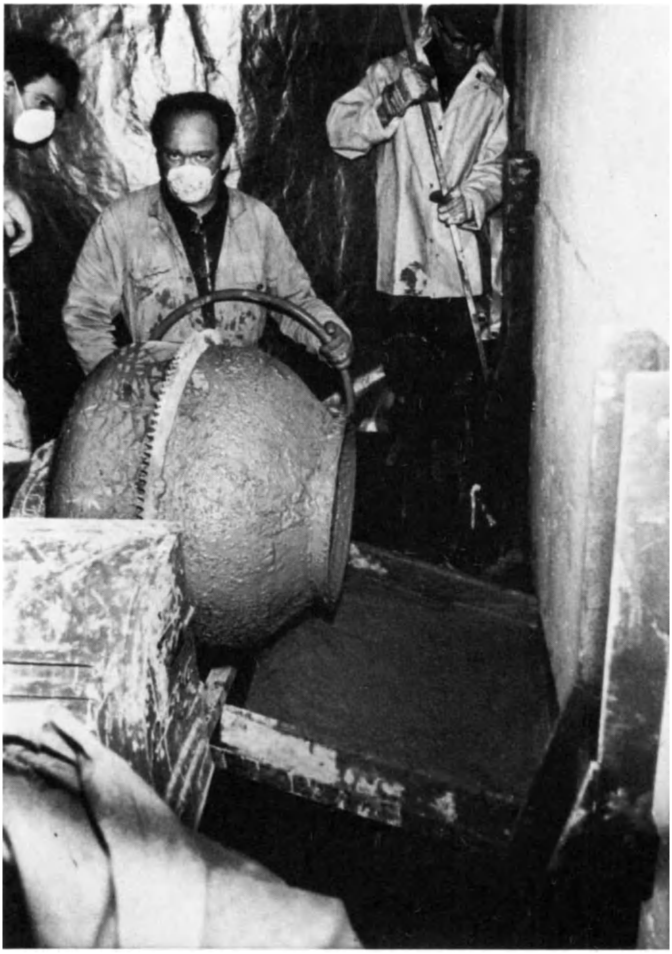

Figure 12.24 Finishing a vertical joint shell segment

was cleaned and oiled and the reinforcing cage prefabricated in a steel fixing shed was placed in the mould. The centering of the two halves of the mould took place on the underside and the top of the mould, outside the concrete cross section. When the concrete for the segments had been poured, compaction took place by vibrating motors, mounted on the formwork. After removing the mould, the segment was lifted with a special frame and put into storage. A few weeks later it was placed in a tilting frame and brought to the vertical position. In lots of ten (the number necessary for one tunnel section of 30 m) the segments were placed on a pontoon and taken to Rotterdam on a specially constructed framework with a three-point support. Loading the pontoon lasted one day, as does the journey from the factory to Rotterdam. One day was also needed for transport of the shell segments into the trench. The rest of the week was used to finish the joint (Figures 12.24 and 12.25).

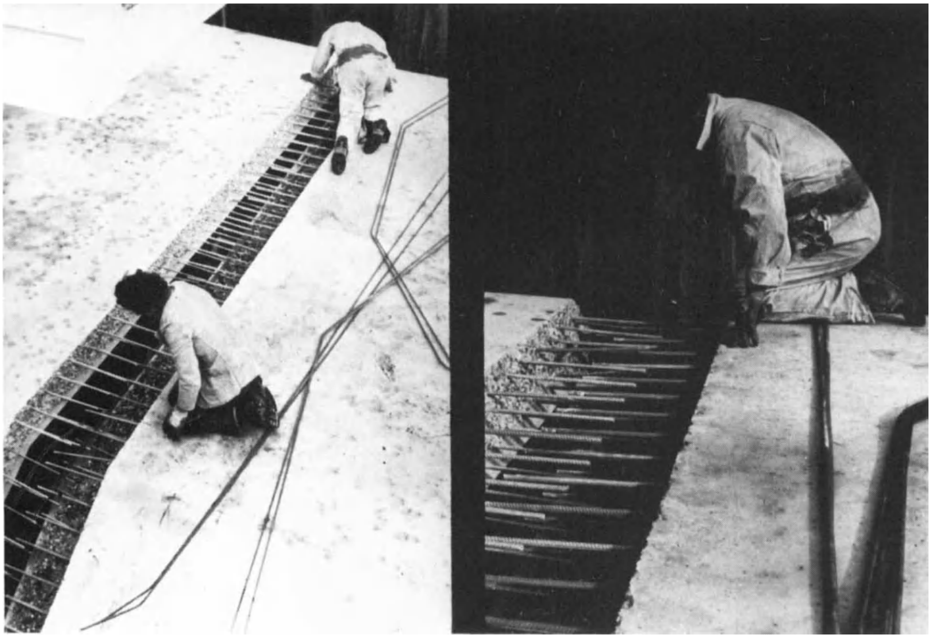

Figure 12.25 Finishing a horizontal joint shell segment

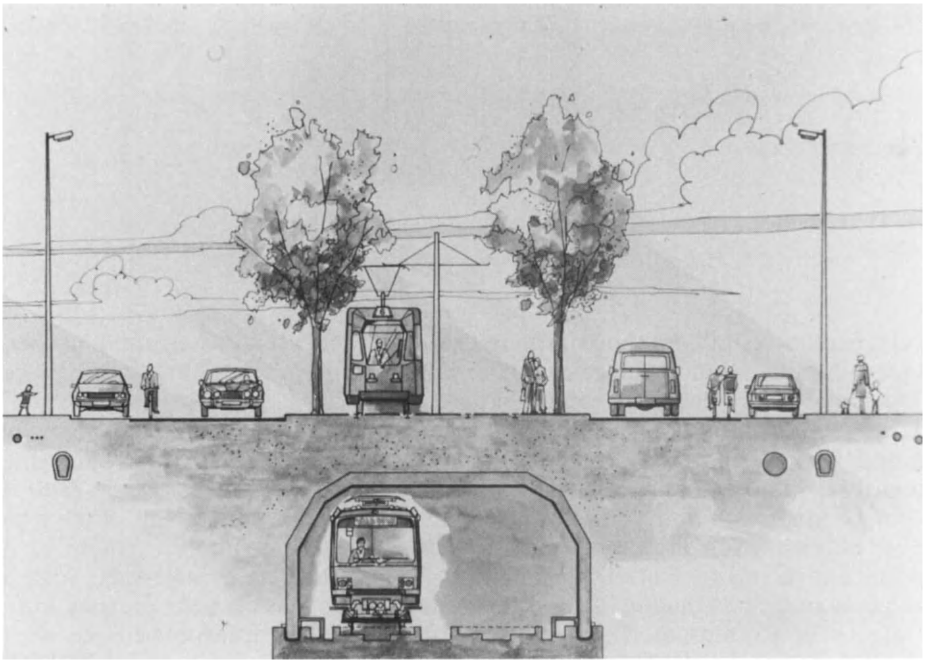

Figure 12.26 The final situation in a shell tunnel; the metro in service

12.9 Conclusion

The shell tunnel is one of many techniques applied in Rotterdam (Figure 12.26). In the present routes for the metro line it also appears to be the most suitable solution in respect of minimum effect on the environment. Prefabrication makes it possible to achieve a high speed of tunnel construction. The tunnel, which has now been in service for several years, remains perfectly watertight. As far as costs are concerned, the building costs of the shell tunnel are the same as those of traditional tunnel construction in an open trench. Detailed cost figures are not relevant; they differ for each part of the line and are subject to changes in the national price index.

Chapter 13

Tunnels in soil and weak rock

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Contents

13.1 The selection of tunnelling method

13.1.1 Scope of soft-ground tunnelling

To the tunneller, there is no sharp distinction between rocks and soils. At depth, a mudrock will deform around a tunnel in a manner and to a degree comparable with that of a clay at shallower depth. A necessary feature of tunnelling in soil and weak rock – referred to subsequently as soft-ground tunnelling – is that exposed ground will not stand indefinitely without support. The physical process of breaking up the ground does not normally present particular difficulties; the problems arise from the need to exclude the ground – and groundwater where present – from the excavated void until an economic and expeditious form of permanent structural support is in place.

13.1.2 Necessary geological data

The first object of this chapter is to guide the reader in an elementary way towards the criteria for selection of the system of tunnelling appropriate to a particular type (or, more usually, a succession or mixture of types) of soil. A primary essential is to have a broad understanding of the relevant geology. This will include a brief geological history of the deposits, their strengths and permeabilities, with the pattern of variability, history of tectonic and other disturbances, knowledge of water tables and their variations, presence of swelling or chemically active constituents. In particular circumstances, e.g. for soils underlain by Karstic rocks (magnesian limestone rocks perforated as a result of chemical action by water percolating joints), the geological survey may need to be extended well beyond the zone to be tunnelled.

The use of one or more experimental shafts, headings or tunnels may be justified:

1. To allay apprehensions about the ground;
2. To permit development and demonstration of an economic innovative form of tunnelling;
3. To permit alternative schemes to be demonstrated and monitored.

There is a need to ensure that experimental excavations explore ground and groundwater in the circumstances adequately representing those to be encountered in the project. It has already been emphasized, for example, that a weak rock near the surface may react, on excavation, differently to the same ground at greater depth.

13.1.3 The two forms of soft-ground tunnelling – informal and formal support

Soft-ground tunnelling may be divided into two different methods: that leading (usually via an intermediate form of support) to an *in-situ* form of lining and that in which excavation leads directly to the final lining, usually constructed in preformed (usually segmental) elements. For brevity, these two methods are designated as 'informal support' and 'formal support', respectively. A segmental lining as formal support will usually be erected immediately behind a shield or (shielded) tunnel-boring machine (TBM) – which thereby provides a form of travelling intermediate support – while informal support will usually utilize less specialized excavation plant, of size and type to suit the particular demand (although there are examples based on the use of a shielded TBM). Formal support also includes the special proprietary system whereby an *in-situ* lining is formed immediately behind the TBM; such linings are classified with formal support (Chapter 14) since the techniques have much in common, the permanent lining providing in each instance the initial ground support of invariable internal geometry with no intermediate process.

13.1.4 Factors affecting choice between methods of tunnelling

The choice between the methods of informal and formal support will include factors other than soil characteristics, such as:

1. The shield and segmental lining will determine a cylindrical (usually but not invariably circular) tunnel whereas informal support with an *in-situ* lining will permit greater diversity and variability in shape, consistent with overall stability.
2. A segmental lining requires greater initial outlay but is likely to achieve higher construction speeds than an informal support system (e.g. in suitable circumstances 500–1000 m/month against, say, 100–200 m/month). Thus a segmental lining will tend to be favoured for long lengths of tunnel of constant cross section.
3. Ground settlements may more readily be controlled by the use of a suitable form of segmental lining. Where it is essential to maintain groundwater levels – often necessary to limit consequential ground settlement – a segmental lining may be required. Such considerations for tunnelling beneath a town may well predominate over any other feature.

Tunnelling costs depend essentially on the nature of the ground and may well vary, for long lengths of tunnel, by a factor of ten or more. The factor may be greater for short lengths of tunnel and where special expedients (Section 13.1.6) are required. Estimates of tunnelling costs should never be attempted without knowledge of the nature of the ground and its probable incidence upon the scheme of tunnelling and rate of advance. When comparing achieved costs of tunnels which make use of formal or of informal means of support it must be appreciated that the former are capable of adaptation to the poorest ground which would necessitate great expenditure on special expedients before being rendered suitable for tunnelling with informal support.

13.1.5 Ground settlement

Most of the published accounts of settlement caused by tunnelling relate to tunnels with formal linings (Attewell *et al.*[1]) while Muir Wood[2] has described the factors contributory to settlement. In uniform soft ground, a settlement trough is experienced at the surface, represented as a normal probability curve with the transverse horizontal distance from tunnel axis to the point of inflexion (i), expressed in terms of tunnel depth (z). The area of the settlement trough may often be approximately equated to the volume of ground loss per unit length. The ratio i/z is dependent principally on the nature of ground, being less for a sand than for a clay, for example Figure 13.1.

The magnitude of settlement is expressed as a percentage dA/A, where A is the excavated area of the tunnel and dA the area of the surface transverse settlement trough. Depending on the circumstances, dA/A may vary between 0.2% and 5% for well-controlled tunnelling[1].

Time-dependent settlement will be caused largely by consolidation on account of drainage into the tunnel. Swelling of the ground around the tunnel may be an additional indirect factor. Where tunnelling leads to lowering of the water table (which may follow, in certain soils, from remarkably low inflow to the tunnel), the seriousness of the consequences depends on the nature of the soils affected. For example, organic soils (e.g. peat) and normally consolidated clays are especially susceptible to settlement of this nature. Depending on the particular circumstances, time-dependent settlement may be localized or may occur over a wider area than immediate settlement.

Due to the differences in nature of immediate and long-term settlement, attempts should not be made to express the latter (in magnitude or distribution, except for familiar situations) as a direct factor of the former.

Ground settlement and heave may result from ground treatments. Settlement has been experienced as a result of disturbance by drilling of loose granular soils; heave has resulted from the use of excessive pressures in soil grouting (especially jet grouting) and from the formation of ice lenses during freezing.

13.1.6 The use of special expedients

If the ground has an open texture and is below the water table, the tunnelling system must incorporate expedients designed to exclude the water continuously, since any water entering the tunnel may bring with it fine-grained soil fractions, leading to cavities and subsequent collapse of the ground. Exclusion of groundwater may be achieved, separately or in combination, by some form of ground treatment (grouting or freezing – see Chapter 4), by using a closed-face pressure-balancing TBM (Chapter 15) with a segmental lining or, possibly, by constructing the tunnel in compressed air. Where ground treatment is to be used, considerations of tunnelling capabilities discussed below should then be applied to the properties of the treated ground, making allowance for the predicted degree of success. Where the success of tunnelling critically depends on the success of ground treatments serious consideration should be given to a form of warranty. (If the quoted cost is excessive, the extent of doubt on behalf of the specialist is apparent.)

13.1.7 Simple geotechnical criteria

The simplest expression of the behaviour characteristics of the ground, which mainly determine the choice of appropriate system of tunnelling, are:

1. Stand-up time; and
2. Stability factor (N_s), the ratio of total overburden stress, at the tunnel level, to soil shear strength.

Stand-up time

Stand-up time requires explanation. As a tunnel is advanced, release of stress in saturated soil in the vicinity of the face leads to a tendency for volumetric expansion of the soil, restrained by the relative incompressibility of the pore water. This in turn gives rise to a *reduction* in pore water pressure and a consequent tendency to maintain effective stress existing prior to excavation and, hence, the pre-existing soil strength which depends on the effective stresses. The pore pressure gradient associated with local reduction of pressure causes accelerated flow of water towards the tunnel and this gradient, in course of time, leads towards a partial restoration of the original pore pressure. As this process occurs, effective stresses decrease with progressive reduction to the safety factor against collapse; 'stand-up time' represents the period which elapses prior to reduction of strength to a degree to provoke ground collapse in the tunnel face.

It will be understood that appreciable stand-up time (for a tunnel below the water table) depends on low-permeability soil (say, 10^{-8} m/s or lower). In fact the controlling soil parameter is the coefficient of consolidation (c_v), which is defined as the coefficient of permeability divided by the coefficient of volumetric compressibility.

Stand-up time is a qualitative measure; not only does it depend on initial condition of soil stress, pore pressure and proximity to aquifers but also on the rate of advance of the tunnel. The more rapidly the tunnel face is advanced, the longer the stand-up time, since the lesser the degree to which original pore pressures will be permitted to recover.

From this association follows the much-observed fact that face collapses are more likely to attend a stopped than an advancing tunnel face. Tunnelling without a shield is only possible where stand-up time is adequate, unless resort is to be made to face support, e.g. by spiling, breast boards or local groundwater lowering.

The above explanation of stand-up time refers to a soil considered as a continuum; stand-up time for open-jointed rock (a discontinuum) requires an alternative explanation (see Chapter 17).

Stability ratio

The stability ratio or stability factor N_s (referred to by some engineers as the 'simple overload factor') – see Figure 13.1 – is defined as:

$$N_s = (\gamma z_o (+q) - p_i)/c_u \tag{13.1}$$

where

γ = unit weight of soil,
q = surface surcharge pressure (if any),
p_i = internal support pressure in tunnel,
c_u = undrained shear strength of soil,
z_o = depth of tunnel.

The stability ratio is another dominant factor which will determine whether a tunnel may be advanced without continuous support at the face. As a first approximation a shielded tunnel may be advanced without continuous face support for a value of $N_s < 6$ while advance of a tunnel without immediate support of the periphery of the tunnel will only be practicable where $N_s < 4$ or thereabouts; the extent of unsupported length will be related to the value of N_s. In reality, the acceptable value of N_s must also depend on factors affecting stand-up time considered above.

The consequence of such criteria for N_s is to set a limit for a soil of particular undrained strength upon the depth of a tunnel if face stability is to be ensured in the absence of support such as a shield. In the absence of provisions to lower the water table, the stand-up time criterion confines the use of an informal support system in soft ground below the water table to a silty clay, clay or a partially cemented

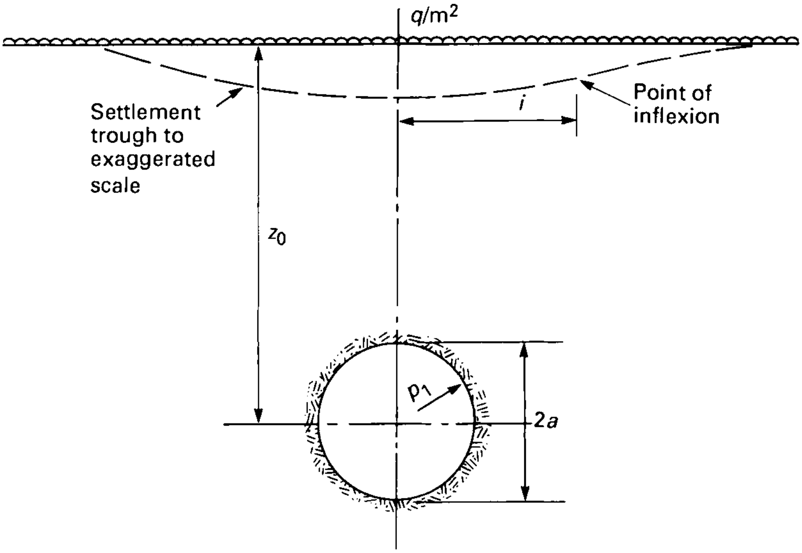

Figure 13.1 Definition sketch for stability ratio

mudrock. It is particularly important to note that stand-up time depends on length of water path from the nearest aquifer; increasing proximity to an aquifer, possibly a relatively insignificant water-bearing lens of soil, should be regarded as a possible hazard where the face is not continuously supported.

13.1.8 Proximity of tunnels: tunnel junctions

The question of the safe and economic spacing between parallel tunnels is one for the specialist. The first consideration concerns the stability ratio N_s, the second that of the practicability of internal strutting of the first tunnel during the intermediate construction phases of the passage of the second. A third consideration is that of total settlement (Section 13.1.5) which will be found generally greater for two closely situated tunnels than the sum of that for each individually [1].

The first tunnel causes a weakening of the adjacent soil, to a system of principal stresses different from that associated with the second tunnel, so initial soil dilatation (i.e. volumetric expansion) may be followed by contraction of the same soil. A first criterion is to consider the adequacy of the dividing pillar of soil to support the total ground load between axes of parallel tunnels at the same level (or the comparable loading for tunnels at different levels). Reduced lateral support provided by this pillar, by comparison with solid ground, will affect the overall deformation and convergence of the tunnels, particularly the first constructed. For a tunnel with formal lining, in particular, design of the pillar must take account of the lateral thrust of the lining of the first tunnel as it tends to deform in consequence of excavation for the second tunnel. As a first rule of thumb, tunnels separated by at least a half-diameter (of the larger tunnel if of different sizes) will not usually need costly additional provisions in construction. Figure-of-eight tunnels have been built for special purposes, sharing an intermediate support, and a 'binocular' shield has been developed in Japan for driving such tunnels, where this form is imperative. Where tunnels are converging rapidly (for example, approaching a junction or a station) the support problem is reduced to that of a short length and, with appropriate precautions, the tunnels may be brought close to mutual contact.

Many designs of tunnel junction have been devised to suit the particular forms of lining; details are often determined to ensure watertightness. Where practicable, junctions should be transverse, to reduce needs for extra support. The concentration of ground loading determines the structural design of the junction [2].

13.1.9 Procedure and contractual basis

The selection of the system of tunnelling must take account of the procedures to be adopted and the appropriate basis for contractual relationships. Where the promoter places all tunnelling risk on the contractor there should be no surprise that the latter needs to base fixed costs on assuming 'the worst of all possible underworlds'!

If the promoter is to obtain the optimal conjunction of low cost and low uncertainty of variability of cost, the object must be to place responsibility for each cause for uncertainty with the party to the contract best able to exercise control [3]. If this aim is to be achieved, there must be provision for continuity in developing the tunnelling scheme; there is no point in time which designates a clear frontier between planning and design or between design and construction. The first should taper into the second into the third. For soft-ground tunnelling in general, attempts at mutually insulating the several stages of studies, investigations, design and construction invariably lead to lack of interaction between the several functions, to extra cost, excessive paperwork and, often, as a result of inflexibility of the approach, expensive litigation between numerous parties.

This is a supreme example of the expensive consequences of short-term accountancy. Continuity in the engineering should lead to an ability to think the design through into acceptable methods of construction, to enable advantageous modifications to be made as experience is gained and to provide for payment for unexpected features to be settled simply and equitably through the exercise of informed engineering judgement.

Where the engineering is undertaken in a fragmented manner on a competitive price basis, with price of this element the main criterion rather than quality and value for money overall, the scope for the exercise of foresight – which is the hallmark of good engineering and which brings important economic benefits to a project – will be much diminished.

For a formal lining, the contract should take account of:

1. The lining design, serving as a basis for tender, which may desirably be modified to suit contractors' procedures with a consequent saving in cost;
2. Alternatively, for a design-and-build contract, a full performance design basis must be provided; performance must be for the life of the project;
3. The merit of encouraging an economic innovative system of tunnelling, possibly needing slight modification as experience accumulates; an appropriate basis for adjustments must then be provided;
4. Where purpose-built machines are to be used, to ensure that all parties have a common understanding of the conditions for successful operation.

In drawing up the contract for an informal lining:

1. It will normally be desirable to 'zone' the ground (see Section 13.2.4) in relation to support needs or to describe how much 'zoning' will be developed from experience with support systems in the early stages of the contract;
2. Thereafter to exercise judgement in the demarcation of zones in the light of experience, determining as a result of monitoring the need, if any, to supplement support prior to the finished lining;
3. Payment must be related to needs for support if full benefit is to be derived from the informal system.

13.2 Soft-ground tunnels with *in-situ* linings (informal support)

13.2.1 Simple analysis

As already discussed (Section 13.1.3), an essential feature of tunnels with informal support concerns the need for ground support in advance of the lining proper. This support may be provided by 'passive' means such as ribs, arches, crown bars and polings or by 'active' means represented by shotcrete, rock bolts and anchors. The difference betweeen the two types stems from the rigidity of the support, best illustrated by a simple convergence–confinement diagram such as Figure 13.2. In

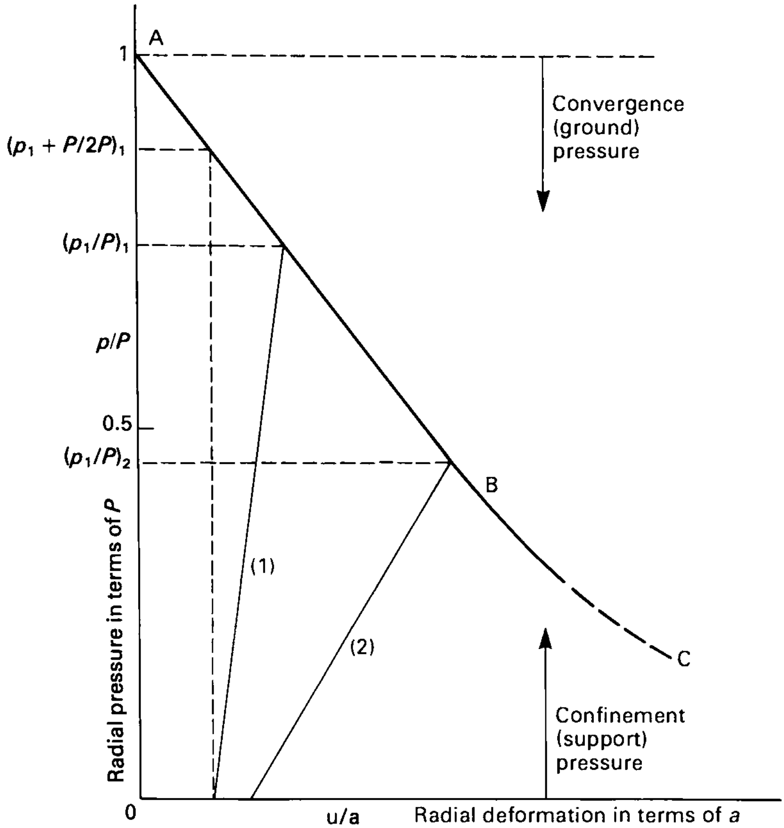

Figure 13.2 Convergence–confinement diagram

this figure a long length of circular tunnel is considered as being in homogeneous ground in a uniform state of stress. The curve represents radial ground stress for stability (as vertical ordinate) plotted against inward radial movement, known as convergence, as abscissa. The curve is known as the convergence line. Ordinate and abscissa are plotted non-dimensionally, the former as a factor of original ground stress, the latter as convergence expressed as inward movement divided by radius, u/a. A rigid support placed at the moment of excavation is represented as the confinement line (1), the slope of the line relating to equivalent radial support stiffness under load. Alternatively, a more flexible support (2) is shown as placed at a selected time after excavation so that a considerable degree of convergence may occur before, and continue after, placing of the support. Such a diagram may be developed to include orthotropic properties of the ground, e.g. where vertical stiffness is different from horizontal stiffness, and the effects of time. The most important feature of Figure 13.2 is the reduction of strength required of support (2) in relation to support (1).

For a tunnel in weak ground, the conceptual convergence line starting from point A in Figure 13.2 is linear (i.e. the process is an elastic one) and subsequently begins to curve convex upwards. For most soft ground, even this degree of elaboration is not justified. The relaxation curve for radial convergence of an advancing tunnel in elastic ground assumes a form illustrated by Figure 13.3. Depending on the degree of axial support of the face, about 50% of the convergence has already occurred ahead of the face. Hence, for support reckoned as providing a uniform internal pressure p_i, supports erected immediately at the face have their origins (Figure

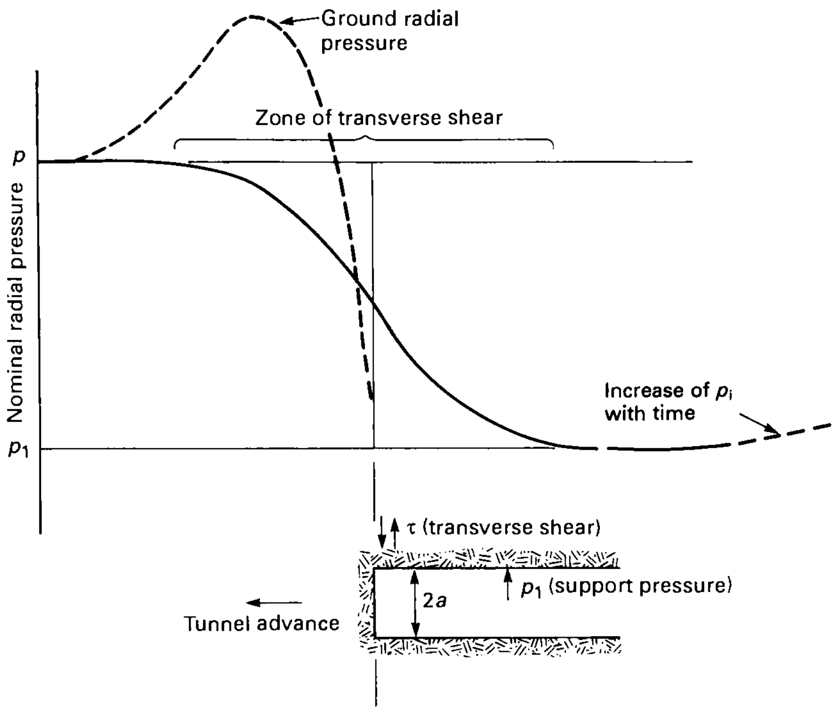

Figure 13.3 Radial convergence in vicinity of tunnel face

13.2) at a point corresponding in time to ground stress of $(P + p_i)/2$, where P is original ground stress at tunnel level.

In practice, there is scope for a considerable measure of judgement as to the extent to which analysis and modelling need to be pursued to result in a safe and economic form of construction.

The engineer may expect to be able to derive benefits in economy and security by adopting an increasing degree of rationality in approach; this must be the justification for so doing. The question that needs constantly to be asked is whether consistency and homogeneity of the ground permit its behaviour to be reliably represented by samples or tests made at relatively distantly spaced intervals. Where the soil is expected to vary, and where this variability does not conform to a predictable pattern nor lend itself to zoning (see Section 13.2.4), there is no object in pursuing a more complex approach to design than is justified by the reliability of the data. Where a soil is variable and the relationships between stress and strain may be expressed in statistical terms, a more complex approach may be justified, aiming for an adequate safety factor at all times, possibly taking account of the beneficial effects of the third dimension.

13.2.2 The observational method and incremental support

A fundamental option in the design of procedures for tunnel support depends on whether:

1. Initial provision of support is to be adequate until the final *in-situ* lining is in place; or
2. Whether criteria are to be adopted whereby initial support may be supplemented, dependent on observation of behaviour of the tunnel.

Procedure 1 may be considered the traditional approach to informal support, based predominantly on timbering and steel ribs. In soft ground the support is designed initially for full ground load or a high fraction (Figure 13.2(1)), with any question of additional support considered as exceptional and only necessary to deal with evidence of incipient collapse. Procedure 2 may be seen as a probable accompaniment to limiting initial support needs (Figure 13.2(2)). Part of such an approach is to establish how the adequacy of support may be assured prior to any inadequacy leading to an irreversible condition of incipient collapse. In simple terms, we need to ensure satisfactory junction between convergence and confinement lines (Figure 13.2), taking account of time-dependent behaviour of the ground and the support, until such time as the final lining accepts a share of ground load support. This is the approach of 'incremental support based on the observational method', for which the term 'incremental support' is used subsequently. The concept equates to that of the New Austrian Tunnelling Method (NATM) [4], but the term NATM has been so grossly misused by those who do not understand the underlying philosophy as to be positively misleading at present, being used by many as synonymous with 'informal support' in general, which it is not. The use of incremental support is more general for tunnels in jointed rock than in soils. Much development of systems has proceeded in Scandinavia and Switzerland, in Austria and in the 1950s–1960s in tunnels for the Snowy Mountains Hydro-Electric project in New South Wales.

Adoption of incremental support requires a fair degree of confidence in the behaviour of the ground, since the essence of the system is to use the load-bearing capacity of the ground to the greatest practicable degree and to ensure that failure mechanisms will give warning of their development. In consequence, another desirable feature of incremental support is that the support/ground system will ensure steady yield and not fail in a sudden brittle mode. Certain types of support (e.g. ribs or shotcrete) will only carry ground load as a result of convergence of the ground subsequent to their emplacement. Rock bolts, on the other hand, will make an immediate contribution on being stressed, and it is essential to ensure that subsequent strains in the soil along the bolt are compatible with longitudinal extensions of the bolt, including the effects of consequent slippage along the bolt which may reduce its effectiveness. There are different strategies to be followed in rock bolting to ensure that slippage does not develop to bond failure.

The need for characteristic curves of convergence against time must be understood, since these will serve as indicators of the nature and timing for incremental support. In interpreting curves of convergence against time the two principal factors concern:

1. Proximity to the face; and
2. Elapse of time.

Close to the face, the degree of convergence will be modified as a result of radial support by the solid ground ahead of the face (Figure 13.3). Well away from the face, convergence will be time dependent, related to swelling of the adjacent ground (see Section 13.1.7). Extrapolation of convergence/time curves needs to consider these two influences separately. For steady rates of advance for a particular tunnel, characteristic curves are developed from observations at several points so that a learning process enables areas of incipient instability to be identified in good time. Generally, the proximity of the face has negligible effect on convergence at a distance of more than three diameters of the excavation [5].

13.2.3 Characteristics of the real ground

A high degree of simplification needs to be made for useful analysis of a soil to be undertaken for tunnelling purposes, but there are certain important features which may need to be considered explicitly. The geological history will determine the initial state of stress tensor. For example, where appreciable surface erosion has occurred during the Quaternary, horizontal stresses in the ground may be appreciably greater than vertical stress. Such a feature will affect patterns of stress change in the ground as a result of tunnelling and the consequent criteria for stability.

Properties of sedimentary soil may be strongly orthotropic on account of its layered texture; the properties parallel to the bedding will be different from those across the bedding. For example, a soil with alternate horizontal layers of clay and silt will be more compressible vertically than horizontally, more permeable horizontally than vertically.

The orthotropic consequences may be considerable, particularly where combined with high initial horizontal stress. One consequence of such orthotropic stress/strain properties is that local stress and convergence will not be uniform around the tunnel. The orthotropic variability in permeability will also affect the time-dependent convergence, resulting from uneven consolidation of the soil around the tunnel.

While much has been written about the basis for design of tunnel supports, for soils the most important feature concerns the containment of the soil around the tunnel so that a continuous arch is formed. The question of delaying initial support does not arise; by the time the theoretical benefit of minimizing the confining stress were to be achieved (Figure 13.2), minor local weaknesses in the soil would lead to incipient collapse. Economy promotes the ability to limit the need for application of further support in increments, while ensuring the possibility of so doing, without excessive cost of initial support.

13.2.4 Zoning the ground for support types

A common expedient is to zone the ground along the tunnel, the zones representing qualitative or quantitative statements on the minimum expected support needs. Initially, the zones may be estimated on known information and geological inference. As tunnelling advances, zoning may be further developed and modified as a result of monitoring and, possibly, of *in-situ* testing ahead of the face.

A zone will be deemed to require particular patterns of support; acceptability of the adequacy of initial support will depend on comparison of the time-based curves of convergence, initially against design, subsequently against design modified by experience. The fundamental requirement is that the rate of convergence should be reducing sufficiently rapidly to establish equilibrium prior to overstress of supports or to conflict with other criteria. Again, it must be emphasized that, with a soil, strain softening may lead to the development of stress concentrations, so that excessive magnitudes of convergence cannot be tolerated. Where unreinforced shotcrete is used for support, convergence in excess of, say, 0.2% may cause incipient cracking. In this respect, where different forms of support are combined they may make varying contributions to the total support for different degrees of convergence on account of varying relative stiffnesses.

Systems of rock support based on rock quality or rock mass rating systems have been developed for jointed rock but are not directly applicable to soft-ground tunnelling, being based fundamentally on the material considered as a discontinuum, with behaviour dependent on jointing patterns and conditions of joints.

13.2.5 Types of support

Tunnel arch ribs provide a traditional means of support in soils, used with poling boards, timber packings and possibly crown bars. A modified arrangement, arising from the greater uniformity of radial load and great stability against lateral deflection, leading to higher axial load safely developed in the arch, uses a porous bagged packing into which a weak grout mixture is placed [6]. The use of arch ribs and polings has survived in North America for traditional reasons (arising largely from the divorce between the design and the construction processes – see Section 13.1.9) where the use of segmental linings would otherwise be preferred.

Shotcrete is applied in layers of 25 mm upwards, for soft ground usually reinforced with steel mesh, although steel or polymer staple fibes may be substituted. Steel mesh will be fixed by rock bolts or by steel pins or staples driven into the ground.

Shotcrete may be used primarily to protect the exposed periphery of the excavation against fretting, spalling or sloughing as a result, respectively, of drying out or wetting. It is frequently used with lattice ribs which have the advantage over ribs in rolled steel section in being lighter and throwing far less 'shadow' in deflecting shotcrete from even application. Arched ribs may be set with a forward inclination to support the crown as close as possible to the face and to permit the face to be inclined without problems in setting footblocks. Lattice ribs are usually encased in shotcrete, partially for protection but primarily to ensure a composite structure.

Rock bolts may be considered in two categories: end-anchored bolts and bolts bonded along the length of the shaft. End-anchored bolts, developing locally high stresses in the ground, are more suitable for strong rock (see Chapter 17). The types of rock bolt are well illustrated by Brady and Brown [7]. Bolts bonded along their length may be grouted, resin bonded or expanded against the ground.

One type of grouted bolt is the Perfo bolt. A perforated expanded metal split sleeve is opened, filled with mortar, closed and inserted into a drilled hole. the bolt is then driven into the hole, displacing mortar to fill the annulus outside the sleeve. Other types of grouted bolt inject the grout by way of a tube through the bearing plate while a second tube releases air.

Resin-bonded bolts insert, ahead of the sharpened end of the bolt, cartridges of 2-agent epoxy resin, selected for appropriate strength and setting time for the temperature of use. The bolt is used to burst the cartridge and then spins to mix and distribute the contents along the shank of the bolt.

The Split-Set bolt has a tubular shank with a longitudinal slit. The bolt is driven into a tight-fitting hole where it is retained by friction. The Swellex bolt is also tubular, with a longitudinal corrugation. Internal pressure by grout filling expands the bolt to fit the hole.

Beyond a length of 4–5 m bolts are normally designated as anchors. Anchors may be based upon a single steel bar or upon multiple strands. Anchorage is generally achieved by oversize drilling and attachment of the head by grouting. Anchors generally have a permanent function and require particular care in protection against corrosion.

Rock bolts are stressed after installation to a predetermined load, threaded nuts transferring the load through shaped washers to bearing plates, the timing of the operation dependent on the type of bolt and the rate of hardening of any bonding agent.

A bolt or bar used without stressing is termed a dowel. The distinction is not precise since, for a jointed mudrock, for example, a bolt may first be stressed to cause joints to close and then released to permit a certain degree of relaxation of the rock without anchor slippage. Dowels may be simple bars driven into holes, and include bamboo or, for example, polymer-bonded cellulose set in mortar.

A distinct form of dowel is the spile, which has much application for soft-ground tunnelling with informal support. A spile is a length of (usually) rolled steel section selected to have adequate stiffness to permit driving into the ground with a percussive hammer and with sufficient exposed perimeter to develop adequate bond with the ground. Spiles may be driven from the tunnel face, inclined to axis, ahead of the excavation.

Drilled horizontal crown bars with reinforcement in concrete or mortar may provide effective support for relatively short lengths of tunnel in critical condition.

All aspects of the tunnelling process need to be considered together and interactively. Adequate safety margins need to be assured at each stage of the operation, in the knowledge that loads carried by supports are dependent on changing patterns of stress and strain in the soil.

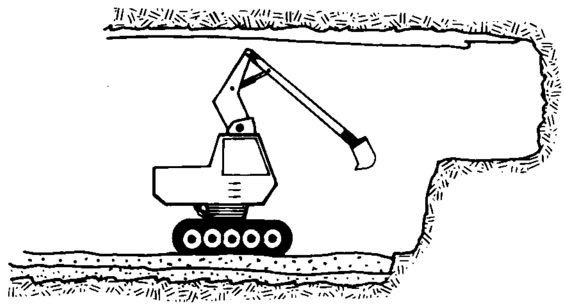

Figure 13.4 Tunnel advance by top heading and bench

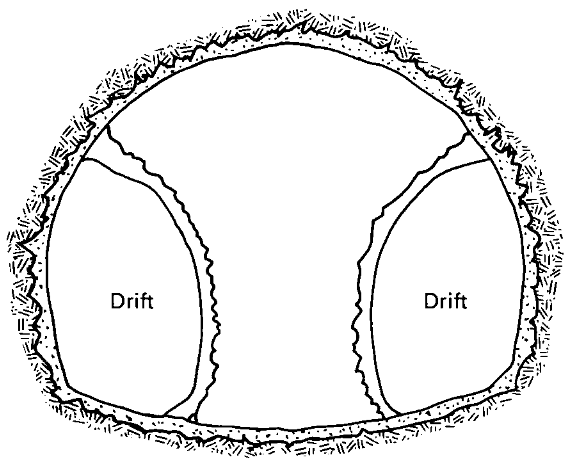

Figure 13.5 Tunnel advance by sidewall drifts

Stability of the tunnel face may require the tunnel to be advanced in headings rather than full face. Depending on the particular circumstances, a tunnel may, for example, be advanced by top heading and bench (Figure 13.4) or by side headings (Figure 13.5) (see also Chapter 17).

The essence of good practice in evolving a scheme of construction is that the stability of each phase of construction should be considered in relation to its contribution to the support needs of the full tunnel section. Particularly in weak ground, stability local to the face may be critical, needing solutions along such lines as:

1. Use of inclined spiles;
2. Use of the expedient of 'decoupage', whereby a slightly convergent annular slot is sawn around the tunnel profile and immediately filled with shotcrete. For larger tunnels, a comparable expedient is to maintain the maximum 'buttress' in place while taking the excavation forward in the form of a horseshoe to enable the placing of arch supports prior to completion of excavating the 'buttress'.

As far as practicable, support introduced for an early phase should contribute to the overall scheme of support for the full-face section. There is much discussion on the timing of the need to 'close the circle' of support, i.e. to provide a completed ring around the tunnel. During the excavation of an upper heading it may be critical to continue support around the invert; alternatively, it may be adequate to provide substantial rock bolts at the (temporary) footblocks in the heading (Figure 13.5). Any support used within the section of the tunnel will need to be excavated at a later stage. Plant access to a top heading is a considerable matter of concern, especially where it is desired to complete the excavation as soon as possible after the heading. A removable inclined ramp or an elevating platform may be provided for such a purpose if the presence of an excavated ramp, advancing with the top heading, is unacceptable, on account of the consequential delay in closing the circle. For certain sizes of tunnel, matched road headers are capable of excavating a top heading and a short bench, while standing at invert level.

13.2.6 Waterproofing

Waterproofing of soft-ground tunnels with informal support follows the pattern of that described in Chapter 17 for rock tunnels, often using a watertight membrane fixed to a protective wool or cotton waste backing between shotcrete and final lining. An informal support will rarely be adopted in association with a need to resist high external water pressure; more often the need will be for an umbrella with lateral drains or an invert drain to ensure adequately dry conditions within the tunnel. Careful detailing is then necessary to ensure, with appropriate maintenance, free operation of the drainage system throughout the life of the project.

13.2.7 Monitoring procedures

In-situ testing

Strength–strain properties of the types of ground suitable for adoption of informal lining systems are generally difficult to determine from tests on samples extracted from the ground. The extraction process leads to unloading of the sample, and subsequent reloading during testing cannot be relied upon to restore the sample to its original state. This usually leads to overestimating strains and hence convergence. In consequence, there is an advantage in making best use of *in-situ* testing techniques.

In view of likely variability of the ground, there is another advantage: whereas sample tests will exploit zones of weakness in the sample (as opposed to rock testing, where the coring process may exclude the weakest zones), *in-situ* tests provide more representative results from a greater mass of soil, surrounding the point of test. The most appropriate means is usually by using a pressuremeter, possibly a self-boring type which collapses to represent the phenomenon of excavation of a cylindrical shaft or tunnel.

Monitoring of performance of supports

Adoption of informal lining requires regular monitoring to detect incipient distress of supports. The incremental support system requires a comprehensive programme of measurements of convergence. It is desirable, at the start of tunnelling, to arrange, by the use of axial extensometers and extensometers transverse to the tunnel line (established from the surface or from an underground vantage point), to measure convergence ahead of the face so that the full convergence/time curve may

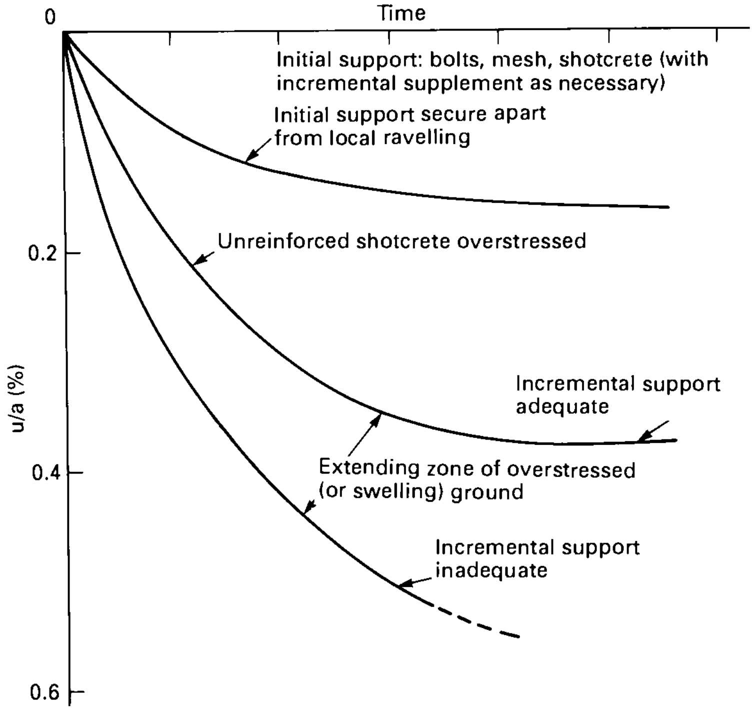

Figure 13.6 Convergence/time curves for tunnel

be established (Figure 13.6). Thereafter it is important to start measuring convergence as close to the face as possible from within the tunnel. The several methods of so doing may be based on:

1. Direct measurement between studs set on the tunnel wall; such measurements provide relative movements only and absolute movements will need precise surveys related to fixed points. A system of triangulation between studs avoids measurement across the tunnel diameter at the expense of reduced accuracy.
2. Use of extensometers set in radial drill holes. These are often multiple-head devices, indicating convergence at anchor points bonded at intervals to the ground relative to the extreme end of the device, whereby an overall picture of radial movement around the tunnel may be developed.

The multiple-head device undoubtedly provides more reliable and more comprehensive records of convergence, obtained with greater facility than direct measurement, also avoiding problems of obstruction from the tunnelling process at critical times. Any system must provide robust protection at the wall of the tunnel. There are likely to be special problems in measuring convergence in the tunnel invert, whichever system is used.

There are several forms of load gauge for measuring loads in bolts. While strain gauges may be used for estimating stresses in supports, a tunnel is not an optimal environment for electronic instruments. Vibrating-wire gauges are more reliable than resistance or capacitance gauges. Piezometers to detect groundwater pressure fluctuations are the most useful means for studying variations in ground stress around tunnels in clay, using appropriate relationships for interpretation. Load gauges between ground and support are notoriously unreliable and should be avoided.

13.2.8 Use of compressed air with informal support

While Chapter 18 describes the general application of compressed air, for informal support methods the use of compressed air may contribute to the total support pressure p_i (see equation (13.1)), thereby economizing on the need for other temporary support. Where, for example, the permeability of the ground is such as to introduce the risk of seepage, locally eroding and weakening the soil, and the water table is not greatly above the tunnel level, a pressure of 1 bar or therabouts may be highly desirable as means of control, with thus a dual purpose. It is necessary to ensure that no interruption may occur since the compressed air will be an essential feature of tunnel support until adequate support is provided to dispense with its contribution. This requirement will have practical consequences on the overall scheme of excavating and lining the tunnel.

A particular benefit of compressed air is that support is provided from the moment of exposure of the ground at the tunnel face so that total support may be considered as provided in two or more phases, with the first phase (compressed air) having zero stiffness (see Figure 13.2).

13.2.9 Future use of informal support

Since 1960 there has been much diversification in the use of informal support and in the ability to design applications specific to the nature of the ground. There has also been a certain over-enthusiasm in applying the concept of incremental support in circumstances for which formal linings were more appropriate – and vice versa – but local traditions promote conservatism in the approach to tunnelling, thereby missing opportunities for economies.

The tunnelling engineer must be pragmatic, learning which method to use for which circumstances, taking account of the ground, of logistics, of the availablity of skills and, above all, of procedures and contract practices. Without appropriate control and allocation of risk, economic tunnelling cannot thrive. Furthermore, it must be understood that economic tunnelling demands continuity of engineering from planning to design to construction, with all the iterations that should occur between each aspect.

Where these conditions prevail, improved understanding of tunnelling philosophy may develop. Current best practices will become more broadly adopted and, in particular, a greater eclecticism may be observed. By this is meant the adoption of ideas developed by different innovations, their adaptation to different circumstances and the increasing combination of methods to provide the most satisfactory solution to each new tunnelling problem, properly analysed and understood.

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Chapter 14

Tunnels in soils – formal linings

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Contents

Chapter 13 provided an overview of tunnels in soils and weak rock. This chapter develops the common case of preformed linings, discussing the linings themselves together with associated structures and temporary works. The topic of ground movements due to tunnelling is introduced and an indication of the wide range of rates of progress attainable is given.

14.1 Choice of linings

Choice of preformed linings is determined by a number of interacting factors, including the type of ground; size, length and orientation of the required tunnels; waterproofing and corrosion protection standards; and the selection of machinery.

The type of ground will affect the loadings on the lining, ground movements and the required stiffness of the lining. These factors are discussed below under lining design. Of primary importance are groundwater, stability factor and the stand-up time of the ground itself (see Chapter 13); these will affect method of construction and form of lining.

The size, length and orientation of the tunnel are of importance mainly in that they affect the type of machinery that can be used. For example, a short length of tunnel may not justify the expense of a tunnelling shield and the lining must be selected for ease of hand building. Similarly, an escalator shaft at 30° to the horizontal, together with its curved machine chambers, must be hand built. These factors should be considered at the planning stage. Modern metros such as Hong Kong and Singapore often eliminate the more difficult hand-dug tunnels by containing all escalators within the cut and cover station boxes, but in London, the age and complexity of the system has led to a large number of sloping and twisting tunnels which do not lend themselves to shield excavation (see Figure 14.1).

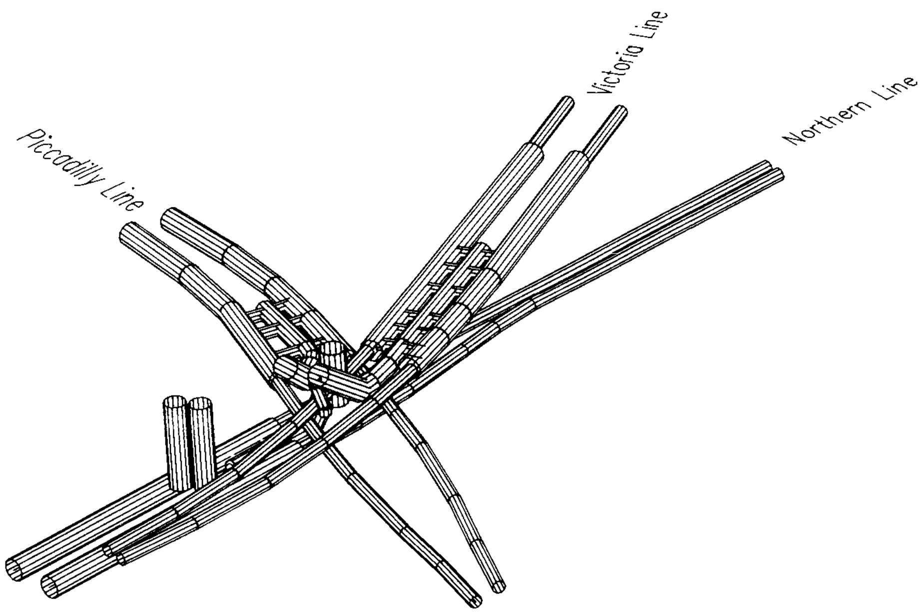

Figure 14.1 King's Cross, London. A complex of hand-built and shield-built tunnels and shafts

Waterproofing and corrosion protection in difficult ground may dictate the form of joints in the lining and the lining material. The machinery selected affects the size and weight of the preformed segments of which the lining is composed.

14.2 Types of lining

14.2.1 Expanded and grouted linings

Almost all precast linings are formed of rings, each ring consisting of one or more segments. Most tunnels for railways have four or more segments per ring. The exception to the ring form is a spiral, which has been used in a few tunnels in West Germany and experimentally (by Bridge) in the UK). The linings may be built by expanding them direct against the ground or may be separated from the ground by an annulus of grout. Figures 14.2 and 14.3 show an expanded concrete lining, while Figure 14.4 illustrates grouted cast iron linings.

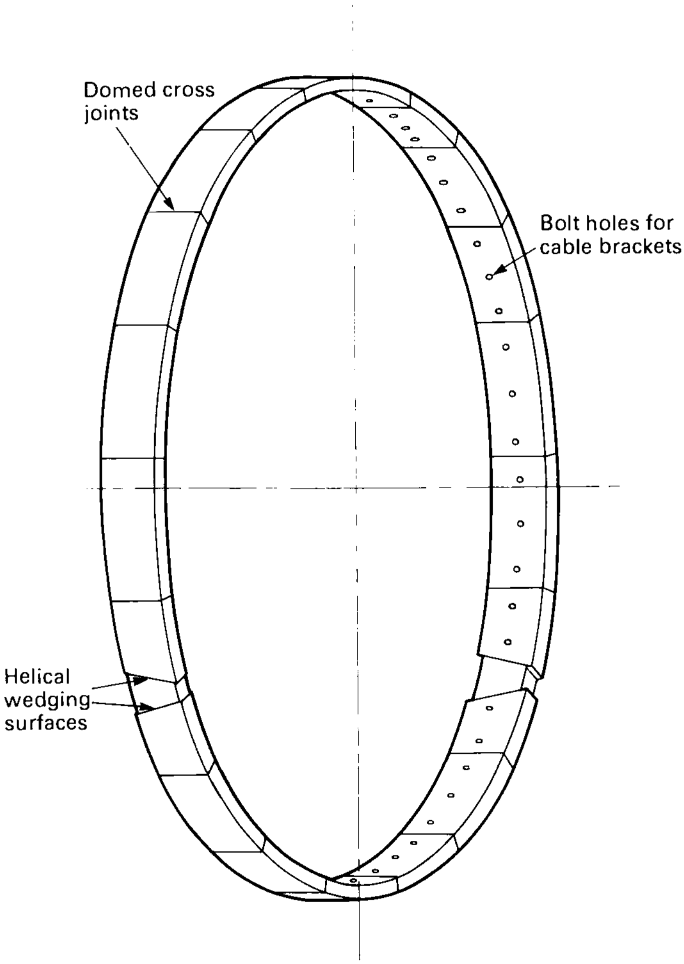

Figure 14.2 Expanded precast concrete lining

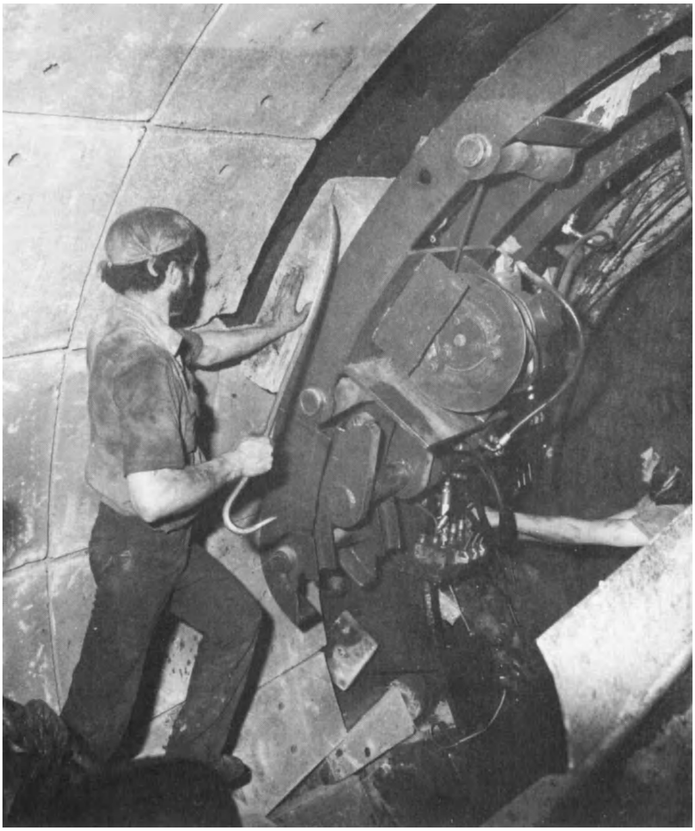

Figure 14.3 Building expanded lining. Note wedge segment not yet driven home

Expanded linings must be built using a shield, which can cut the ground to a clean and accurate cylindrical shape. The ground itself must be able to stand unsupported for a certain length of time behind the shield while the lining is erected. A cohesive ground with few hard inclusions is required; London clay is an example where expanded linings have been successfully utilized.

If the ground cannot stand unsupported behind a shield, grouted linings built within the tailskin of the shield must be used. These linings are also used for hand-dug tunnels, where the uneven cut shape of the ground precludes full contact with expanded linings. Temporary support for the ground is provided by the tailskin of the shield or by temporary timbering, if required. Grout is pumped behind the lining after erection, filling any overbreak and providing continuity between the lining and the ground it supports. This grout is termed primary grout.

14.2.2 Materials

Cast iron

Two forms of cast iron are used for tunnel linings; grey cast iron (CI), which has been in use since 1869, and spheroidal graphitic or ductile iron (SGI), which was introduced in the late 1940s.

Figure 14.4 Grouted cast iron tunnel linings, including curved rings

Grey iron is a brittle material having high compressive strength and low tensile strength. Spheroidal graphitic iron has greatly increased tensile strength and is less brittle. Some typical strength values are given in Table 14.1[1].

CI segments have a ribbed form to enable the thrust line for permanent loads to be kept within the depth of the segment, avoiding tension in the iron. Temporary loads may determine some of the segment dimensions; these include handling and shove loads from the rams on the shield. Designs using SGI make use of its tensile strength, leading to thinner and lighter segments which are less costly and are easier to build. However, thin linings may lead to problems in transmitting loads from shove rams on the shield.

Table 14.1 Strength of cast iron and spheroidal graphitic iron

Iron	*Tension* (MN/m^2)			*Compression* (MN/m^2)		
	0.1% proof	*0.5% proof*	*Ultimate*	*0.1% proof*	*0.5% proof*	*Ultimate*
Grey	100–170	NA	160–260	200–340		620–880
Ductile	320–380	360–460		340–400	360–470	500–700

The corrosion resistance of CI has proved to be excellent, with the use of appropriate protective coatings. SGI has a shorter history but also appears to have good resistance.

Steel

Steel has the advantages of high tensile and compressive strengths and lack of brittleness. It is used in two forms: pressed liner plates and fabricated segments.

Liner plates are manufactured from comparatively thin sheet steel (3 to 7 gauge), corrugated to provide bending strength. Joint details are simple, with laps being sometimes used at longitudinal joints. Such plates are light and therefore easy to build. In the United States it has been common to build liner plates for immediate support in hand-dug tunnels, followed by an *in-situ* concrete lining.

Fabricated steel segments are manufactured by cutting and welding plate material. Sections can be much thicker and stronger than for the liner plates noted above. However, the manufacturing process is more labour intensive than for cast iron linings or steel liner plates. Fabricated segments are normally used only for small quantities of a particular design (for example, for a short length of tunnel carrying concentrated heavy building loads). They are also frequently used for special segments at openings in cast iron or concrete tunnels.

Steel is subject to corrosion and segments must be provided with high-quality protective coatings.

Concrete

In recent years, concrete has become the principal material for lining railway tunnels. Its advantage over cast iron and steel is low cost, of the order of one-third of that of cast iron. Disadvantages are its lack of tensile strength, comparatively low compressive strength and brittleness. It is much more subject to damage during construction than either iron or steel. Many forms of lining have been used, with variations in ribbing or plain sections, type of bolts (if any), number and shape of segments and form of joint. It is used for both expanded and grouted forms.

Reinforcement is commonly, but not always, incorporated into concrete segments to provide tensile strength for bending, bursting stresses near joints or for temporary loadings during construction. Where reinforcement is used, the designer must be aware of the need to provide protection against the corrosive effects of groundwater. A variety of measures have been used depending on the particular conditions. A protective coating to the outside of the lining may be specified, together with high-quality concrete of low permeability and with adequate cover to the steel reinforcement.

14.3 Lining design

14.3.1 Internal diameter

The factors determining the internal diameter of the tunnel are the railway requirements and design and construction tolerances.

The principal railway requirement is, of course, the train itself. Dimensions are stated as a kinematic envelope or kinematic gauge, which is an outline beyond the actual shape of the vehicle. This represents the extreme position any part of the vehicle may take, due to movements on its suspension, wear on the vehicle and on the track. On curves this must be increased by vertical and horizontal throw, which is the offset of the vehicle between its wheel bogies. When traction current is supplied from overhead wires, electrical clearances must be added over the train. A further tolerance or clearance is allowed to derive the structure gauge; all structures must be outside this.

In the tunnel invert, space is required for the track. A common requirement is for a walkway to evacuate passengers in the event of an emergency breakdown of the train. In some metro systems (Hong Kong and Singapore are examples) the walkway is provided between the running rails, with no increase in tunnel diameter. This requires that the trains be designed to allow passengers to walk their full length and to disembark from the front or back of the train. In loco-hauled railways this is not normally possible and walkways may be specified along one side of the tunnel. Not all railways require these provisions.

In tunnels for high-speed trains, aerodynamic considerations may control the tunnel diameter. Transient pressure pulses occur when two trains pass in a two-way tunnel, when entering tunnels or passing various changes in the tunnel section. These pulses must be limited to comfortable levels. Drag can be a factor; more tractive power is required to drive trains when there is little clearance between the train and tunnel and a minimum area may be specified. Ventilation requirements can also affect tunnel diameter, where large quantities of surplus heat must be extracted.

A design tolerance is required to limit ground movement. This results in specifying a maximum calculated distortion of the lining, which will be small (perhaps 1–2% of diameter).

Construction tolerances are larger than this. Tunnels for London's Underground were traditionally built with 25 mm tolerance on position of lining, leading to a specified diameter 50 mm larger than the minimum 3.8 m required for railway and design purposes. This tolerance is achievable in London clay but must be regarded as an absolute minimum. In Singapore, contractors' designs allowed 130–300 mm increase in diameter over the minimum of 5100 mm required for railway and design [6].

14.3.2 Structural design

The preformed lining is required to maintain the stability of the cavity formed in the ground while minimizing ground loss and to control water movement into the tunnel.

Analysis of tunnel linings is complex because of the nature of the interaction with the surrounding ground. Most simple analyses assume that the lining is introduced into the ground with no changes in ground stresses. This simplification allows a two-dimensional plane strain analysis to be performed, with initial loading approximating to undisturbed ground pressures. The lining then distorts, with the vertical axis normally shortening and the horizontal one lengthening. There is a consequent readjustment of ground stresses. The amount of distortion of the lining is a function of the excess of vertical over horizontal pressure, the elasticity of the lining and of the ground. Flexible linings, with many segments per ring, tend to allow stresses around the ring to equalize and carry comparatively little bending moment. Stiffer linings control ground movements but tend to carry more moment. The moments are induced as the thrust line in the ring moves away from its centre of area. A particular effect is at joints; if a flat joint rotates, it tends to open on either the inner or outer face of the lining, moving the thrust line in or out to the edge of the segment, inducing moments.

In practice (see Chapter 13), the stress paths in the ground are more complex than assumed above, depending on the ground itself and the construction method. With open-faced shields or hand-dug tunnels without a shield, ground moves in towards the excavation in a three-dimensional manner before the lining can be built. Water drains into the face, drying the ground and changing its properties. It is not until the lining is in place that it can begin to control the ground movements. Such movements can continue for ten years or more in some clays.

The simple analysis can be varied to take advantage of stress release in medium-hard rock or highly cohesive soil. Ground stresses can also be determined empirically, based on experience in similar ground. The last-mentioned approach requires high-quality observations and monitoring of the previous tunnels, if maximum advantage is to be gained from experience. In softer grounds, stress release during excavation is a temporary condition and a simple analysis assuming full overburden loading may be appropriate.

Hydrostatic loading can be an important element of the total loads on the lining. If the lining is impermeable it will eventually carry full hydrostatic loads.

An important aspect of lining design is related to the longitudinal joints between the segments forming a ring. As noted above, rotation of the joint is possible, appearing in the tunnel as a visible opening or 'birdsmouthing'. In a flat joint, full contact between the mating faces is no longer possible, the thrust line moves and moment is induced. In some designs the joint is shaped to limit the displacement of the thrust line. For example, expanded linings in London clay in the 1970s utilized double convex joint surfaces. The theoretical point contact between such faces becomes an ellipse under load. Rotation results in a small movement of the thrust line and minimum induced moment. However, the concentration of load on the joint face induces a bursting force in the segment which may require reinforcement. Another approach is to place a deformable packing in the joint. Moment in the segment also depends on the degree of uniformity of stress imposed by the ground. There may also be an element of shear stress between ground and lining.

14.4 Waterproofing

14.4.1 Reasons for controlling water

Since waterproofing a tunnel can be a difficult process it is necessary to have a clear understanding of the reasons for controlling water inflow. There are four main reasons:

1. Pumping costs for inflow water should be minimized.
2. In the event of pump breakdown, water must be stored safely. Large inflows require large and expensive sumps.
3. Electrical short circuits must be avoided. Note that damp and dirty or salty conditions in a tunnel may be troublesome for, say, signal currents in the rails while rainwater in surface sections of the railway causes few problems.
4. Large inflows into a tunnel may lower the groundwater table and carry fine soil particles into the tunnel, causing settlement and damage to buildings and infrastructure. Inflows causing settlement are of concern during construction, where face collapse can be triggered by inflows in some circumstances.

14.4.2 Waterproofing standards

Waterproofing standards often combine a general permitted inflow with a larger inflow in limited areas. This recognizes the difficulty of obtaining a uniformly low value. CIRIA[7] lists seven classes, as noted in Table 14.2. These are compared with the 1983 specification of the Singapore MRT.

Table 14.2 Waterproofing standards

CIRIA class	0	Nothing visible	
	A	1	litres/square metre/day
	B	3	
	C	10	
	D	30	
	E	100	
	U	Unlimited	
Singapore		0.12	average litres/square metre/day
		0.24	in any 10 m length

CIRIA class 0 is probably impossible to achieve in most types of ground. The Singapore figures are difficult to obtain; Copsey and Doran note only one method which succeeded with little trouble, as described below.

14.4.3 Waterproofing methods

Waterproofing methods have to cope with the realities of tunnel construction. Dirt is a constant problem. Tunnel lining segments are usually slightly misaligned when built in position. Packings are often used between rings to correct misalignment of the tunnel as a whole. After construction, the tunnel will continue to move for some time, depending on the type of ground. A number of waterproofing methods are in use, which have varying success in dealing with these problems.

1. Caulking is the traditional method. In cast iron tunnels lead strip is rammed into grooves preformed in the joint surfaces (see Figure 14.5) using small compressed-air tools. This may then be held in place with a compound of iron filings and sal ammoniac which expands on rusting. This method is very effective and has the merit of being capable of repair if the tunnel moves after caulking, opening up the joints. However, it requires too much force in placing to be used in concrete linings; the lining would be damaged. An asbestos cement compound was used for caulking in past years but this has now been replaced by modern cementitious flexible compounds. Caulking in concrete linings is not easy to repair if damaged when the tunnel moves. Note that the caulking is normally placed near the front of radial joints, so bolts which are nearer to the water source must be sealed with grommets (pronounced 'grummets'), a kind of flexible washer.
2. Bitumen sealing strips can be incorporated into the joints between segments but lack the ability to recover their shape and cannot respond to movements in the tunnel. Packings between rings can render them ineffective.
3. Neoprene gaskets have the necessary resilience to follow tunnel movements but can allow passage of water as a result of misalignment of the segments and use of packings.

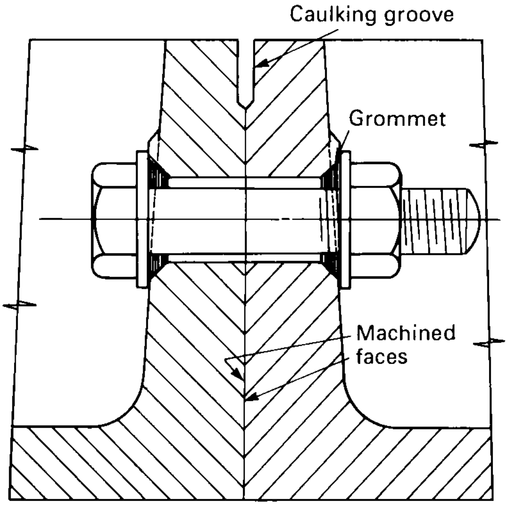

Figure 14.5 Radial joint in cast iron lining

4. The Hydrotite gasket is a recent invention which is proving very effective where groundwater has a low salt content. The gasket is glued into a shallow groove round the segment, where it is reasonably safe from damage. After building, it absorbs groundwater and swells to a maximum of ten times its original volume. Provided the gasket has not reached its limit of absorption, it can adjust to tunnel movements. Limited packing between rings can be used. These gaskets were used in Singapore without additional measures being necessary[6].
5. The additional measures needed when the primary waterproofing fails to control the inflow to the specified level consist mainly of backgrouting. The primary grout pumped in behind the lining may not control water, since it is not always complete, contains cold joints and possible dirt, and may crack when the tunnel moves after construction. The quantity and pressure used are generally not sufficient to penetrate far into the surrounding ground. Backgrouting carried out some time after construction will penetrate the ground, filling voids and decreasing permeability. Water inflows can be substantially reduced by this method but it must be realized that a considerable investment of time and labour is required. It is preferable to provide effective first measures in the form of a suitable caulking or gasket.

Finally, it must be mentioned that not all tunnels require waterproofing measures. Tunnels in London clay, for example, are often dry. Expanded linings with no caulking are on occasions successful with insignificant levels of inflow. On initial construction, however, they may appear to sweat, drying as a result of air movement and heat once the tunnel is taken into service.

14.5 Special structures

So far, tunnels have been considered as continuous drives. This is a simplification of real situations; almost all tunnels have some sort of special structures associated with them. These may range from a shaft to a complex such as occurs in many of

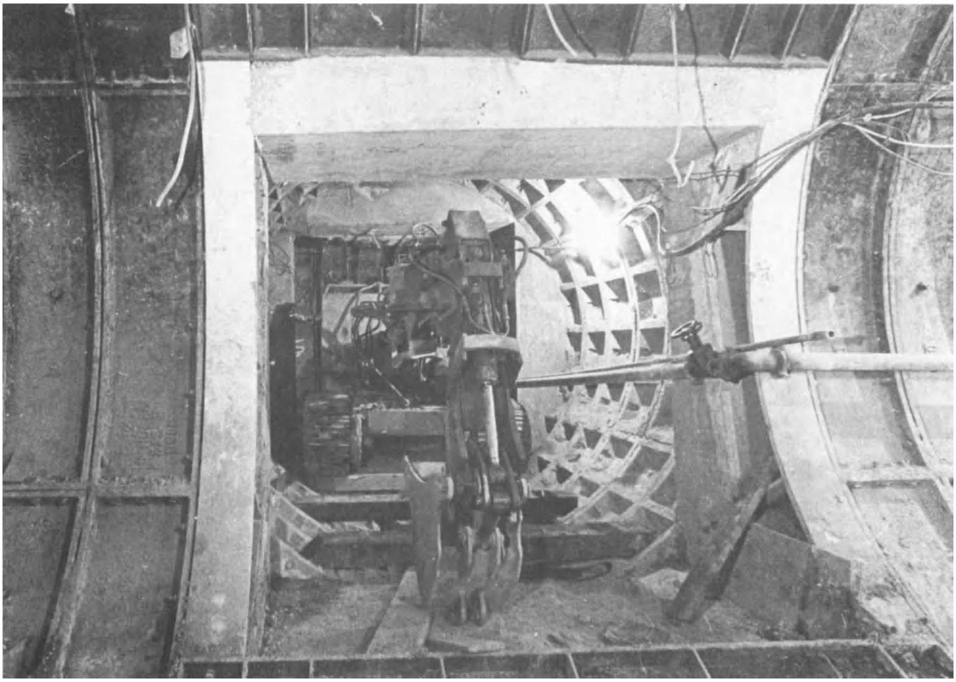

Figure 14.6 Cross-passage opening in cast iron tunnel

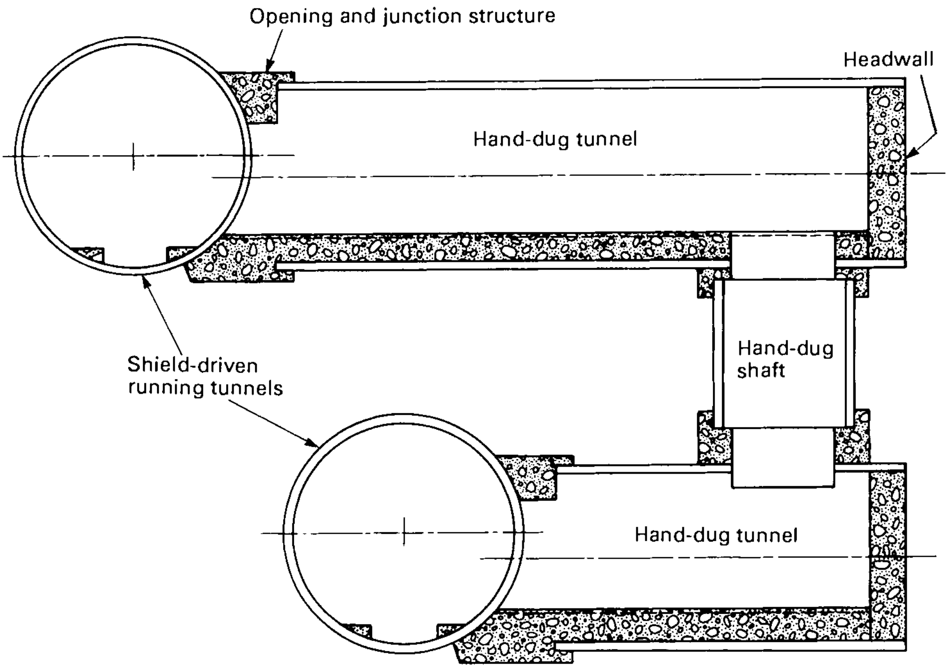

Figure 14.7 Hand-dug special tunnels connecting shield drives

London's underground stations. Some of the structures which may be needed are:

1. Shafts
2. Sloping shafts
3. Curved iron (Figure 14.4)
4. Openings from shafts
5. Openings from tunnels (Figure 14.6)
6. Ring walls between tunnels of different sizes
7. Headwalls at tunnel ends (Figure 14.7)
8. Overbridges (Figure 14.8)
9. Underbridges

Shafts with preformed linings may be built by constructing a cutting edge on which the linings are built. This is thrust into the ground using kentledge – many tonnes of dead weight on a frame above the linings. Excavation can be by hand or machine. The cutting edge may require lubrication by a bentonite mixture which may also be used to fill an annulus outside the shaft as it is sunk. Alternatively, the shaft may be built by underpinning, erecting each ring underneath the previous one. There is no tunnel face to support but ground failure by heave up into the shaft can occur. Water may be a major problem if the ground is permeable and ground treatment or compressed-air working may be required.

Sloping shafts are often required for escalators, where these cannot be incorporated within a cut and cover station box. It is not possible to use a shield for

Figure 14.8 Overbridge cut across top of cast iron station tunnel

such a shaft, although some machinery for excavation or erection of the lining may be used if space permits. These shafts are more akin to hand-built tunnels than shafts, in that a face of excavated ground has to be supported, although not a vertical face. Note that the escalator shaft will normally be driven downwards to simplify face support.

Curved iron is required when there is a sharp bend in the tunnel (for example, at the bottom of an escalator shaft) or for one of the service tunnels which is often associated with urban underground railway works. The latter is shown in Figure 14.4.

The remaining structures are not supported with preformed linings, although overbridges may have a combination of part rings of cast iron or concrete as a roof structure, with *in-situ* concrete walls. An example is shown in Figure 14.8. Where the structure involves making an opening in a tunnel with preformed linings a variety of techniques is available to preserve the structural integrity of the tunnel around the opening. Steel lintel beams may transfer the thrusts from several rings to steel jambs at the sides of the opening. In some cases, the lintels have been made of special segments constructed with the original tunnel rings and fastened together with bolts or welding to form a beam. In an expanded concrete-lined tunnel, shear pins have been placed between rings over and under an opening to transfer the thrusts to the sides of the opening. In London, the permanent lining for junction structures is *in-situ* concrete but shotcrete could be used as an alternative in some situations.

14.6 Temporary works

The general principles of ground support have been discussed in Chapter 13, and some of the applications to preformed tunnels and special structures are described below.

For shield drives, it is evident that the shield itself provides support around the periphery of the excavated ground before the lining is erected and expanded against the ground or grouted. Support required for the face depends on the ground and techniques adopting face rams, breastplates, full-face support using earth pressure or bentonite slurry are available. Compressed air is another common technique. These methods are covered elsewhere in this book. An addition to some of these may be required when the shield is stopped for any time, for a weekend or due to breakdown. For example, in a clay with a reasonable stand-up time normal tunnel driving may proceed with little or no face support but, when the shield is stopped, timbering will be required. This may have to be close boarded and grouted to delay drying out of the clay, which could lead to face collapse.

Hand drives of short, sloping or curved tunnels require face and top support to the ground. Rings should be built and grouted one at a time to provide the permanent support quickly. In permeable ground, or where there is an aquifer close over the tunnel, ground treatment or compressed air may be necessary. Even in good clay, the slower rate of advance will entail close timbering to reduce drying out of the clay in many cases. Experience in the particular ground, combined with an appreciation of the mechanisms at work (see Chapter 13) are vital to safe working.

Figure 14.9 shows temporary works required for a ring wall connecting a tunnel to a cut and cover station box.

Figure 14.9 Temporary works for ring wall connecting to cut and cover structure

14.7 Ground movements

Tunnelling is always accompanied by movements of the ground over the tunnel. In soft ground, with the tunnels at common railway depths, the effects will reach to the ground surface. A three-dimensional settlement trough is normal, commencing some distance ahead of the advancing tunnel face. The cross section of the trough is approximated by the normal probability curve and the longitudinal section ahead of the face by the cumulative probability curve [10]. In some grounds, and particularly if an earth pressure shield is used with high pressures, there may be some heave ahead of the shield. When two tunnels are constructed close to each other their settlement troughs are additive, although the second one may tend to lean into the first [8].

The factors influencing settlement are discussed in Chapter 13. It is noted that settlement has two major elements; immediate or ground loss settlement and time-dependent or consolidation settlement. Prediction of settlement is difficult. O'Reilly [9] notes that the greatest uncertainty is in characterizing the strength of the ground. Shirlaw and Doran [8] also discuss the influence of different construction methods. Immediate settlement was minimized using earth-balancing shields but these increased consolidation settlement. Inflow of water into the tunnel face and through the precast linings was seen as a major factor increasing settlement.

It will be appreciated that very different values of settlement have been experienced in different situations. Values can be from 10 mm to 350 mm or more. It is essential to assess the likely spread of settlement values with different possible construction methods, in conjunction with an understanding of the type of buildings and infrastructure over the tunnel, and the likely effects of movement. This will enable decisions to be taken on the specification for the tunnel construction method and the contractor's proposed method.

14.8 Rates of progress

This section attempts to give some general guidance on the subject of possible rates of progress for straightforward tunnel drives. Special structures such as openings and enlargements are not discussed, since progress is affected markedly by local factors.

Rates of progress achieved on various projects are available from a number of sources. However, before attempting to use these for planning a new tunnel it must be realized that every tunnel is a prototype, unlike all previous tunnels in many significant details. Progress data from previous tunnels must be used with caution. Differences in the ground, machinery, lining, working practices, haul distances, etc. can result in different progress in nominally similar tunnels.

Figure 14.10 shows a typical progress curve for a tunnel drive, the distance tunnelled being plotted against time. It will be seen that there is a start-up time at the beginning of the drive, which may last several weeks. Progress is slow, due to setting up and commissioning equipment (the drive often has to start with less than its full equipment due to space limitations) and the necessary learning curve. Once

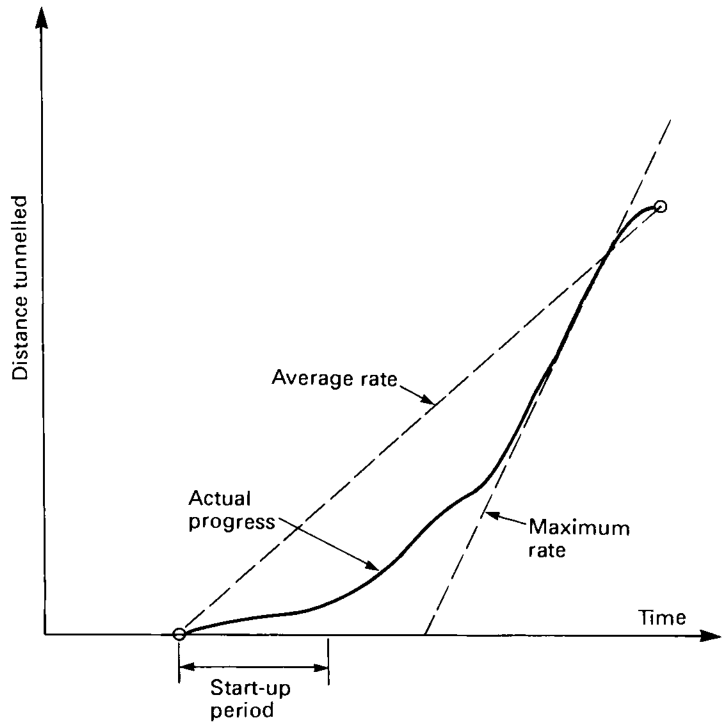

Figure 14.10 Typical progress for tunnel drive

Table 14.3 Rates of tunnelling progress

Tunnel	*Metres/week*	
	Av.	*Max.*
(1) London Underground to Heathrow, 3.8 m dia. in London clay:		
Shield drive, mechanical excavation	51–52	
Shield drive, hand excavation	23	
Hand drive, no shield	14	
(2) Jubilee Line, 6.5 m dia. in London clay:		
Shield drive with hand excavation	14	
Ditto, enlargement of pilot tunnel	17	
(3) 10.4 m dia. bolted concrete-lined tunnel in mudstone, shield with mechanical excavation	24	33
(4) Singapore MRT, 5.3–5.4 m dia. in variable firm to soft ground. Typical drives[11]:		
Full-face tunnel-boring machine	35–45	155
Shield, mechanical excavation	22–45	80
(5) Channel Tunnel[12] Drive T4: Full-face earth-pressure machine	75	206

all the equipment is installed and working, and full shifts are being worked, progress will settle down to a reasonably steady rate, with variations due to changes in the ground, holidays and breakdowns and other factors. At the end of the drive, progress will flatten out while the final rings are built. The average progress over the drive is obviously less than the maximum, depending on the length of the drive. The drive itself is preceded and followed by periods of time needed to install and remove equipment.

Table 14.3 lists some progress achieved on a few projects. All rates assume working 24 hours a day, for 5 or 6 days a week.

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Chapter 15

Tunnelling machines in soft ground

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Contents

Shields and tunnel-boring machines (TBMs) are used to tunnel in soft ground. In the shields the soft ground is dug by hand or excavator, while in the TBM, the ground is excavated with special equipment such as a road header or cutting wheel.

15.1 Geological operating range of shields and TBMs in soft ground

Shields and tunnelling machines which are driven in soft ground vary considerably in their design, as they must be equipped for a specific set of ground conditions. The term 'soft ground' includes geological features ranging from soft slurry-like soil to rock-hard marl. The corresponding mechanical properties of the soils are therefore very different, yet they influence the design of the machine, the method of ground excavation and the tunnel lining. The machine itself is surrounded by a steel skin, the shield.

The geological properties, the stability of the face or the shear strength of the ground determine the method of soil excavation. The requirement for accurate excavation, i.e. to remove only the volume of the design cross section, is often linked in softer ground with the need to simultaneously support the face.

·However, in mixed ground, when large, hard stones are embedded in soft soil, it is very difficult to achieve the twin aims of soil excavation and simultaneous face support. Only in recent years have tunnelling machines been developed which could perform both tasks even in difficult conditions.

Meanwhile, reliable face support has become increasingly important. The aim is now to drive tunnels in soft ground without disturbing the primary stress state of that ground. There would then be no settlement at the surface nor damage to overlying buildings. The tunnels could be built more economically and would, due to optimal bedding in the ground, scarcely be deformed by the loading. This reduction in stress would lead to lower maintenance costs and a longer service life.

15.2 Shields and TBMs for soft ground classified according to method of face support

The shields and tunnelling machines which are built for use in soft ground are classified according to the method by which the face is supported:

1. Shields without face support, or inclination of the face below the natural angle of repose, possibly with subdivision of the face by platforms (Figure 15.1(a));
2. Shields with fixed sealed face and closable apertures or TBMs with nearly closed and closable cutting wheel (Figure 15.1(b));
3. Shields and TBMs with face support by compressed air in the entire tunnel or only in the working chamber at the face (Figure 15.1(c));
4. TBMs with face support by a pressure-regulated fluid in the working chamber (Figure 15.1(d));
5. TBMs with face support by a pressure-regulated soil slurry in the working chamber (Figure 15.1(e)).

15.2.1 Shields and TBMs with open face

Face support

If the shear strength of the ground is sufficient to provide a stable face during excavation, then a shield or TBM can be driven with an open face. However, it

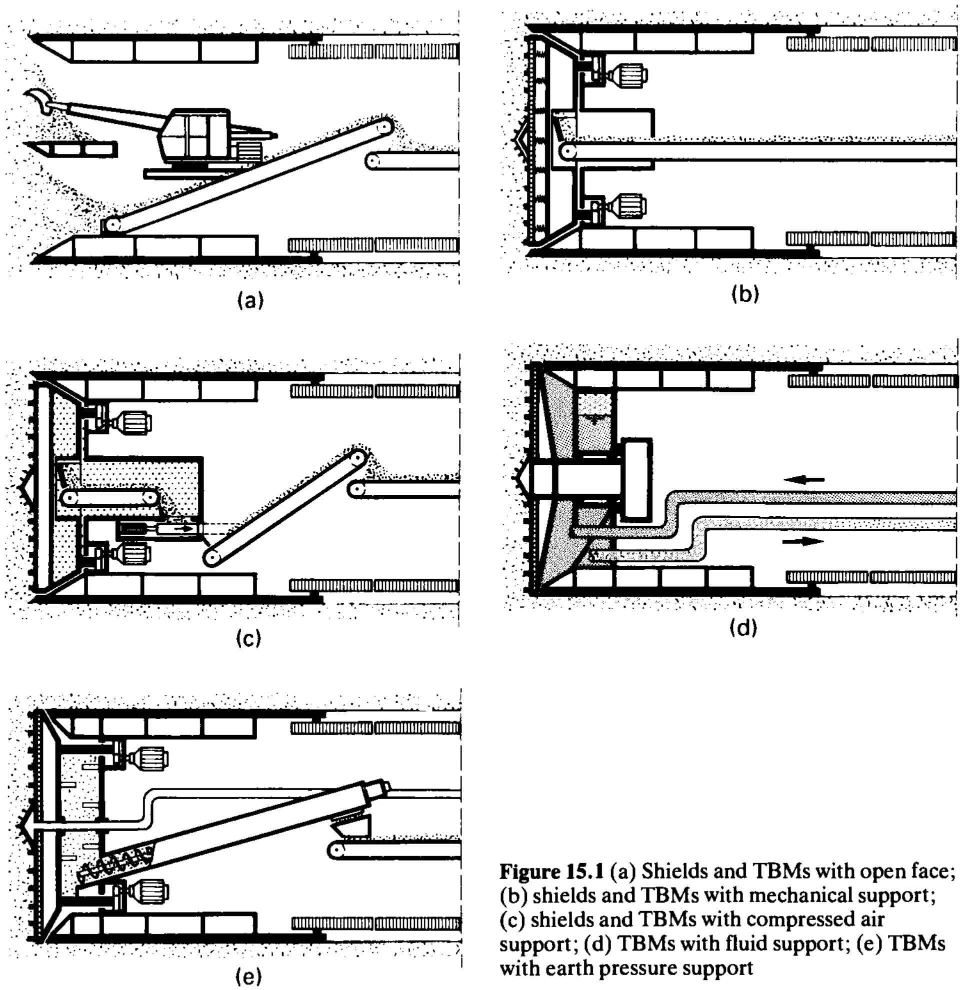


Figure 15.1 (a) Shields and TBMs with open face; (b) shields and TBMs with mechanical support; (c) shields and TBMs with compressed air support; (d) TBMs with fluid support; (e) TBMs with earth pressure support

must be noted that the span width of the ground arch, which carries the loading on the face and thus the shield diameter, are far more important parameters for the deformation of the face and its stability.

A Memco shield with an open face and a diameter of 12.2 m was used in 1973 in the construction of the Seelisberg Tunnel, Switzerland, in fissured rock. However, open-faced shields have also been used in soils with low shear strengths and even in sand or gravel. Here the face is supported by the ground itself, ramped back below the natural angle of repose. On shield diameters above 2.5 m the face is divided by platforms so that the ramps do not reach too far back into the shield. Otherwise the thrust required to advance an open shield (in any case, very high) would become so large that it could not be carried by the tunnel lining. The platforms which carry the face support ramps have a front cutting edge and can often be slid forward hydraulically. This design concept is identical to the first Brunel shield, which drove the first Thames tunnel in 1826–1841. The Suez Canal tunnel in 1979–1980 was also driven with a platform shield, diameter 11.8 m. In order to avoid uncontrolled removal of ground, the shield and platform cutting edges must be continuously pressed into undisturbed ground. For this alone, thrust forces of 150–200 t/m of cutting edge are required. With a shield of 6.4 m diameter for the Nürnberg metro it was necessary to provide a thrust of up to 4800 t.

Excavation

TBM Material can be excavated from a stable face with an open cutting wheel. The cutting wheels are open spoked and operated within the protection of the shield cutting edge. They are mounted on sliding bearings so that on a change to harder strata the cutting wheel can be advanced to cut in front of the shield cutting edge.

Telescopic or hand-mounted cutting tools at the end of individual spokes (the 'profile cutters') ream the profile for the cutting edge in order to reduce the required thrust forces. The profile cutters also aid steering. In more stable ground they are quickly worn out, and in these conditions it is therefore worth using cutting wheels with a rim which encloses the ends of cutting spokes. These rimmed cutter wheels operate in front of the shield cutting edge and maintain a clear profile for the shield.

A number of excavating tools are mounted on the rim. Scraper blades are fixed on the sides of the spokes which shave off 2–4 cm layers of soil; while small hoes, on the front of the spokes in advance of the scrapers, limit the width of the shavings.

Steel plates are also mounted on the rim which can seal off or partially close the space between the spokes. In non-cohesive ground simple square bar picks suffice as excavation tools. Tests on cohesive soil were carried out under a contract for Transport and Road Research Laboratory of the UK Department of the Environment in 1972 at Chinnor, which produced useful information for the development of scraper blades. Round-shafted picks are often mounted on the rim and are simple to replace.

Disc cutters should reduce harder strata or stones in softer ground which could destroy the scraper blades. There is, however, the danger that they do not rotate in soft ground, due to reduced friction, and then become worn on one side and thus ineffective. This can be avoided by careful adjustment of the bearing friction.

Cohesive soils tend to adhere when the material under 0.06 mm diameter is more than 30% and the water content is between 30% and 45%. This can paralyse the performance of a TBM. Only the choice and positioning of tools to remove material in small pieces, and the design and assembly of the machine to permit a direct and unimpeded material flow, can ensure a satisfactory advance rate in cohesive ground.

The advance rate of a TBM with open cutting wheel is very high. The spoil is loaded by scoops on the cutting wheel onto a conveyor belt and continually removed.

The design of the cutting wheels for the Channel Tunnel TBM, which is to be discussed below, essentially corresponds to the principles outlined above.

Shield In shields with an open-ramped face, such as are used in soil of low shear strength, the ground is removed by excavators. These have a telescopic boom to permit precise cutting at the face despite the restriction of space by the support ramp. In cohesive ground road headers are often used for excavation.

There are no particular preconditions to be met by the soil properties. An excavator can cope with anything, from sand to weathered rock. Large boulders are also fairly easy to remove. Only water causes a major disruption to the excavation process as the support ramp becomes ineffective, and the groundwater must first be lowered by sinking wells.

During the construction of the motorway tunnels under the river Elbe in Hamburg in 1969–1972 the three tunnels were driven with a platform shield of 11.5 m external diameter. The groundwater was lowered only to about 15 m above the crown. The remaining water was kept out of the tunnel with compressed air.

Even with extreme care it is scarcely possible to avoid excavating more than is theoretically necessary. In addition, the ramp which supports the face cannot prevent deformation and relaxation of the face. This results in settlement at the surface and loosening of the soil around the tunnel.

Lining

All types of tunnel lining have been built behind open-faced shields and TBMs: from masonry linings, tubbings in cast iron or reinforced concrete, shotcrete and *in-situ* reinforced concrete to extruded steel fibre concrete. Ideally, linings should be chosen which are not damaged by unfavourable bedding conditions and where the bedding can be improved by prestressing or grouting.

The thrust forces required to advance the shield or TBM are transferred to the tunnel lining. These can be considerable, and thus determine the form and dimension of the lining. However, a 'blade' shield does not transfer the thrust forces to the lining but directly to the surrounding ground via friction at the shield skin. The skin is divided into strips, 60–120 cm wide, the 'blades', which are mounted on a support frame. Each blade can be advanced about 60 cm by a hydraulic jack attached to the support frame. The friction between the stationary blades and the surrounding soil provides the reaction for the thrust forces. When, one after the other, the blades have all been advanced one cycle, the support frame is drawn up by reversing the blade jacks (Figure 15.2).

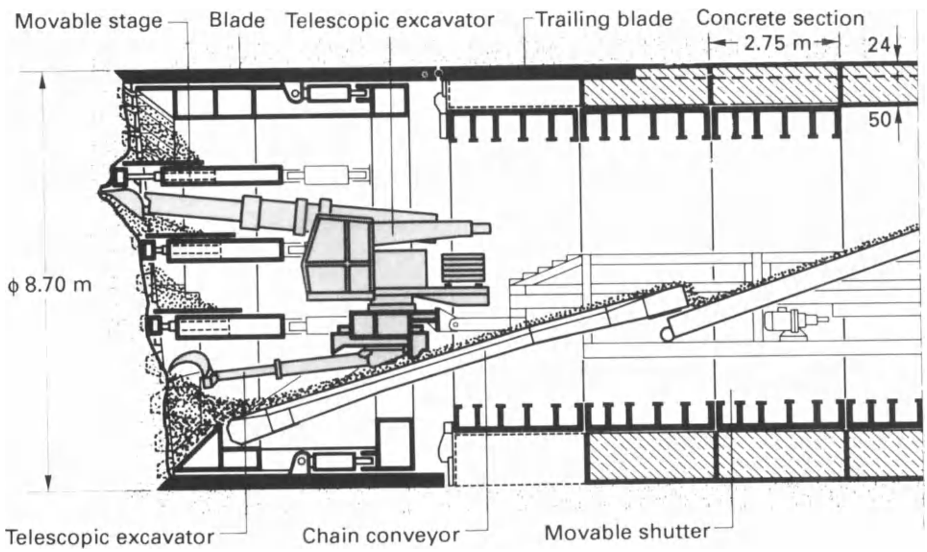


Figure 15.2 Blade shield

Behind such a blade shield a lining can be built which has to carry only the loading of the surrounding ground. Thus linings of shotcrete, extruded steel fibre concrete or *in-situ* reinforced concrete have been placed behind blade shields, Whereas shotcrete and extruded steel fibre concrete can be placed in a continuous process directly against the surrounding rock, *in-situ* reinforced concrete can only be placed in 2.5–4.5 m sections within the protection of the trailing blades. Therefore the advance rate is correspondingly low.

15.2.2 Shields and TBMs with mechanical face support and excavation

Face support and excavation

Blind shield In very soft cohesive soils, shields with a fixed sealed face are used ('blind shields'). In certain applications, such as the tunnel under the Huan Pu river in Shanghai in 1965–1970 with a shield of 10 m diameter, no soil was removed but all was displaced by the sealed shield. The thrust forces which had to be carried by the lining increased to the order of 10 000 t. The shield was difficult to steer and the river bed heaved by up to 3.5 m.

To avoid this heave an attempt is made to excavate the soft ground into the interior of the shield. Particularly in Japanese construction, screw conveyors are used which remove soil from the centre of a tunnel-shaped face support. However, this method is confined to very soft ground and a limited shield diameter.

Thus for the second tunnel in Shanghai (1985–1988) an 11.3 m diameter blind shield was used with many hydraulically operated apertures in the flat fixed face support. The soil flowed into the shield through these controllable apertures. If the shear strength was too high the soil was removed with high-pressure water jets. The front of the shield was separated from the rest of it by a bulkhead, and so formed a working chamber which could be set under compressed air. The soil/water slurry was removed from the tunnel by centrifugal pumps.

TBMs with cutting wheel In non-cohesive or stiffer cohesive soil a TBM can be used with a cutting wheel that is more or less closed and which supports the face during excavation. On the Holzmann–Bade TBM the cutting wheel supports the face with sprung-hinged support plates. A cutting plate edge on the support plate scrapes off the soil. The spoil passes to the shield interior through gaps between the support plates. Water make and stones disrupt controlled soil excavation and economical advance rates.

The turning moment required for such tunnelling machines with mechanical face support is very high in order to overcome the friction between the face and the supporting cutting wheel. The drive unit installed in the Bade machines for metro tunnels of about 6.2 m diameter produces a turning moment of about 950 mt. On the Lovat TBM the cutting wheel also mechanically supports the face during excavation. The cutting wheel is almost closed between the spokes (Figure 15.3).

Hydraulically operated apertures ('flood doors') between the spokes permit the passage of the excavated material. By adjusting the flood door openings, an experienced shield operator can maintain the required pressure on the face. Even with a moderate water inflow, the face support pressure can be maintained. In the tool gap (i.e. the space at the face bounded by the tips of the cutting tools and the face of the cutting wheel) the excavated soil and water form an earth slurry. Tentative operation of the flood doors can maintain the pressure of this soil slurry

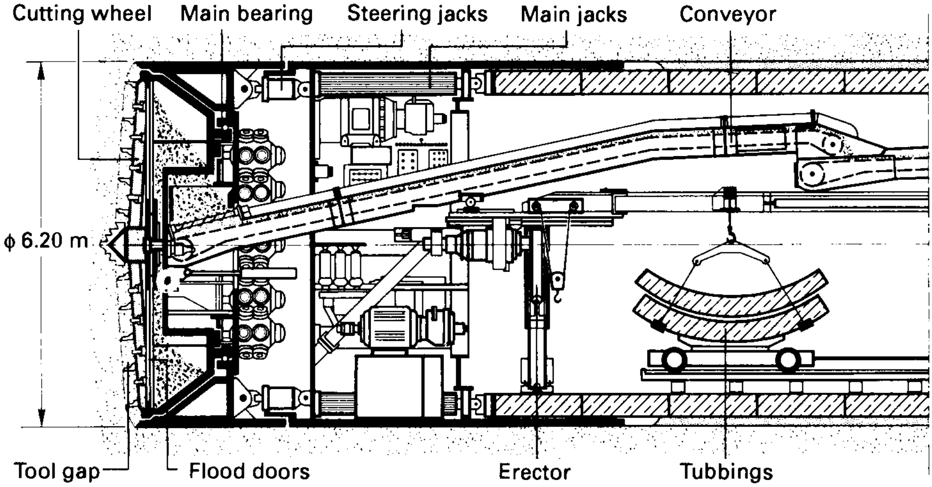


Figure 15.3 Shield with mechanical support

and so support the face. This technique can be practised with success only by experienced crews.

Lining Linings behind shields and TBMs with mechanical support are mainly formed with precast elements, such as cast iron or reinforced concrete tubbings, to accommodate the high thrust forces. Extruded steel fibre concrete is also appropriate behind these machines as a tunnel lining, when it is formed within a fixed steel shutter which can accept these forces.

On the Lovat TBM a temporary wooden lining is often erected behind expandable steel profile rings. The complete lining is erected within the protection of the shield tail, and then behind the shield it is pressed against the ground by expanding the rings with the help of hydraulic jacks. The necessary thrust forces can be transferred without damaging these wooden linings. A second lining of reinforced concrete accommodates long-term loading.

15.2.3 Shields and TBMs with compressed air support

Face support and excavation

Compressed air support of the face during excavation is a suitable method because excavation is not delayed and groundwater is kept away. The disadvantage is that the crew must work under compressed air if the compressed-air area has not been restricted to the working chamber. This method is also restricted to particular soils.

The face is supported by the compressed air because the flow pressure in the ground pores at the face surface is greater than at depth. This means that the more impermeable a face is in relation to the deeper zones, the easier it is to support. If this precondition is not met, problems can arise through an unstable face or blow-outs.

New shield techniques avoid these dangers by spraying the face during excavation with a bentonite suspension to form a sealing membrane at the surface and to raise the flow pressure to a stagnation pressure (Figure 15.4).

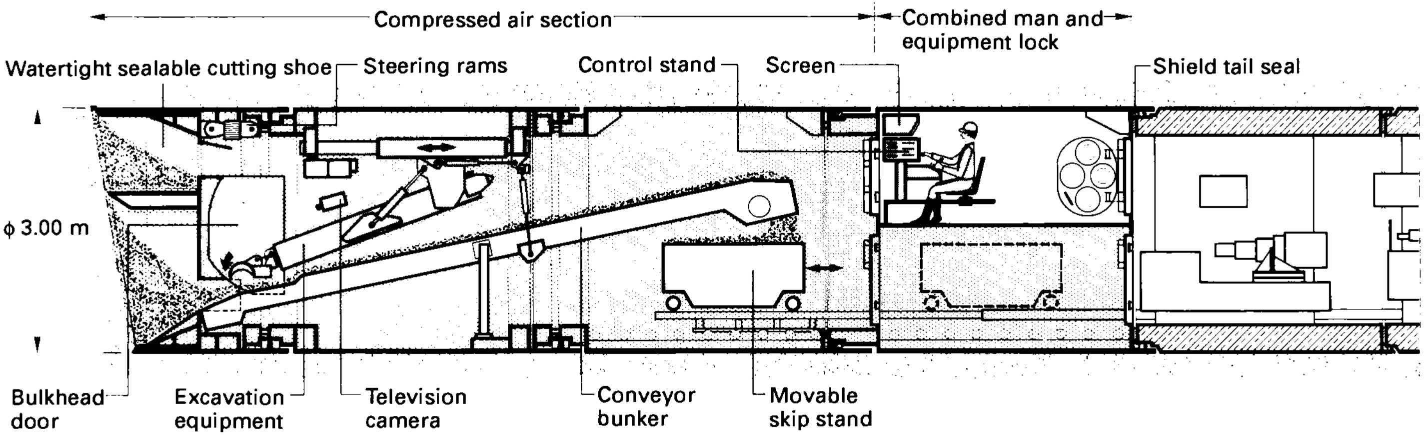


Figure 15.4 Shield with compressed air support

In order to permit the crew to work in mainly atmospheric conditions compressed air could be limited to the working chamber at the front of the shield. However, there is then the danger that the compressed air flows around the shield cutting edge into the steerage gap, which exists around the shield due to overbreak at the shield edge. As the shield tail seal is only partially airtight, the air can flow through this seal into the atmospheric part of the tunnel. The relatively small volume of the working chamber is not a sufficient reservoir to compensate for the outflow of air without large pressure drops, which would then endanger face stability.

So far, shields with only the working chamber under pressure have been used for pipe jacking, for it is only on pipe jacking drives that simple effective shield tails can be employed.

15.2.4 TBMs with fluid support

Face support

The ideal method of working is to drive a tunnel in loose ground without altering the primary stress state. A modern way of approximating this is to support the face during excavation with a fluid, which then serves as the transport medium for the excavated material.

Water serves as the support medium, mixed with an additive which can filter out. The support medium is maintained under a constant pressure, so that the additive filters out at the face and forms a membrane. The support pressure is then transferred onto the face by the membrane. If the surface of the face is sufficiently impermeable the membrane and thus the additive can be dispensed with. In Europe bentonite is usually used as the additive, at about 30–50 Kp/m^3 suspension in gravel conditions. In Japan natural clay is usually added.

To support the face effectively, the support pressure must be held constant. The dynamic effect of variations in the support pressure causes the face to collapse. However, because the support medium is also used for spoil transport, from the working chamber of a 6.5 m dia. TBM, for example, up to 1000 m^3 of fluid can be removed each hour and replaced with cleaned suspension. The extracted volume of fluid must be replaced by an equal volume if no pressure differential is to occur.

Volume flow is monitored by flow meters and controlled by pumps and valves. However, pumps and valves are sluggish control elements, and cannot compensate for momentary volume differentials, which in a fluid lead to immediate pressure changes. An elastic spring element is required to absorb the pressure changes from volume imbalance due to the input and extraction of support fluid. Containers filled with gas can serve most effectively as the spring element when they form part of the working chamber or are integrated close to it in the fluid circuit. Volume changes in the gas result in relatively small pressure changes.

The Hydroshield (Figure 15.5) has proved itself to be a reliable design. The working chamber is divided by an immersed wall and in the upper section to the rear of this wall an air cushion, whose pressure is regulated, compensates for volume changes in the flow of fluid. Pressure variation in the support fluid can thus be generally held to ± 0.05 bar.

Other designs control the pressure without an elastic spring element and employ only pumps, valves and computers. Pressure variation is absorbed by the momentum of the pressurized fluid flowing in the circuit. With such a design, pressure variation is greater, the less the volume of fluid in motion, i.e. during the start phase.

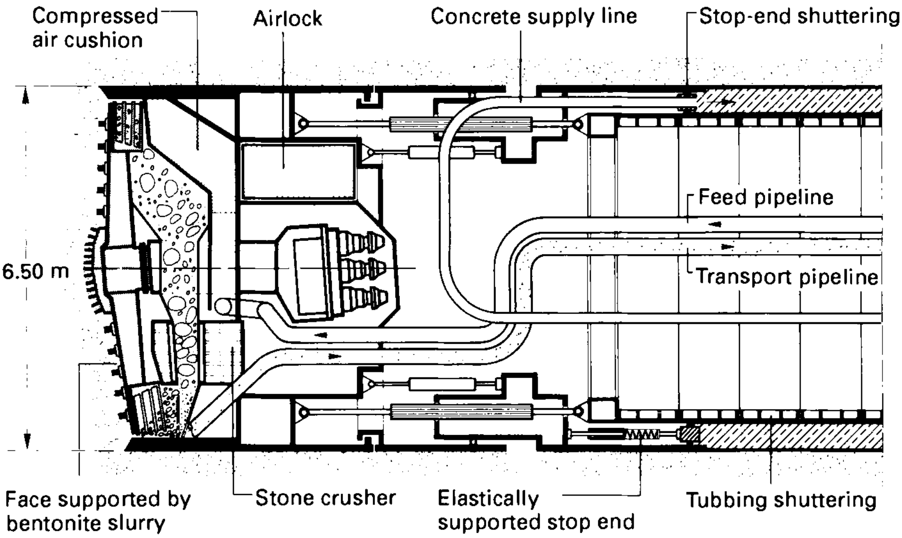


Figure 15.5 Hydroshield with extruded steel fibre concrete lining

Excavation

A cutting wheel, with or without spokes, is almost exclusively used for excavation. Cutting wheels are sometimes open, as purely spoked wheels, and sometimes almost closed. The picks, scraper blades and roller cutters employed on TBMs with mechanical support are used.

The excavated material is hydraulically transported with centrifugal pumps. The edge length of the solids is therefore limited to less than 12 cm for a 300 mm pipeline. This precondition is difficult to maintain if there are many boulders in the ground. Occasionally, in loose ground, the cutting wheels are mounted with roller cutters to break up boulders in the face. The results, when certain requirements are complied with, are encouraging.

Stones up to an edge length of 40 cm can be removed from the face by a cutting wheel and must then be taken by hand from the working chamber or fed, in the working chamber, into a crushing mill. Crushers in the working chamber have proved themselves when they are so arranged that only coarse material passes through them.

In order to remove stones by hand from the working chamber or face, tunnellers must enter the working chamber. For this, the support fluid is pumped out and replaced by compressed air. The membrane, which has filtered out of the support fluid, prevents the compressed air from escaping even in very permeable ground. This procedure can become so routine that even with a considerable number of large stones acceptable advance rates are possible (in Cologne under the Rhine there were 682 stones per running metre with a TBM of 3.6 m diameter).

In impervious ground there is a certain degree of risk involved in supporting enclosed sand or gravel meltwater pockets with compressed air. In such glacial deposits it should be possible to close the cutting wheel.

Spoil transport

The excavated material is moved by centrifugal pumps through a pipeline to a separation plant where the spoil is separated from the transported suspension. The cleaned fluid is returned to the TBM working chamber. The solid content during pumping is about 10% and the flow speed about 3 m/s. On a 6.5 m diameter TBM the diameter of the transport line is 250–300 mm.

Separation

Separation of the excavated material from the support and transport medium is determined by the ground's geology and, in particular, the grain distribution curve of the spoil. If the amount of material of less than 0.06 mm is below 25% the solids are separated by passing coarse-grained material of less than 0.35 mm over a coarse vibrating screen.

Finer material of grain size 0.06–0.35 mm is extracted by the first stage of a hydro-cyclone, and then passed over a dewatering sieve with a mesh size of 0.3–0.6 mm. Even finer material down to a grain size greater than 0.03 mm is removed by a second hydro-cyclone stage and then passed over a high-frequency dewatering sieve with a mesh of 0.1 × 7 mm. When the amount of material of less than 0.06 mm grain size is above 25%, additional centrifuges or filter presses must be used. Depending on the grain distribution curve and local conditions, separation can take place in simple settling ponds. Chemical additives accelerate the process of separation.

15.2.5 TBMs with earth pressure support

Face support

TBMs with earth pressure support have been developed in Japan to simplify or avoid separation of the excavated material from the transport fluid (Figure 15.6).

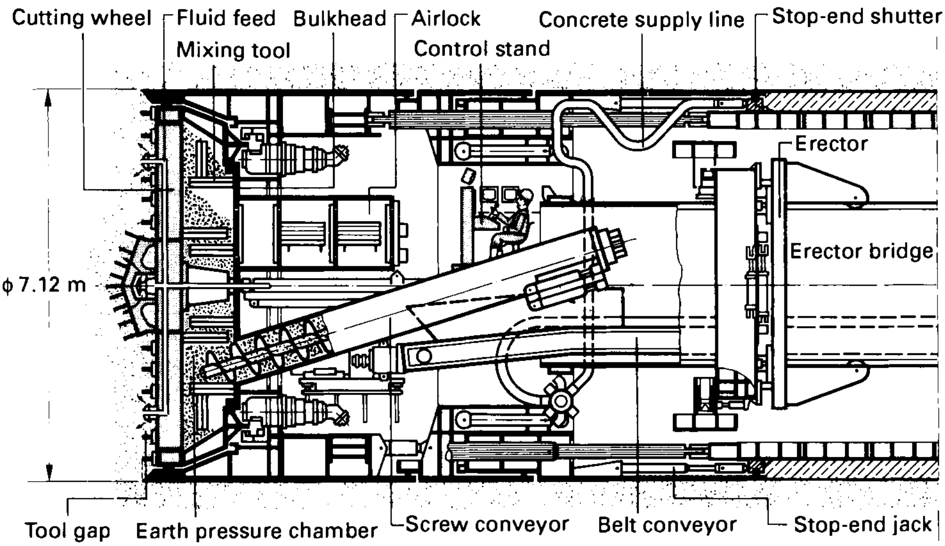


Figure 15.6 Earth pressure-balanced shield

In these TBMs the face is supported by a pressure-regulated earth slurry, which consists of excavated material and inflowing water. The solids content is about 50–80%. If there is not sufficient water make to produce a slurry capable of pressure transfer, water is pumped into the working chamber, occasionally with additional bentonite or natural clay. The earth slurry is mixed in the working chamber if it has not already been scraped off as an homogeneous material. Mixing paddles are mounted on the back of the cutting wheel and on the fixed-pressure bulkhead. Mixing tools which operate independently of the cutting wheel are also mounted in the working chamber.

The pressure of the earth slurry on the face is controlled by regulating the speed of the shield advance, dependent on the rate of the spiral conveyor which removes the slurry from the working chamber. Sensors on the bulkhead monitor the support pressure. However, due to the high viscosity of the support medium, the pressure transfer to the face is imperfect. In the working chamber the pressure drop between the cutting wheel and the rear pressure bulkhead alone is estimated at about 1 bar.

Therefore the pressure of the earth slurry in the tool gap is of the utmost importance for face stability. New TBMs with earth pressure support therefore pump additional high-density slurry through the hub and spokes of the cutting wheel directly into the tool gap. The pressure in the tool gap can be controlled by regulating the amount that the segments between the cutting wheel spokes are opened. In practice this method of support pressure control, which is not as simple as on fluid support shields, leads to greater relaxation of the face than with fluid support shields. However, experienced crews can compensate for this disadvantage by sensitive operation. In addition, the shear strength of the soil is generally higher when TBMs are driven with earth pressure support.

Excavation and spoil transport

Loose ground is generally worked with a rimmed cutting wheel which rotates in front of the shield cutting edge. Scraping tools are used, often in combination with disc cutters. It is difficult to remove obstacles such as stones or blocks by hand from the working chamber, for the substitution of compressed air for earth slurry in the working chamber is a complicated procedure.

The slurry is generally locked out from the working chamber by a screw conveyor. This is mounted as low as possible so that, when necessary, it can clear most of the chamber. In order to transport stones, screw conveyors with a large diameter are used, with or without a core.

Screw conveyors maintain the pressure differential between the support pressure in the working chamber and the atmosphere. They are therefore often very long and have no screw at the end sections. There a body of soil should form (the sand plug) as a barrier against pressure loss. Often two screw conveyors with different diameters and turning speeds are mounted one behind the other. The earth slurry is fed through a hydraulically operated ground flap onto a conveyor belt and removed in normal skips. The long bulky screw conveyors can be kept much shorter when a reliable locking-out system, such as a double piston pump, is mounted at the screw exit. This has been done on the Robbins TBM T1 for the Channel Tunnel.

Earth slurry is often pumped from the tunnel with reciprocating pumps. A mixer is then installed behind the screw conveyor to render the slurry pumpable. The solid content is then about 70%. Water is injected into the pipeline for lubrication. The operating pressure in such a pipeline is about 20 bar.

The drive torque for a TBM with earth pressure support must be considerably higher than that for a fluid-support TBM. Whereas a drive of 240 mt is sufficient on a fluid-support TBM of 6.5 m diameter, a TBM of similar diameter with earth pressure support requires about 650 mt.

15.3 Construction of the tunnel lining

The stress state of the soil is influenced not only by the method of excavation and face support but also by the way the tunnel lining is built. This stress state acts upon the lining and determines load distribution, size and nature of the loading and thus the cost of maintenance and life span of a tunnel. One must therefore attempt to optimize the bedding of the tunnel lining, either by stressing the lining directly against the ground with minimum disturbance or by grouting the gap between the soil and the lining.

15.3.1 Erection behind the shield

Geological preconditions
In order to erect the lining behind the shield, loose ground must, for a limited period, be stable. There must not be a high water make. In addition, stratified material, which leads to local collapse, hinders lining erection behind the shield.

Lining
Precast units (tubbings) of cast steel, concrete or reinforced concrete are mounted with erectors to form a lining behind the shield. To attain higher building rates, several erectors are used to build a ring. The ring erection time is decisive for the rate of advance. It is not so dependent on the tunnel cross section as on the mechanical equipment. Erection time can be reduced to 15 min, but is generally 30 min.

The tubbings which are erected behind the shield form a pin-jointed chain in the plane of the ring and are generally stressed against the ground by forcing in a wedge-shaped keystone. These are not bolted but, at most, dowelled. The jointed ring can also be stressed by hydraulic jacks between the tubbings. The resulting gap is then concreted. Instead of tubbings, shotcrete reinforced with reinforced mesh is used as a lining behind a blade shield.

The rates of advance for a TBM where tubbings are erected behind the shield are very high when thrust forces are transferred not to the lining but to a special thrust anchorage. The soil can then be excavated without being interrupted by ring erection. The structure of the ground may, however, be destroyed by such thrust anchorages.

15.3.2 Erection within the protection of the shield tail

Particularly in loose, water-bearing soil, the lining is erected within the protection of the shield tail. As the shield is advanced, the resulting gap between the tubbing and the soil must be simultaneously pressure grouted if loosening of the surrounding soil is to be avoided. The front of the gap on the shield side is sealed by a sealing unit.

Grouting and tubbing lining
Linings in loose, water-bearing soil are formed from tubbings of reinforced concrete or cast steel, or from extruded concrete. Tubbing elements are very accurately cast and have peripheral seals. They are bolted together to form the ring; adjacent rings are usually only dowelled together. Thrust forces which are transferred to the tubbings compress the ring-joint and seal. The hardened grout later holds the rings in the compressed position.

The tubbings are grouted with cement grout to which bentonite has been added to improve flow characteristics. The equality of the lining bedding depends on the care with which the 10–12 cm thick shield skin gap is grouted. The usual method of grouting through holes in the tubbings is replaced in newer TBMs by grouting through several pipes, parallel to the tunnel axis, which pass through the sealing unit into the shield tail gap. Thus the grout can be injected into the gap as soon as the shield starts to move. However, even this improvment should not obscure the fact that, in unstable loose ground, water and soil will immediately penetrate the shield tail gap if the grout is not maintained at a constant pressure above that of the water pressure and ground load. Generally, there are no mechanical systems provided to ensure this.

Sliding-seal system
Only a sealing system which slides along the shield axis and is elastically supported can provide the spring element required to compensate for variations in volume. These occur when, during the shield's advance, the resulting volume behind the shield tail is not immediately filled with grout at a certain minimum pressure. Without such a spring element, volume imbalance leads to an immediate pressure drop in the grout and penetration of the shield tail gap by soil and water. The elastic support of the sealing unit is provided by hydraulic jacks whose oil circuit is coupled to a gas reservoir.

Seals
The mobility of the seal at the shield tail gap will not maintain a constant grouting pressure if the seals themselves fail and allow the grout to escape. This often happens with inaccurately placed tubbings, when the fairly stiff neoprene seals cannot close the gaps formed at the joints. Japanese brush seals have proved to be effective in such cases. These are arranged in several rows and the spaces between are filled with grease. The grease is maintained at a slight excess pressure to prevent grout entering the seals, which, on hardening, would destroy the sealing action.

If, instead of grout, concrete is pumped through the elastically supported sealing unit, the tubbing bedding is extruded at a constant pressure. Even with an imperfect seal, the concrete itself seals the gaps and joints. The coarser aggregate matrix of the concrete forms a filter which closes even larger gaps.

Extruded concrete
Apart from the tubbing lining, the extrusion method can also produce a tunnel shell within the protection of the shield tail. Concrete is continuously pumped through an elastically supported stop-end into the ring-shaped void formed behind the advancing TBM. This void is bounded on the outside by the surrounding soil, on the inside by fixed steel shuttering and at the front by the stop-end. Control and steering elements ensure a constant pressure in the concrete, so that even in the fluid state it can support the pressure of non-cohesive water-bearing soil.

To achieve this, the elastically supported stop-end is pushed forward by the pressure of the concrete itself. A special hydraulic linkage ensures a parallel advance, a gas reservoir the necessary elasticity and a friction-compensation system correction of obstruction or jamming of the stop-end between the shield tail and the shuttering.

This process is very economical because normal concrete is used. The rheological properties merely have to be improved by the addition of a super-liquidizer. Instead of steel bar reinforcement, steel fibres are added. Microsilicate improves the bond of the steel fibres in the concrete.

Unfortunately, the tunnel lining of extruded concrete is not watertight, and shrinkage cracks in the continuously formed tunnel shell cannot be avoided. Thus a second sealing lining, which can be installed afterwards, is required in water-bearing soil.

15.4 Channel Tunnel machines

The tunnelling machines for the Channel Tunnel are TBMs in loose soil. Because of the significance of this project these are discussed in detail (Figure 15.7).

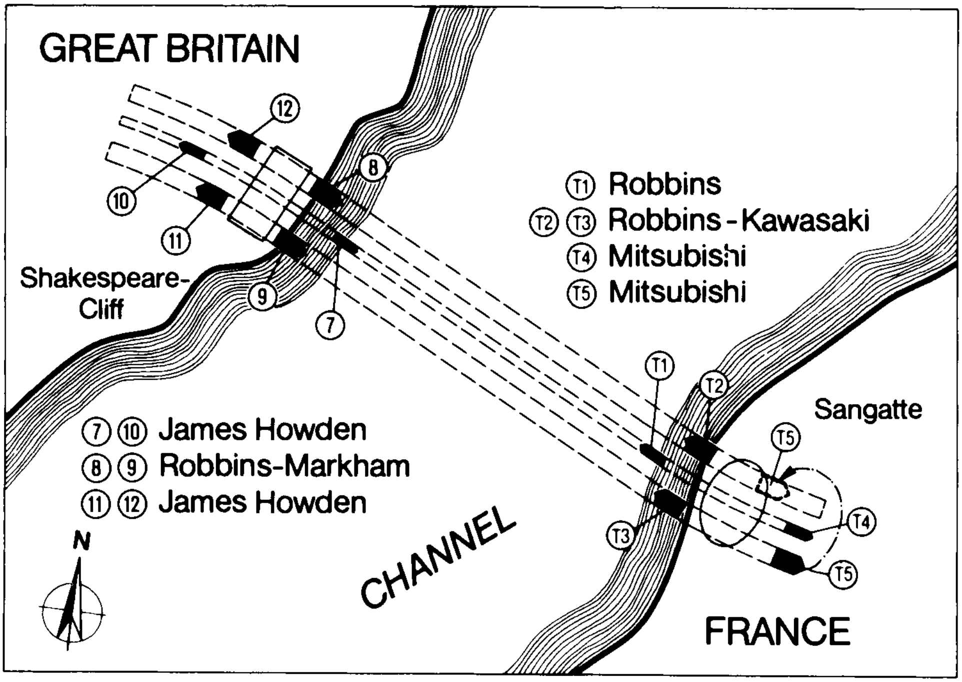


Figure 15.7 The Channel Tunnel machines

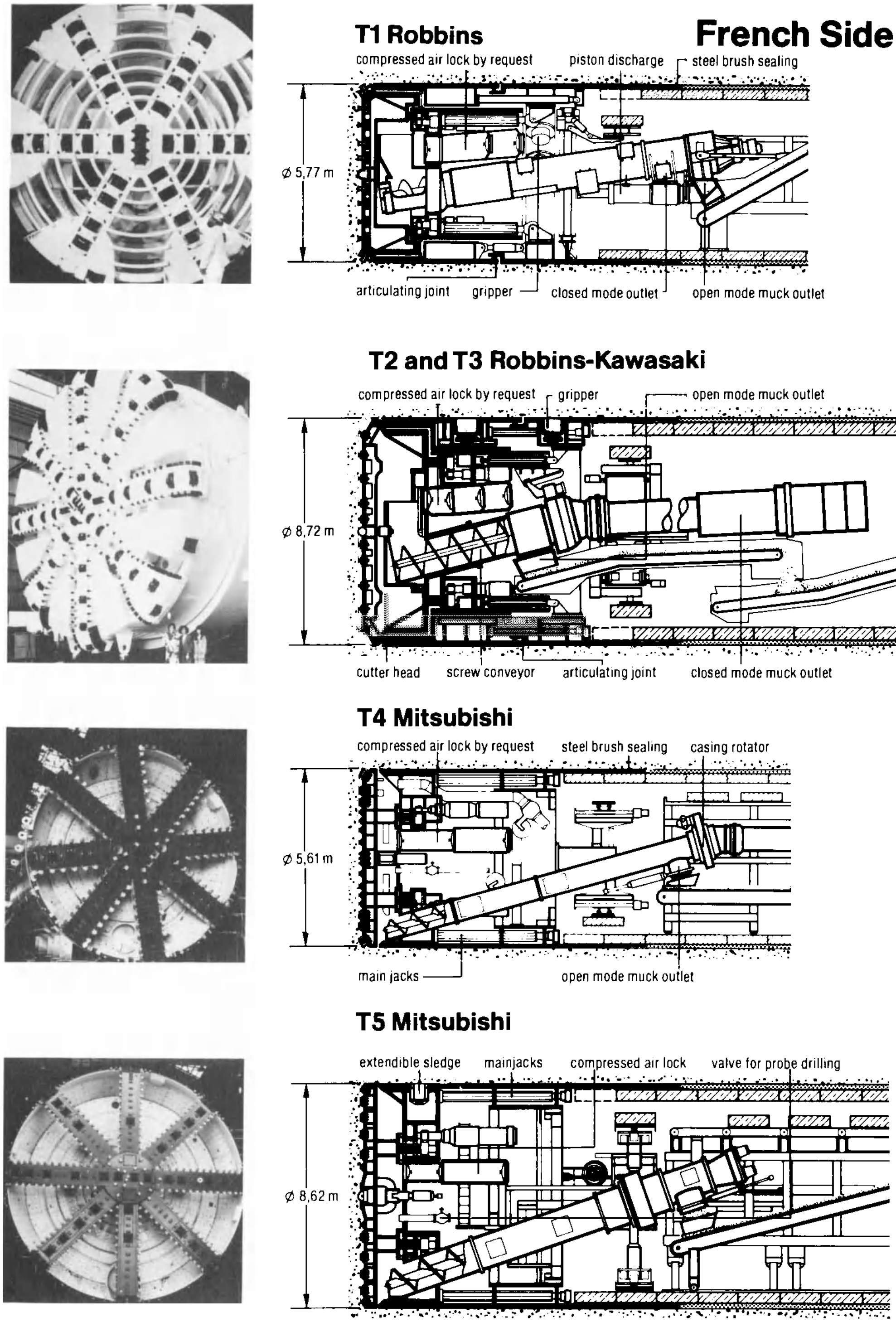
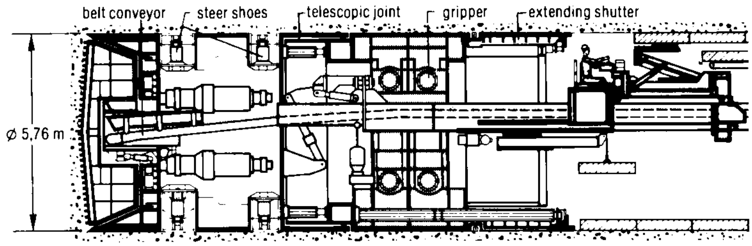
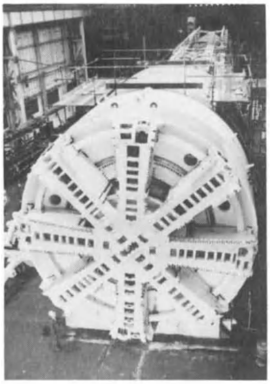


Figure 15.7 (Continued)

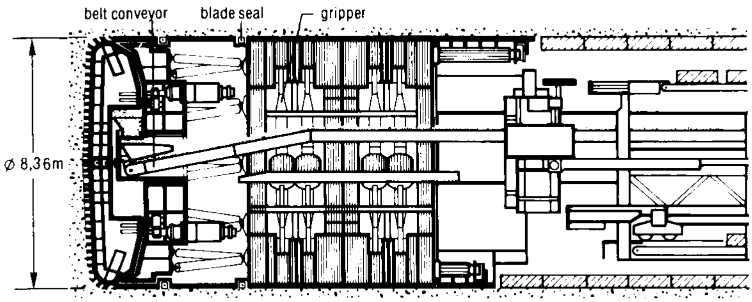
7 and 10 James Howden

British Side





8 and 9 Robbins-Markham



11 and 12 James Howden

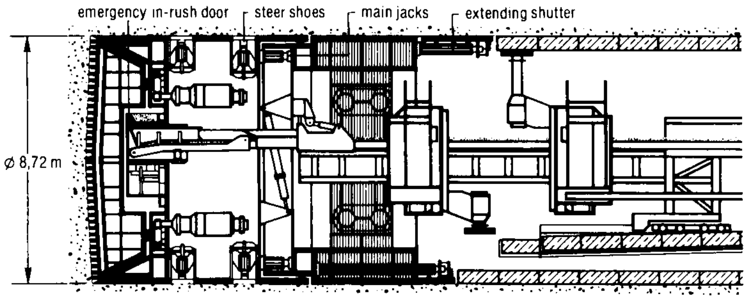


Figure 15.7 (Continued)

Technical Data

		Shield outside diameter	Length of shield	Total shield thrust	Cutter head			Screw conveyor Belt conveyor			Total electric energy
					Torque	Revolutions	Electric energy	Diameter Width	Torque Power	Revolutions Speed	
		m	m	t	m x t	rpm	kW	mm mm	m x t kW	rpm m/sec	kW
French Side	T1	5.77	11.000	4,000	172/ 344	2.5/5.0	882	1,000	8	0/20	2,700
	T2	8.72	13.745	11,500	650/1,300	1.5/3.0	2,160	1,400/1,200	30/14	18/16	4,650
	T3	8.72	13.745	11,500	650/1,300	1.5/3.0	2,160	1,400/1,200	30/14	18/16	4,650
	T4	5.61	10.595	4,000	407	0.9/1.8	750	750	5	0/20	2,350
	T5	8.62	12.610	9,000	664/1,304	1.0/2.0	1,440	1,200	30	0/15	4,110
British Side	7	5.38	13.520	2,080	213/ 142	3.0/4.5	760	800	45	2	1,200
	8	8.36	14.106	7,833	215	3.4/1.7	1,320	-	-	-	2,300
	9	8.36	14.106	7,833	215	3.4/1.7	1,320	-	-	-	2,300
	10	5.76	13.520	2,080	213/ 142	3.0/4.5	760	800	45	2	1,200
	11	8.72	14.030	3,120	349/ 526	1.9/2.9	1,140	1,200	2 x 55	2	2,000
	12	8.72	14.030	3,120	349/ 526	1.9/2.9	1,140	1,200	2 x 55	2	2,000

Figure 15.7 (Continued)

15.4.1 Machine types

The TBMs on the French side of the Channel Tunnel initially drive through the geological strata of the water-bearing white and grey chalks before they descend into the virtually impermeable blue chalk, which is very favourable for tunnelling. These are TBMs which can first be driven as earth pressure shields and later, in the stable, dry blue chalk, can operate without face support. The lining is formed from 1.4 m and 1.6 m bolted steel tubbings, mounted within the protection of the shield tail and sealed with a neoprene comb profile.

On the English side of the tunnel the drive commences in blue chalk and follows this stratum for the entire length of the drive. Therefore the TBMs have no mechanical arrangement for face support during excavation. The 1.5 m wide reinforced concrete tubbings are not bolted but designed as a pinned ring with a swelling Hydrotite seal in the joints. They are erected behind the shield tail and stressed against the soil with a wedge-shaped keystone.

15.4.2 Machine advance

In order to drive the long tunnel in the shortest possible time, the designers of the TBMs have created a system which enables continuous excavation, without interruption during the ring-erection period. On the English side of the tunnel the TBMs have gripper units which are hydraulically extended and bear against the soil. The cutter head is thrust forward against the gripper unit which itself advances in step. Thus the lining can be erected independently of the shield advance. Only soft ground strata, which impede thrust transfer by the grippers, cause lower drive rates as the thrust forces must then be transferred to the tubbings by hydraulic jacks.

On the French side, due to the initially soft and water-bearing soil, the thrust forces are not transferred to radial grippers but onto the tubbings. To ensure, in spite of this, an advance independent of the tubbing erection the TBM shield tail is extended to allow two rings to be built within it. However, the seaward TBMs T2 and T3 are also equipped with gripper systems which can be used in the blue chalk. The rate of advance can then be increased when excavation and tubbing erection can be carried out simultaneously and independently.

15.4.3 Face support and protection against water inflow

The machines on the French side of the tunnel are driven in the white chalk as earth-pressure balanced (EPB) shields. The spoil is removed from the working chamber by screw conveyors. The support pressure is reduced along the screw conveyor, the different TBM manufacturers using various ancillary equipment. On the Robbins T1 a double-piston discharge pump is flanged onto the end of the short screw conveyor as the locking system. The Robbins-Kawasaki T2 and T3 use a very long two-part screw conveyor in the EPB mode. The Mitsubishi T4 and T5 have, in addition to a two-section screw conveyor, a so-called 'casing rotator' as a pressure-reducing system. Subsequently, in the blue chalk, where face support is not required, T1, T2 and T3 will continue to remove the spoil with the screw conveyors but a second opening will be used to feed a conveyor belt. The pressure-reducing systems will thus be bypassed.

The initial EPB mode on the French side of the tunnel has led to a design which will also be safe during the blue chalk drive – where the machine operates without earth pressure support – in the event of sudden water inflow. On the English side, where the TBMs are driven without face support, the spoil is removed from the cutting head by conveyor belts. Various procedures have been adopted to stop unexpected water inflow. The conveyor can be retracted in a few seconds and the opening in the bulkhead wall closes automatically. At the same time, steel blades are thrust out of the shield radially into the surrounding soil to seal the steerage gap around the shield. As the tubbings are erected behind the shield, water could penetrate between the completed tubbing and the shield tail. Thus in an emergency steel shutters would be advanced from the shield tail to close this gap.

15.4.4 Machine guidance

All Channel Tunnel machines are equipped with an electronic optical guidance system from ZED Instruments, London. This provides shield operators with constant information on the position and alignment of the TBM to enable them to steer it. As on the Villejust Tunnel in France, this guidance system will probably be developed for a few machines into automatic TBM steering. On the French side of the tunnel T1, T2 and T3 have a hinged joint less than one diameter back from the face. It is therefore simple to steer them with thrust rams. T4 and T5 are also steered with thrust rams. The body of the shield is, however, very long, exceeding that which would be indicated by the classical relationship of shield diameter to shield length for manoeuvrability, which is generally not less than 1.0.

On the English side the long TBM shield bodies supplied by James Howden are divided by telescopic joints. The front half of the shield body can be pushed into the required direction by radial steering pads. The two seaward Robbins-Markham machines are steered by thrust rams directly behind the cutting head. These are arranged around the perimeter in pairs, each forming a V open at the front, and operated by two overlapping hydraulic pump systems.

15.4.5 Shield tail seal

The tubbings are only erected on the French side of the tunnel within the protection of the shield tail seal. A five-row steel brush seal is used in the gap between the back of the tubbings and the shield tail. Grease is forced between the rows of brushes at a pressure higher than that of the grout behind the shield tail.

An attempt is made to grout effectively the shield tail gap, even when soft grout and high-pressure groundwater occur, with a two-component grout. One component, which sets slowly, is circulated in the feed line at a constant pressure.

On the English side of the tunnel the tubbings are pressed against the ground by a wedged keystone, so that no shield tail gap (which would have to be grouted) remains. However, in water-bearing zones the tubbing is subsequently grouted.

15.5 Future development

Future development of TBMs in soft ground will be accelerated by large projects, such as the Channel Tunnel or the tunnel under the Great Belt, and through increased use of electronics in construction equipment. Application of electronics in construction equipment opens up new areas for TBM operators. These machines can reliably perform delicate and constantly changing tasks. Thus the contractors can master even difficult and variable geological boundary conditions. Today TBMs are already steered automatically; electronic signals from the survey equipment are not only converted into optical aligment information, but are also directly translated into hydraulic control commands. As a result, the TBM follows a more even course than with manual guidance. Electronic control systems in future will also enable the face support pressure to be better maintained within tighter limits, possibly by also introducing an elastic spring element.

Certainly, in the not-too-distant future robots will construct the tubbings, which will not improve the quality so much as the use of slip form systems to seal the shield tail joint. This is the only way to ensure a reliable seal of the shield tail joint, which is the precondition for complete filling of the shield tail gap between the tubbings and the surrounding soil. If concrete is used instead of cement as the grouting material, then even groundwater under high pressure cannot penetrate to the inside of the shield.

The development of tunnelling machines has almost reached the point where we can excavate soft ground from the face without affecting the primary stress state, and where we can install an economical and technically high-quality tunnel lining with slip formwork in a continuous extrusion process.

Chapter 16

Tunnelling machines in hard rock

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Contents

16.1 Introduction

Visualize a mountain of solid granite without a crack or fissure and a powerful machine slowly but steadily grinding its way through the rock, boring a railway tunnel 8 m in diameter, leaving behind it a round, smooth, cylindrical tunnel. This vision, a dream in the 1960s, became an economic reality in the 1980s.

Mechanization of rock tunnelling has been progressing since 1846, when Henri-Joseph Maus mounted a gang of mechanical rock drills on a frame to speed the excavation of the Mont Cenis tunnel between Italy and France. Drill and blast methods are progressing today with high-speed hydraulic-powered drills and computer-controlled jumbos, but drilling a tunnel continuously without the cyclic blasting has obvious advantages. The challenge has been to develop a machine and cutting tools capable of the job.

In 1851 an American engineer named Charles Wilson developed a machine which was to become the first successful continuous tunnel borer for rock (Figure 16.1). However, problems with disc cutter tools and other difficulties made it not competitive with the developing techniques of drill and blast tunnelling.

Other attempts were made and abandoned both in the United States and in Europe from the late 1850s to about 1920, when mechanical tunnelling in rock seems to have gradually disappeared. Practically no serious attempts were made for a period of 30 years until the 1952 developments by James S. Robbins, which combined drag pick cutters with rolling discs. These early attempts were also unsuccessful in hard rock until Robbins built a machine in 1956 which used only

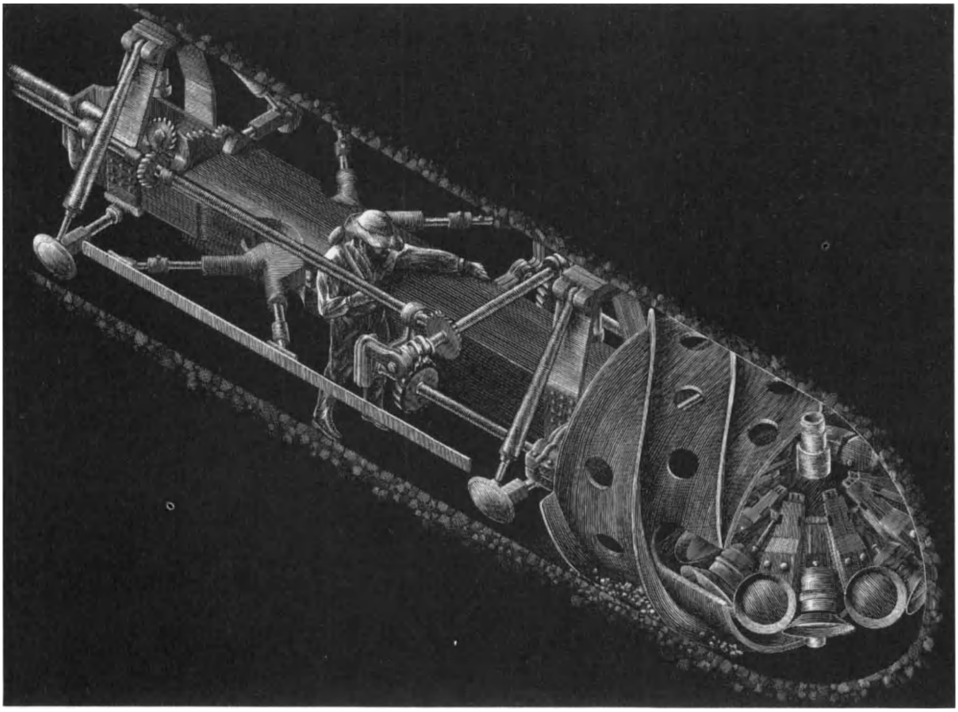

Figure 16.1 Charles Wilson's tunnelling machine, 1851

rolling disc cutters for crushing the rock in a manner similar to Wilson's design of a hundred years earlier.

During the next 30 years more than 200 rock-tunnelling machines were used successfully in soft and medium-hard rocks and more recently in very hard ones. Finally, the cutters and other mechanical systems were developed to a point where hard rock could be bored at a rate of advance higher than could be achieved by blasting methods (Figure 16.2).

The key objective of mechanical tunnel boring has remained high-speed advance of the heading, but safety and efficiency are also important components of driving tunnels at a low cost.

Blasting accidents and rockfalls have cost many tunnellers' lives and a special skill and experience is required to train, manage and staff a crew of drill and blast tunnellers. Those skills have retreated mainly to Scandinavia, where conventional tunnelling is still practised with efficiency, but even there modern hard rock tunnel-borers are being used on an ever-increasing amount of the work due to the higher rates of advance which can be achieved. This is especially true in rock which is not entirely uniform or somewhat fractured and unstable. The rock fracturing not only gives the tunnelling machine an extra advantage of higher speed, but unstable rock can be more safely and quickly supported in a machine-driven tunnel.

An old adage says that good rock makes good tunnellers, and this applies to both conventionally blasted and to machine-bored tunnels, but poor rock does just the

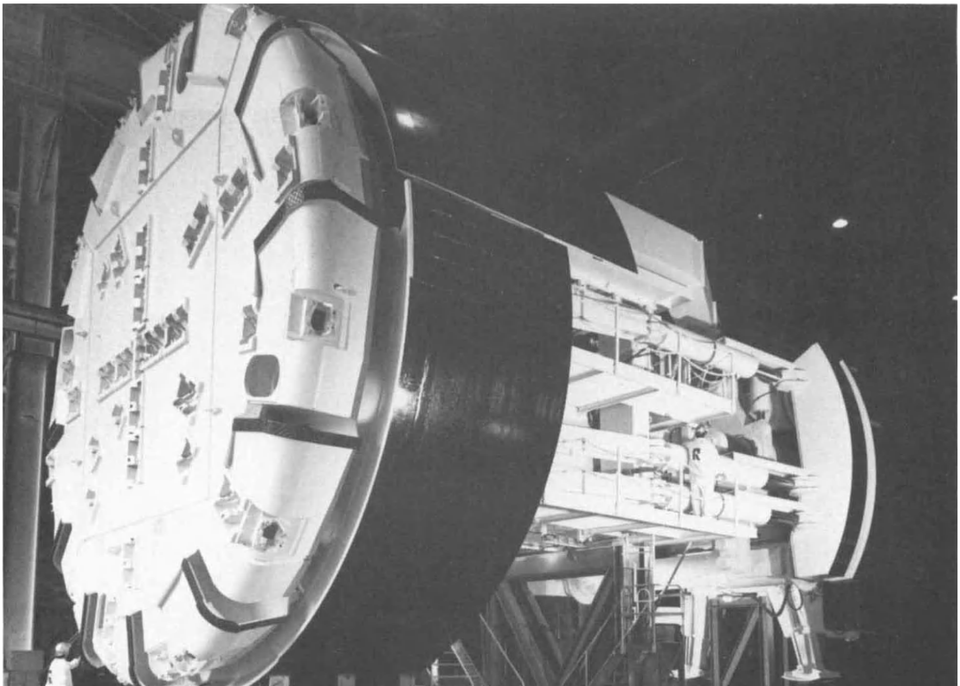

Figure 16.2 A modern hard–rock tunnel borer

opposite. Ironically, it is in the poor rock that the true measure of the tunnel crew is really demonstrated.

Open, unshielded gripper-type rock-boring machines have been applied to many tunnels which, to the surprise of both the owners and contractors, have contained sections of very poor ground. These open machines are often built with some form of roof canopy to provide a degree of protection against falling rock, but they are not well suited to very unstable rock. As a result, tunnelling machines have developed an undeserved reputation of being a method to be used in only good rock. With drill and blast tunnelling more room is available for handling and installing the various types of face and crown support required in caving or running ground. However, some contractors and machine designers have concentrated on the development of poor-rock machine-tunnelling systems which provide the possibility of systematically supporting the rock and the higher advance rates which can be achieved with machines in good rock.

Contractors with experience in the use of these specially designed machines and rock support systems feel that as the rock conditions get worse, the advantage of machine tunnelling over conventional drill and blast methods becomes greater. Unfortunately, only a small number of tunnel builders have the experience to back up this conviction.

16.1.1 Machine-tunnelling limitations

Limitations of machine tunnelling can be placed into two categories: very hard rock and poor ground.

Very hard rock

Only a recent few hard-rock tunnel-borers are designed to cope with the hardest of rock types with penetration rates which are competitive with drill and blast techniques. The machine design constraints can be enumerated starting from the machine front:

1. The cutter rings, which are made from a high-strength alloy steel (usually a tough nickel alloy), can fail when they are subjected to the highest loads attainable, either by fracturing or by plastic deformation. High thrusts are essential in boring hard rock since the rock directly under the cutter must be crushed before the efficient splitting of rock chips can occur.
2. This high thrust requirement must be supplied through a main bearing supporting the rotating cutterhead of the machine. High-capacity main bearing systems are a point of development and may prove to be a limitation in the future.
3. The drive system consisting of motors, gear reducers, drive pinions and the main bull gear mounted to the machine's cutterhead is required to withstand high torque and power together with serious shock loading.
4. Structural integrity and fatigue life of the machine's main elements are increasingly important in the high thrust environment of hard-rock boring.

Poor ground conditions

As described earlier, the improper application of hard-rock tunnel-boring machines to poor ground conditions has eroded the confidence of tunnel builders in the use of machines. In the days when the earliest serious attempts were being made to overcome these limitations the Komatsu Company Limited of Japan, a Robbins Company licensee, designed a hard-rock tunnel machine incorporating a full shield for the Ena highway tunnel in Japan. This complicated machine was designed to withdraw its cutterhead inside the shield when faulted conditions were encountered. The contractor, a joint venture of Kumagai Gumi and Kajima Construction Companies, the tunnel owner Japan Highway Corporation and Komatsu, are all to be commended for their efforts, which proved to be less than successful. However, it appears that the result of this experience coming on the heels of other unsuccessful machine-boring attempts in Japanese rocks seems to have resulted in a serious loss of confidence in machine tunnelling in Japanese rock from that time until fairly recently.

One must recognize that Japan is a country with very little good rock. Their experience led to a period of more than ten years in which, with a few exceptions, Japanese tunnel owners and contractors lost touch with progress and developments in the rest of the world. We must also keep in mind that during this same period Japanese tunnel builders concentrated on leading the world in the development of soft-ground tunnelling techniques. However, in about 1986 a new awareness of progress in poor-rock machine tunnelling was indicated by several contracts being let involving the use of telescopic shielded rock-boring machines.

Construction experts from around the world are now recognizing that machines designed for poor ground conditions can achieve a shorter construction time than any traditional method, but only if the nature of the rock conditions is reasonably well understood or anticipated before the start of construction and if an appropriately designed tunnelling machine and ground-support system are employed.

16.2 Geology

16.2.1 The tunnel route

Tunnel owners and designers are aware that the cost of their project can be substantially reduced by taking great care in selecting the tunnel route and grade which provides the most favourable rock conditions. This is particularly true of urban transportation tunnels such as transit and light rail systems, which are, for the most part, restricted to a near-surface environment due to access requirements. Rock deterioration due to weathering can be difficult to assess, but avoiding the mixed conditions of surface soils with a partial rockface by lowering the tunnel can be especially cost effective.

16.2.2 Tunnel diameter

The tunnel diameter must be considered in the light of its effect on the cost and time to construct. Many modern city metro systems have been constructed in recent years accommodating spacious cars and cavernous stations. The costs have been several times higher per kilometre of completed tunnel than could have been achieved with a system the size of the small-bore tubes of London Underground.

Planners may conclude that the extra cost of a large-diameter system is justified by system efficiencies in peak commuting periods, but they and the tunnel designers should be aware that in unstable rock conditions the problems of the builder (and therefore, presumably, the cost to the tunnel owner) will increase by the square of the difference in diameter. Both the face stability and the crown and wall rock are adversely affected by the increase in diameter.

16.2.3 Rock supports

The geology determines the degree of required ground constraint and control. Tradition, which varies around the world, seems to be as much a determining factor in selection of ground supports as are the engineering features of the rock. Perhaps that is as it should be, since there is more than one way to accomplish a task, and the owner usually must depend on locally available resources of designers and tunnel contractors to get the job done. Most firms do a better job applying a technique with which they have recent experience to solve ground-support problems.

The result is the emergence of what appears to be regional preferences such as the use of steel ribs and wood lagging as a primary lining in the United States (Figure 16.3), shotcrete with rock bolts and wire mesh in Austria, precast concrete segments as a primary lining in Italian rock tunnels and precast concrete invert segments with steel ribs in Switzerland.

Figure 16.3 New York City 63rd Street Subway. Superimposed tunnels supported by steel ribs

All these techniques can be used effectively and somewhat interchangeably in a given rock, but all have certain advantages and disadvantages in various rock conditions. Tunnel designers should be aware of the effect their selection of rock support will have on the bid price and advance rate of the tunnelling system as well as the objective of primary or final rock support. They should also be aware of the effect on the type of machine which can be used to apply the chosen rock support.

16.3 Performance in various rock conditions

16.3.1 Solid rock

Massive unfractured rock must be considered an extreme rock condition and one which is not often found over long stretches of tunnel length. However, this condition must be faced occasionally in limited lengths even at locations where tunnels are built in jointed rock. The problems encountered by tunnelling in those conditions are (1) advance rate and (2) cutter costs or (3) other spare-parts costs. These, of course, depend on the strength and abrasivity of the rock, but it must be emphasized that completely massive fresh rock, particularly crystalline rocks such as granites, granitic gneiss or basalt, will be bored at a significantly slower rate of penetration if the rock is truly joint free than will a rock with the same strength with minor jointing spaced at one metre centres. Natural rock fractures have a larger effect on machine rate than most analysts or formulas give them credit for.

Machine design factors

Looking at a truly massive rock, as may be found, for example, in Scandinavia, the machine design factors affecting machine penetration rate are:

1. Total machine thrust;
2. Cutter spacing (i.e. cutter density);
3. Cutter diameter and edge geometry;
4. Machine cutterhead turning speed in revolutions per minute (usually a function of the bore diameter);
5. Cutterhead drive torque.

These factors, when determined, also affect other rock-breaking and machine design specifications such as the ratio of crushed rock to split or chipped rock. This ratio can be expressed in rock as specific cutting energy in J/cm^3. The main machine specifications also determine such items as the cutter bearing design and machine main bearing requirements. Machine designs have progressed slowly but steadily over the past 20 years. Figure 16.4 illustrates the improvment in the penetration rate of tunnelling machines in recent years.

In Figure 16.4, steps which occur in the curves in 1980 and 1988 result from the introduction of a new cutter with increased capacity. It can be expected that the trend shown will continue as new materials and mechanical systems are developed.

Cutter usage

The rate at which cutters are consumed is one of the most important variables in rock boring, probably the most important in hard rock. Cutter costs may vary from \$1 to \$20 per cubic metre of rock *in situ,* depending on the cutter design and rock characteristics. Shale, marl or chalky limestone, with a strength of about $350\,kPa/cm^2$, may provide a cutter ring life of 1800 km of rolling before the ring is worn out, resulting in a 1989 cost of about \$1 per cubic metre, including rings and

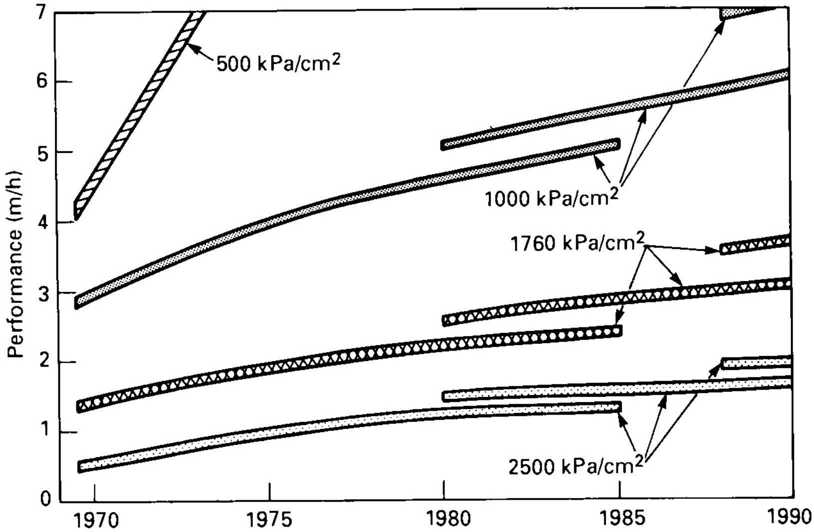

Figure 16.4 Historical development of TBM penetration showing effect of rock strength

other cutter parts such as hubs, bearings, seals, etc. A hard limestone or slate or moderately hard sandstone or mudstone of approximately 1050 to 1400 kPa/cm^2 could result in a rolling life of between 700 and 1100 km yielding a cutter cost of $3–6 per cubic metre.

Figure 16.5 illustrates some typical cutter costs in 1989 US dollars in average rocks of the type and strength specified. These curves are based on steel disc cutters with a thrust capacity of 25 metric tons applied to the full-rated thrust only in the hardest rock types. Thrust per cutter will be approximately 7 tons in rock with compressive strength in the range of 350 kPa/cm^2. Assumptions must be made as to

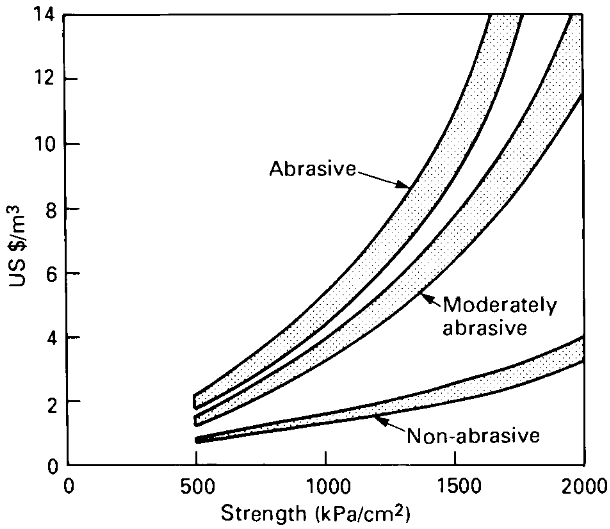

Figure 16.5 Average cutter costs in 1988 US dollars versus rock strength

the other variables which affect cost and performance. In this example the average cutter spacing on the face of the TBM is 75 mm. Of course, other values will yield other results.

Utilization

The key to lower tunnelling costs is more metres per month. High average advance rates can be achieved by combining a good instantaneous penetration rate with many hours per day of actual cutting time. In both hard and weak rock the number of hours boring per day is often more important than the actual rate of penetration.

High utilization can be somewhat easier to achieve in hard rock than in softer rock because less time per day is required for car changing, muck-haulage delays, and for other causes such as installation of track and extending the ventilation column and power and water lines. On the other hand, hard abrasive rock may cause much more frequent cutter changes, and downtime for changing cutters can be one of the most significant causes. Downtime for both major and routine repair and maintenance is also higher in hard rock due to the serious shock loads which result from the rock-fracturing action.

A reduction in the boring-machine utilization can result from designs which are too complex even if they are directed at improving efficiency. Apart from unpredictable poor ground which causes a change in the routine, a machine-bored tunnel should be mainly an exercise in logistics. The challenge is to organize the workers so that the muck can be loaded and hauled out while the tunnel supports, tracks and vent pipe are hauled in and emplaced in a manner which results in the minimum downtime.

The more complex the back-up system, the more attention must be given to mechanical, electric and hydraulic features to make it work effectively. A complex trailing gantry system may cause more downtime in maintenance than it saves in efficiency. A simple single-track trailing gantry was used on the Oso Tunnel, Colorado, in 1966, followed by a series of movable California switches at intervals of about a mile. The tunnel heading was advanced 6600 ft (just over 2 km) in one 30-day period on that job for a world's record, which stands today. More locomotives and muck cars were required for that achievement than would have been the case with a more complex double-track muck-loading system, but the simplicity was at least one of the important contributors to the consistently high advance rates.

Utilization and availability are of prime importance to the tunnel contractor and will determine the speed, efficiency and cost of the construction. Availability may be determined by the quality of maintenance and the operating procedures, but it is also the responsibility of the equipment manufacturers. Since utilization is a function of availability, among other factors, the equipment suppliers are also partly responsible for high utilization. Tunnel-boring machines and trailing systems are custom designed, and will therefore have a much higher incidence of mechanical difficulties to be corrected in the early stages of the tunnel than would be expected from standard heavy construction equipment. How this factor is estimated before construction and the risks shared with the equipment suppliers is often a matter of delicate negotiation.

Operating statistics

It is important for the tunnel builder to know what is happening in the tunnel heading in order to be able to focus on those items which can have the greatest impact on improving utilization. Experience shows that one should not rely only on the shift report of the tunnel machine operator. Several computer-aided data-collection systems have been devised and some are available as software programs. Other systems utilize a detailed report usually produced by both the machine operator and another junior engineer, to record the duration and cause of every downtime, even those lasting only a few minutes. Careful analysis of these reports will reveal the opportunities for improvement.

In developing the utilization statistics the following factors should be recorded:

1. Boring time (hour meter on the machine);
2. Shift time made available for boring;
3. Total time, i.e. 24 hours per working day (removing planned holiday time);
4. Scheduled maintenance;
5. Planned labour-related delays (travel time, lunch, etc.);
6. Other planned delays such as gripper and propel resetting, scheduled cutter inspection, probe hole drilling, planned cutter inspection and changing.

The factor of machine-caused unplanned downtime should be further broken down into subcomponents as follows

The machine
Cutterhead drive
Unplanned cutter change time
Hydraulic and lube oil
Electrical
Machine conveyor
Mechanical general
Miscellaneous (electronic, etc.)

Back-up trailing system
Surveying
Track installation
Conveyor systems
Vent line and scrubber
Power cable and lighting
Water pumping
Grouting
Car handling
Water and compressed air system

A typical hard rock tunnel with little or no requirement for tunnel support may have a utilization breakdown, as shown in Figure 16.6.

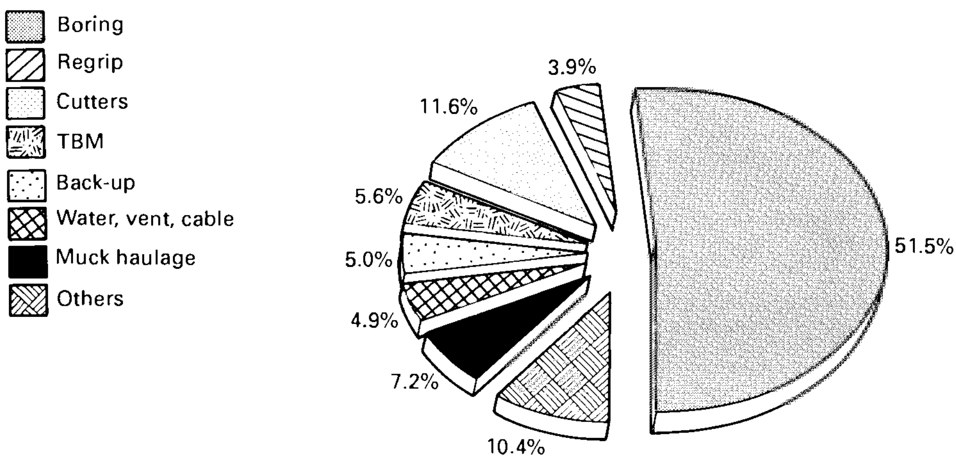

Figure 16.6 TBM utilization on two Norwegian tunnels at Nedre Vinstra in 1987

16.3.2 Limitations of size and shape

The largest tunnels being built are highway tunnels. In poor rock or soils they may be built by constructing a series of small peripheral tunnels which are filled with concrete as work progresses, thus constructing the lining in advance of excavating the core of the tunnel. They may also be driven by excavating a top heading and benches (see Chapters 13 and 17).

In relatively good rock one can consider a full-face tunnel borer. These machines have been built to more than 12 m in diameter, but in very good rock conditions there is no reason why they cannot be built to 15 m or more. The rate of advance of a tunnelling machine is in inverse relation to the diameter. In other words, a machine of 5 m diameter will advance at double the rate of a machine 10 m diameter, assuming both machines are designed to have their peripheral cutters turning at the same maximum speed. In very hard rock, the penetration per revolution of the cutterhead may be as little as 5 mm (depending on the design specifications of the machine and its cutter system). It is apparent that if a machine of 12 m diameter is turning its cutterhead at only 4 revolutions per minute, the penetration rate will be only 2 cm/min or 1.20 m/h. While this may be a satisfactory advance rate in some circumstances, it requires an excellent utilization to achieve a competitive average advance per day. Unstable ground (which should be expected in a larger-diameter tunnel) will reduce the utilization significantly.

Non-circular tunnels

In 1983 the Norwegian Public Road Administration let a contract to construct twin highway tunnels bypassing the city of Bergen. The owner Hordaland Vegkontor, chose to bore tunnels with a 7.8 m diameter hard-rock tunnel-borer to achieve a short construction schedule. As a result, they brought the tunnels into operation 12 months earlier than would have been possible if conventional drill and blast tunnelling had been employed. However, in designing the tunnel it was recognized that the 7.8 m circular section was not quite large enough to provide the clearance required for traffic. They therefore employed drill and blast techniques to cut out lower corners of the tunnel, converting it from a circle to a horseshoe shape.

This blasting was quickly accomplished following completion of boring but left the tunnel somewhat more ragged than had been anticipated due to the rock jointing. Similar enlarging work will almost certainly be accomplished mechanically on future tunnel projects with large drum-shaped cutter wheels fitted with disc cutters. Mechanical enlarging can be accomplished simultaneously with the advance of the circular bore, thus completing the excavation sooner and with smooth wall contours.

An early Robbins tunnel borer used in Pittsburgh in 1956 utilized invert cutting drums to create a flat floor and in the 1980s both the Wirth Company and Mannesmann-Demag built machines for tunnelling in West German coal mines with wall-cutting arms to alter the shape of the circular bore.

In 1985 the Selkirk Tunnel Constructors, joint venture of S. A. Healy, Atlas Construction and Foundation Company, constructed the Rogers Pass railway tunnel in the Selkirk mountains of British Columbia. This was an 8.2 km tunnel bored at 6.8 m diameter by a hard-rock tunnel-borer through micaschists, quartzite, marble and phyllite. Once again, the objective of the joint venture was speed of excavation, which they achieved in completing the job on schedule in June 1986. However, a circular section is not well suited to a single-track rail tunnel. A tall horseshoe is the shape most commonly employed. In this case a circular bore of 6.8 m diameter was chosen for a top heading. The lower half of the tunnel was blasted as a bench following completion of the TBM bore.

This was again an opportunity to bore the lower bench with a specially developed mechanical cutter to achieve the required shape with smooth walls and in a continuous advance together with the circular bore. One should expect to see this technique employed in the 1990s.

Other non-circular rock-cutting systems

Road headers or boom miners were developed for cutting development headings in coal mines through sedimentary rock and coal. These are successful excavating tools when applied to weak or moderately hard rock types and have enjoyed some success applied to transportation tunnels in appropriate geological conditions. They are, however, as a rule, not suited for cutting hard rock and are therefore not treated in this chapter.

After a two-year study developing the requirements for a hard rock road header in 1982, the Robbins Company selected a concept of their consultant, David B. Sugden of Tasmania, for full development. This idea was directed at developing a non-circular hard rock cutting system to be applied to underground mine development and was dubbed the 'Mobile Miner' (Figure 16.7).

A boom mounts a large cutter wheel with a transverse axis having rows of cutters arranged only on the periphery of the wheel. As the boom is swung from side to side an excavated shape is generated with a flat roof and floor and curved walls. Although the prototype machines have operated only with this side-swinging action, in order to cut openings which are better suited to vehicular tunnels the cutterhead boom must be elevated up and down and at the same time swung from side to side. In this way a horseshoe-shaped excavation can be generated.

A side-sweeping Mobile Miner has been boring in hard quartzite rock at Mount Isa Mines in Australia, boring tunnels 3.7 m high by 5.5–7.4 m wide. A second machine is under construction for delivery in 1990 which will cut 4.1 m high by 5.5–8.0 m wide.

Softer or weaker rock types

Modern hard-rock tunnel-boring machines have the power and thrust available to penetrate weak rock at extremely high advance rates. However, their muck-removal systems, including the cutterhead muck-gathering buckets, are often not designed to collect and pass muck at the rate such a machine can penetrate the rock.

Machines designed for high-speed advance rates in weak rock have large bucket openings which are susceptible to over-excavation in unstable, caving rock, but they can penetrate the rock at rates of 8–10 m/h. Such machines have achieved advances of 120 m/day, and, in the case of the US Bureau of Reclamation's Oso Tunnel in Colorado, have bored 2000 m in one 30–day period (see Section 16.3.1).

Clearly, the problem which had to be solved in these cases was not high-speed boring but the logistics of providing support to such a machine. The back-up system or trailing equipment behind the machine must be designed to load the muck efficiently into muck cars for haulage out of the tunnel, and at the same time provide for the installation of tunnel supports, track, ventilation, power cable, lighting, etc. Bringing in miners and materials and carrying out tunnel muck must be done with a minimum delay to achieve good machine utilization. Both the

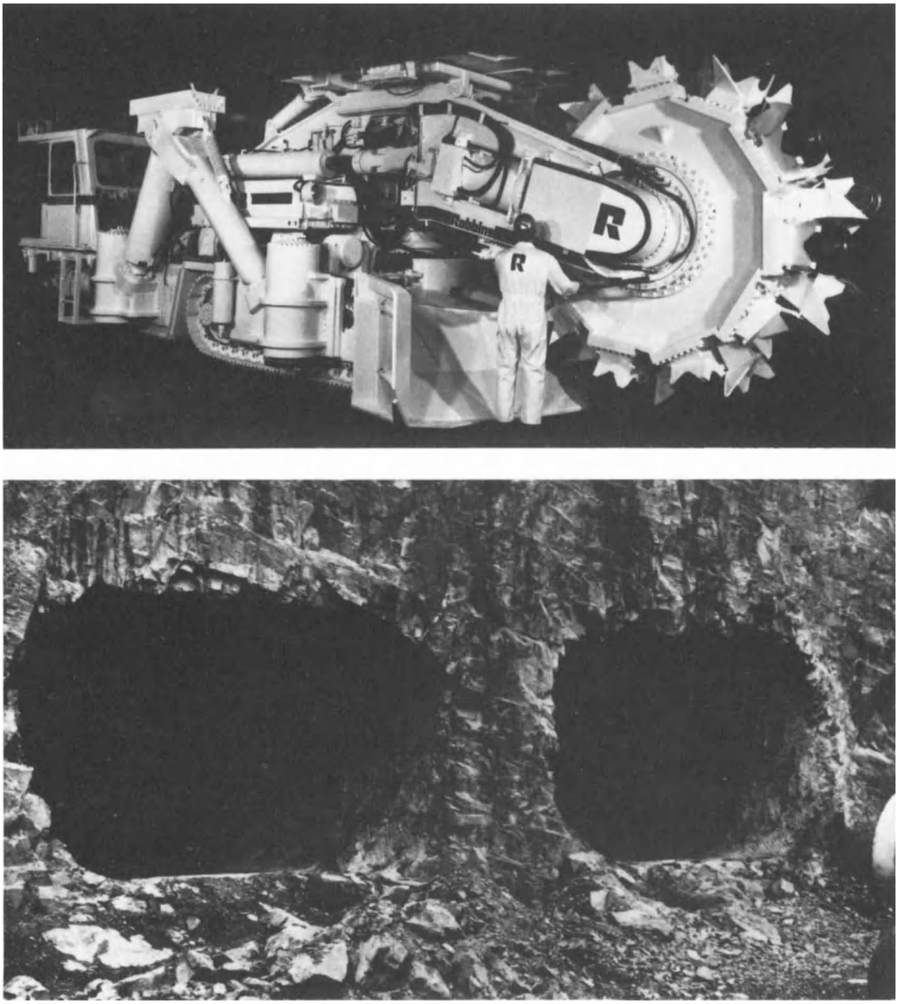

Figure 16.7 The Mobile Miner

Blanco and Oso tunnel machines on the San Juan Chama Tunnel project in Colorado achieved days of 20 h of machine-operating time while boring at average rates up to 6 m/h.

Among the reasons for the achievement of these considerable advance rates on a daily, weekly and monthly basis was a very simple design of the back-up system. Downtime due to repair and adjustment of complex car-handling mechanisms and operations requiring close vigilance of the tunnel crew will usually not produce the results hoped for. On the other hand, a simple system may require more workers in the tunnel than one which utilizes remote or automated systems with television monitoring and a multitude of limit switch activators.

A world record advance must be planned carefully and the back-up system designed for that achievment. A record is a worthy accomplishment, and contributes to tunnelling technology as well as to the reputation of the contractor.

However, it may not be the most cost-effective way to drive a tunnel. It will almost certainly require extra workers and supervision in the tunnel for logistics control, high-speed track laying and installation of tunnel services. It will probably also require more locomotives and cars to ensure having a train always at the proper position to avoid muck-haulage delays.

In recent years the European contractors, notably the Austrians, Swiss and Italians, have developed very efficient tunnel systems which, although somewhat sophisticated, allow high advance rates in good rock with better than average utilization while keeping a very small crew in the tunnel. They have found this to be a cost-effective approach, but have used more highly qualified personnel than would be found in most other parts of the world: that is, more fully qualified engineers and highly trained mechanics, electricians, welders and hydraulics experts.

Careful thought should be given to matching the machine to be used with the anticipated geological conditions and then developing a back-up system which will achieve a specific set of objectives for the entire tunnel job. This should result in a completely integrated tunnelling system.

16.4 Poor rock

Most tunnels of a length sufficient to justify the use of a tunnel-boring machine have some poor rock. Some have just a small amount which is isolated and can be planned for. Others, especially those under high cover where exploration is expensive, may encounter poor rock much of the way or where it is totally unpredicted.

For the purpose of this discussion we will refer to poor rock as unstable rock which will cave in at the face, crown or walls due to faulting, jointing, weathering or ground stress. Poor ground may be so unstable that it caves in as the cutters touch it or the cave-in may run against the machine's cutterhead, creating a chimney or cavity ahead of and above the machine. It may be temporarily stable at the cutterhead but fail a few minutes later from the crown of the tunnel of the upper quarter points, or it may disintegrate gradually and, if unrestrained, cave in a diameter or two behind the tunnel face with large pieces of rock falling in from as low as the tunnel midpoint.

16.4.1 Predictable poor rock

In some tunnels the geologists and rock mechanics specialists can predict fairly accurately where the rock will be unstable and what type of instability will be found. That may be where the geology is well known and previous tunnellers in the area left reliable records of their experiences. In that case, a contractor can select a machine design which is specially suited to deal with those geological conditions. For example, it may be determined on a subway project that a programme of ground consolidation from the surface in two critical areas will provide a stable rock formation allowing the use of a standard open-gripper TBM and that the best choice will be a used machine.

On a different job it might be determined that the geological conditions can best be handled with minimum risk by use of precast concrete segment lining and a double shield. The double-shield design has the versatility to be able to place other types of primary tunnel supports such as ribs and boards or wire mesh and rock bolts in better ground, and in the one section of the tunnel known to have very good rock the machine can progress somewhat faster, leaving the rock completely unsupported while placing only a precast invert segment to support the track system.

16.4.2 Variable ground and unknown conditions

In case of high rock cover in complex geology it is often extremely difficult to get a good picture of what to expect in advance of construction. A few strategically placed boreholes may provide much-needed information, and analysis of satellite photography can indicate probable faults with great accuracy, but geologists are often not in a position to predict how the ground is going to react to the tunnel opening at depth. Rail and road tunnels have been driven in these conditions in the European Alps, the Rocky Mountains of Canada and the United States and in the Japanese Alps.

Under these circumstances, a prudent planner would try to imagine what conditions might exist based on the best information that can be gleaned particularly from local geotechnical firms, who may have experience as near to the area as possible. One can list all sorts of problems which would be difficult enough to force a conclusion that the tunnel was not feasible.

If conditions are listed for each segment of the tunnel that can be identified as a distinct length in a particular geological block, then probabilities can be assigned to the various conditions considered to be possible and what percentage of those conditions may be expected. A worst case, most optimistic case and most probable case can be developed for each segment of the tunnel.

Tunnel builders must select a method which will allow them to overcome any obstacles and to pass through with reasonable efficiency any conditions which have more than a very small percentage of probability. If the conditions are really expected to be quite bad, the most important step for the tunnel owner will be to select a contractor with recent successful experience in similar conditions, and not necessarily a low bidder. This may call for a restrictive pre-qualification for bidders on such a difficult job. The tunnel contractor must make the choice of whether to select equipment and a system which will produce the best advance rates in the conditions known to be the majority of the job and develop plans for getting through the minority of conditions to which the basic tunnelling system is not well suited. Alternatively, the contractor may choose a more conservative approach with a very flexible or versatile system which can be adapted to whatever is found to exist as the tunnel is being driven.

16.4.3 The versatile solution with a tunnel-boring machine

It must be assumed that a special-purpose TBM will operate more efficiently in a given set of conditions for which it was designed than would a more general-purpose machine designed for adaptability. The versatility that comes with being adaptable causes many compromises and reduced capacity of certain machine functions. However, the total time to complete construction of the tunnel must be evaluated. If variable geological conditions exist, the tortoise may beat the hare.

A striking example is the 7.6 km Oso Tunnel, referred to earlier. The machine bored the first 350 m of tunnel in good uniform shale rock and accelerated to a pace of 20 m/day before encountering broken and wet rock, then alluvium with boulders, clay, water and sand. The machine was surrounded with loose materials and hand mining through the 270 m of unexpected conditions, which was found to be a buried river course, required 7½ months. The machine was put back into operation and accelerated to an average rate of progress of 60 m/day. It was then that the machine bored over 2 km in one month, but the overall average for the tunnel from beginning to end was 21.5 m per calender day.

If the conditions had been known to exist and a suitable machine and method had been employed to bore through this material at a reduced rate of, say, 15 m/day then a rate of advance in the balance of the tunnel of 30 m/day (one half of that actually achieved), this would have resulted in a quicker completion time for the job by 2½ months.

The Oso, Blanco and Azotea tunnels were driven through air-slacking shale which deteriorated rapidly when exposed to the atmosphere. The final cast-*in-situ* concrete lining was placed with a great deal of difficulty and expense, and required considerable extra quantities to fill ravelled overbreak, which could have been avoided if the rock had been covered by a sealer coating during the drive, close behind the TBM. Experimentation was carried out but the contractors did not have the advantage of today's materials. An alternative solution which would be seriously considered today is the use of either primary or final precast concrete segments as a means to cover the rock immediately to provide a quicker and cheaper final completion of the tunnel. These methods would, however, preclude the ultra-high-speed monthly advance rates that were actually achieved in the drives.

What type of versatile or adaptable TBMs might be considered for variable or unknown geological conditions? The answer to that depends on a guess as to what conditions may be encountered and the primary support system to be employed.

16.4.4 The shielded TBM for rock

A normal soft-ground combined with a relatively closed or closable cutterhead may be a good solution for rock boring in poor conditions. However, rock boring provides the opportunity to grip the tunnel walls for a thrust reaction. A double shield or telescoping shield with grippers in the second shield allows the front shield to bore forward while the rear one remains in a fixed position. This permits the installation of tunnel-lining segments within the protection of the rear shield while the machine bores ahead. The ground remains covered completely by the telescoping shields as they extend and retract.

One of the advantages of the telescoping double-shield designs is that they permit the use of any rock-support system from sealed and bolted segments to no supports at all. This versatility allows the tunnel builder to select the type of supports which best match the ground conditions at any point in the tunnel. However, this creates two practical problems.

First, one must see the rock to be able to judge its condition. This requires leaving some rock unsupported for at least a short length if segmental lining is being used. Alternatively, one can try to inspect the rock from within the cutterhead cavity but with only a very restricted view.

Second (and most important), the routine is broken every time a change is made in the rock-support system. Speed is the real cost saver in tunnelling, and the way to achieve speed is to develop a routine so that everyone in the tunnel knows exactly what to do at all times and can anticipate any problem that will create a delay, such as a shortage of segment bolts or the need to extend the fan line.

A change in the routine from placing full-circle ring beams and wood lagging, for example, to installing top arches and wire mesh with rock bolts will require a completely new routine to be developed. It is usually surprising how long an experienced crew will take to adjust to the new routine and operate at maximim efficiency. If, meanwhile, the rock conditions change again, the former support system might again be required. However, one must not assume that the crew will drop quickly into the same performance they had two weeks before. They will probably need at least a week or more of sorting out the tasks again to achieve the former efficiency.

The versatile tunnelling system may result in the saving of support costs over long stretches of tunnel, but it usually pays to continue the routine of placing primary supports if short sections of tunnel are encountered where they may not be required.

16.4.5 Moving rock – when the mountain fights back

In-situ rock pressure may be the result of various geological phenomena such as mountain-building crustal movements which can be traced back to plate movements on a regional or global scale, simply from high cover over the tunnel and the mass of rock to be supported, or chemically induced swelling of minerals which are permitted to hydrate due to water flows caused by the tunnelling operation. In large areas of the northern hemisphere swelling may be the result of relief of locked-in stresses introduced when the land was covered by thick glaciers. The rock and soils were over-consolidated by the ice pressure and movement. When the ice melted, the earth rebounded, relieving some of its vertical stress but the horizontal stress remained locked in.

High natural rock stress may be a near-surface phenomenon as well as deep rock and it is harder to predetermine stress levels accurately in advance of construction. The rock fabric may be strong enough to resist the natural stress and in most cases it is, even considering the stress multiplication at the surface of a bored tunnel. If the rock is fractured or jointed the rock's resistance to movement will be a function of both the rock strength and the friction on the fracture surface.

In the cases where very little friction exists or where the fracturing is intense, as it may be in a fault zone, the rock stresses are automatically relieved by movement which may be gradual as in a rock squeeze. The movement may also open joints and permit gravity induced failure of the rock at the tunnel face or crown.

If the rock stresses are high and significant movement can be expected before reaching a condition of stability one must be very cautious about the use of a shielded tunnelling machine. Double-shield machines are particularly susceptible to being trapped by ground convergence due to their relatively long shielded length (Figure 16.8).

Examples of jobs where double-shield machines have been trapped in squeezing ground are the Vat Tunnel and the Stillwater Tunnel, both built for the US Bureau of Reclamation for their Central Utah Water Project in the 1980s. The Stillwater tunnelling machine was converted in the tunnel to a walking gripping shield which could yield with the converging ground and, at the same time, restrain the rock to the point where supports could be installed.

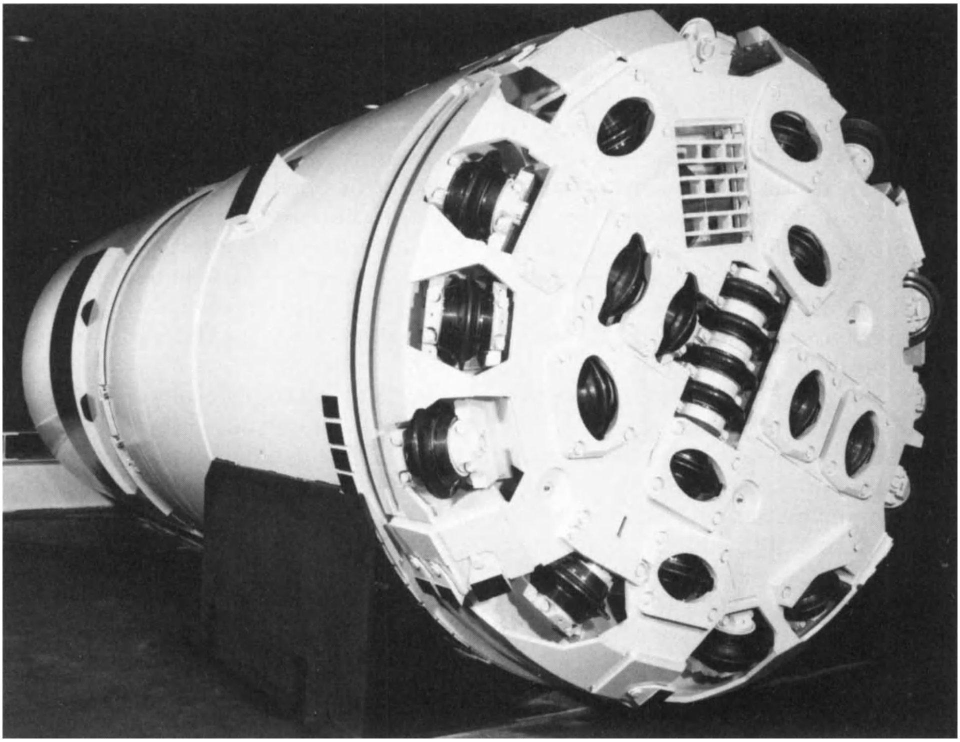

Figure 16.8 Telescoping double–shield tunnel borer

16.4.6 NATM and tunnel boring

The technique of choice in much of Europe for tunnelling through highly stressed crushed rock is known as the New Austrian Tunnelling Method (NATM). It has been pointed out at length that the system is neither new nor Austrian and it is more often art than method. However, the Austrians should be credited with the development of an understanding of how to analyse, design and apply a system of interactive tunnel supports which yield with the rock as it converges and create a stable and secure support which has the minimum thickness and strength to do the job effectively.

Since the rock condition, its fabric strength and its internal friction coefficient and general rock mass properties may vary considerably from one point to another (even from one side of the tunnel to the other), the support system used in NATM is applied with varying degrees of restraint as the rock movements are measured. The usual support system uses shotcrete, rock bolts, wire mesh and yielding steel arches. One of the important principles is to catch the rock with shotcrete as quickly as possible after excavation to prevent ravelling and failure of the rock due to gravity, while at the same time allowing controlled convergence.

Can this technique and these tools be applied with a tunnelling machine? One example can be cited where the technique was applied with success in a near-surface tunnel in Melbourne, Australia. In 1975 the Melbourne Underground Rail Loop Authority (MURLA) built 2.5 km of 7.12 m diameter rail tunnel through a Silurian series of sandstones and mudstones which had been intruded by basalt dikes. The material was highly weathered and blocky. MURLA specified the application of shotcrete immediately behind the cutterhead of a TBM. The contractor chose to use precast segments to cover the lower half of the tunnel and applied shotcrete, wire mesh and steel arches over the upper half. The machine was specially designed to permit the application of shotcrete at a position further forward than had been possible before, and in cases where the face caved in against the cutterhead shotcrete could be used to control the caving even above and ahead of the machine. This technique will no doubt be developed further for transportation tunnels of the future. It has not yet been applied as a complete system to highly stressed squeezing rock.

A second method which accomplishes many of the desired results of NATM is the Stillwater solution, referred to earlier. This technique, using a walking, expanding gripper shield, was also employed in 1986 to build a pilot tunnel for the Freudenstein Rail tunnel of the German Federal Railway. The machine provided the gripping method of immediate restraint of the ground from a point just behind the rotating cutterhead to a point at the tail of the blade shield, where a pumped continuous concrete liner was placed.

For weak, squeezing rock, the continuously cast concrete liner may not be as appropriate as a lining system which can restrain the rock while continuing to yield after the passage of the machine. The machine and other equipment involved in this solution for squeezing ground are somewhat more complex than most other machine solutions, but it will take a sophisticated approach to solve this difficult problem.

16.5 Mixed-face tunnelling

Urban transportation tunnels are often situated near the surface. In some cities such as Washington, DC, they may pass in and out of a full face of rock or soils, sometimes below the water table. Metropolitan planners are faced with the difficult task of laying out a system of transportation which moves people along optimum routes while trying to take advantage of geological features which will ensure that the tunnels and stations are buildable at minimum cost. Many compromises are required in both the system layout and the most geologically favourable route. The result is that tunnels are sometimes forced to pass through sections with substantial lengths of mixed face where rock and soil are both present.

Open-faced shields are traditionally employed with face breasting. Small-charge blasting may be used to break the rock at the shield face, and the excavation work can be difficult and dangerous.

In recent years shielded slurry tunnelling machines have been employed for boring through water-bearing soils containing large, hard rock boulders. The Japanese have pioneered the use of a combination of drag picks and disc cutters to break up boulders while they are removed under a pressurized environment along with the soil surrounding them. These techniques are covered in Chapter 15.

When a machine is used to bore a mixed face many of the same problems are encountered as in boring through soil with boulders. Hard-rock rolling cutters must be employed, and for years the assumption was made that cutters for rock would not operate in soft ground. However, practice has shown that specially designed disc cutters will work effectively in such conditions.

If a tunnel is laid out so that it has substantial lengths of full face of hard rock and then passes through zones of transition or mixed-face conditions into a full face of soft ground it may be approached by tunnel contractors as several short tunnels, each of which requires different methods and equipment. This may require extra shafts for access points or time-consuming delays to convert from one system to another.

The new challenge for tunnelling machines is to bore through the solid rock portions of such a tunnel with reasonable efficiency, then move into a mixed-face geology and, perhaps a short distance later, bore a full face of soil. If this tunnel is situated below the water table the excavation and muck removal must be done under confined water pressure and muck must be removed from the pressurized zone into atmospheric pressure.

This is the situation which exists on the French side of the Channel Tunnel (see Chapter 15). An important point is that without any conversion of the equipment used, the machines can operate in a submerged, pressurized mode and also in an open mode with advance rates comparable to specialized machines for each of the conditions. One can envisage the application of similar machines boring in mountainous conditions which are able to pass through faulted and wet conditions without significant delays.

16.6 Used tunnelling machines

Since the early 1980s a large number of machine-bored tunnels has been bored by previously used tunnelling machines. The worldwide recession of the early 1980s followed a period of rapid growth in tunnel demand. Many machines remained idle for years, and contractors were forced to the conservative stance of lower risk and lower capital investment whenever possible. This situation encouraged the re-use of existing machines. A relatively sophisticated capability has developed in both the machine-manufacturing and tunnel-contracting businesses to redesign, remodel, modify and refurbish existing machines for new jobs.

Of course, some used machines have been misapplied to jobs where the geological conditions required special features, but that has also been true with new machines. One must evaluate the ground conditions and the cost and effectiveness of building the required features or upgrades into an existing machine. A number of specialist businesses have been organized to operate primarily in this field of used tunnelling machines. These locate used machines which will become available for imminent projects, act as selling or purchasing brokers and concentrate on the engineering, manufacturing and shop testing of rebuilt machines. Most original machine manufacturers are also in a position to offer these services.

An interesting result of the re-use of tunnel borers is the attention some owners have placed on their availability in the planning and design of tunnel projects. Several European owners have planned the issuance of bid documents and the schedule of their work to coincide with the availability of several used TBMs which could be applied to their job. These machines have also provided a relatively large degree of flexibility in the finished diameter of the tunnel. A surprisingly large number of tunnels will perform their intended function well if built somewhat larger than their optimum size.

16.7 Conclusions

Tunnel-boring machines for hard rock have been an evolutionary development and are at the point of being able to bore any type of rock at speeds competitive with or higher than can be achieved by blasting techniques. The major developments which will have an important impact on the costs of rock tunnelling are those directed at greater versatility and adaptability to poor-rock tunnelling. In many cases ground conditions cannot be well defined in advance of construction, and machines are being developed to make the excavation less sensitive to changing ground conditions. The substantial advantages of speed and versatility can be expected to continue the emphasis on machine tunnelling in the future.

Chapter 17

Hard-rock tunnels

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Contents

17.1 Introduction

This chapter will discuss tunnels and tunnelling in so-called 'hard rocks'. These are rocks that are so hard or strong that they can only be excavated by specially designed tunnel-boring machines (as described in Chapter 16) or by the conventional drill and blast method.

In hard rocks the rock itself is normally stronger than concrete, which is the most important support and lining material in tunnels. It is therefore the existing discontinuities in the rock, i.e. the joints, fissures and cracks, that greatly influence the quality of the total rock mass. Stability problems in hard rock tunnels are often related to discontinuities.

Hard rocks will normally also have a very low porosity, and may, for all practical purposes, be regarded as watertight. The water in these rock masses always flow along the discontinuities. Successful tunnelling in hard rocks therefore depends on a good understanding not only of the rock itself but also of the jointing of the rock mass.

All rock masses, as well as hard rocks, contain faults and other zones of weakness. Such zones may vary in thickness from a few centimetres to several metres, and in extreme cases to several tens of metres. In addition to heavily jointed and crushed rock, they also contain minerals that may easily be dissolved (for example, calcite) and/or minerals with a very low strength such as clay minerals [1]. Some clay minerals (the so-called 'smectites') have even a high swelling capacity when in contact with water. Tunnelling in such zones can be extremely difficult, and tunnels should therefore be planned so that they intersect as few zones of weakness as possible.

In mountainous areas railway tunnels may encounter high rock stresses due to heavy rock mass overburden or to tectonic stresses set up by geological forces in the earth's crust. For softer rocks high stresses may cause squeezing of the rock surrounding the tunnel. In the really hard rocks these stresses may cause violent rockbursting or popping. The harder and stiffer the rock, the more violent the rockbursting will be [2]. As rockbursts often occur suddenly, they can be a serious hazard to tunnelling.

Tunnels for metro lines in and near cities will normally run at shallow or moderate depths. The kind of stress problem one may encounter in such tunnels is the converse – a lack of stresses. It is important to understand that hard rock is a discontinuous material with joints and fissures in different directions. A certain level of compressive stress is therefore necessary to keep together all the blocks that make up the rock mass. Where such stresses are lacking, they must be compensated for by the use of rock support.

Also typical of tunnels near the surface is that the rock mass is more jointed and has more open joints than for the deeper tunnels. Tunnels near the surface will therefore more easily drain the groundwater in the area. This can have several negative effects both on the tunnel and on the surrounding land.

Even small changes in the alignment for a hard-rock tunnel may have a significant effect on final costs. For railway tunnels, where revision of alignment during construction is almost impossible, proper site investigations and planning are therefore of the utmost importance.

17.2 Site investigations

17.2.1 Investigation stages

Geotechnical investigations for hard-rock tunnels can be divided into two main stages:

1. *Pre-investigations.* Tunnelling has not yet started and all information must be collected on or from the surface.
2. *Post-investigations.* Through tunnels being excavated the rock masses are accessible for inspection.

As shown in Table 17.1, both these main stages can be divided into two substages. The characteristic investigations for each of the four stages are briefly listed in the table and types of reports are indicated.

Not all kinds of investigations will be carried out for all tunnels. A short tunnel through rocks which can easily be mapped on the surface does not necessarily need a two-stage pre-investigation phase. On the other hand, for a complicated subsea tunnel a subdivision of the pre-investigations into more than two stages may be considered [3].

17.2.2 Preliminary site exploration

Preliminary site exploration is carried out in the early phase of the planning of a project. The aim is either to study the feasibility of a planned tunnel or, more often, to evaluate and reduce the number of possible alternatives in a scheme based on geotechnical information. Few decisions are made at this stage and sketches are more common than detailed drawings. This is a highly challenging phase, in which important decisions are taken, often based on limited information. Experience from similar projects and sites is therefore of particular value.

At this early stage it is important to collect all existing relevant information from the literature. Topographical and geological maps are studied as well as aerial photographs. These will give the first answers to questions about where the bedrock is covered with soils, what are the locations and directions of the more important weakness zones and what may be the stress situation in the area.

These studies will normally be followed by a walk-over survey to investigate certain key points in the area. Rock sampling for simple classification tests is carried out and the most important joint information collected. Depth of weathering and groundwater conditions are also studied during this survey.

In the report that concludes the preliminary site exploration all information collected is presented and alternatives are discussed. Plans and cost estimates for further investigations are drawn up and requirements for maps are given.

17.2.3 Detailed surface investigations

With the preliminary report as a basis, the client, in cooperation with consultants, will decide if further planning should be carried out, and if so, what alternatives should be investigated. Additional aerial photographs are taken if required and more detailed maps drawn. The engineering geologist will normally need aerial photographs and maps that cover a larger area than is strictly necessary for other planning operations.

Table 17.1 Site-investigation stages

Pre-investigations: information collected on or from the surface

Preliminary site exploration	*Detailed surface investigations*
Desk studies of: – geotechnical literature – topographical and geological maps – aerial photographs Walk-over survey for preliminary mapping of soil cover, rocks, jointing and weakness zones Geophysical investigations at key points for tunnels: – entrances – intakes and outlets in lakes, fjords and rivers – areas of low rock cover – check of soil thickness in critical points	Engineering geological mapping along tunnel alignment: – types and quality of rocks – orientations, spacing and quality of joints – orientation, thickness and type of weakness zones – groundwater condition Special investigations: – refraction seismic survey – core drilling Sampling and laboratory testing of rocks: – strength – drillability – blastability
Preliminary report: – review of geological and geotechnical conditions – evaluation of feasibility for different alternatives – plan and cost estimate for detailed investigations – need for more maps and aerial photographs	Pre-investigation report: – description (with maps and cross sections) of all topographical and geological factors that may influence construction and use of tunnels and openings – estimates and preliminary plans for excavation requirements, rock support and lining – plans for use of rock material

Post-investigations: rock masses can be inspected in the subsurface

Detailed subsurface investigations	*Tunnel mapping*
Sampling and testing of rocks and infilling materials from joints and faults Supplementary investigations: – rock stress measurements – permeability tests of rock masses – convergence measurements of openings Control and revision of reports from pre-investigations Recommendations of permanent rock support and lining Recommendations for grouting Recommendations for excavation through highly unstable rock masses	Mapping in tunnel of: – types and quality of rocks – orientation, spacing and quality of joints – orientation, thickness and type of weakness zones – seepage of water – stress-induced problems Registration of all rock support, lining and rock improvement Evaluation of excavation performance
Supplementary reports from post-investigations Report on permanent rock support and lining	Final report with tunnel-map and review of rock support Evaluation of pre-investigations

At this stage the most important task for the engineering geologist is to produce maps and cross sections that cover different parts of the project. For this purpose, mapping, sampling and analysing of representative specimens of the different rocks and soils are required. It may also be necessary to supply a purely surface-based mapping with special investigations such as core drillings and various geophysical and other measurements from drill holes.

The results from the detailed surface investigations are collected in a report which normally is a part of or an appendix to the tender documents. This report contains engineering geological descriptions, as well as evaluations of construction and stability problems in different parts of the project. Results from field measurements, sampling and laboratory testing are presented and also evaluated.

17.2.4 Detailed subsurface investigations

When construction work has begun and the tunnel can be entered, the possibilities for the engineering geologist to obtain better information increase considerably. The post-investigations are therefore initiated as early as possible. During the planning of an underground project, important decisions must be taken about the investigations to be carried out before the start of operations and those that can be postponed.

A high degree of flexibility and relatively simple pre-investigations have often characterized tunnelling operations in the hard rocks of Scandinavia. Under such conditions it is particularly important that post-investigations be started early in the construction period. In many cases expensive pre-investigations (for instance, deep-core drillings) can be replaced by the much cheaper pilot borings from the face of the tunnel during construction.

Stress measurements in rock masses will normally have to be made in underground openings and tunnels. Such measurements are therefore good examples of the type of detailed investigations which must be delayed until tunnelling has started.

Detailed subsurface investigations should not be delayed pre-investigations but planned as a control of and a supplement to pre-investigations. For subsurface work a pre-investigation report will always have to be based on a number of assumptions. The sooner these are verified, the better for the remaining part of the underground work. The reports should thus be under continuous revision during the entire construction period.

17.2.5 Tunnel mapping

Some underground openings and tunnels will be difficult to inspect after they have been put into operation and for the owner it is therefore useful to have maps and drawings describing inaccessible areas of the project. Such maps should contain all geological elements that may influence the stability of the tunnel such as major joints, faults, crushed zones, water leakages and areas with rockburst problems, in addition to rock types and information on support work.

The post-investigations, especially tunnel mapping, are important elements in the process of requiring engineering geological experience. The method of pre-investigations can be improved only if the prognosis is carefully monitored through the post-investigations.

17.3 Excavation

17.3.1 Drill and blast systems

During the last few decades there has been considerable technological development in the field of drill and blast systems in tunnelling. Three improvements which are worth mentioning are hydraulic drilling, which has recently replaced the less efficient pneumatic drilling, granular explosives with no nitroglycerin and non-electric initiation systems, which are replacing the old sensitive systems.

This chapter will outline the design of drill and blast systems. A description of explosives and drilling equipment is beyond the scope of this book. Detailed information on these issues can be obtained from the various manufacturers and suppliers or from specialized textbooks.

17.3.2 Drilling

Conventional drilling in hard rock is carried out by the percussion method. The principle is to force a drill rod with a suitable drill bit against the rock in the bottom of a drill hole, generate a stroke in order to obtain rock spalling, then rotate the rod before it is once again forced against the bottom of the hole and a new stroke is generated.

In tunnelling, blast hole diameters of 45–50 mm are normally used, and hard metal button bits are now replacing the older cross-bit inserts. The reason for this is the considerable increase in drilling rate as well as the fact that the lifetime of a button bit may be more than 50% longer than for a 'conventional' one.

For tunnels larger than 25–30 m^2 two- or three-boom rubber-wheeled drilling rigs are normally used. With larger cross sections the rig is also usually equipped with a special basket for scaling, rock support, loading, etc. On a modern rig the power for the drilling thrust and rotation is transmitted by hydraulic systems. An example of a modern three-boom hydraulic drilling rig operating in a 68 m^2 tunnel is shown in Figure 17.1.

Drilling rate is governed not only by the capacity of the equipment but also, to a great extent, by the character of the rock mass. For hydraulic drilling in

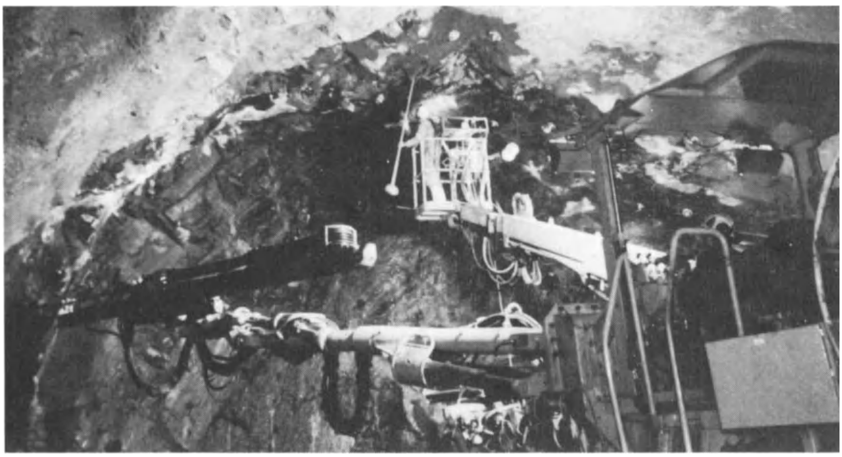


Figure 17.1 A modern drilling rig operating in a 68 m^2 tunnel (a rock bolt is being installed)

Scandinavian hard rocks a variation of drilling rates between 90 cm/min and more than 200 cm/min has been measured (45 mm diameter). For bit wear, an even greater variation caused by rock properties has been experienced. Depending on the mechanical character of the rock, and in particular the quartz content, the lifetime of drill bits is found to vary between 150 and 700 m.

17.3.3 Design of a blast

For all rock blasting the basic principle is to break the rock and push the rock fragments towards a free surface. In a tunnel the degree of confinement of the blast volume is far higher than in a quarry. In order to obtain a satisfactory result from a tunnel blast it is therefore necessary to include a so-called 'cut' in the blast design. To a great extent, the result of the blast will depend on the design of this cut.

A cut may consist of either loaded or unloaded holes. The wedge-cut shown in Figure 17.2 is an example of the former type. The wedge-holes are loaded with a

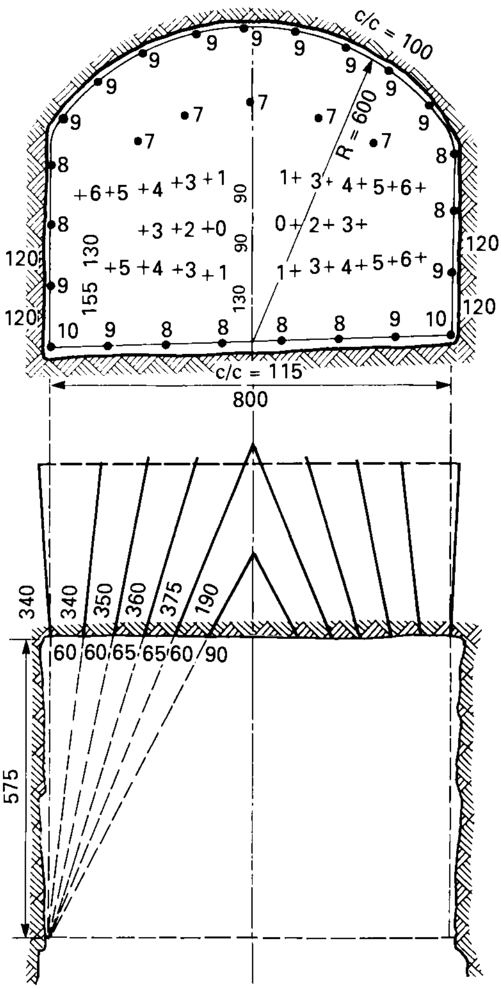
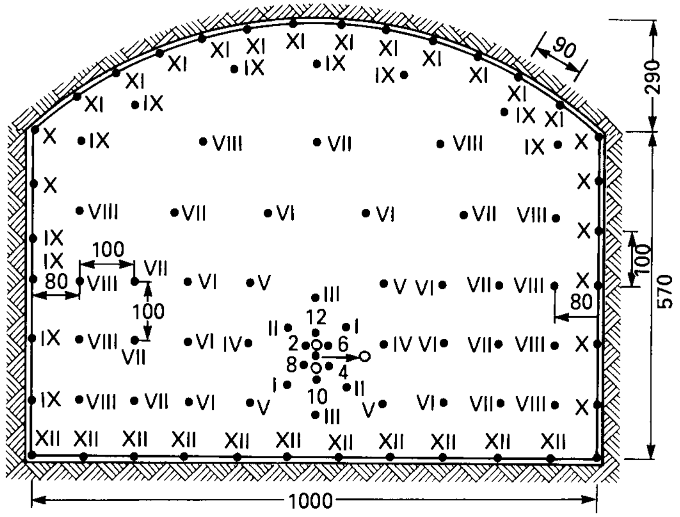


Figure 17.2 Blast design of a 43 m^2 tunnel with wedge-cut (all dimensions in cm). + Cut holes, • parallel holes. Drill hole diameter 1⅞ in (45 mm). Net advance assumed: ~ 3 m. Number of holes: 56 Numbers on face are the order of igniting the detonators



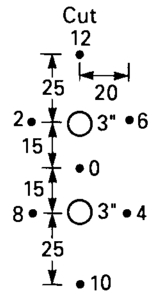


Figure 17.3 Blast design of a 70 m^2 tunnel with parallel hole-cut. All dimensions in centimetres. Diameter of blast holes: 45 mm. Diameter of unloaded cut holes: 3 in (75 mm). Drilled length: 16 ft. Net advance assumed: ~ 4.5 m. Number of holes: 89 + 2. Numbers on face are the order of igniting the detonators

high-strength explosive and detonated as the initial stage of the blast. A parallel hole-cut is shown in Figure 17.3. This cut gives a higher advancement per drilled round and is faster to drill than a wedge-cut. As shown, the parallel hole-cut includes unloaded, large-diameter drill holes in addition to holes loaded with high-strength explosive which are detonated initially.

In tunnelling, half-second detonators are generally preferred. This is mainly because these reduce the spreading of the blasted rock and hence make the mucking easier than the alternative millisecond detonators. The new non-electric detonators are becoming increasingly popular, particularly in tunnels in cities and in other situations close to electrical equipment or installations.

A blast for a medium-size tunnel may have about a hundred blast holes. When the explosive is detonating in a hole a compressive shock wave will spread radially from the hole and a fissure will be generated by the tangential tensile stresses. As a next stage, the slower-acting gas pressure will open the fissures, break the rock and move it towards the free surfaces (towards the cut and the tunnel face). The specific charging for a tunnel blast (necessary explosive consumption in kilograms of explosive per cubic metre of mass) will, to a great extent, depend on the cross-sectional area of the tunnel and also on the drilled length of the blast. This general trend is illustrated in Figure 17.4.

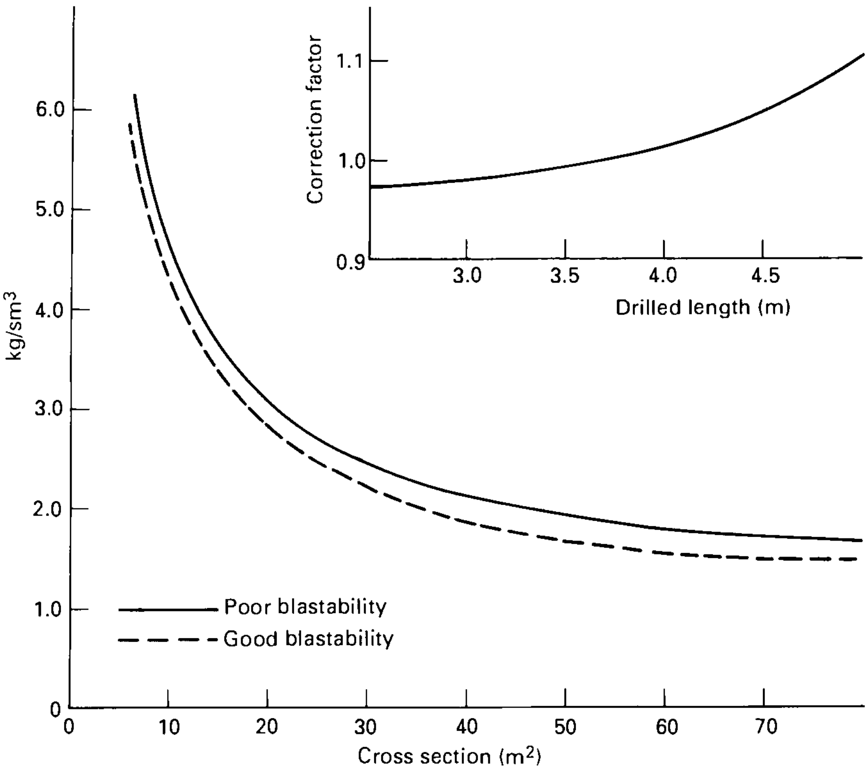


Figure 17.4 Empirical diagram illustrating the relationship between specific charging and the tunnel cross-sectional area (after Norwegian Institute of Technology, Division of Construction Engineering) [4]

17.3.4 Blastability

The result of a blast will depend not only on factors such as type of cut, drill hole pattern, types of explosives and detonators. Geological factors may also have an influence on the result, although in most cases, as indicated in Figure 17.4, the variation of specific charging between 'good' and 'poor' blastability is within a range of ±5–10%.

In certain cases, the character of the rock may, however, have a considerably greater influence on blastability. The net advance per round for 'normal' rock conditions will be in the order of 90–100% of the drilled length. In very difficult rock this may be as low as 50%, thus slowing down tunnelling considerably.

The main geological parameters which will influence blastability are:

1. The mechanical strength of the rock;
2. The degree of jointing;
3. The density of the rock mass;
4. The anisotropy of the rock mass.

Rock types with a distinct foliation (for instance, mica schist and phyllite) are those that most frequently create blastability problems. For a hole drilled along the

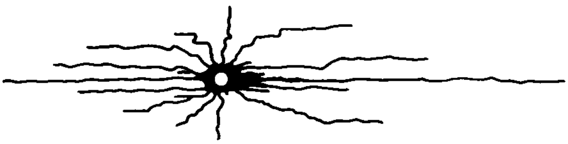


Figure 17.5 Fissures resulting from detonation in a hole drilled along the foliation of an anisotropic rock

foliation, the situation will, in principle, be as shown in Figure 17.5. In the direction normal to foliation the compressive shock wave will be strongly attenuated, and the tangential stress will have to overcome a high tensile strength. In the other direction there will be a smaller attenuation, and the tensile strength in the tangential direction is also smaller. The final and unfavourable result may be that only one or a few new fissures are initiated along the foliation, and very few normal to this direction.

The following are typical characteristics of rocks having favourable blastability properties:

1. Low to moderate anisotropy (a ratio between maximum and minimum sonic velocity lower than 1.3);
2. Moderate mechanical strength (a point load strength within the range 9–14 MPa);
3. A low density (lower than 2.75 g/cm^3).

17.3.5 Perimeter blasting

If the holes close to the planned contour of the tunnel are too heavily loaded, a considerable 'overbreak' and a rough, uneven contour may be the result of the blast. The secondary cracking due to overbreak will greatly increase the need for scaling and rock support. Hence, the result of careless perimeter blasting will be a poor-quality product and increased support costs.

In order to minimize overbreak in a tunnel's walls and roof, reduced charges are used close to the contour and the drill-hole pattern is specially designed. In most cases low-strength pipe charges (explosive in tubes much smaller than the drill-hole diameter) are used along the contour.

For the examples in Figures 17.2 and 17.3, 25 mm and 22 mm pipe charges are used, and a spacing of 1.0–1.2 m and 0.9–1.0 m, respectively. In other cases 17 mm pipe charges are used with a drill-hole spacing down to 0.5–0.6 m. In order to avoid overbreak it is important to reduce the charges also in the drill holes next to the contour row.

To a great extent, the optimum design of the perimeter blast will depend on local geological factors, some of which are difficult to quantify. The final design should therefore be based on experiments during the first stage of tunnelling and not on theoretical anlyses only. In general, highly anisotropic and heavily fractured rocks represent the conditions when a successful contour blast is most difficult to obtain.

Precise drilling is essential for the contour. The final sequence of contour holes are fired as simultaneously as possible. Figure 17.6 shows successful perimeter blasting: overbreak is negligible and all contour holes can be identified.

The use of a mixture of ANFO and polystyrene beads as an alternative to pipe charges is the most recent development in the field of perimeter blasting. Good

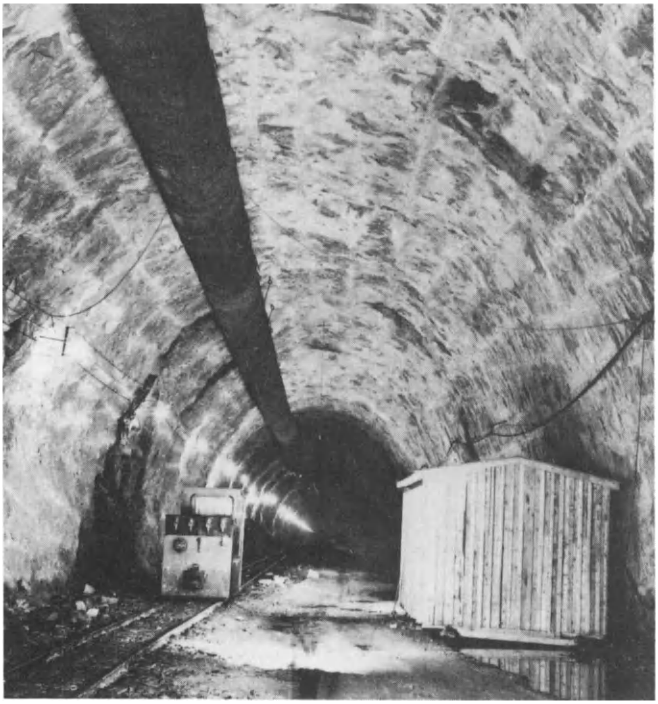


Figure 17.6 Smooth tunnel contour resulting from a successful perimeter blasting (courtesy Statkraft)

results have been obtained for ANFO mixtures (ammonium nitrate with 5–6% fuel oil) with up to 75% polystyrene beads. This is a faster and much cheaper alternative than pipe charges.

17.3.6 Mucking and hauling

Selection of equipment for mucking and hauling will depend on the cross-sectional area of the tunnel. The following guidelines for combinations of equipment have been developed at The Norwegian Institute of Technology [4]:

1. *Railborne transport.* Railborne loading for cross sections smaller than $20\,m^2$, shovel loading for cross sections exceeding $20\,m^2$.
2. *Trackless transport.* Shovel or excavator loading directly into trucks for cross sections larger than $25\,m^2$.
3. *Load-and-carry system (niche loading).* Shovel loading from niches (recesses), normally for cross sections between $16\,m^2$ and $40\,m^2$.

For the load-and-carry alternative, loading niches are normally required for every 100–150 m. For trackless transport, niches for turning are required for cross-sectional areas smaller than approximately $50\,m^2$ (depending on the size of the hauling equipment). For railborne transport niches will also be needed (Figure 17.7).

Trackless transport during excavation is normally the most efficient and economical alternative. The relatively large cross-sectional area of such tunnels is a

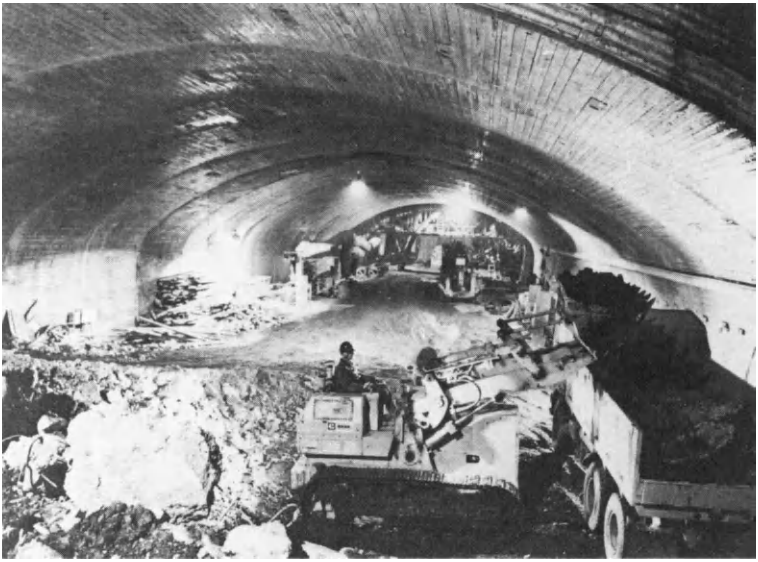


Figure 17.7 Mucking and hauling during excavation of the Nationalteatret Railway Station, Oslo (courtesy Norwegian State Railway – NSB)

major reason. For the recent extension of the Oslo metro system (1984), the mucking and hauling equipment both in a single-track 37 m^2 tunnel and in a double-track 75 m^2 one consisted of two side-tipping shovels loading a fleet of 5 m^3 capacity dump trucks.

17.3.7 Costs

The costs of drilling, blasting, mucking and hauling varies considerably from one country to another, as well as for apparently similar projects within the same country. It is therefore difficult to define general cost figures for such operations.

Some general trends can, however, be defined. One example is shown in Figure 17.8, which is based on Norwegian hard-rock tunnelling data. The curves in this figure represent normalized tunnel costs (1 NOK = 7.0 US$), excluding rock support, as a function of cross-sectional area. 'Normalizing' in this case mainly refers to a tunnel length of 3 km, an approximately horizontal tunnel, a distance to spoil heap of 600 m and 'average' drilling and blastability conditions. For other conditions, correction factors are defined.

17.4 Ground vibrations

17.4.1 Shock waves

Most of the explosive energy in a blast is absorbed in the process of breaking the rock. One effect of the remaining energy is to cause air shocks and shock waves in the surrounding rock. The amount of energy which is transferred through the rock depends on the character of the rock mass and the history of the blast.

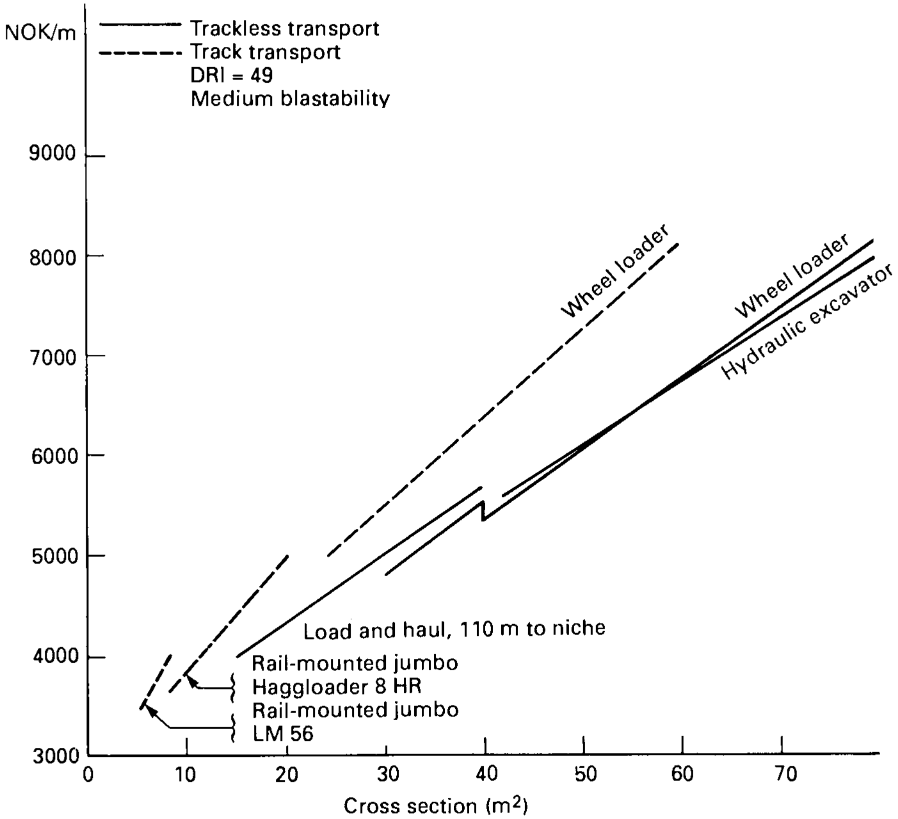


Figure 17.8 Basic curve for normalized tunnel costs as defined by Norwegian Institute of Technology, Division of Construction Engineering[5]

On the surface the following types of waves from a subsurface blast may easily be recognized:

1. Longitudinal waves (S-waves), causing oscillation of particles in the direction of wave propagation;
2. Transversal waves (P-waves), causing particle oscillation perpendicular to the wave-propagation direction;
3. Surface waves, of which Rayleigh waves (R-waves) are the most important. The particles have a retrograde, elliptical movement.

The P-wave (Primary) has the highest velocity. The velocity of the S-wave (Secondary) is normally in the order of only 50% of the P-wave velocity, and the R-wave velocity is even slower. The P- and S-waves, however, are more easily damped.

Different waves also have different frequency, amplitude and phase characteristics. Therefore, the magnitude of the resultant will, in a few cases, be greater than the maximum magnitudes of individual components. Usually the vertical component of the surface wave has the maximum amplitude, and is therefore the main objective for recording. In some cases, particularly when buildings are located on slopes, the horizontal component may be the critical one. Ground-vibration characteristics can be recorded with special instruments (vibrographs).

At the planning stage of a blast the assumption is normally made that the vibrations can be represented by harmonic oscillations. For harmonic oscillations (sinusoidal waves) the following relationship exists between vibration velocity (v), maximum amplitude (A) and frequency (f):

$$v = 2\pi \, . \, A \, . \, f$$

Vibration velocity, v, is the distance per time unit (i.e. mm/s) which a surface particle has travelled around its point of equilibrium. This velocity is different from the seismic velocity, which in hard rocks is normally in the order of 4500–6000 m/s for P-waves.

The resultant wave has a wide range of different frequencies, depending on ground conditions, distance, detonator characteristics, etc. In most hard rocks the dominant frequencies are in the range of 10–100 Hz. In soil the frequency is generally lower, in most cases 5–20 Hz.

17.4.2 Design criterion for blast vibrations

Most criteria used for defining 'allowable vibrations' are based on critical values of v or A. When critical values are defined, it is important to bear in mind that human beings are particularly sensible to vibrations. For instance, at a frequency of 50 Hz an amplitude of only 2 μm is easily recognized by human beings, while the critical value for building damage is 200 μm or more.

Many countries have their own criteria for critical vibrations, and there is a considerable variation in 'critical values'. Figure 17.9 is an attempt to define a 'general criterion' which may be used as a first indication of damage risk. In each individual case, however, a more thorough evaluation should be carried out based on local conditions. For buildings containing computers or other sensitive installations, critical values of vibration velocity of 10 mm/s or lower may be required.

The planning of a blast is in many cases based on empirical equations of the following type:

$$v = k \, . \, \frac{\sqrt{Q}}{R}$$

where

v = vibration velocity,
k = 'k-value',
Q = weight of simultaneously detonating charge, and
R = distance from detonation.

The k-value is not constant, but a parameter which is dependent on the ground conditions and the distance from the blast.

As a part of a general planning procedure, small-scale test detonations are carried to evaluate the k-value. The next step is then to calculate the maximum permissible charges as a function of distance from the blast using this k-value and the maximum vibration velocity allowed.

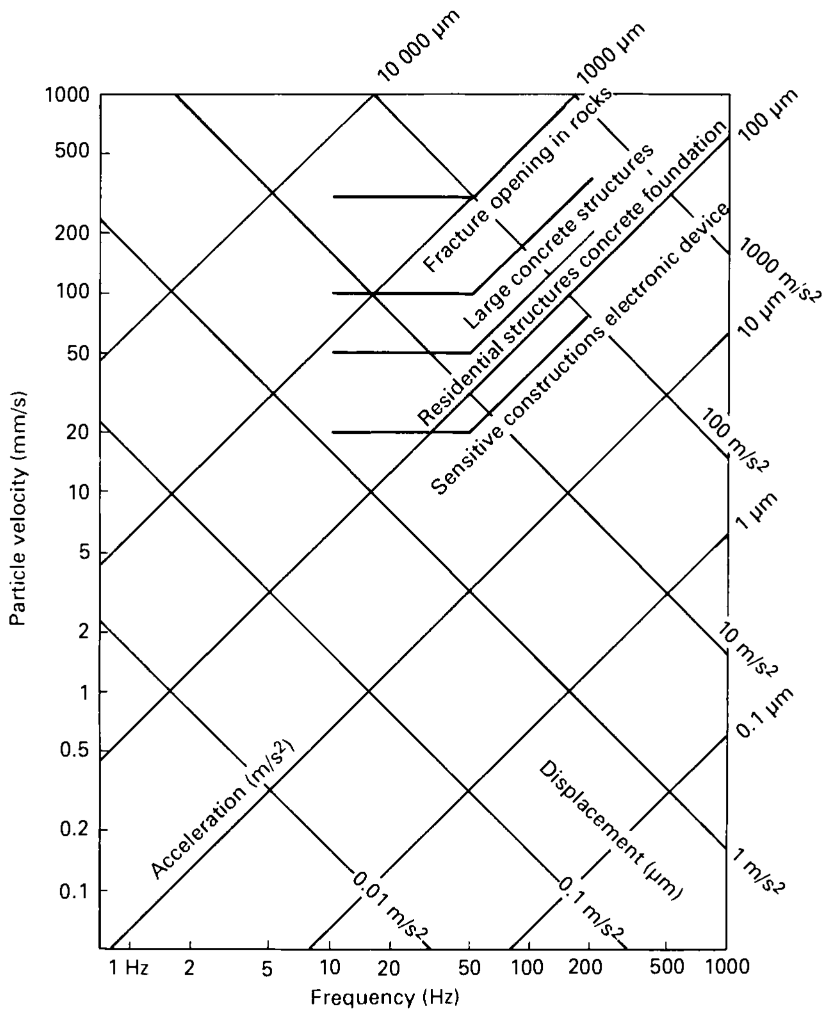


Figure 17.9 Critical values of vibration velocity and maximum amplitude as a function of frequency (after By [6])

17.5 Support and lining

17.5.1 Support philosophy

The various geological factors that may influence the excavation and stability of a hard rock tunnel were briefly described in the introduction. They are:

1. The strength and quality of the rock itself;
2. The number and character of the discontinuities (joints) in the rock mass;
3. Faults and other weak zones;
4. Stress-induced rockbursts;
5. Water inflow.

In hard-rock tunnels the use of *in-situ* cast concrete or steel sets as support during excavation is restricted to particularly difficult zones of weakness. Other stability problems are normally solved by the use of rock bolts and/or shotcrete. In difficult situations a combination of light steel sets and shotcrete may be used.

By its nature a railway tunnel will normally be conservatively supported and at least locally lined with concrete or shotcrete before put into operation. To obtain the most economical result it is advisable to use a contracting system which allows the necessary temporary support during excavation to be accounted for when the final support is decided[7] (see Chapter 13). Thus temporary rock bolts should be protected against corrosion and shotcrete and concrete should have proper thickness and quality.

17.5.2 Ground freezing

Exceptionally, and in very poor-quality rock conditions, special precautions must sometimes be taken which will act as a temporary support only. Ground freezing is the most obvious example. This is a ground-strengthening technique which is used, for instance, if the tunnel must cross major zones of weakness or soil-filled depressions in the bedrock, or if the rock cover is insufficient.

In Helsinki, ground freezing had to be carried out when the twin 6.5 m diameter metro tunnels were driven through a soil-filled cleft at a depth of 25 m (Figure 17.10). The frozen section had a length of about 30 m, and was created by vaporizing freon in the freeze tubes. A 2.5 m thick ice wall in sand and moraine resulted from a freezing period of 55 days. The tunnels were excavated under protection of the frozen arch by careful drilling and blasting, and cast-iron segments were used for permanent lining[8].

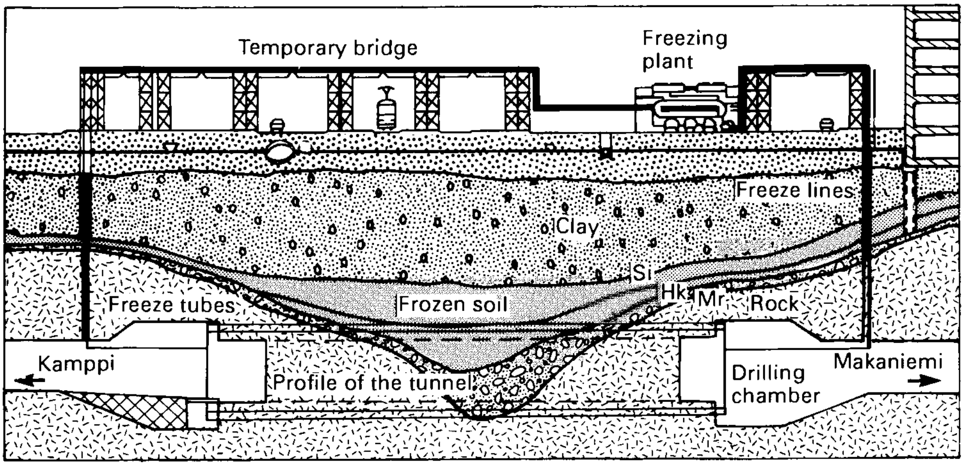


Figure 17.10 Horizontal freezing in the Helsinki metro tunnel (after Vuorela and Eronen[8])

In a 3.6 km railway tunnel under the city of Oslo which was opened in 1980 the problem had a different character (Figure 17.11). Close to the Royal Garden there is a major depression in the rock surface, reducing the rock cover for the railway tunnel to only 0.5 m, and thus making normal tunnelling impossible. The canyon is filled with sandy gravel. In this case drilling of 56 freezing pipes was carried out

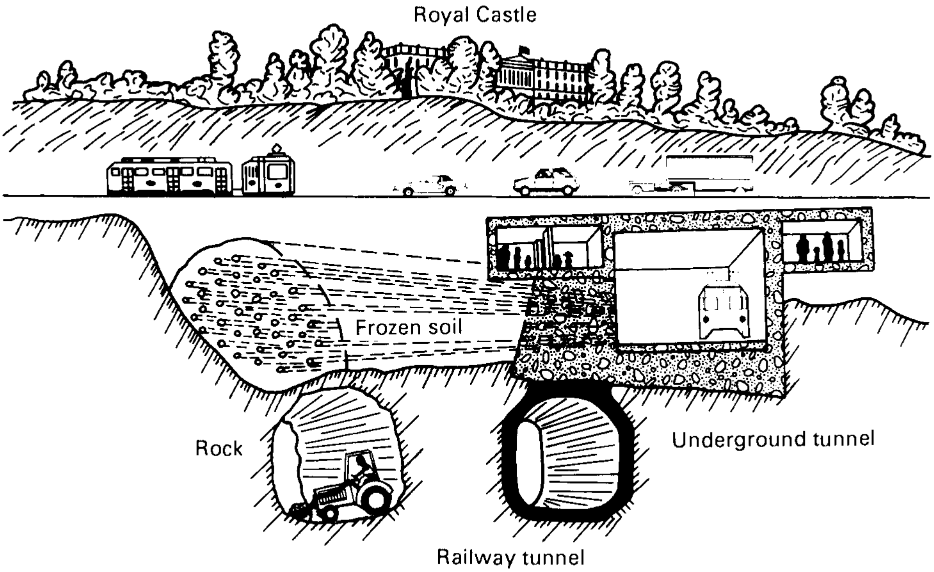


Figure 17.11 Ground freezing in the Oslo railway tunnel (after Jøsang[9])

from a nearby metro tunnel. The thickness of the frozen zone was 5 m after a continuous freezing time of 8 weeks. When freezing was completed, this 25 m section of the tunnel was excavated by careful drilling and blasting, and *in-situ* case concrete was used for the final lining[9].

17.5.3 Pregrouting

Large inflows of water are a considerable problem for tunnelling in general. In urban areas even minor inflows to a tunnel may, over time, result in a widespread drawdown of the groundwater table. As illustrated in Figure 17.12, this may cause a particularly difficult situation with uncontrolled settlements when the rock is overlain by fine-grained soils such as silt and clay. This is a common situation in areas located close to the sea, as are so many of the world' major cities.

To avoid such problems, pregrouting is carried out as an integrated part of the excavation cycle for urban tunnels. In many cases only very small leakages can be

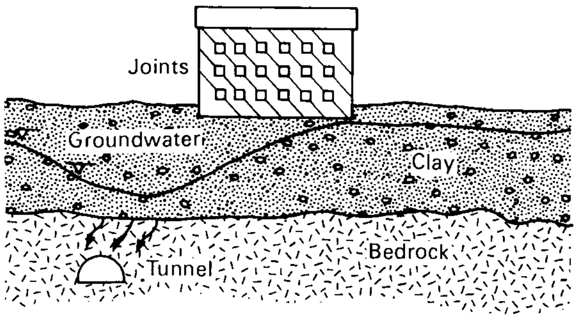


Figure 17.12 Damage to a building due to lowering of the groundwater table

tolerated. In some cases the grouting procedure also includes drilling and testing of the permeability in probe holes ahead of the tunnel face. The final decision whether and how to grout is based on recordings from these tests. In cases where the criterion for acceptable leakage is very strict, continuous pregrouting is carried out as shown in Figure 17.13.

It is important that enough holes are drilled to give a continuous grout barrier. In the Oslo railway tunnel approximately 40 holes with a spacing of 1–1.5 m were drilled for every round of grouting. Initially, each round had a length of 15 m and was drilled for every 9 m of excavation. Later, each round had a length of 12 m and was drilled for every 6 m of excavation, as shown in Figure 17.13 ('double-barrier system').

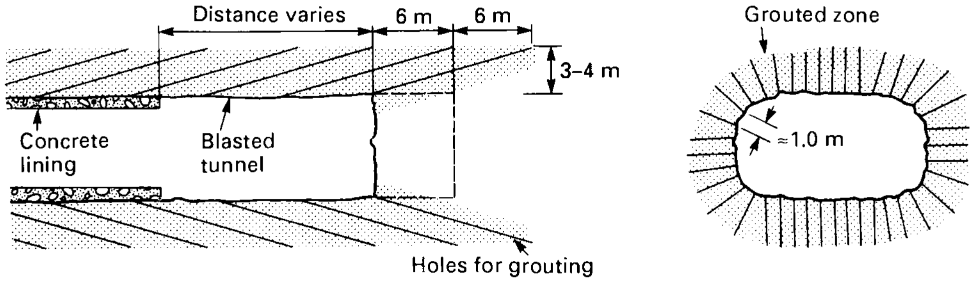


Figure 17.13 Grouting procedure for the Oslo railway tunnel (after Karlsrud [10])

In most cases cement-based suspensions are used for grouting. When the leakage criterion is strict, chemical grouts such as silicates, acrylic products or polyrethane foams are used. Penetration into thin joints is far better for these products than for cement grout, which normally has a lower limit for penetration corresponding to a joint aperture of 0.1–0.2 mm. Because of the often toxic character of chemical grouts, microcements are gradually replacing chemical grouts.

The optimum grouting pressure will, to a great extent, depend on local rock conditions. In urban, shallow tunnels grout pressures between 1 and 4 MPa are commonly used. The final decisions should in each case be based on experiments during the early stage of tunnelling.

Results from the Oslo railway tunnel are shown in Figure 17.14. In this case most of the grouting was cement based, and grouting pressures of up to 3–4 MPa were used. For a 100 m tunnel section the grout consumption in litres per metre varies between 40 and 2900. Sections of high grout consumption clearly correspond to deep clefts in the bedrock.

For urban tunnels the leakage criterion is often very strict. In Oslo, for instance, leakage per 100 m of tunnel exceeding 1–5 l/min was not tolerated. This is a criterion which, in a few cases, is obtained by pregrouting alone. In most of the Oslo railway tunnel the leakage after pregrouting was between 10 and 60 l/min per 100 m [10]. Further measures therefore had to be taken (see Section 17.5.7).

17.5.4 Scaling

Scaling of loose rock is carried out after each blast round, and in many cases also periodically at later stages. Manual scaling is still commonly used, but mechanized scaling, which is making the work safer and more efficient, is gradually replacing it.

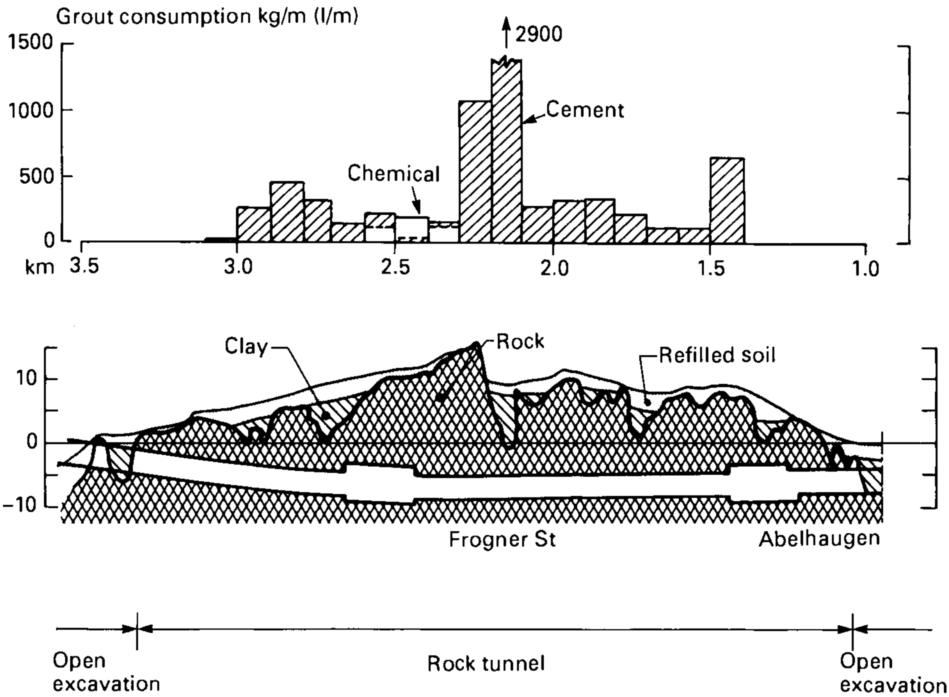


Figure 17.14 Longitudinal section and grout consumption for the Oslo railway tunnel (after Karlsrud[10])

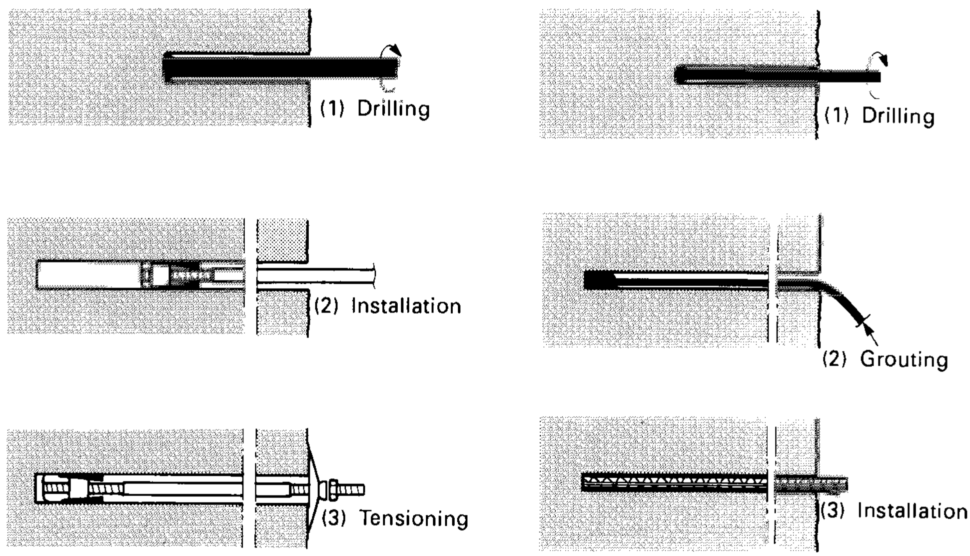


Figure 17.15 The principle for installation of tensioned expansion bolts (*left*) and untensioned grouted bolts (*right*)

Thorough scaling is important for reasons of safety. Also, in many cases it may reduce the need for more expensive rock support.

17.5.5 Rock bolting

Rock bolting in tunnels is carried out by either:

1. Spot bolting of individual, unstable blocks; or
2. Systematic bolting of a tunnel section in a defined pattern.

For immediate support at the working face, tensioned rock bolts are used. These bolts may be mechanically or resin anchored, and are widely used as protection against falling blocks and rockbursts. The principle for installing an expansion bolt is shown on the left of Figure 17.15. This is a very common bolt for immediate support. In a corrosive environment pipe bolts, which have a central hole for later grouting, are often used.

For rock bolting behind the working face, untensioned grouted bolts (dowels) are commonly used. At the right of Figure 17.15 the procedure for installation of such bolts is shown. These are completely surrounded by grout, and hence well protected against corrosion.

In combination with rock bolting, wire mesh and straps are often used, particularly in closely jointed rock and rockburst conditions (Figure 17.16). In

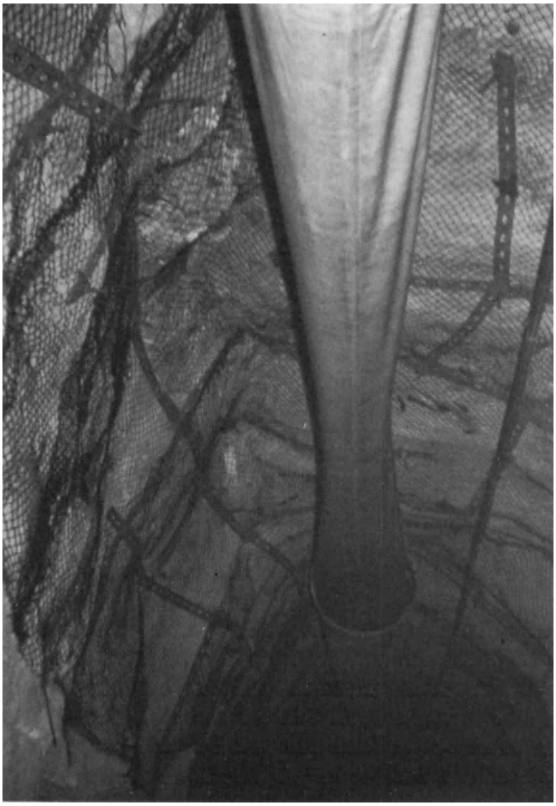


Figure 17.16 Rock mesh and straps between rock bolts in rockburst conditions

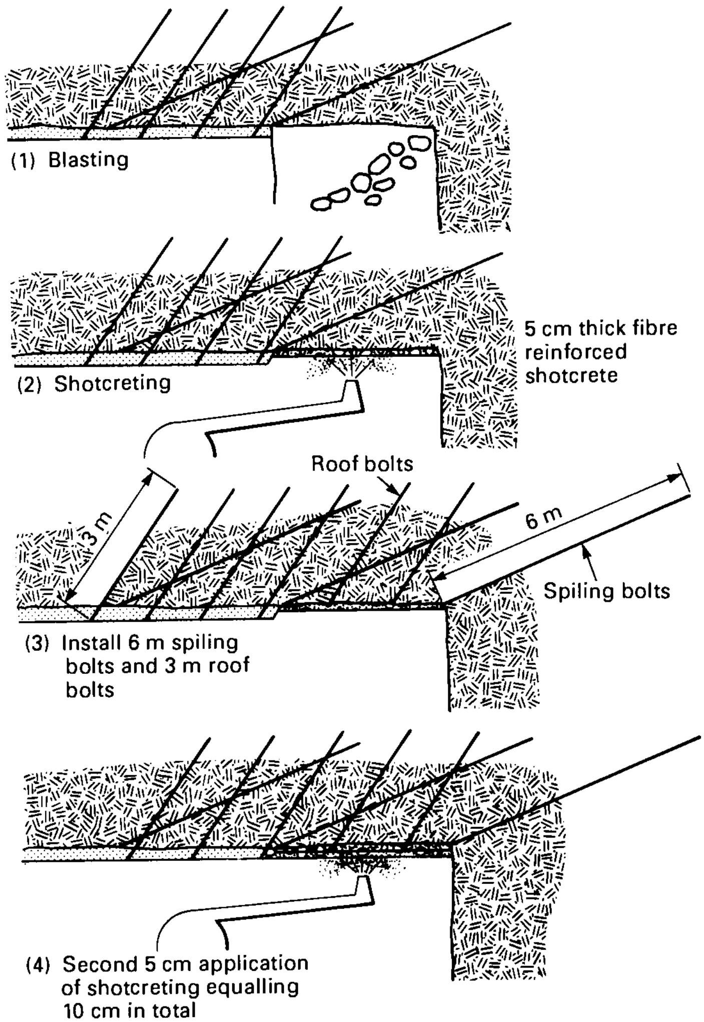


Figure 17.17 Rock support system used in poor-quality rock conditions for the Oslo metro (after Wallis and Martin [11])

some cases rock bolting is also used as an integrated part of a more complex rock-support system (Figure 17.17).

On average, the length of rock bolts in railway tunnels is 2–4 m and the diameter 20–25 mm. A typical yield strength for a 20 mm rock bolt is 125 kN. For systematic bolting automatic, high-capacity rockbolting jumbos are used in some cases. Usually, however, the tunnel rig is employed for the drilling of bolt holes (see Figure 17.1).

17.5.6 Shotcrete

Shotcrete is widely used for temporary as well as for permanent support. There are two different methods for the application of shotcrete; dry mix and wet mix. In the dry-mix method cement and sand is mixed in a dry condition and water is added at the nozzle. The wet-mix shotcrete is mixed as a low-slump concrete which is pumped to the nozzle. In most countries the dry-mix method still has an important position. The main reason is that the wet-mix alternative has been assumed to give a rather limited period between mixing and application and hence a considerable risk of stoppage. By proper use of additives this problem can be eliminated. In many countries the wet-mix method is therefore now mainly used.

In Norway, the wet-mix method accounts for more than 95% of all shotcrete used in tunnels. Considerable improvements in the working environment, and the very good results of steel fibre application when using the wet-mix method, are the main reasons. This method has virtually no loss of fibres while dry-mix shotcrete may have losses of as high as 50%.

Adding steel fibres into the mix has the favourable effect of increasing the strength and the energy before failure of the shotcrete. In most cases the effects of fibre reinforcement are even more favourable than those of the far more expensive and time-consuming alternative of wire mesh reinforcement. While hand-held nozzles were previously mainly used, today robots are being employed for shotcreting.

For immediate support, shotcrete is primarily used in heavily jointed rock masses. A thickness of 5–10 cm is normally applied in such cases, and has the favourable effects of sealing and locking joints and reducing deformations. Shotcrete is often used as an alternative to rock bolts, straps and wire mesh, which in most cases represent a more expensive alternative.

Shotcrete is often used in combination with rock bolts as a permanent lining and rock support. Against rockbursts steel-fibre reinforced shotcrete has been employed for immediate rock support, and for permanent support this has been supplemented by systematic rock bolting[2]. In very difficult rock conditions a combination of multi-layer shotcrete and rock bolts has been used as an alternative to concrete lining. An example of the recent extension of the Oslo metro system is shown in Figure 17.17. This support system involves spiling, two layers of reinforced shotcrete and systematical roof bolting ahead of the working face. In this case the system allowed the safe advance of a 12 m wide tunnel through areas of completely crushed rock.

Adhesion of the shotcrete to the rock surface is the most important factor determining the failure mechanism and load capacity. Sheet minerals such as chlorite, graphite, talc and clays will, to a great extent, reduce this adhesion, and hence influence the capacity of the shotcrete. However, the most important restriction on the use of shotcrete is represented by zones of weakness containing swelling clay (smectite). If shotcrete is applied on such zones there will be no room for expansion of the swelling clay, and, as can be seen in Figure 17.18, the shotcrete lining will be subjected to the maximum swelling pressure. This will destroy the shotcrete lining. Hence, if smectite is present, shotcrete should only be used in narrow zones and low-activity clay or if possibilities for expansion are included in the design of the support system.

17.5.7 Concrete lining

For stabilization in hard-rock tunnels concrete lining is used only in areas of exceptionally poor rock. The lining is normally cast in place at the tunnel face by the use of steel formwork (Figure 17.19). The length of the formwork is normally

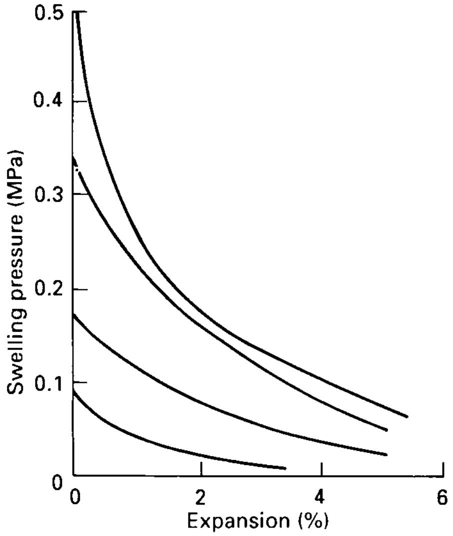


Figure 17.18 Swelling pressure as a function of expansion

between 4 and 6 m, and the concrete lining is designed with a minimum thickness of 30 cm.

The formwork may also be used temporarily to protect workers in ground where the use of other support measures is difficult. Rockfalls during the pumping of concrete will give a lining of poor quality. In the few cases where concrete linings in hard-rock tunnels have failed, this is believed to be the main cause.

When swelling is the major stability problem, a cast-*in-situ* concrete lining has an advantage over a shotcrete lining in that it will be subjected to a considerably lower swelling pressure. The main reasons for this are the incomplete filling of concrete against the crown, the less tight infilling of the joints and the higher shrinkage of a

Figure 17.19 Mobile 6 m steel shield for use in a 68 m^2 tunnel

cast-*in-situ* lining. These factors will give the clay some room for free swelling before the pressure is built up directly against the lining. The resulting swelling pressure will therefore be considerably reduced (see Figure 17.18), and failures of such concrete linings caused by swelling ground are uncommon. In some cases concrete lining is carried out primarily to reduce water leakage into the tunnel. This will be discussed in the next section.

17.5.8 Water protection

The character of a water-leakage problem will, to a great extent, depend on the restrictions that are made on the water inflow and the potential lowering of the groundwater table. If some leakage can be tolerated the problem is mainly reduced to the diversion of the water into trenches and drains. Sheet vaults of aluminium, steel or fibre-material are used in most cases. In cold climates double, insulated vaulting has to be employed to prevent freezing of the water. The cheaper polyethylene plates can be used in some cases (Figure 17.20).

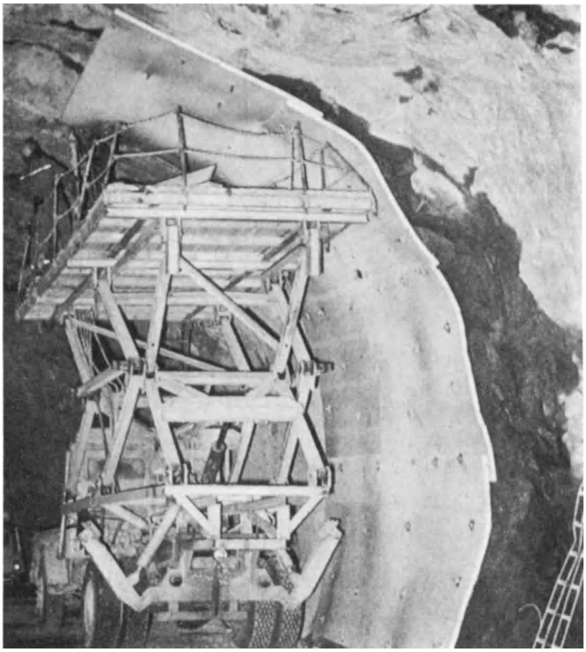


Figure 17.20 Polyethylene plates for water leakage protection in a cold climate

If only very small leakages can be tolerated, the problems will be to make the rock mass impervious, or alternatively, to construct an impervious lining. Pregrouting, as described in Section 17.5.3, will normally reduce the leakages considerably, while postgrouting will, in most cases, have only a limited effect. This is mainly due to the destressing and the difficulties in obtaining sufficiently high grouting pressures.

With very strict leakage criteria, the construction of a final lining will be needed. This was also the case for the Oslo railway tunnel (Figure 17.13). By using high-quality impervious concrete and high-pressure contact grouting of the discontinuities, the final leakage per 100 m tunnel section was reduced to approximately 1 l/min. Temporary injection of water was necessary to prevent damage to nearby buildings due to settlement[10].

In some cases a double lining with a watertight membrane in between has been successfully used. This, however, is an expensive and time-consuming alternative. In other cases where, in spite of all efforts, the watertightness of the final lining was not satisfactory, an inner coating of polyurethane or other chemicals has been used with satisfactory results.

17.5.9 Costs

For many railway tunnels the costs of rock support and lining will far exceed those of excavating, particularly if the leakage criterion is very strict. The total cost of the rock support will depend not only on the type of support but also, to a great extent, on the timing (Figure 17.21). Support work carried out close to the tunnel face will interfere with tunnelling operations and hence be very expensive. Although Figure 17.21 is based mainly on data from Norwegian hydropower tunnels in hard rock, it is believed to be fairly representative of drill and blast railway tunnels.

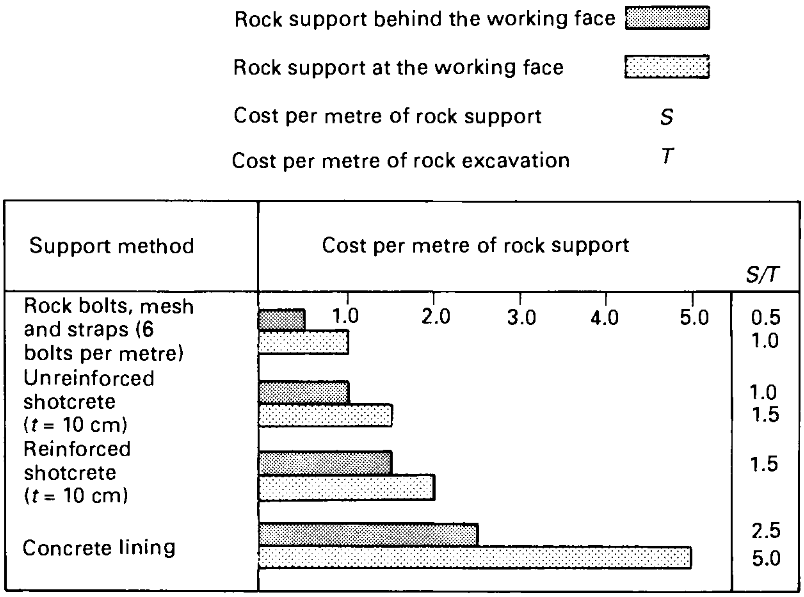


Figure 17.21 An indication of relative costs of rock support and lining compared to tunnelling

For water protection in difficult situations the cost per tunnel metre will far exceed the cost of lining (Figure 17.21). In some projects the costs of water protection have been considerably higher than the total costs of excavation and stability support.

If ground freezing has to be carried out, the cost will always be very high. For the cases discussed in Section 17.5.2 the total costs of the freezing for the Helsinki metro (converted into 1989 figures) was close to US$6 million[8] and almost

US$1.5 million for the Oslo railway tunnel [9]. The cost figures for rock support and lining illustrate the importance of comprehensive pre-investigations to define the tunnel's optimum location.

17.6 Conclusions

Tunnelling with the conventional drill and blast technique is by far the most-used method in hard rocks. For well-organized operation in good rocks weekly advances in the order of 50–75 m are within reach. Long parts of hard-rock tunnels may be without any support at all, or with only a few occasional rock bolts installed to secure blocks.

For both rate of advance and a general costing point of view excavation of a hard-rock tunnel by the drill and blast method will still have much to offer in the future. At its best, hard-rock tunnelling is a relatively easy and cheap operation; at its worst, it can be extremely difficult and expensive. The latter is normally a result of unforeseen adverse geological features due to improper site investigations.

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Chapter 18

Tunnels in compressed air

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Contents

18.1 Introduction

Compressed air tunnelling, i.e. tunnelling with the workings pressurized to help overcome the problems of dealing with water-bearing strata, has been carried out for over a century. Despite the later development of techniques such as freezing, groundwater lowering, injection of chemical grouts and of bentonite shield or slurry shield tunnelling methods, it remains an important weapon in the tunnelling armoury.

18.2 Different geological strata

The effects of compressed air vary for different types of soft ground mainly according to particle size and water content:

1. In stiff impervious clay, compressed air is unnecessary, but in softer clays the supporting action of the air may be useful and if there are silt bands (as, for instance, in many glacial tills), compressed air may be virtually the only way of stabilizing the face.
2. Many silts react very favourably to compressed air, and are transformed from an uncontrollable flowing mud into a self-supporting dry material with a long 'stand-up time', but support of the working face should not be neglected, as small changes in soil properties may have major effects on the stability of the face.
3. In sands and gravels the greater permeability of the ground alters the whole mechanism of compressed air working. It becomes very important to decide what air pressure to apply and the various factors are examined later in the chapter. If, as is often the case, it is decided to maintain air pressures sufficient to balance the water pressure at (or slightly above) the axis level of the tunnel, then air may escape in large quantities through the top of the working face, and water will flow in through the bottom of it. Various measures can be taken to reduce both these undesirable features, such as pre-treating the open ground by grouting methods or applying a sealant to the top of the face. Judicious pumping and face supports can reduce the adverse effects of the inflow through the bottom of the face.
4. The use of compressed air in rock is more limited. In fissured or porous rock below the water table, clearly the application of compressed air will reduce the inflow of water. In self-supporting rock, it is usually better to grout the fissures and pump the water.
5. It is a different matter when the rock is unstable because of soft intrusions or weathering. The upper chalk in the London area is a rock in which the use of compressed air has often proved to be useful or necessary.
6. An additional circumstance in which compressed air may be used in fissured or permeable rock is where it is important not to lower the water table. Consider a city where valuable old property, with inadequate foundations on alluvium, is underlain by permeable rock through which it is desired to tunnel. While pumping from the advancing tunnel, or from deep wells, might be the most economical tunnelling method, it would be quite unacceptable if it were to cause damaging settlement above the tunnel. The use of low-pressure (LP) compressed air would be one way out of the dilemma.

18.3 A typical compressed air tunnel

To understand the general process, let us examine the construction of a typical length of tunnel in compressed air. The sequence is as follows:

1. Sink a working shaft, and as the shaft approaches a level which requires air pressure to exclude water . . .
2. Construct an 'airdeck' (airtight surface strong enough to resist maximum air pressure and weighted or anchored to resist uplift) in the shaft, with working access through vertical airlocks set in the deck.
3. When necessitated by ground conditions, pump in air to pressurize the shaft below the airdeck to balance water pressure in the strata through which the shaft is being sunk.
4. Complete the shaft sinking, including plugging the bottom of the shaft which will probably conveniently include a sump.
5. Caulk the shaft and test by removing air pressure, which also provides an opportunity for shaft survey work through the airdeck.
6. After restoring air pressure, the complicated tasks of breaking out of the shaft and constructing the immediately adjacent lengths of tunnel are carried out, still working through the limited access facilities provided in the airdeck.
7. As soon as sufficient lengths of tunnel have been completed, bulkheads and airlocks are constructed in the tunnels, the airdeck in the shaft is broken out and construction proceeds at the faster pace made possible by the new plant arrangement with the shaft 'in free air'.
8. The ongoing work will often include the construction of shield chambers, the erection of shields and the driving of considerable lengths of tunnel.
9. In various circumstances, including accumulating large air losses through completed lengths of tunnel, it may be best to build new airlocks close to the face and take the original locks out of use.
10. On completion of tunnel construction in strata requiring compressed air, return the workings to atmospheric pressure, an operation which, like most of those preceding it, requires forethought and care.

There are many possible variations to the above sequence of operations. The use of this method has important implications at the planning, design and construction phases.

18.4 The planning phase

The local geology is only one of several factors which determine the best level at which to build a metro (See Chapter 2). Earlier construction of sewers, services and other metro lines may leave little or no choice. The railway-operating gradients of adjacent sections will limit the choices even in the absence of existing infrastructure.

Stations close to the surface reduce journey times: no-one wants long escalators unless they are essential (See Chapter 3). The best advice to a metro planner contemplating the construction of station work in compressed air is to avoid it if conveniently possible. However, if it is essential to use compressed air, then consideration must be given to the detailed feasibility of construction, and an especially thorough investigation of ground and water conditions becomes necessary (See Chapter 4).

A tunnelled station will include the construction of the platform tunnels, the adjacent running tunnels, various cross-passages and signal rooms, ventilation shafts, concourse tunnels, escalator shafts, machinery rooms and, quite often, access tunnels to other facilities such as existing metro stations. How will all this be constructed under compressed air? If there were to be only one working face at a time, the job might be relatively simple, since the workings could be kept at an air pressure to suit the single working face. However, the construction time would become quite excessive, so the planner must study potential construction methods, and be confident that there is a possible sequence of construction which does not, for instance, entail applying excessive air pressures to some shallow workings in order to allow completion of other work where high pressures will be needed.

The development of bentonite, and slurry and earth pressure balance tunnelling machines (see Chapter 15) has been so successful that an increasing number of metro running tunnels and even station platform tunnels are being constructed by such methods rather than by using compressed air. Such machines cannot, however, construct the numerous associated tunnels needed in a metro station, so the use of compressed air remains essential in some circumstances.

18.5 The design phase

The tunnel designer will naturally have in mind the difficulties which may be encountered during the construction phase. Until quite recently, many engineers would have designed and specified cast-iron or steel linings for any soft-ground compressed air tunnel. They would have argued that the additional cost of a metal lining was desirable or necessary to deal with the unexpected stresses which can occur in difficult ground (for instance, when a serious escape of air occurs, causing cavities behind segments), and also that in such ground it would not be possible to guarantee a watertight tunnel with concrete segments.

Improvements in reinforced concrete segments and in caulking and gasketing techniques have given sufficient confidence to allow relaxations to what had been virtually a universal rule, but it is essential to judge every case on its merits, and in some circumstances it remains necessary to use ductile iron or steel segments.

Designers can assist constructors in various other ways. Their layouts can either facilitate the positioning of horizontal airlocks or make them very difficult and expensive to accommodate. Even at the planning stage, preliminary estimates of working and maximum air pressure will have been made. Designers should take this work a stage further, and make preliminary assessments of the probable rate of air consumption, and therefore of the plant requirements.

The maximum air pressure is normally assessed by the depth below the water table of the lowest point of the new structure, but allowances may have to be made for possible floods, spring tides and the higher density of salt water or liquid mud. The probable working pressures may be estimated, having regard to the character of the ground; the maximum working pressure may be limited by the proximity of cellars or the surface of the ground: an upper limit is effectively provided also by economic considerations, as the cost of compressed air work rises with pressure steeply at about 1 bar (when medical facilities become important) and becomes virtually prohibitive at about 3 bar, because of short working times and long decompressions. It is, however, worth noting that almost all the world's metro tunnels are less than 30 m deep and that is the height of a column of water balanced by 3 bar air pressure.

The figure for air consumption is far more difficult to estimate. British regulations require a minimum 5 l/s per person measured at the working pressure, and in a long heading in impervious clay it might be necessary to check that sufficient fresh air was being supplied for human consumption. Normally, however, the air escaping through the working face ensures ventilation greatly in excess of human needs. Consumption is increased by air escaping from completed lengths of tunnel, by use of the airlocks and often by using a 'snorer' pipe to evacuate water from the invert of the tunnel near the working face. Various rule-of-thumb methods of estimating air consumption or compressor capacity will be found in the references at the end of this chapter [1,2] which also list professional papers giving experience on numerous contracts, which may be of greater value. In the past, consumption has varied from very low figures, such as 20 l/s (per square metre of tunnel cross section) through 65 l/s in 'moderately open ground', such as coarse sand, up to well over 100 l/s in open gravel. In any case, to any estimate of normal consumption should be added a generous contingency in arriving at the capacity of the LP plant to be installed at the site.

In designing the various connections between one tunnel and another, the designer should visualize the construction phases and the techniques likely to be used, and should minimize the area of ground likely to be exposed, to reduce air losses. In extreme cases, special segments have been designed to give continuity of lining, such as conical rings between changes in diameter. Today the cost of such work is often prohibitive, and steel curtain plates may be a better solution. With deeper tunnels in water-bearing ground the cost of a deep sump, which will require higher air pressures than are used in the rest of the project, should be appreciated and alternatives sought.

To reduce cost and health hazards, the designer should seek to minimize the amount of work carried out in compressed air. It may be possible to plan adjacent contracts to minimize the lengths of tunnels driven in compressed air. The construction schedule is also important. Certain operations, such as caulking the tunnels, are obviously best carried out in compressed air, but with careful planning it should be unnecessary for subsequent construction work in the tunnels, such as building railway track or station platforms, to be carried out before air pressure can be removed from the workings.

The designer should be knowledgeable about the constructor's plant requirements, so that, for instance, there is a suitable surface site available for the compressor house, etc. One of a designer's many problems is whether or not to specify a minimum capacity for the compressed air plant which may, of course, prove inadequate in the event, and how to specify the extent to which standby plant should be made available under the contract.

There is a strong case for designers to play an active role in making these decisions, since they generally have longer to give careful thought to the project than do contractors during a brief tender period. On the other hand, some clients insist that contractors should take more or virtually all of the risk associated with the work.

18.6 The construction phase

Although the principles of compressed air work have not changed radically for many years, the undertaking of a major contract in compressed air (such as is likely to be needed for metro tunnel construction) grows increasingly demanding and complicated. There are very specific medical and legal requirements to be met, special plant to be assembled, skilled staff to be found and, very probably, operatives to be trained.

18.6.1 The provision of compressed air

A final assessment of the air consumption and pressures must be made, and the plant installation, including standby arrangements, designed accordingly (Figures 18.1 and 18.2). Virtually all tunnel sites have high-pressure (HP) compressed air installations for winches, grout pans, pumps and other tools on the site and in the tunnel. The compressors for pressurizing the workings are termed the low-pressure (LP) compressors. They must be capable of compressing large volumes of air up to pressures generally less than 3 bar. Even if a single large LP compressor were available and sufficient to supply the estimated capacity, it would be preferable to have several smaller machines, so that cessation of work by any one machine, either for maintenance or because of a breakdown, becomes less critical. For normal working, the compressors are usually electrically driven, and if power is available from more than one reliable source, that helps to meet the requirements for standby capacity to be available. Electric power generators and transmissions can fail and it is usual to have some diesel-driven compressors and/or diesel generators to supplement other arrangements.

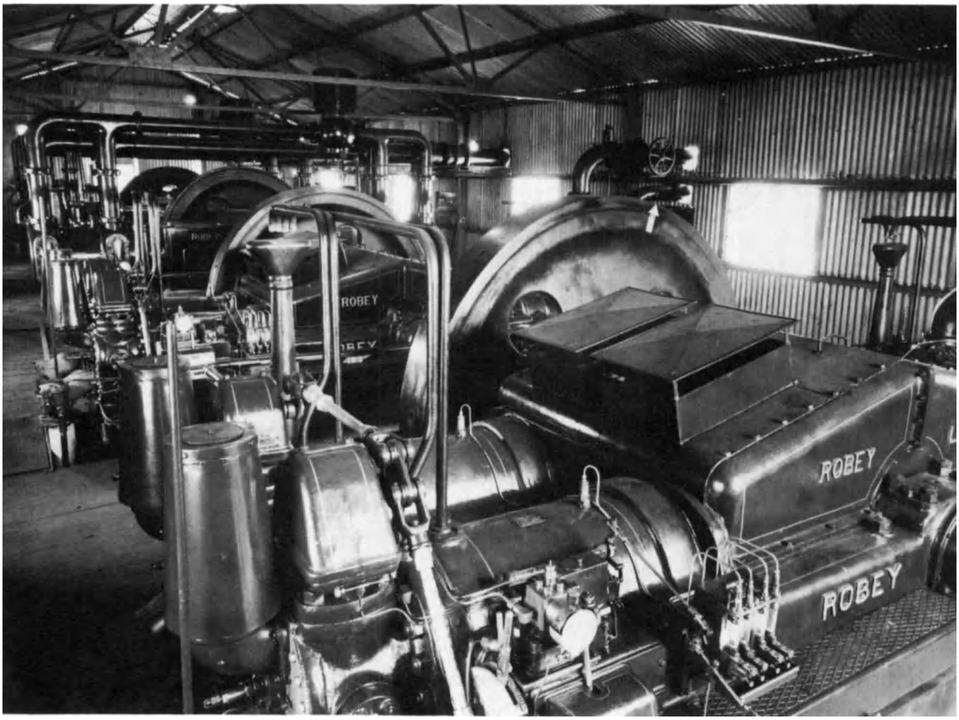

Figure 18.1 Compressor house: Dartford, Road Tunnel pilot 1938, Robey Compressors with power output of 1000 HP, delivering 5000 ft^3/min, 40 lb/in^2

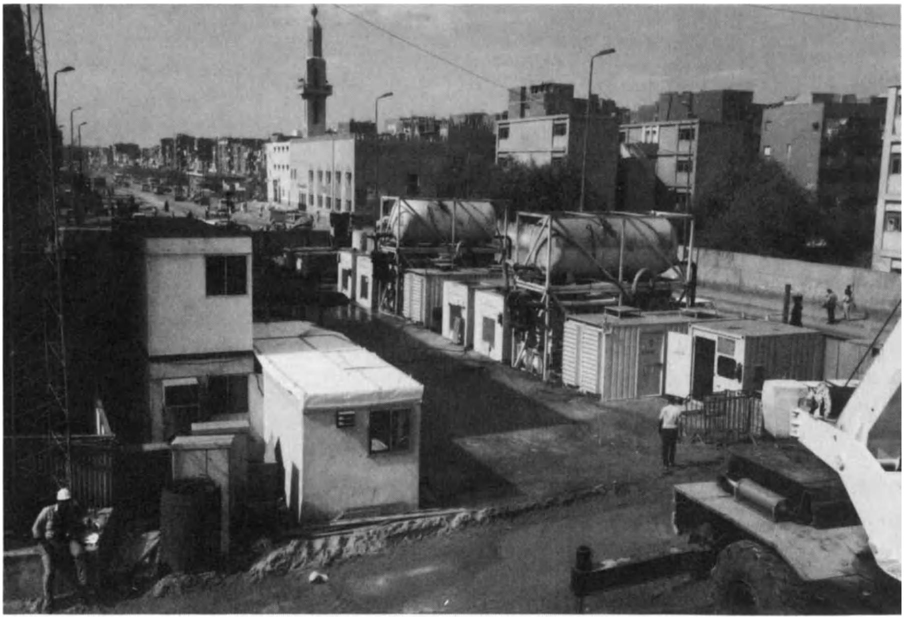

Figure 18.2 Compressor installation; Cairo Wastewater Project 1985, Compair-Broomwade containerized LPCA sets providing a total of 4000 ft^3/min, 50 lb/in^2

The compressors should be sited to have a clean air intake. After compression it will be necessary to cool the air, remove excess water and pass it through an oil separator to remove any traces of oil. The LP air will then pass through large-diameter pipes duplicated where possible, down the shaft, through the airlock and into the working chamber, terminating at a non-return valve. The whole installation has to be capable of maintaining the desired pressure in the working chamber, with a rapid response to any variations in the volume of air consumed. If water pressures in the face are tidal, it will be necessary to vary the tunnel air pressures to suit.

One of the side effects of using LP compressed air is that a higher than usual air pressure must be provided by the HP compressors, to operate the HP air tools in the working chamber which normally require about 7 bar pressure.

18.6.2 Airlocks

The siting of airlocks, whether vertical in the shaft or horizontal in the tunnel, requires investigation of the strength and stability of the shaft and tunnel linings (Figure 18.3). If a vertical airlock is required near the top of a shaft then the shaft has to be able to resist the bursting pressure of the air below the airdeck, as well as the downward weight of the counterbalancing kentledge on the deck. Some shafts are sunk as caissons with bentonite lubrication; if an airdeck is subsequently constructed in such a shaft special consideration must be given to its stability.

In constructing a horizontal airlock near the start of a tunnel care must be taken to ensure the stability of the few rings of tunnel lining which will be in place when the airlock is first commissioned (Figure 18.4). On occasion, it has been necessary

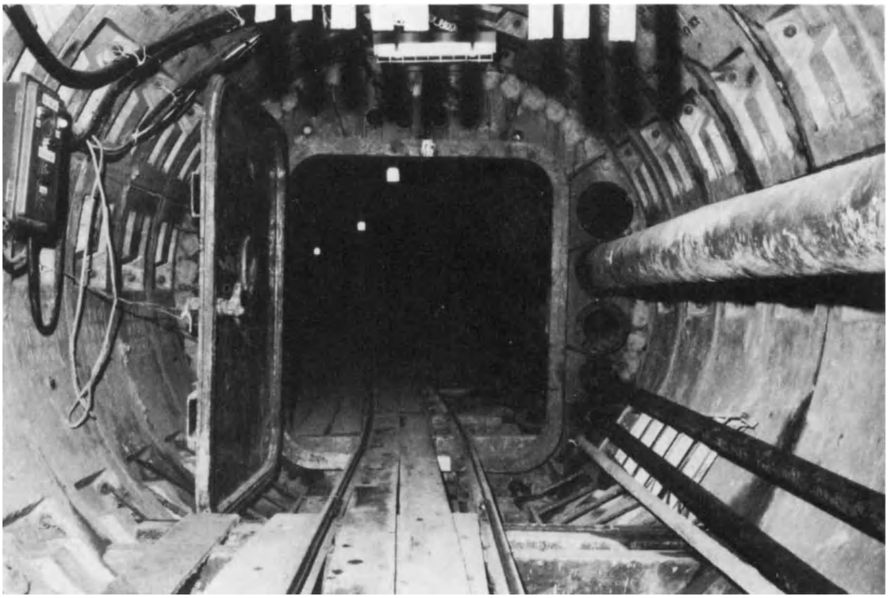

Figure 18.3 Airlock: water-transmission tunnel system, first-stage tunnels, for Metropolitan Works Authority, Bangkok, Thailand

to build anchor plates into such a length of tunnel to prevent its being forced bodily back towards the shaft by air pressure.

In small-diameter tunnels it is normal practice to use a short length of the tunnel itself as the airlock, with bulkheads (and doors) at each end. In larger-diameter tunnels, steel 'boiler' locks are used mounted in a single bulkhead usually consisting of mass concrete. Two or three locks may be included in the one bulkhead so that the passage of spoil, materials and operatives is delayed as little as possible. Careful building, caulking and grouting is required in the vicinity of airlocks to minimize air leakage from the pressurized to the free air side.

All the tunnel services need to be accommodated through the bulkheads, including water supply, fire mains, electricity, telephones and control cables, survey lines, pump or snorer pipes and LP and HP air supplies. In some tunnels provision must be made for hydraulic power lines or spoil-transmission pipes. It is advisable to provide a number of spare pipes with blank flanges. 'Pipe locks' are often provided so that lengths of pipe or rail or various tools can be passed into the workings without occupying the main lock. The size of the airlocks must be adequate to deal with the flow of operatives and materials, but not unnecessarily large, which would be wasteful of time and power.

To pass into the pressurized workings the lock is entered and the outer door closed. Air is then allowed to enter the lock from the working chamber until the pressures are equal, whereupon the inner door is free to open. To return from the workings to free air, the process is reversed. The lock is operated by a specially trained lock-keeper who works on the free air side but who has a window into the lock and telephone communication.

One of the classic accidents of compressed air tunnelling is for the inner door to swing open, or fail to seal while the lock is pressurized, when there is no-one in the

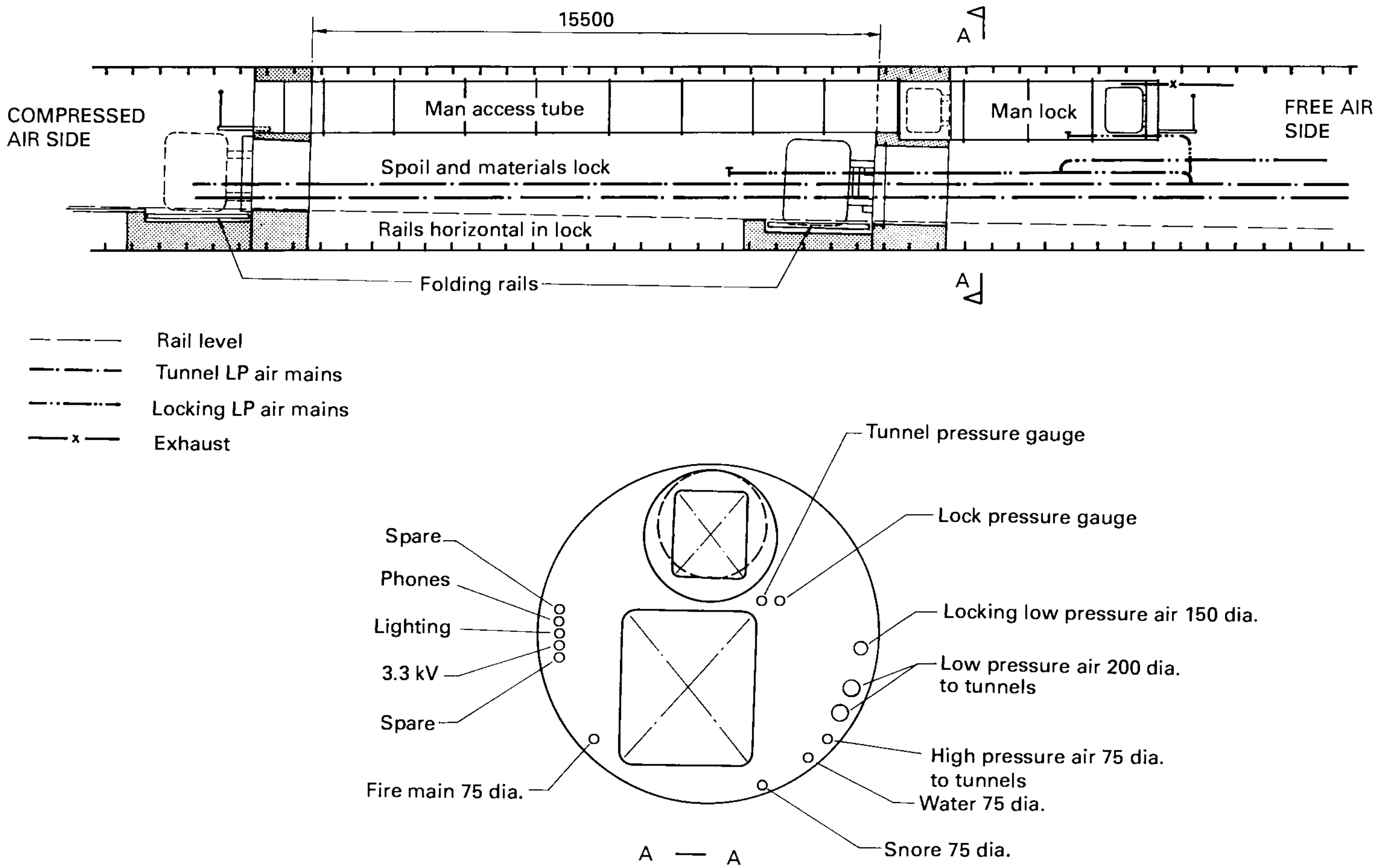

Figure 18.4 A typical horizontal airlock in a tunnel

tunnel. Unless there is a second airlock into the tunnel the resulting dilemma is a good illustration of the need for thoroughness and discipline in compressed air work!

The narrow-gauge railways which normally serve tunnel driving have to pass through airlock doors and airlocks. The removal of short lengths of rail allows the lock doors to be shut.

The airlock doors often put an upper limit on the dimensions of any item which can be taken into the tunnel before completion of compressed air work. Forethought is therefore needed to store any large items on the pressure side of the bulkhead during its construction. One obvious example of such items is the frames and doors for any airlocks that may be needed further along the tunnel!

18.6.3 Tunnelling operations in compressed air

Working practices

Once the compressed air plant is working, the actual tunnel construction is in many respects similar to work in free air. In Britain, a full 8-hour shift can be worked up to 2.8 bar pressure, and the methods of excavation, supporting the face, building the lining, grouting and caulking proceed as in free air, except that the last two operations mentioned are as much to reduce the escape of air out from the tunnel as to prevent the inflow of water.

Control of pressure

Selection and control of the working pressures are most important. There is a temptation to use higher pressure than necessary, to dry up the face: but higher pressures increase the risks of a blow, air consumption and medical hazards. It is often only by experimenting with air pressures that the best pressure can be found, and in most cases that means the lowest acceptable pressure.

Mixed faces

Quite often the face will consist of a band of water-bearing sand or gravel above an impermeable clay stratum. In these circumstances water will almost invariably flow into the tunnel just above the clay, and although raising the air pressure may give some temporary alleviation, the tunnel face is, in effect, draining an underground lake (which may be recharged by an underground stream). No matter how high the air pressure, that water will continue (except in the very short term) to flow into the tunnel face.

Adverse effects of excess pressure

Excessive air pressures can have various adverse effects not mentioned so far. They can increase the pore water pressures in an unstable clay river bank; drive air and water into the normally waterproof basements of city office buildings (where the most valuable documents are liable to be stored); or cause air to travel along an open stratum for a mile or more and emerge to form a geyser in, say, a farmer's field. If the air breaks into a sewer, it might blow every water trap in a district, and the claims, physical, decorative and psychological, can be imagined.

Minimizing air escape

Apart from the waste of power involved when air is allowed to escape unnecessarily, it is highly undesirable from the tunnelling viewpoint. Escaping air can develop an open channel through the ground, following the line of least resistance, and eventually and sometimes quite suddenly becoming a 'blow', with very large quantities of air escaping, possibly into a riverbed, leaving the tunnel without air pressure and facing inundation. In a straightforward tunnel drive, even in open ground, the danger is much reduced by keeping the working face advancing 24 hours a day and 7 days a week. Timber and puddled clay both support the face and reduce air losses. Whenever a face is left unattended it is essential for it to be 'fully boxed up and grouted', even if that might not be deemed necessary purely for support of the ground. Plastic sheets and bentonite can also play their part in reducing air escape: regrouting of any tunnel linings where air can travel back towards an escape into 'free air' may be needed from time to time.

Effect of air on vegetation

One of the more unusual complaints about compressed air tunnelling occurred in a Toronto park, where the air escaping from shallow tunnels in sand caused the grass to grow twice as fast in two unsightly lines across the park.

Snorers

Despite the use of compressed air, it is often to be expected that water will flow in through the working face as previously described. One of the incidental advantages of using LP air is that a pipe can be led from the free air side of the bulkhead to the working face, where a cock and a filter head can be fitted. The air pressure makes this 'snorer' or 'blow-out pipe' into a potent pump for discharging water from the workings to the free air side of the bulkhead. Its use also encourages a flow of fresh air towards the working face to improve ventilation for the miners. The snorer pipe is typically of 75–150 m diameter.

Fire hazards

The additional fire hazards in compressed air are threefold. First, compressed air has more oxygen per unit of volume available to feed any incipient fire, and therefore what might be an innocuous smoulder in free air rapidly becomes an inferno in compressed air. Second, the creation of fire and smoke in a compressed air tunnel, with the airlock interposed between the miners and the outside world, can exacerbate additional difficulties such as darkness, confusion and loss of communication. Third, although in normal tunnels firefighting is readily reinforced by the regular fire brigades, in LP air tunnels it is seldom sensible or practicable to train the local firemen to enable them to be effective in the compressed air tunnel. It is therefore essential to:

1. Insist upon excellent 'housekeeping' to minimize the presence of unnecessary combustible materials in the working chamber;
2. Enforce a no-smoking rule in compressed air;
3. Have a special watch or inspection whenever welding or burning is carried out;
4. Check the availability of sand, water and fire extinguishers at strategic points;
5. Ensure that all those involved know or can easily find out the current layout and accessiblity of the workings;
6. Train and have available at all times an adequate firefighting workforce capable to dealing also with any casualties likely to occur.

Restoring tunnels to atmospheric pressure
When the tunnels and their caulking have been completed and carefully inspected, the air pressure can be removed. Any sudden removal of high air pressure naturally puts a new loading of air and/or water pressure onto the tunnel. Caulking may be disturbed, and cases have been known of grout plugs blown out 'like bullets'. A gradual reduction of pressure is desirable, with frequent inspections so that any important leakage can be remedied at an appropriate pressure. It is advisable to monitor the return of groundwater around the tunnel structure to its normal level, which may be almost immediate or which may take months.

Deoxygenated air
Some strata contain oxidizable material (such as iron pyrites or some organic substances). If such strata are fissured or permeable, any air in the cracks and interstices is liable to have less than the normal proportion of oxygen. There is one old cable tunnel in London which is inspected only at times of high atmospheric pressure, because during depressions the air drawn in from the surrounding ground is not fit to breathe. If compressed air drives the groundwater from an oxidizable stratum the air will be particularly susceptible to losing its oxygen content to the ground. This phenomenon emphasizes the need to monitor the quality of the air in the tunnel. There is a danger of deoxygenated air being drawn into the tunnel whenever air pressure is reduced, whether intentionally or by failure of the LP compressed air plant to meet demand, or when the tunnels are finally being restored to free air. It was during the latter operation that a number of miners and inspectors were asphyxiated in a Melbourne sewer tunnel some twenty years ago.

There can also be a danger to others: during compressed air work on the Victoria Line at Euston in London, LP compressed air from one tunnel forced deoxygenated air into another heading being built at lower pressure, fortunately without fatal results. However, the possibility must be faced that compressed air from a tunnel could drive foul air into an inhabited basement or cellar, or even into an unventilated ground-floor room.

18.7 Sub-aqueous tunnels

18.7.1 Precautionary air

Railway and metro lines often pass beneath rivers, and in doing so compressed air may be needed. In London several lines cross under the River Thames close beneath the riverbed. Even though the Victoria Line tunnels near Vauxhall were expected to be built in London blue clay throughout, it was felt to be worth specifying that compressed air at a precautionary pressure of about one-third bar should be applied while tunnelling under the river. Otherwise, if the working face had unexpectedly encountered a ready connection to the river, such as a buried channel filled with gravel or even perhaps some old screw-pile barge moorings, inundation of the works might have been rapid. It was felt to be necessary to have the compressed air plant in place and actually working, so that pressure could be increased rapidly to meet any requirement. In other instances it may be obvious that compressed air is required in under-river drives, although the recent development of bentonite shields and other tunnelling machines increasingly offer safe and economical alternatives.

18.7.2 Cover above tunnels under rivers

It is recommended that regular soundings should be taken to establish the nature and profile of the riverbed and to record any variations. The surveys of port authorities are not always appropriate; they tend to record the shoals where vessels may ground, whereas the tunneller must find the low spots which are of no concern to the mariner. Echo sounders can be deceptive if silt, in thick suspension, travels along the bottom.

The very successful recent completion of tunnels for the Lyons metro was at one time in jeopardy because the notorious Rhone scoured its gravel bed down to tunnel level. Fortunately, a slurry shield was in use, and the situation was retrieved by dumping material into the river. If soundings reveal dangerous reduction of cover above a tunnel under construction the solution may be to place a blanket of material on the riverbed over and ahead of the tunnelling face. In at least one case, where an early compressed air tunnel blew into the Hudson River, this was a sheet (of strong sailcloth) rather than a blanket, with clay on top of it which was drawn down into the tunnel face by pumping from the flooded tunnel. However, in the more usual case, of an unacceptable reduction in cover discovered by soundings, some authorities recommend placing a blanket by dumping clay from barges to restore a safe profile. It is perhaps assumed that impermeable clay will minimize the escape of air, but the stability of the blanket itself, as affected by air and water, must be considered. It is suggested that weight and the certainty that it will stay there may be more important than impermeability, and that graded rock or a mixture of rock and stiff clay may be suitable materials, especially in fast-flowing rivers or tidal streams.

18.7.3 Measures to facilitate escape

In large sub-aqueous compressed air tunnels, where there is any likelihood of a major blow leading to the inundation of the workings various safety measures to facilitate the escape of the miners have been applied in the past. These include catwalks in the crown of the tunnel, steel curtain plates built in to retain a bubble of air near the crown and emergency man locks placed as high as possible in the bulkhead. In terms of metro railways, such measures are unlikely to be required – or effective – except perhaps in station tunnels, but they should be borne in mind for large-diameter railway tunnels under rivers.

18.8 Compressed air and groundwater lowering

For some projects there may be a choice between LP compressed air and groundwater lowering (whether by deep wells or by wellpoints). Some of the very large underground stations being built or planned clearly favour groundwater lowering of a whole area to facilitate not only tunnel construction but excavation and construction to considerable depth over large areas. For other projects it is tempting to contemplate a combination of groundwater lowering (to allow subsurface construction, say, of a station ticket hall) with LP compressed air (to construct the associated low-level tunnels). The groundwater lowering may, at the same time, allow an important reduction in the air pressures needed for the said tunnel construction.

Such a scheme needs very careful planning and an unusually comprehensive understanding of the local groundwater characteristics. If there is a ready supply of groundwater at tunnel level, either artesian or from a horizontal aquifer of high permeability, then the reduction in necessary air pressures may be illusory. Furthermore, the escape of air from the tunnels to the pumps or wellpoints may so reduce their efficacy that the groundwater-lowering programme is also jeopardized. On the other hand, there are circumstances where a ring of deep wells could successfully lower the water level with little or no risk of being affected by LP compressed air.

18.9 Reduced exposure to compressed air

In recent years, as the danger to health of compressed air working has become more fully understood not only have the medical regulations and decompression standards been improved to make exposure to LP air much safer but there have been various moves to reduce the overall exposure to compressed air. Miners, like divers, are well paid and the consequent mechanization has helped to reduce the workforce. In addition, various attempts have been made to limit the compressed air to a small chamber at the tunnel face, as opposed to the normal 'plenum' process, whereby the tunnel (and all those working in it) is pressurized almost to the shaft bottom. The aim then was to apply compressed air to the face alone, in front of a bulkhead in the shield or TBM. Pioneering work in this field was carried out in Paris in the 1960s, which focused attention on the difficulty of forming a seal around the tail of the shield to prevent LP air in the face of the shield from escaping into the tunnel immediately behind. With the subsequent development of the bentonite shield and its derivatives the attractions of 'air at the face alone' were reduced except in the context of getting into the face of a bentonite shield to inspect, make repairs, remove boulders or change tools. The difficulty of forming a seal around the tail of the TBM was found to be considerable, even when using a bentonite slurry rather than LP air to support the face, but solutions have now been found.

In early attempts to develop the techniques for 'air at the face alone', for the bentonite shield, and for its West German derivative (the Hydroshield, for instance) it was prudent to build an orthodox bulkhead and airlocks so that the whole tunnel could be pressurized in an emergency. The Hydroshield has its own interesting compressed air feature in that it has an air-filled chamber which acts like an air receiver to reduce fluctuations in slurry pressure, into which miners or divers can be locked to carry out necessary work.

The latest development in this field is, perhaps, to be seen in the current contract for tunnelling under the Eastern Channel of the Storebaelt in Denmark. Howden tunnel-boring machines will be working through marl and glacial till (including boulders) but with the probable presence of some water-bearing sand and gravel. When necessary, they will work in an 'earth pressure balance' mode and miners can be locked in through the machine's bulkhead to change tools, etc. The machines will be working at depths where the water pressure will be up to about 8 bar.

When such operations are required at a pressure exceeding 4 or 5 bar it is planned to bolt a hyperbaric divers' life-support system onto the TBM's airlock and for divers to carry out the necessary work in the cutting head of the machine.

18.10 Medical aspects of compressed air working

To work in compressed air a miner should be fit and should not have blocked Eustachian tubes (which prevent the equalization of pressure in the inner ear). Apart from this need to clear the ears, compression from atmospheric to working pressure can be quite fast and the body is unaffected. If the Eustachian tubes are blocked, ear-ache becomes painful at 0.4 or 0.5 bar, and unless the ears can be cleared by swallowing or yawning actions, or by holding both nostrils and blowing, compression must be stopped and the sufferer returned to atmospheric pressure. People suffering from a cold in the head, a sore throat or ear-ache should not attempt compression.

The amount of gas which dissolves in water until a state of equilibrium is reached increases with pressure. By a similar but doubtless more complicated process, when a person is pressurized the blood and tissues dissolve additional nitrogen from the air. Nitrogen is particularly soluble in fatty tissue. The less accessible parts appear to take several hours to become saturated. The individual is quite unconscious of this process, and indeed the only physical indication of being pressurized (in the range of pressures we are considering) is the change in the wavelength of airborne noise, which alters people's voices somewhat, and makes it difficult or impossible to whistle!

To return to atmospheric pressure, it is necessary to re-enter the airlock, and to have a controlled decompression the form of which depends on the working pressure and the length of exposure. As the pressure is reduced, the excess nitrogen dissolved in blood and tissue becomes gaseous. If the decompression is sufficiently gradual, the bloodstream apparently carries small and harmless bubbles of nitrogen to the lungs whence the gas disperses innocuously.

However, in a proportion of people, despite lengthy decompression periods, the bubbles of nitrogen (which continue to be formed until equilibrium is reached an hour or so after decompression) can cause discomfort or damage. This 'decompression sickness' is classified as being of Type I or Type II. These sicknesses have been known to divers and compressed air workers for many years; our understanding of them is far from complete, but is advancing steadily. They rarely occur in pressures of less than 1 bar.

Type I sickness has, for a century, been called 'the bends' or, in a less painful manifestation, 'the niggles'. It is esentially a pain or irritation in a joint or joints. It sometimes happens during the later stages of decompression but has been known to occur as much as 12 hours later. Although the pain can sometimes be severe, it disappears upon recompression (the only sensible cure) and does not recur during subsequent very gradual decompression.

Type II compressed air sickness is more serious and much more rare. It appears to be caused by bubbles of nitrogen in important control centres such as the spinal column and the brain, resulting in a variety of symptoms, including extreme giddiness (known by miners and divers as 'the staggers'), paralysis, stomach pains, collapse or loss of consciousness. The immediate initial treatment is urgent recompression, often to pressures somewhat higher than those to which the patient has been exposed, in order to accelerate re-absorption of the bubbles. The decompression of a patient suffering from Type II sickness can, in extreme cases, take several days, with recompression needed whenever symptoms reappear.

18.10.1 Medical locks

The need on a large contract for separate muck locks and man locks has already been mentioned. In the man lock, compression brings heat and a dry atmosphere (a lower relative humidity) and decompression inevitably causes the opposite; a cold damp misty atmosphere. In man locks, as well as the requirements for gauges, clocks, valves, windows and communication equipment, it is necessary to add heaters.

To enable those suffering from compressed air sickness to be recompressed and treated properly (and, incidentally, to avoid disrupting the tunnel driving) it is necessary to provide a 'medical lock', which should be sited as close as reasonably possible to the man locks. Indeed it is obligatory to provide such a lock if the working pressure exceeds 1 bar.

A medical lock, in addition to all the features of a man lock, requires a bed, additional communications equipment, a chemical closet, a food lock and the facility for a doctor or attendant to get in without altering the pressure.

18.10.2 Acclimatization

It is a fact, so far unexplained, that people can become acclimatized to working in compressed air (CIRIA Report 44). In other words, after a few days working at pressure, and resting or sleeping at normal atmospheric pressures, the incidence of sickness falls significantly. At the risk of giving psychiatrists a field day it might be suggested that this merely confirms our oceanic origins! There is also the associated phenomenon known as de-acclimatization, whereby people who stop work for a few days are more prone to compressed air sickness when they return.

The phenomena of acclimatization and de-acclimatization are also discernible when working pressures in the tunnel are varied, or when people work for a time elsewhere in a different pressure.

There is another type of damage to which divers and compressed air workers are susceptible, i.e. aseptic bone necrosis. It seems that gaseous bubbles liberated during decompression can attack bone. When bearing surfaces of knee, shoulder or, more especially, hip joints are damaged over a period of months or years, use of the joints can become painful or, in extreme cases, impossible. As with the other types of compressed air sickness, the incidence of bone necrosis is reduced but not totally eliminated by careful decompression procedures and other recommended precautions. It is unusual for any of these undesirable medical conditions to manifest themselves in people working in pressures of less than 1 bar, i.e. less than 1 bar above atmospheric pressure.

The Health and Safety requirements in Britain are set out succinctly and thoroughly in CIRIA Report 44, *Medical Code of Practice for Work in Compressed Air*, first published in 1973 but already twice revised and with further amendments in train. The requirements include the engagement of an 'Appointed Doctor' suitably qualified to supervise the medical aspects.

18.11 Other uses of LP compressed air

Apart from the tunnelling techniques covered in this chapter, compressed air is used in the sinking of shafts and of 'pneumatic caissons' which may form elements of a tunnel, or the basements or foundations of any major structure, and of bridges in particular. Compressed air is, of course, used in diving bells, underwater 'habitats', compression and decompression chambers, and indeed in virtually all diving operations in one form or another. Note that in all these operations other than tunnelling there is no great tendency for the air to escape in large quantities (except unintentionally). There is therfore no need for compressors of high cubic capacity, but the ventilation aspects, and the supply of sufficient good air for breathing purposes, are of increased importance.

At very high pressures divers have to breathe not compressed air (which becomes too dense and toxic) but mixtures of oxygen and helium. The expense of this special technique is acceptable for certain operations at great depth. The escape of such expensive gas mixtures through a tunnel face would be unsustainable in virtually all circumstances.

18.12 Summary

Progress in compressed air working and diving can in some ways be compared with that in aviation and space travel: or, in less technical terms, with hillwalking and mountaineering. In all these cases the air traveller, the hillwalker or compressed air worker is increasingly safe as scientific knowledge advances and standards, equipment and procedures improve. However, the dangers to the deep diver, the astronaut and the mountaineer can actually increase as they seek new challenges to overcome.

In recent years our increased understanding of the medical aspects of compressed air work has led to:

1. The development of alternative techniques;
2. Safer working practices for compressed air;
3. Reduced exposure to compressed air for those working on a given project;
4. A lower incidence of medical damage to those involved.

Compressed air tunnelling remains the safest and most economical way of tackling various challenging situations which would otherwise prevent the satisfactory completion of important human enterprises. At present, at pressures of up to 1 bar compressed air is often the best way of building tunnels through various water-bearing strata. At pressures from 1 bar to 3 bar, or slightly higher, compressed air is an expensive technique, to be compared, if possible, with alternatives. It will, in some circumstances, be the most economical and in others the only satisfactory technique to use. Where the required pressure is above 3 bar, the search for alternative methods should become more persistent. While the use of a few people for special operations can be justified, the prospect of gangs of miners driving a tunnel in such high pressures is increasingly unusual. Nevertheless there are currently more aviators, more mountaineers and more deep divers than ever before. Who, then, will predict the demise of compressed air tunnelling on grounds of safety, when the advance of scientific knowledge is making it safer as time passes?

References

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Chapter 19

Large-diameter and non-circular tunnnels

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Contents

19.1 Large-diameter tunnels

Previous chapters have described tunnelling methods and linings for running and station tunnels in all ground conditions. They have outlined the methods of excavation by tunnel-boring machines or excavators, of providing face support, such as compressed air or ground treatment, of ground support and of lining tunnels in precast segments or cast-*in-situ* concrete. In particular, Chapter 13 has described methods of ground support. Those chapters have been concerned with medium to long lengths of running tunnels and for shorter drives for station tunnels.

This chapter covers short lengths of large-diameter or non-circular tunnels where the use of a shield is generally impractical and methods of excavation, ground support and linings have to be adapted to suit the particular tunnel. In the case of station tunnels the tunnel drive may be of a single diameter, while for crossovers or step plate junctions, where two running tunnels join into one tunnel, there will be short lengths of tunnel of different diameters to suit the track alignment and the structure gauge. In stations there are other large-diameter tunnels required for escalators, machine chambers for escalator equipment and shield chambers for the erection of station tunnel shields. In all these instances there are also special problems at the junctions between tunnels of different diameters.

19.2 Crossovers and step plate junction tunnels

19.2.1 Choice of tunnel shape

A crossover is the location of a junction where two running tunnels join to allow trains to pass from one running tunnel to the other running tunnel to enable single-line working to be used for maintenance reasons or in case of an emergency when one of the lines is out of use. Figure 19.1 shows the crossover, with a 1 in 6 diamond crossing, to the east of Heathrow Central Station on the London Underground, Piccadilly Line extension to Heathrow Airport. The crossover consists of a central section of 54 m of 9.500 m internal diameter flat-bottomed lining and two 22 m trouser-leg tunnels at each end[1]. A step plate junction is where two running tunnels join into one tunnel. Figure 19.2 shows a step plate junction on the London Underground Victoria Line between Euston Station and King's Cross Station[2], which consists of tunnels of 5.64, 6.45, 7.62 and 9.15 m internal diameter.

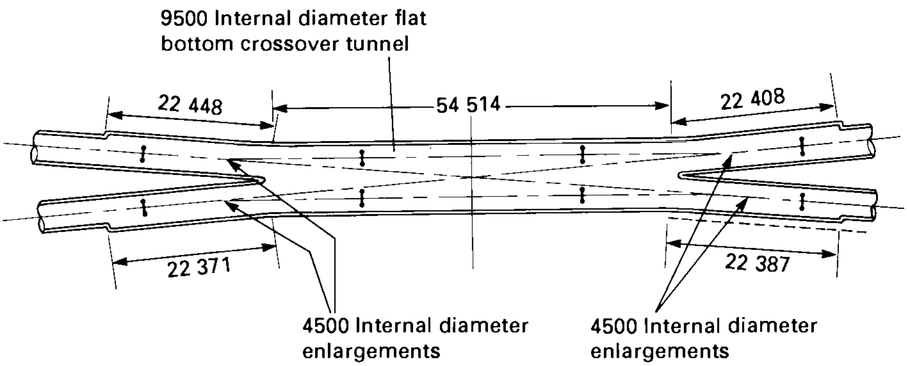

Figure 19.1 London Underground crossover tunnels

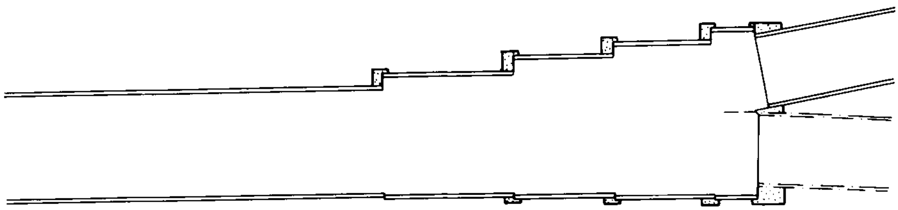

Figure 19.2 London Underground step plate junction

The cost of the construction of a unit length of tunnel is related to:

1. The volume and the method of excavation;
2. The type and method of erection of the tunnel lining;
3. The extent of any internal civil engineering works to fit out the tunnel for its use.

It is therefore important to keep the extent and cost of the internal works to a minimum. For example, for a running tunnel the structure gauge is relatively square and will fit closely to the circular or other shape of the tunnel. Figure 19.3 shows the structure gauge for a London Underground running tunnel. Circular, 'D'-shaped or horseshoe tunnels will be suitable and economical shapes for running tunnels. The internal works to provide the concrete infilling below the trackbed will be small. In the case of large-diameter tunnels for a crossover or a step plate junction the width or horizontal dimension of the tunnel is the critical dimension. Figure 19.4 shows the rectangular shape of the structure gauge for a large tunnel for a crossover or step plate junction. For a crossover or step plate junction the combined structure gauges for the two tracks will give the horizontal dimension with the height similar to that for each of the structure gauges.

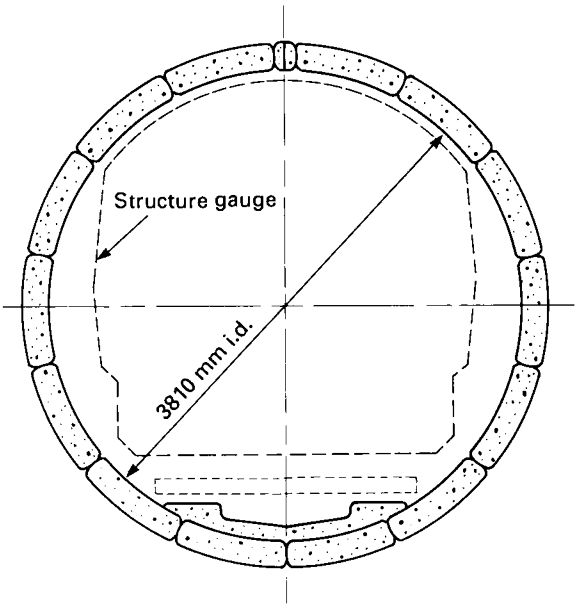

Figure 19.3 London Underground running tunnel and structure gauge

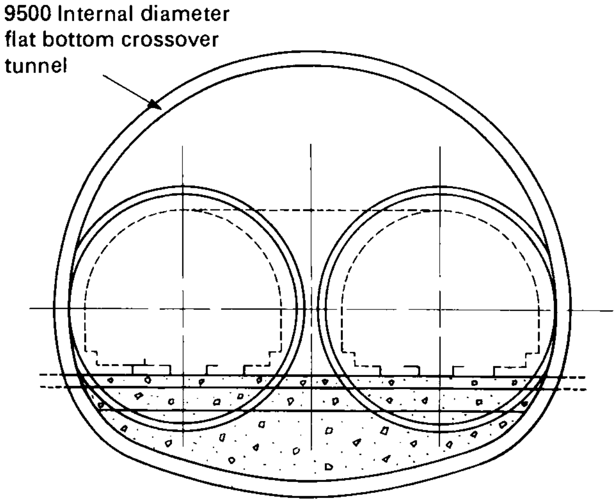

Figure 19.4 Structure gauges for crossover or step plate junction

The ideal shape for a crossover or a step plate junction is therefore a rectangular tunnel, but this is a difficult shape to design and to construct for a bored tunnel. The alternative is to construct the crossover or step plate junction by cut and cover methods (see Chapter 6). For the Singapore Mass Transit System the alignment was not restricted by existing tunnels and it was possible to design the vertical alignment so that all the underground crossovers could be closer to the surface and constructed under roads or open areas by cut and cover methods [3]. Typically, cut and cover crossovers or step plate junctions may be between 30% and 60% of the cost of a similar bored tunnel scheme. This major cost saving is partly on account of the closer spacing of the tunnels (and thus a reduced length of the crossover or step plate junction) and partly on account of the higher unit cost of excavation and lining for short lengths of large-diameter tunnels in bored tunnel compared to cut and cover methods.

A circular shape for the large tunnels for a crossover or a step plate junction will therefore not normally be economical. For a circular tunnel the structure gauge will be symmetrical about the horizontal axis of the tunnel. The infill concrete required up to the underside of the trackbed may be equivalent up to 20% of the volume of the tunnel excavation. In addition, there is the volume of this over-excavation to be considered. It can therefore be seen that the high cost of the internal works to the underside of the trackbed should be kept to a minimum.

For non-shield-driven large tunnels for crossovers or step plate junctions the method of construction, the lining and the internal works must therefore also be taken into account when deciding on the tunnel shape. For a crossover or step plate junction the largest tunnel internal diameter may be in the range of 10–13 m to allow for the horizontal dimension of the structure gauge. The vertical dimension of the structure gauge, in contrast, may be only 4–5 m. For these larger tunnels flat-bottomed shapes have often been provided where the vertical diameter is one or more metres shorter than the horizontal diameter. Figure 19.5 shows a flat-bottomed-shaped tunnel lining for a 10.0 m tunnel for the Hong Kong Mass Transit Railway [4]. The design of flat-bottomed tunnels incorporates a circular top

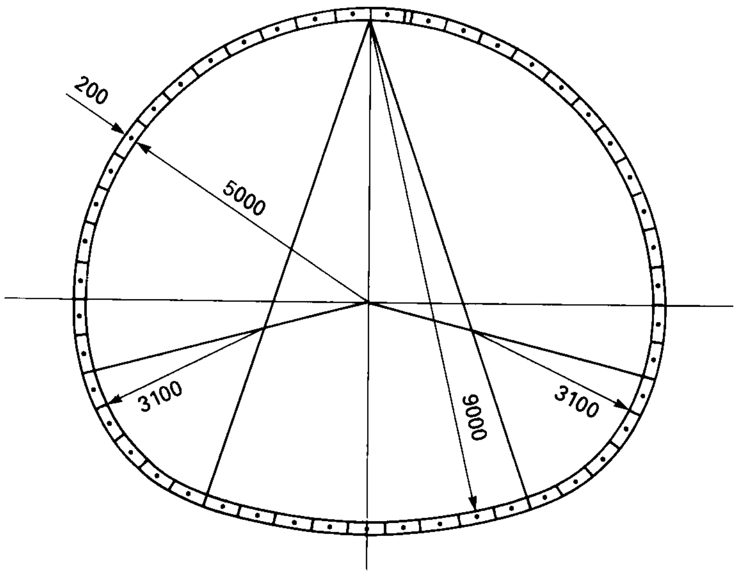

Figure 19.5 Flat-bottomed-shaped tunnel lining

half of tunnel equivalent to the required horizontal diameter, a radius of 5 m for the tunnel shown in Figure 19.5, and a larger radius for the invert which may be up to twice the radius for the top half of the tunnel (9 m in Figure 19.5). These two radii are joined by a short length of smaller radius (3.1 m in Figure 19.5).

In the detailed design of a flat-bottomed-shaped lining the hoop loading has to be transferred from the top half of the tunnel, with the smaller radius, to that in the bottom part of the tunnel, with the larger radius. The segments in the top half of the ring are designed in the normal manner for a tunnel lining as described in Section 14.3.2 for the hoop loading and moments. For the segments in the bottom part of the ring the hoop loading is the same as that for the top half, the segments are of the same profile as those in the top half (but of a different radius) and the moments are calculated in a manner similar to those of the top half. The special segment between the top half and the bottom part of the ring has to be designed to take additional moments. The ground loading on the segment will act as a uniformly distributed loading and the hoop loading will be similar to the rest of the lining. The special segment is at the 'knees' of the ring, and there will be little bending moment, caused by squatting of the ring, in the normal design, as the segment will be at the point of contraflecture. The line of thrust will be at the centroid of the section at the two ends of the segment. However, the line of thrust along the segment will not be along the centroid, as in the other two parts of the ring, but will be parabolic. This will cause a bending moment equal to the hoop loading times the eccentricity of the line of thrust compared to the centroid of the section. Linings with flat bottoms have been used for large-diameter tunnels in metros worldwide.

In the design of the Hong Kong Mass Transit Railway crossover tunnels were provided of up to 10.0 and 11.6 m internal diameter [4]. The tunnels on one of the contracts were to be constructed through decomposed granite under compressed

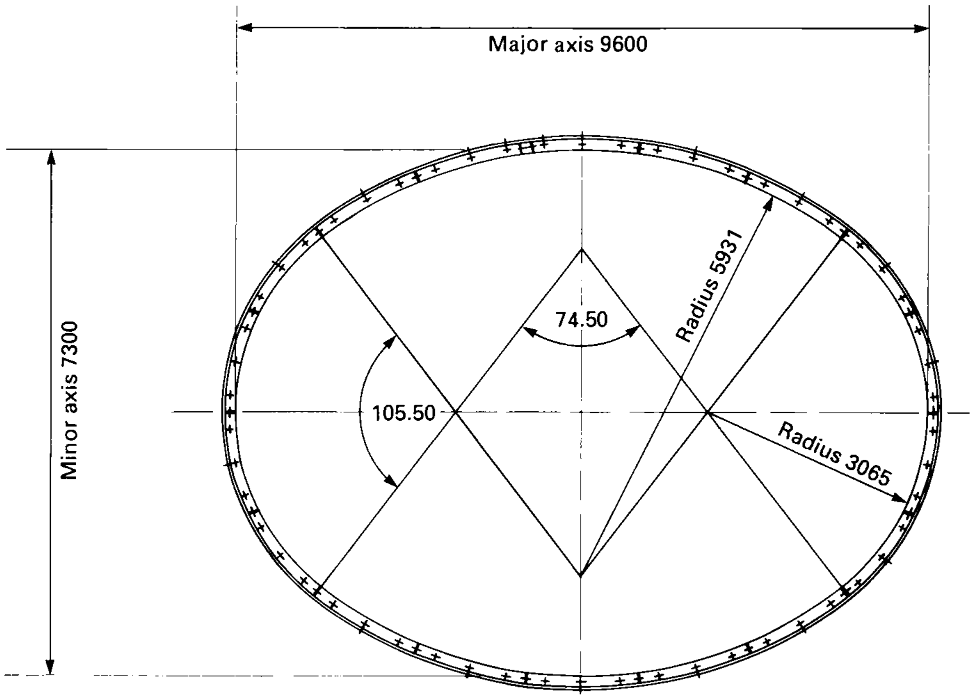

Figure 19.6 Near-elliptical or oval-shaped tunnel lining

air. The successful contractor in his tender proposed flat-bottomed tunnel linings on account of his concern about the height of the tunnel face to be supported and the high excess compressed air pressure at the crown of the excavation. Further consideration, after the award of the contract, led to the design of a near-elliptical or oval-shaped lining for two of the crossovers. Figure 19.6 shows the elliptical or oval-shaped lining used in the contractor's alternative. The elliptical shape has a horizontal, major, axis of 9.6 m and a vertical, minor, axis of 7.3 m. This vertical axis was nearly 2 m shorter than that for the flat-bottomed lining.

In the design of the elliptical lining the hoop loading was established for a circular tunnel having a diameter equivalent to that of the major axis of the ellipse [4]. This load was then assumed to act within a circular lining of radius equal to that of the smaller, minor, radius of the ellipse. An analysis was then carried out for the loadings and the deflections on the smaller diameter in a manner similar to a circular tunnel. This analysis was checked using a structural analysis computer program. Two of the crossovers were in medium-strength decomposed granite, while the third was in a low-strength granite. Analysis showed that only in the medium-strength granite was there sufficient ground support to limit the deflections along the horizontal axis of the lining to within the allowable limits. These two crossovers were therefore constructed with elliptical-shaped linings with the third crossover constructed using the conventional flat-bottomed-shaped lining. In the construction of the elliptical linings the rings were built to a high standard with a resulting minimal settlement of the ground surface.

19.2.2 Linings for crossover and step plate junction tunnels

Traditionally, tunnel linings for large-diameter tunnels in weak or soft ground conditions have been in cast iron. The early London Underground tunnels were in grey iron, but with the introduction of spheroidal graphitic iron in the 1960s and 1970s more recent large-diameter tunnels have been lined in this form of cast iron. The crossovers and other large-diameter tunnels for the Hong Kong Mass Transit Railway and the step plate junctions in weak rock and soft ground for the Tyne and Wear Metro were lined in spheroidal graphitic iron[4, 5].

Precast concrete segments have been used for large-diameter tunnels for roads but not for crossover tunnels. Various schemes have been designed for their use but they have not reached the construction stage. Future tunnels for crossovers and step plate junctions will be lined in spheroidal graphitic iron or in concrete segments, their choice being related to the ground conditions, the requirements of the watertightness and the cost. Until recently it was possible to make cast iron lined tunnels considerably more watertight than concrete segmental lined ones. With the introduction of gaskets and other forms of sealing materials, segmental concrete linings will now be suitable for these tunnels in many ground conditions. The increased weight of large concrete segments, compared to cast iron segments, may be the main reason why cast iron segments may be preferred, despite the large cost saving of concrete segments.

The alternative to precast linings for large-diameter tunnels is the initial use of ground support and an *in situ* concrete lining. Details of the method and the sequence of excavation for large tunnels, the methods of ground support with rock bolts, mesh, shotcrete and arches are discussed in Chapter 13. Generally, a waterproofing membrane is attached to the inside of the shotcrete prior to casting an unreinforced or reinforced concrete primary lining.

19.2.3 Methods of construction

The method of construction of short lengths of large-diameter tunnels is dependent to a large extent upon the ground conditions, the depth of tunnel, the allowable settlement and its likely effect on any property above, and the type of lining to be installed. The following sub-sections discuss a number of construction methods.

Pilot tunnels

Short lengths of large-diameter tunnels are often constructed using one or more pilot tunnels. Where it is uneconomical to use a shield for a large-diameter tunnel, pilot tunnels are invaluable as they restrict the area of the face which is open, and thus the support required to the face is greatly reduced. Pilot tunnels may be 2–4 m in internal diameter, which may be increased as a second pilot to an internal diameter of 5–6 m for the larger-diameter tunnels. Alternatively, two or three 2–3 m pilots may be used within the face of a large-diameter tunnel. The timbering for a full face of 9–14 m is a massive operation requiring very large soldiers, struts and walings in timber or in steel.

For the construction of a crossover the two running tunnel shields may be driven through the area of the crossover, thus forming the pilot tunnels. For the crossover adjacent to the Central Terminal Station at Heathrow Airport, shown in Figure 19.1, the two 3.81 m internal diameter shield-driven tunnels were driven through the area of the 9.5 m internal diameter crossover tunnel as pilot tunnels[1]. For larger internal diameter running tunnels, of 4.5–6 m internal diameter for larger

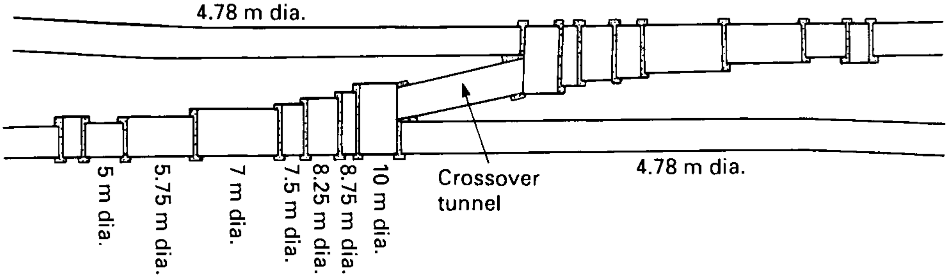

Figure 19.7 Single crossover in a double step plate junction

rolling stock, it may be possible to drive only one of the running tunnels through the crossover. A smaller tunnel of, say, 2.25–2.5 m internal diameter may be hand driven on the opposite side of the crossover to provide a pilot tunnel to reduce the area of the face.

For a step plate junction it will be possible to drive only one of the running tunnels through as a pilot, with a smaller internal diameter pilot of 2.25–2.5 m driven adjacent to the running tunnel for part of the length of the enlargement. Figure 19.7 shows a single crossover in a double step plate junction on the Tyne and Wear Metro[3]. The two running tunnels of 4.78 m internal diameter were constructed as pilot tunnels through their respective step plate junctions. These pilots were then enlarged to give the 5.0, 5.75, 7.0, 7.5, 8.25, 8.75 and 10.0 m internal diameter tunnels for the step plate junctions. The crossover tunnel was constructed between the two step plate junctions. Figure 19.7 also illustrates the two shield chambers at the ends of the running tunnels and the number of ringwalls, nine for each of the step plate junctions, between rings of different diameters.

In all pilot tunnels the lining will be recoverable and, if undamaged, may be re-used for further pilot tunnels. Second-hand cast iron tunnel linings are therefore often used for pilot tunnels. Precast concrete segmental linings are likely to be damaged if used for a pilot and only a small proportion of the segments are likely to be re-useable. Expanded concrete or cast iron linings have been used as pilots as they are quick to erect, thus allowing the pilot tunnels to be constructed at a considerably faster rate of progress. In the case of the cast iron pilot expanded linings they were standard grey iron linings expanded with wedges or keys to the circular excavated profile.

Compressed air

Large-diameter crossover tunnels and step plate junctions have been constructed in compressed air. The compressed air may be provided to assist in supporting the face to improve the stability ratio (see Section 13.1.7) or to hold back the water table. (Chapter 18 discusses compressed air tunnelling in detail.) For large-diameter tunnels with an excavated face of 10–13 m, the difference in water pressure across the face may be the equivalent of between 1.0 and 1.3 bar. If compressed air is used in more permeable ground with the air pressure balanced near the crown there is likely to be a large inflow of water in the lower part of the tunnel. Alternatively, if the air pressure is balanced nearer to the invert there may be the possibility of a blow of the ground above the tunnel, thus resulting in a collapse of the face. Compressed air is therefore more likely to be used for large-diameter tunnels for the improvement of the stability of the face.

For the crossover adjacent to Heathrow Central Station on the London Underground extension of the Piccadilly Line to Heathrow (see Figure 19.1) the face was in London Clay but there was only a cover of 2 m of clay above the excavated crown and a total cover to the ground surface of 9 m [1]. Above the London Clay were waterbearing open gravels. Compressed air was installed and used for the construction of the crossover to provide an increase in the stability of the face. The excavation for the crossover was carried out using a mechanical excavator from the invert to the level of the shoulders and by hand from a staging for the excavation of the crown.

A second example of the use of compressed air was the construction of a number of the crossovers for the Hong Kong Mass Transit Railway, where compressed air was used to improve the stability of the face [4]. Ground treatment was also carried out where there was reduced cover to the crossover. The height of the face was reduced in the elliptical-shaped tunnel by up to 2 m compared to a flat-bottomed-shaped tunnel and up to 3 m for a circular one, thus reducing the out of balance of the air pressure and the associated risks.

Horizontal piles

Contiguous piles have been used successfully for many years for the construction of shafts. The Mount Baker Ridge project in Seattle in the United States was opened in 1989 [6]. The project, for five lanes of road, has an internal diameter of 19.4 m with a low cover. The tunnel was constructed by excavating with a shield twenty-four 3 m nominal internal diameter or horseshoe-shaped tunnels or drifts along the periphery of the tunnel and backfilling each tunnel with concrete to form contiguous horizontal piles (see Figure 19.8). The mass excavation was then carried out and the concrete for the internal construction cast. A similar form of construction has been considered for the crossover on the French side of the Channel Tunnel. The Seattle scheme was constructed through a hill from shafts at either end of the tunnel. For a scheme to be constructed completely underground the horizontal tunnels forming the contiguous tunnels will be driven from underground caverns and will be considerably more difficult to construct.

A similar form of construction has been used in Japan, for a canopy over the crown, for a road tunnel and a river-diversion tunnel. In both these cases a piped roof was formed above the tunnel prior to excavation using steel pipes of 400 mm or 700 mm diameter. The piped roof formed a canopy above the tunnel excavation to reduce the surface settlement as there was little cover to the tunnels. In the case of the Mount Baker Ridge project the minimizing of the surface settlement in the soft ground and low cover were major factors in the choice of the method of construction.

An alternative method utilizing horizontal piles is the use of spiles which are drilled and installed from the tunnel face [8]. Inclined perforated pipes, grouted or injected, are installed to form a canopy above the tunnel excavation. The spiling system may be constructed 10–20 m ahead of the face, depending on the size of the tunnel and the ground conditions, allowing excavation of 5–15 m of the face prior to the installation of the next set of spiles.

*Ground support and cast-*in-situ *concrete lining*

Details of the method of construction of tunnels using ground support and cast-*in-situ* concrete linings have been discussed in Chapter 13. The tunnelling method entails the construction of a series of side, top, intermediate and bottom

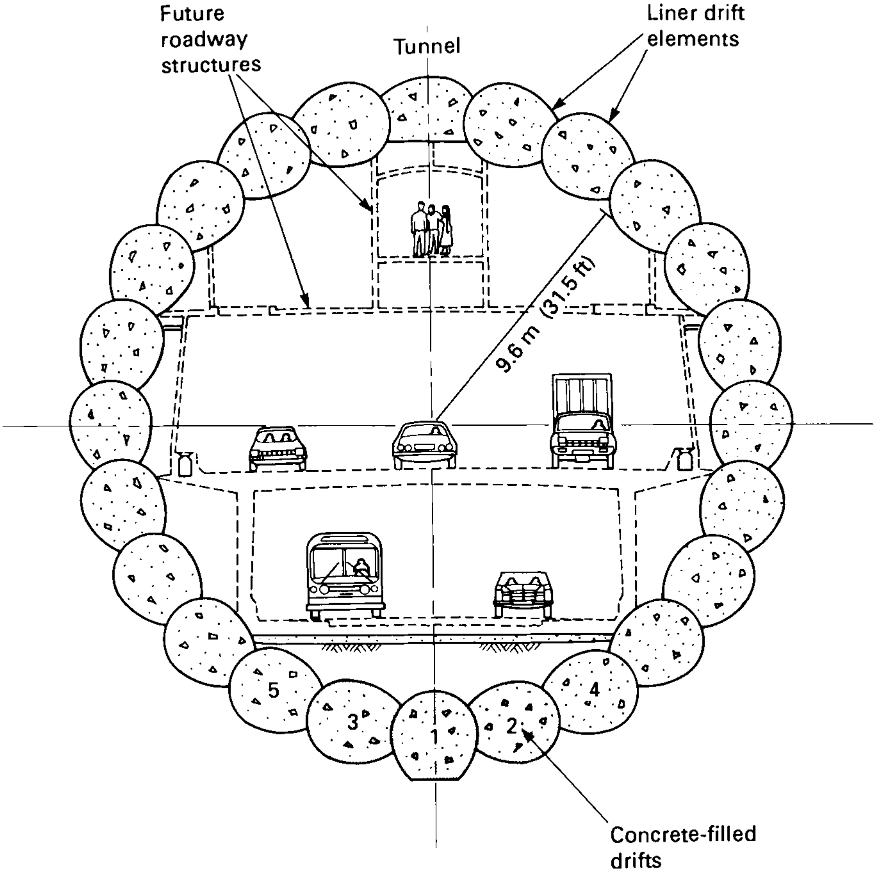

Figure 19.8 Mount Baker Ridge Project with horizontal contiguous drifts

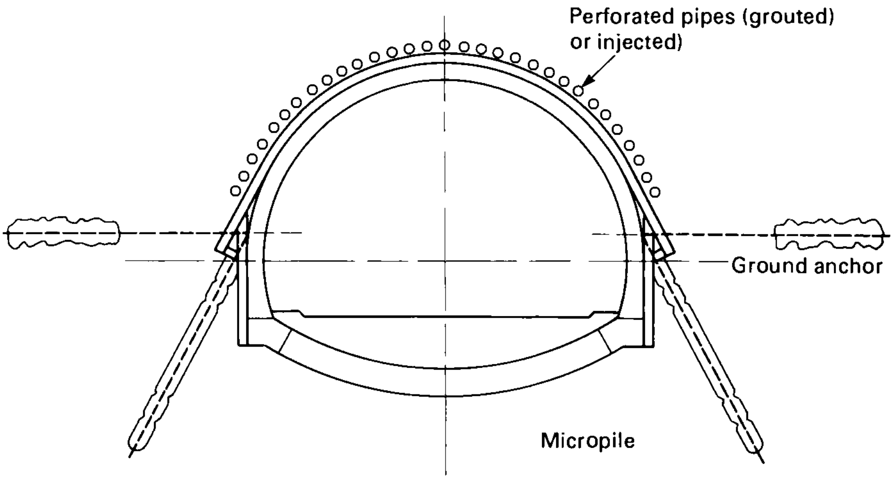

Figure 19.9 Spiling to form a canopy above a tunnel

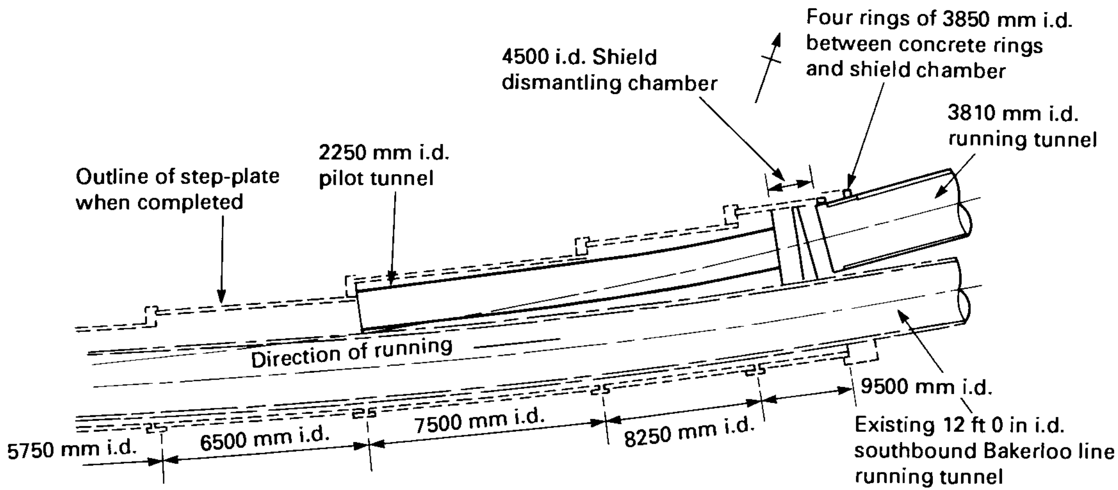

Figure 19.10 Jubilee Line, Baker Street, step plate junction

heading, depending on the size of the eventual tunnel and the ground conditions. The headings are excavated using an excavator or by drill and blast. Ground support of mesh, shotcrete, rock bolts and arches is provided to suit the ground conditions and to prevent the ground from weathering. The large development tunnels and the crossover tunnels for the English side of the Channel tunnel are being constructed using these methods of ground support.

19.2.4 Enlargements around existing tunnels

In the construction of extensions of underground railways it is sometimes necessary to form crossovers or step plate junctions around existing tunnels which have to be kept in use with trains running during the construction. Such enlargements were necessary both in the construction of the Victoria and Jubilee Lines on the London Underground[2, 9, 10].

Near Baker Street Station a new step plate junction was constructed for the Jubilee Line around the existing Bakerloo Line while trains were kept running[9]. Figure 19.10 shows the tunnel layout. The new Jubilee Line running tunnel was constructed as far as the step plate junction and a 4.5 m internal diameter shield chamber constructed to dismantle the shield. A 2.25 m internal diameter pilot was then constructed to the end of the 7.5 m internal diameter enlargement. During the construction of the enlargement for the 9.5, 8.25, 7.5, 6.5 and 5.75 m internal diameter tunnels the existing running tunnel was supported using saddles and steel frames. Figure 19.11 shows a section through the 9.5 m internal diameter tunnel enlargement during the construction as well as the 2.25 m internal diameter pilot tunnel and the support of the existing running tunnel.

On the Victoria Line the step plate junction between Euston and King's Cross was within the Woolwich and Reading beds with sand lenses[2]. The construction around the existing running tunnel included a 2.13 m pilot tunnel below a sand lens

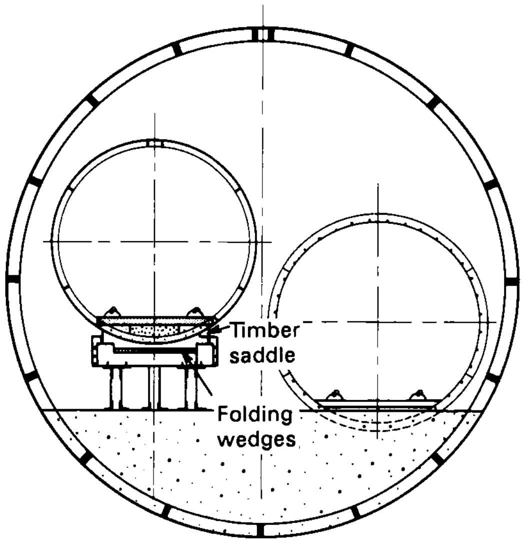

Figure 19.11 Jubilee Line, Baker Street, temporary support to existing running tunnels. Cross section through step plate junction

with probes to dewater it. Compressed air was not appropriate in the ground conditions, but bentonite cement, followed by resin grouting, was carried out to help stabilize the ground.

19.3 Station and concourse tunnels

In general, station and concourse tunnels are circular in shape. For the London Underground and other underground systems the station and concourse tunnels have generally been between 6.5 and 8.25 m internal diameter. For the Tyne and Wear Metro the station tunnels were 7.0 m internal diameter. For larger rolling stock and where more station space is preferred (and for island platforms) station tunnels may be up to 9.0–10.0 m internal diameter.

The structure gauge for a station tunnel is based upon the running tunnel structure gauge and the width and height of the platform. Figure 19.12 shows the structure gauge for a London Underground station tunnel. The structure gauge is approximately symmetrical about the axis of the station tunnel. In the invert a substantial quantity of concrete has to be placed to the underside of the trackbed and the platform. Voids can be provided for ducts, etc. for cables, pipes and ventilation. In the crown any additional space can be usefully employed in the architectural features for the station.

Most of these tunnels have been constructed using shields requiring shield chambers of 8.25–10 m internal diameter. These shield chambers are normally excavated by hand.

The majority of station tunnels are constructed as enlargements of the running tunnels. The area of the face is therefore reduced and thus the requirements of the temporary support to the face. Where the programme is very tight it may be necessary to construct the station tunnel with a full face, and in these instances it is essential that a shield is considered so that in weak rock or soft ground the necessary face support can be provided.

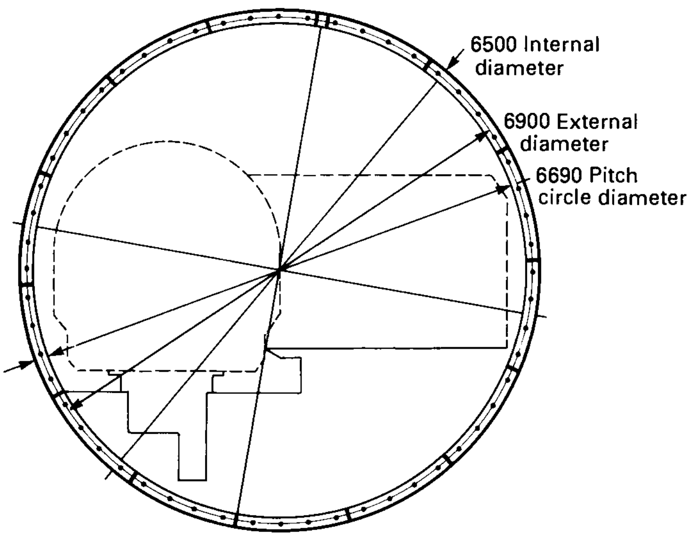

Figure 19.12 Structure gauge for station tunnel

Station tunnels constructed in weak rock with the use of ground support and *in-situ* concrete primary linings will generally be non-circular and will be constructed using the methods described in Section 19.2 and Chapter 13.

Circular tunnels for station tunnels have generally been lined in cast iron. The earlier tunnels were in grey iron but, as with the linings for crossovers and step plate junctions, they have been lined over the last twenty years in spheroidal graphitic iron. Station tunnels are, however, now being lined in concrete segments. The choice of linings in the future will either be in concrete linings or in spheroidal graphitic iron, depending upon the ground conditions and the watertightness required.

Most station tunnels are constructed using a shield, and thus the size and the weight of the concrete segments is not of such importance, as equipment can be incorporated behind the shield for the erection of the segments. In the assessment of the choice of lining, consideration must also be taken of the considerably higher cost of the spheroidal graphitic iron linings.

In two instances on the London Underground Victoria Line station and concourse tunnels, special linings were used which were fabricated in steel [2]. The linings were of the expanded lining type and were erected behind a tailless shield against the ground. They were expanded with jacks at approximately axis level.

At Oxford Circus the southbound station tunnel of 6.67 m internal diameter, passed within a metre or so below the foundations of the third basement of Peter Robinson's department store. It was important that settlement of the foundations was kept to a minimum. The basement was underpinned with a prestressed raft to carry the loading and to distribute it uniformly over the tunnel. An expanded lining was chosen so that the ground loadings could be reinstated as soon as possible after the excavation. Figure 19.13 shows the tunnel lining provided for the station

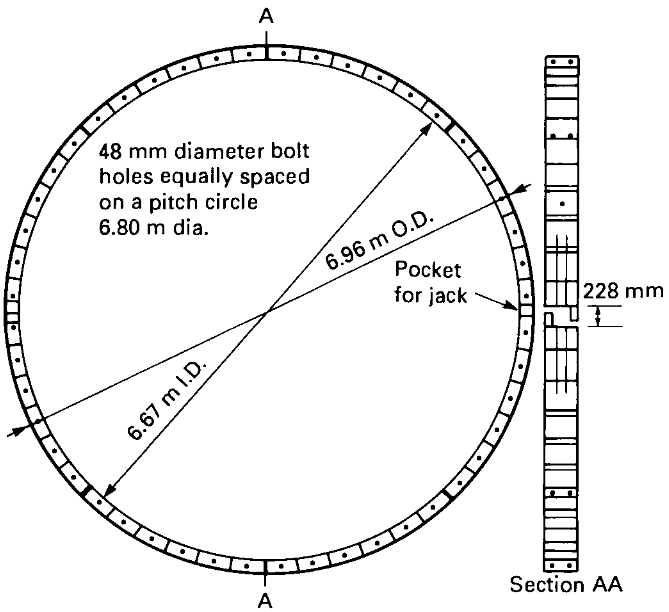

Figure 19.13 Victoria Line, Oxford Circus station, expanded lining

tunnel. The lining was a conventional segmental one above and below axis but was strengthened at axis level to take the jacking forces. The method of construction was important and the shield was of a heavy construction to take the concentrated vertical loading of 500 tonnes and a face support of 20 tonnes/m^2. The shield incorporated six face rams and six tables with a total face pressure of 800 tonnes which was maintained at all times. No bead was added at the cutting edge of the shield.

At King's Cross the station tunnels and the concourse tunnel passed immediately above the existing Northern and Piccadilly Line tunnels and beneath the Metropolitan and Circle Line ones. In addition, the tunnels passed below the British Rail Midland Curve tunnel. It was important to keep settlements of the brick-lined British Rail tunnel to a minimum. The Victoria Line station and concourse tunnels passed the British Rail tunnels at a skew and a special lining had to be designed to take the very high loads. The expanded lining designed used a conventional lining below axis. The segments were of fabricated steel of the standard bolted lining type (see Figure 19.14). Above axis a special lining was designed with an increased sectional area to withstand the high concentrated loadings from the footings of the tunnel above. The lining was designed as a semi-circular two-pinned arch for the various loading cases as the high loadings passed over the tunnel. The bending moment envelope contained both hogging and sagging moments around the semi-circle with maximum values of 4.5 and 3.5 tonnes/m, respectively. The joints between segments were designed to maintain full continuity of section with two rows of high-tensile bolts.

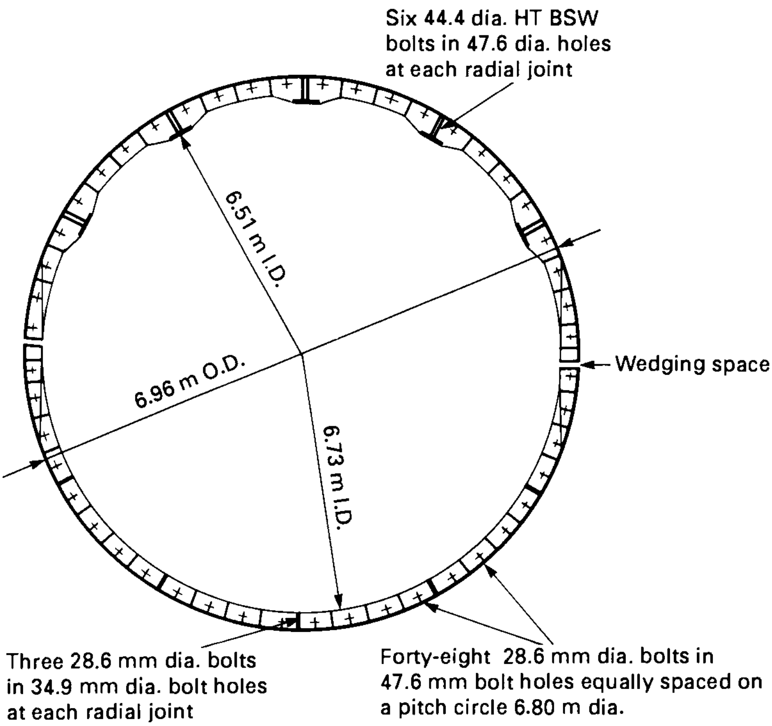

Figure 19.14 Victoria Line, King's Cross station, expanded lining

19.4 Escalator and machine chambers

Escalator tunnels generally vary between 5.75 and 7.5 m internal diameter. In soft ground the tunnels are normally of circular shape. Space is required below the escalators for equipment and any additional space above can be used in the architectural features. The machinery for escalators is now housed in the machine chamber at the top of the escalator where a diameter of 7.5–10.0 m will be required. In the older escalators a similar chamber was needed at the bottom of the escalator.

The linings for escalators and machine chambers have in the past been in cast iron. Linings of concrete segments are now being used and future linings may be in spheroidal graphitic iron or in concrete segments, depending upon the ground conditions.

Escalators are on an incline of approximately 30°. They therefore traverse a mixture of ground conditions. Escalators for London Underground developments pass from the superficial deposits, through the London Clay and sometimes into the Woolwich and Reading beds below. The upper section of an escalator may therefore give major construction problems. Pilot tunnels are normally used in the construction of escalator tunnels to reduce the area of the face. Escalators are often constructed below existing buildings which may have to be underpinned or special measures taken to reduce settlement.

These special measures may include ground freezing or grouting of the ground to assist in the stabilization [10]. The escalator tunnel, for example, at Tottenham Hale Station on the Victoria Line passes through water-bearing gravels with lenses of silty material. Previous experience showed that ground treatment might not be successful, and ground freezing was installed and used successfully. Initially, it was planned to install a pilot tunnel, but the excavation and lining for the first few rings without a pilot showed that the ground had been stabilized and the whole length of the escalator was constructed full face. At Vauxhall Station ground freezing was also used and again the escalator was driven full face.

The construction of escalators is therefore slow and costly. For new mass transit schemes where there are few constraints the construction of stations is preferred in cut and cover. As in the case of crossovers and step plate junctions, this requires the vertical alignment to be closer to the surface. However, there are major cost savings in the construction in cut and cover and particularly in the overall programme.

19.5 Junctions

In the construction of stations, crossovers and step plate junctions there will be many junctions between tunnels of different diameters. Figures 19.1, 19.2, 19.7 and 19.10 show a number of crossovers and step plate junctions and illustrate the number of ringwalls between tunnels of different diameters. For the Baker Street works on the London Underground Jubilee Line ringwalls were required between each of the tunnels of different diameters. For the works for the Charing Cross low-level station there were eight ringwalls under one of the large buildings above, between tunnels of 10.0, 8.25, 7.5, 6.25, 4.5 and 3.85 [9]. In addition, there were a total of 24 junctions under the building for cross passages, ventilation headings and overbridges.

In the design of these ringwalls and the junctions particular attention must be paid to the method of excavation and lining to keep the settlement of the buildings above to a minimum. In many cases it is possible to design the ringwall such that the larger tunnel encompasses the smaller tunnel, thus allowing the lining for the larger tunnel to be erected against the ground and the ringwall between the inside of the larger tunnel and the outside of the smaller one. Figure 19.7 shows step plate junctions with tunnels increasing in internal diameter in small steps. The ringwalls between these tunnels therefore required excavation behind both tunnel linings. Temporary support to the ground will be needed with steel or timber headboards installed and concreted in.

Ringwalls, headwalls at the end of a tunnel and junctions between tunnels at right angles are cast in mass concrete. Reinforced concrete may be used for headwalls. As there are difficulties in casting these walls there are potential paths for the ingress of water. If there is ingress of water at these locations there may be great difficulty in sealing the paths.

Under London there are two aquifers, one in the superficial deposits and a deep aquifer, sealed from the upper aquifer by a layer of clay, the London Clay or the clays in the Woolwich and Reading beds. Over two hundred years there has been extraction of water from the deeper aquifer which has depressed the water levels. In the last thirty years this extraction has been considerably reduced and the water levels have started to increase at rates of up to 1 or 1.5 m a year [11]. If these water levels are allowed to continue to rise for the next twenty or thirty years the levels would be above many of the tunnels constructed in the dry. If this occurs it will be the junctions between tunnels which may be the main areas of concern on account of the difficulty in sealing the ingress of water.

The rising of groundwater is a potential problem in many cities around the world. Where it is a potential problem in the future these junctions should be cast with waterbars or other seals to extend the water paths.

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Chapter 20

Multi-bore station tunnels

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Contents

Demands on handling traffic and applying up-to-date mechanized tunnelling methods often lead to the construction of large-diameter tunnels enclosing tracks and platforms. Generally, large underground areas can be built only in hard or strengthened soils or by special construction methods. These conditions are rare and are very costly. Therefore since the beginning of deep-level metro construction there has been a demand for constructing stations consisting of two or more smaller-diameter tunnels depending mainly on tunnelling methods applicable to moderate or poor ground or on the integration of the construction with adjacent running tunnels.

In this chapter different types of metro stations consisting of multi-bore tunnels constructed in some cities are described as well as their development.

Selecting the best method for the construction of a metro station is a very complex task. Tunnelling (especially the construction of a metro station) changes the state of the surrounding soil in a finite region as described by Greschik *et al.* [1] (see also Chapter 13 and 14). The construction process results in a change:

1. In the initial conditions of equilibrium;
2. In the initial kinematic conditions, (i.e. the state of rest); and
3. In the physical properties of the soil (e.g. dewatering, watering, grouting, etc.).

Depending on the construction method applied to soil conditions (and on many other factors), the field of displacements may vary and may be affected by the work.

In urban areas it is very important to know the permissible settlements on the surface for avoiding damage to the buildings situated above the tunnelling area. The type of station selected should be optimal, depending on a combination of:

1. Soil conditions;
2. Structure; and
3. Construction method.

20.1 Station with one large central tunnel enclosing tracks and platform

These types of station are generally built in hard or strengthened soil, their cross section consisting of one large arch. In some cases there may be small adjoining tunnels for transmitting abutment pressure distributed to the surrounding soil to avoid or minimize lateral displacements. Examples of these stations are:

1. *Étoile Station* (Paris, RER Line). Constructed in the 1960s in limestone, marl and clay layers, this has a width of 21 m and a height (above the rail) of 7 m. The two tracks have been built in the centre of the arch inside the platforms. The arch was constructed of precast concrete blocks.
2. *Auber Station* (Paris, RER Line). This was constructed in the 1960s in strengthened ground. The large central arch has a width of 24 m and it is supported by two smaller tunnels of 7.7 m width and 10.3 m height. Soil strengthening was carried out by chemical grouting from three smaller drifts placed inside the tunnels. The two tracks have been built in the centre of the large arch between the side platforms. The inside height of the central tunnel is 18 m. The arch was constructed of precast concrete blocks.

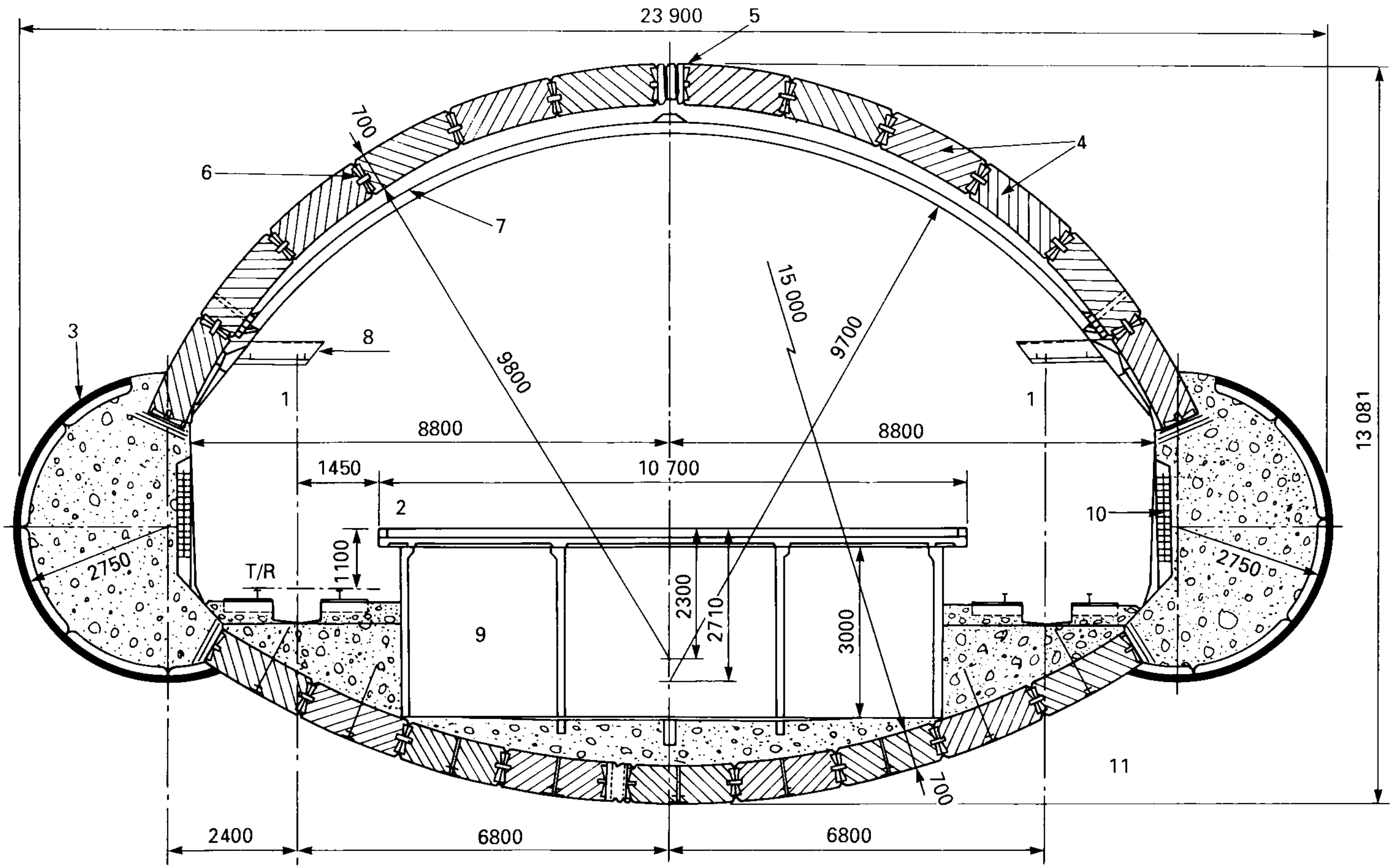

Figure 20.1 Station with one large arch (Leningrad). 1, Track centreline; 2, platform level; 3, precast RC elements (parallel displaced running tunnel); 4, precast RC blocks; 5, disc press (Freyssinet system); 6, PVC sheet; 7, 'armocement' ceiling; 8, cantilever for lighting; 9, service rooms; 10, cables; 11, dimensions (mm)

3. *Metro station in Leningrad* (USSR). In the 1970s a newer type of station in dry, hard clay soil was built. The cross section consists of three tunnels (Figure 20.1). Escalators leading to the surface are placed at one end of the platform.

   First, the two external concrete blocks for supporting the arch were prepared. These were placed inside tunnels of 5.5 m outside diameter constructed by the erector method as parallel displaced running tunnels along the station. The arch consists of 12 precast RC. Blocks and one keystone. The joints of the blocks are hinged by plastic sheets. During construction the arch was pressed against the ground by a disc press or jack (using the Freyssinet system) placed in the keystone.

20.2 Station with tunnels enclosing one track and platform

In moderate or poor soils the dimensions of the space excavated during tunnelling generally have to be restricted or an appropriate construction method (e.g. soil strengthening, shield method, etc.) must be chosen according to the rigidity and physical parameters of the ground. Connections to surface or underground ticket halls are generally made by escalator tunnels from the ends of the platforms or from cross passages. Examples are:

1. Many stations of London's Underground deep-level, tube lines. These were built by the shield method and lined with cast-iron segments (inside diameter 7073 mm).
2. Marienplatz Station (Munich Underground, GFR). This was constructed in clay in the 1960s, and Figure 20.2 shows the cross section [2]. First, the sidewalls were built from a bottom drift, then the upper arch and finally the inverted arch.

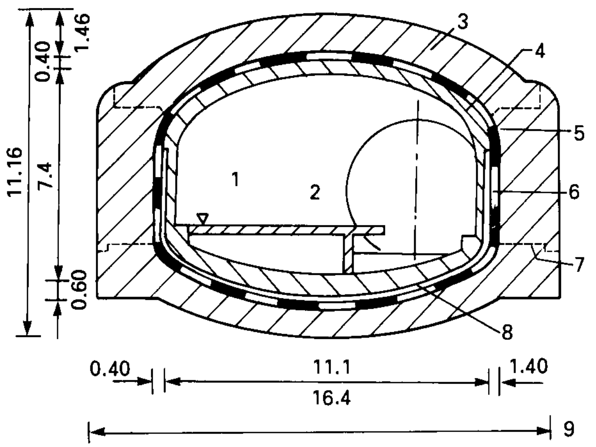

Figure 20.2 Station with single tunnels (Munich). 1, Platform level; 2, track centreline; 3, tunnel construction; 4, cast-*in-situ* RC lining; 5, bituminous waterproofing; 6, 3 cm gunite layer; 7, joint; 8, 3–10 cm protecting concrete layer; 9, dimensions (m)

20.3 Stations with single tunnels connected

With this type of station first, the tunnels are constructed as closed rings and then connected by passages.

20.3.1 Two-bore tunnels

This type of station was built at the end of the 1950s in Hamburg, and the cross section is shown in Figure 20.3. From the station tunnels driven by shield and lined with cast-iron segments, the strengthening of the upper sand layer was carried out for the construction of passages by mining[3]. During this operation the station

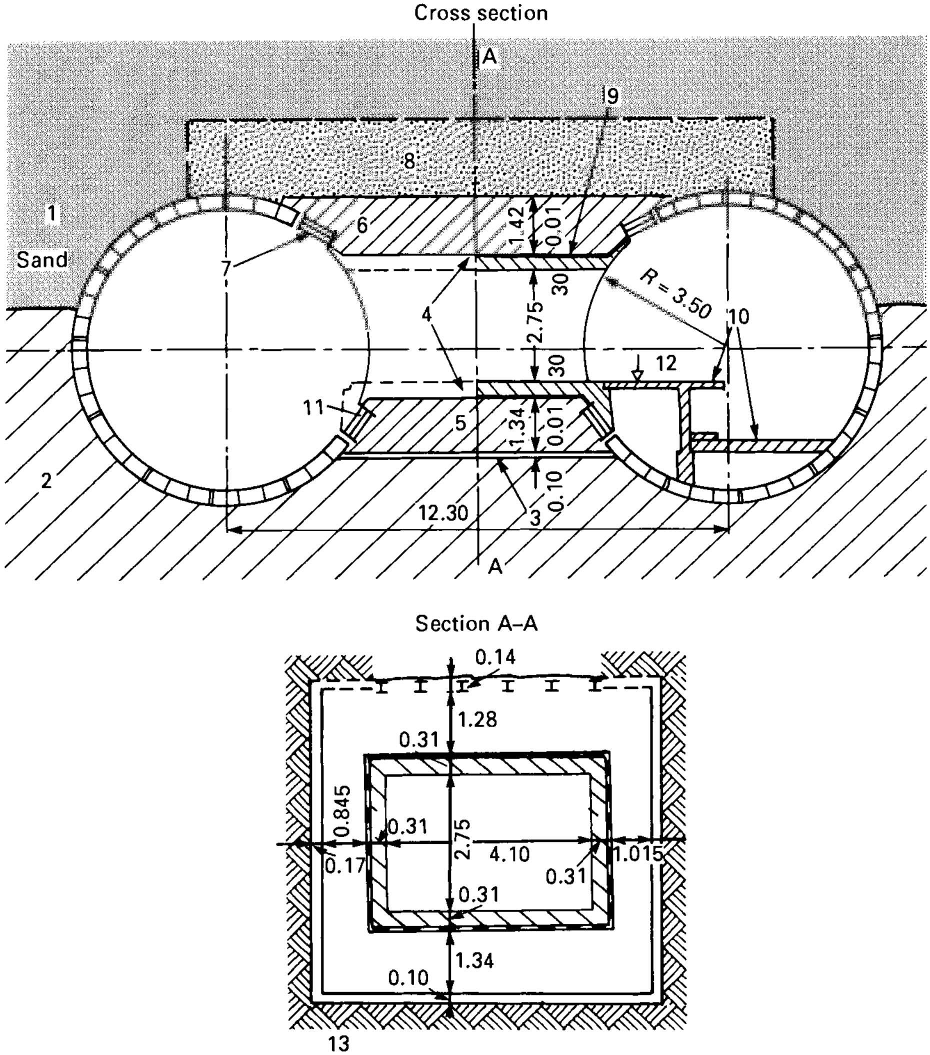

Figure 20.3 Station with two single tunnels connected (Hamburg). 1, Sand; 2, drifted marl; 3, floor slab; 4, inner RC lining; 5, foundation floor; 6, slab; 7, top beam; 8, grouted zone; 9, waterproofing; 10, internal structure; 11, bottom beam; 12, platform level; 13, dimensions (m)

tunnels were temporarily supported by a special steel structure. The depth of cover over the station tunnels is about 7 m.

20.3.2 Three-bore tunnels

This system of station has been frequently used since the 1940s. It consists of two (generally) shield-driven outer tunnels of a large diameter enclosing the tracks and the entire length of the platform. Between the outer tunnels is the central tunnel, the concourse, for movement of passengers. This is either shield driven or constructed by other methods. The central tunnel is connected to the outer ones by passages. Its length is generally shorter and at one end it leads from the escalator tunnel to the surface. At the other end there are either elevators or a second escalator tunnel and service rooms are placed in its extension (e.g. electrical substations, etc). Beyond the central tunnel the outer tunnels run as individual ones. Examples are as follows.

Tunnels with fully or partly cast-iron segments

1. *Keleti pályaudvar Station* (Budapest, East–West Underground Line). The three tunnels were built by the shield method in the 1950s and 1960s, in silty clay ground, using compressed air. The tunnels are of equal diameter (Figure 20.4). The lining consists of bolted cast-iron segments with special arching elements for the passages. During construction of the central tunnel the outer tunnels were temporarily supported by vertical struts to enable material to be transported through them. While the four passages at each side were being built, horizontal tie rods were laid in the central tunnel to prevent deformation. The length of the platform is 120 m for eight-car trains.

   Similar stations were built in Moscow, Leningrad and Kiev and in Prague.
2. *Station in Toronto.* For passenger interchange a shorter and smaller central tunnel was built between the outer station tunnels[4]. The cross section of the station is shown in Figure 20.5.
3. *Karlsplatz Station* (Vienna Underground). This was built at the end of the 1960s. The outer tunnels were shield driven and lined with bolted cast steel segments with a platform length of 115 m (Figure 20.6)[5]. The central tunnel is shorter and smaller than the outer ones and was constructed with the New Austrian Tunnelling Method (NATM).

Tunnels with precast RC elements

Station systems with three-bore tunnels were built in Kiev and Prague in the 1970s–1980s by using cheaper precast RC elements.

By 1990 the Prague Underground (under construction since 1967) will have 19 deep-level stations[6]. These are in schiste and slate layers with an overburden of 23–40 m and the length of platforms is 100 m (Figure 20.7). One tunnel consists of 10 precast bolted RC elements, the largest weighing 2040 kg. Before erection, lining segments are coated on the outside with epoxy mortar. Excavation was carried out mainly by blasting techniques.

The segments were placed by an erector mounted in the constructed tunnel. After the three single tunnels were completed the openings for the passages were temporarily filled with precast RC elements (see section A-A, Figure 20.7). The horizontal steel beams of the openings were placed by a small hydraulic crane into the upper and lower segments. Temporary vertical struts in the outer tunnels were needed during the construction of the shorter central tunnel.

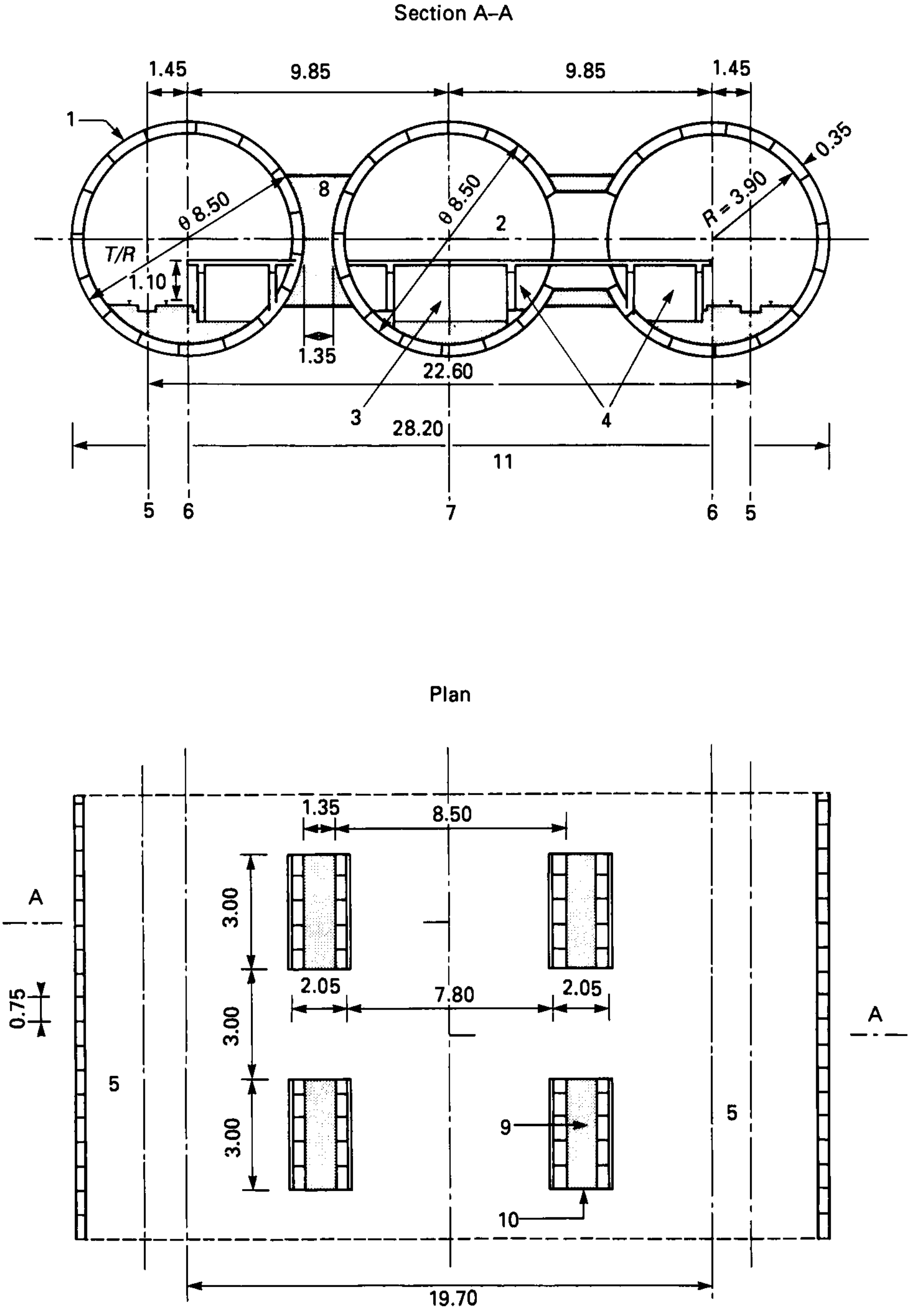

Figure 20.4 Station with three tunnels connected (Budapest). 1, CI segments (bolted); 2, platform level; 3, service rooms; 4, ventilation ducts; 5, track centreline; 6, station tunnel centreline; 7, station centreline; 8, concrete pier; 9, pier; 10, steel plate waterproofing; 11, dimensions (m)

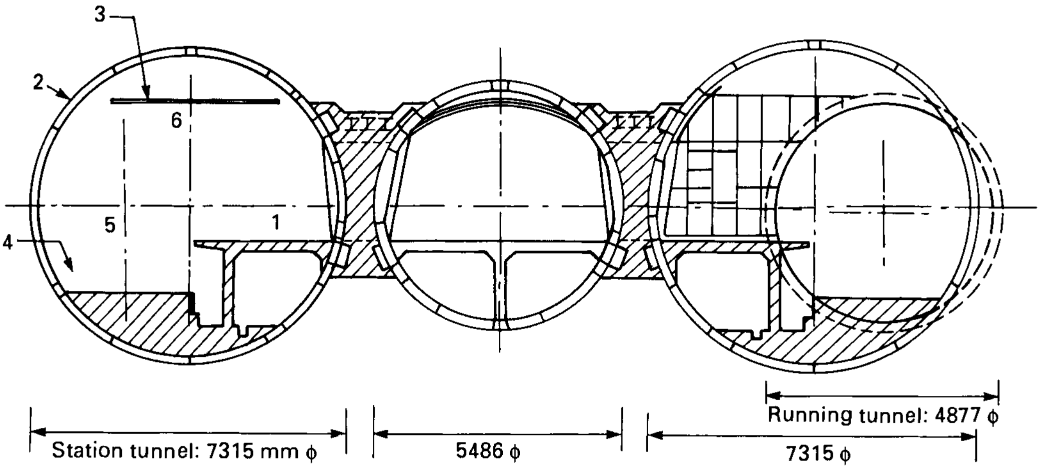

Figure 20.5 Station with three tunnels of different diameters, connected (Toronto). 1, Platform level; 2, CI segments; 3, suspended ceiling; 4, power rail; 5, track centreline; 6, station tunnel centreline

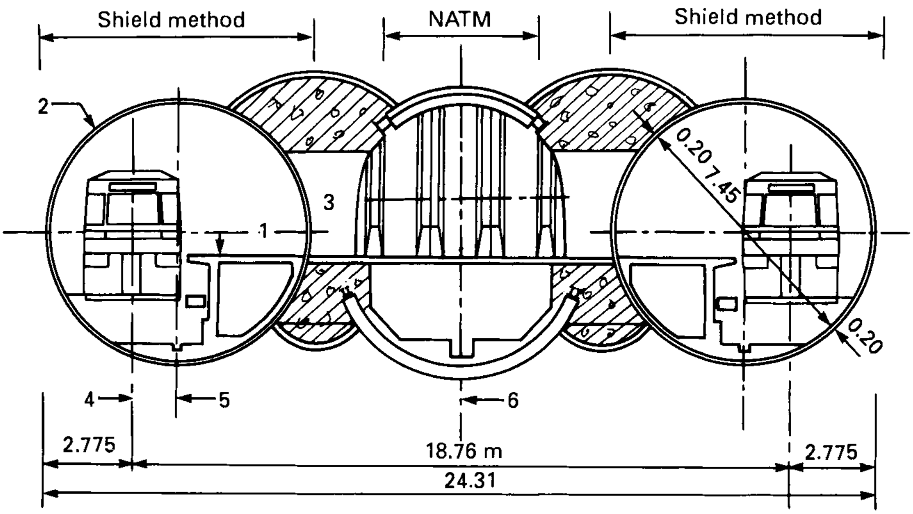

Figure 20.6 Station with three tunnels of different sizes, connected (Vienna). 1, Platform level; 2, cast steel segments; 3, escalators; 4, track centreline; 5, station tunnel centreline; 6, station centreline

20.4 Station with intersecting tunnels

The aims of minimizing the mass of ground being excavated and the platform width for the passenger traffic have led to the shaping of the cross section with a smaller total station width by intersecting the tunnels.

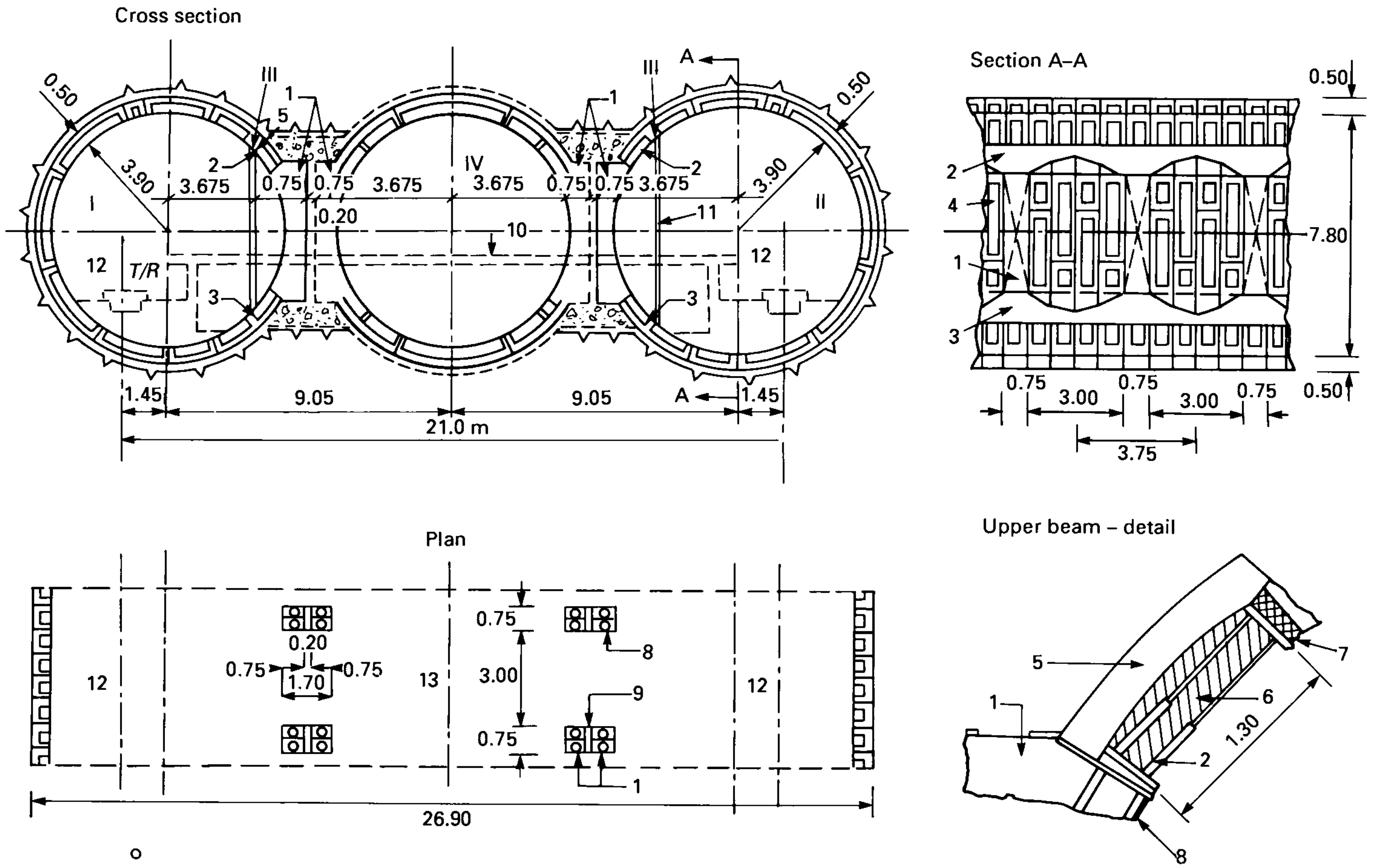

Figure 20.7 Station with three tunnels connected (Prague). 1, RC pillar; 2, welded steel beam (upper); 3, welded steel beam (lower); 4, temporary filling segments; 5, precast RC tunnel lining segment (bolted); 6, concrete filling; 7, steel fibre concrete; 8, steel pipes of 324 mm diameter and wall thickness of 36 mm as rigid reinforcement; 9, grouting; 10, platform level; 11, temporary struts; 12, track centreline; 13, station centreline. Sequence of operations: I, Driving the first outer tunnel; II, driving the second outer tunnel; III, placing the steel beams; IV, driving the central tunnel

20.4.1 Interior supporting system

In good soil conditions and larger tunnel diameters the vertical clearance of the passage enables the placing of the supporting system inside the tunnels. This has the advantage of constructing the complicated supporting system in a previously built space. Examples are:

1. *Metro station in Leningrad.* This was constructed in the 1970s in hard clay ground (Figure 20.8). First, the outer tunnels were built by full-face excavation and the precast RC lining was placed by an erector, temporarily as a closed ring with the bottom element of the vertical support and the upper connection element. The central tunnel has the same length as the outer ones. Before construction the outer tunnels had been temporarily supported and thereafter the supporting system, the central upper arch and finally the central invert arch were constructed.
2. *Metro station in Tokyo.* The cross section of the three-bore station [7] is shown in Figure 20.9. The outer tunnels were shield driven and lined with cast-iron segments.
3. *In Bochum* (GFR) a three-bore tunnel City Railway station was built in the 1970s by the New Austrian Tunnelling Method.

20.4.2 Exterior supporting system

In moderate or poor soil conditions smaller tunnel diameters are preferable, but with these the vertical clearance of the passage does not allow the placing of the supporting system inside the tunnel. Therefore it must be arranged partly outside it.

Two-bore (twin) tunnels

The first application of this system was in the 1960s at the East–West Underground Line in Budapest. A station had to be built along the line where a section of the two running tunnels had previously been constructed [4]. For the central part of the station (about the half of the 120 m platform length) a five-bore tunnel construction had been developed while the end parts of the station – where only the platform was required – were built as the twin tunnels (Figure 20.10). The running tunnel part of the final twin construction had been previously constructed as a closed ring. From it, in short drifts, the bottom and top RC beams with supporting steel pipe columns were built at 4 m crs. Thereafter the twin part of the concrete ring was built by underpinning.

The NATM was applied to the construction of a two-bore (twin) tunnel station in Munich. The twin-tunnel cross section includes both tracks and between them the platform, divided by the row of columns supporting the RC beams at the intersection of the two arches.

Similar to the above was the construction of a twin-tunnel system by the NATM proposed by Wagner and Schulter [8] for the Washington Underground Station (Figure 20.11). It was stated that savings of almost 40% of construction costs (compared with an alternative construction by cut and cover methods) had been achieved.

As shown in Figure 20.11(b), the cross section is constructed by dividing it into smaller segments, driven as drifts with temporary shotcrete linings. A great advantage of this method is that surface settlements can be reduced.

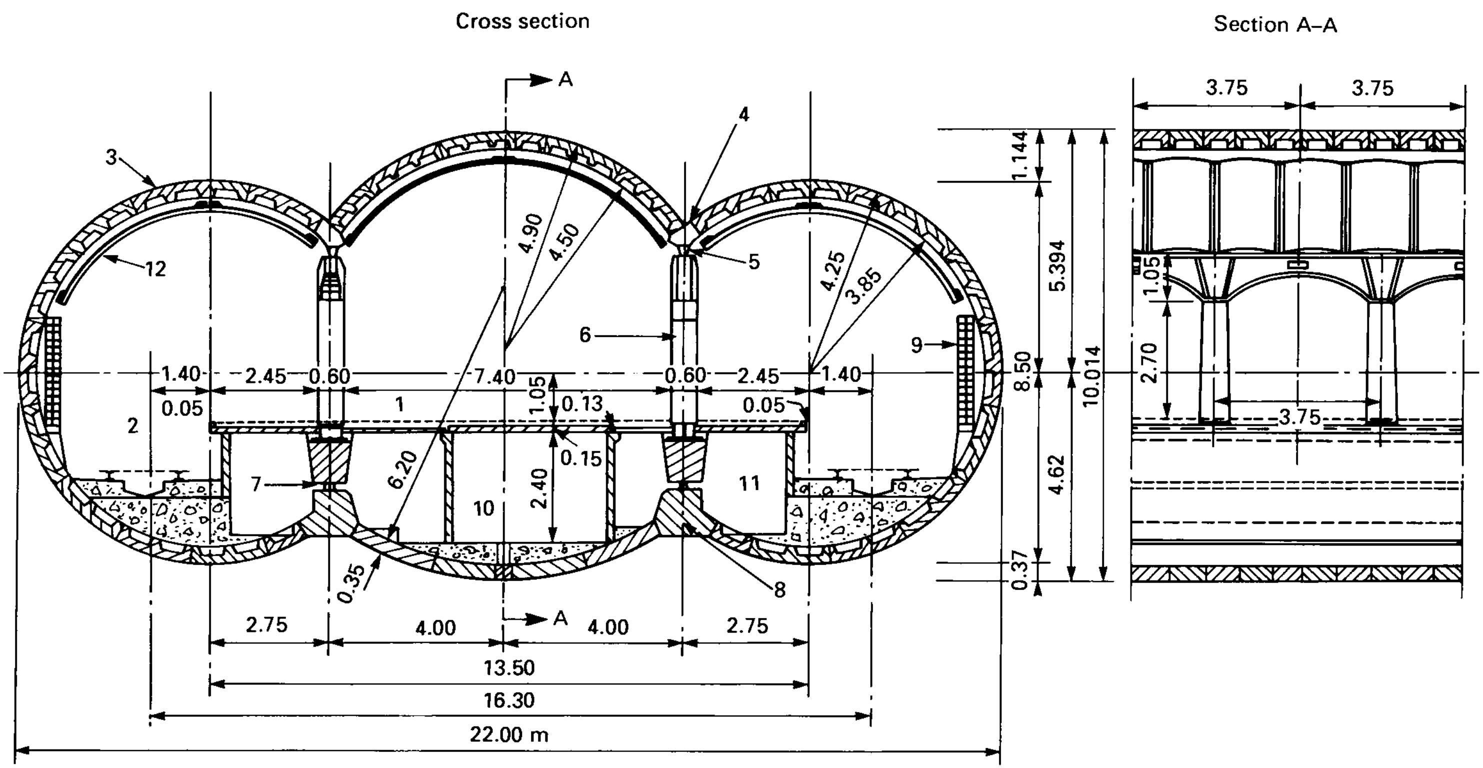

Figure 20.8 Station with three intersecting tunnels of different sizes, interior supporting system (Leningrad). 1, Platform level; 2, track centreline; 3, precast RC segments (bolted); 4, CI segment; 5, steel hinge (top); 6, steel column; 7, steel hinge (bottom); 8, precast RC beam element; 9, cables; 10, service rooms; 11, ventilation duct; 12, 'armocement' suspended ceiling

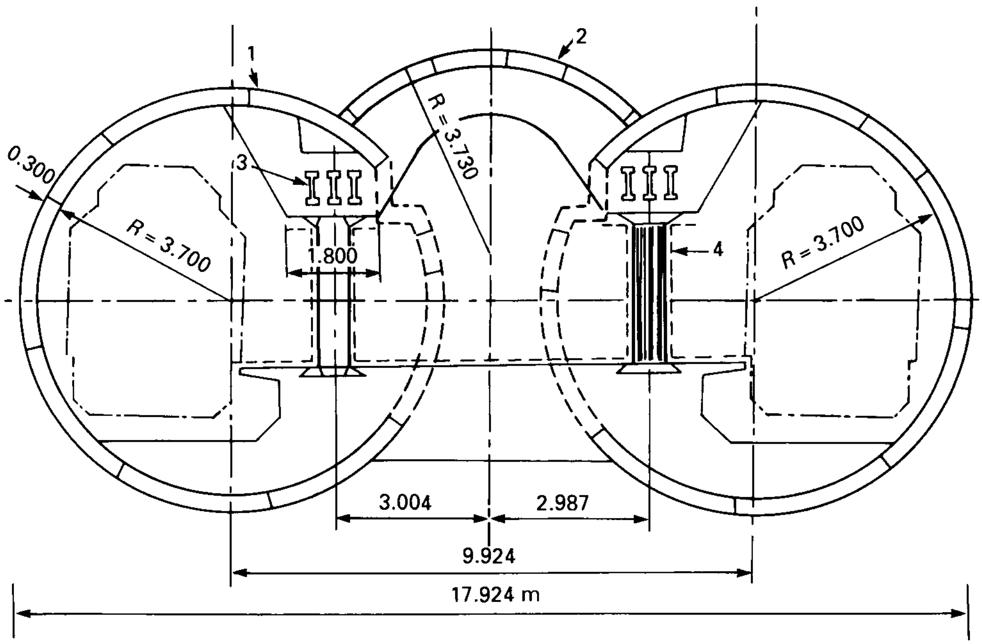

Figure 20.9 Station with three intersecting tunnels of different sizes, interior supporting system (Tokyo). 1, CI segments; 2, CI segments; 3, beam; 4, steel column of 650 mm diameter

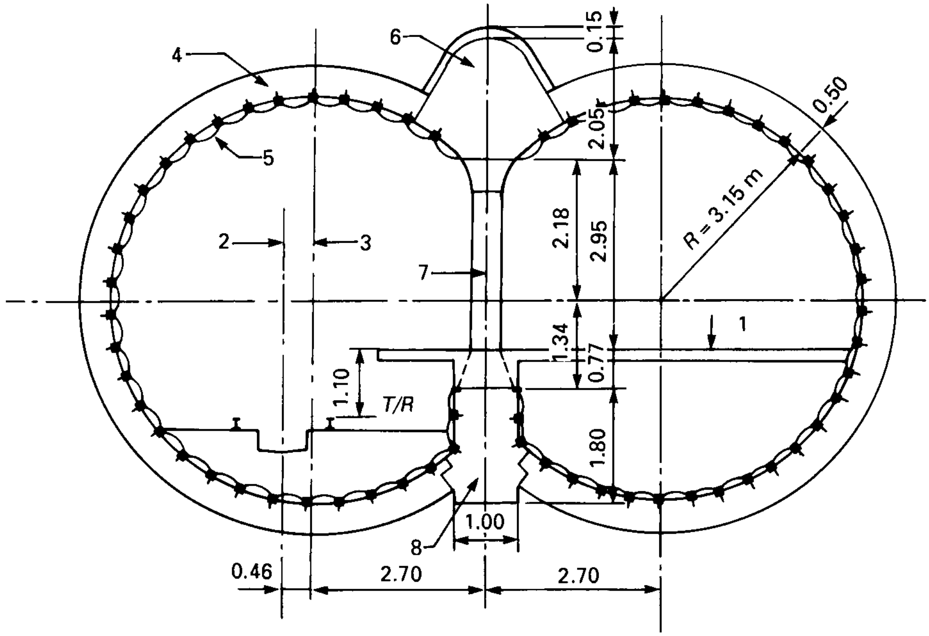

Figure 20.10 Station with twin tunnels. Exterior supporting system (Budapest, part of Astoria Station). 1, Platform level; 2, track centreline; 3, tunnel centreline; 4 concrete tunnel lining; 5, steelplate waterproofing (3 mm thickness), anchored in the lining; 6, RC beam; 7, steel pipe column of 460 mm outer diameter at 4 m; 8, RC beam

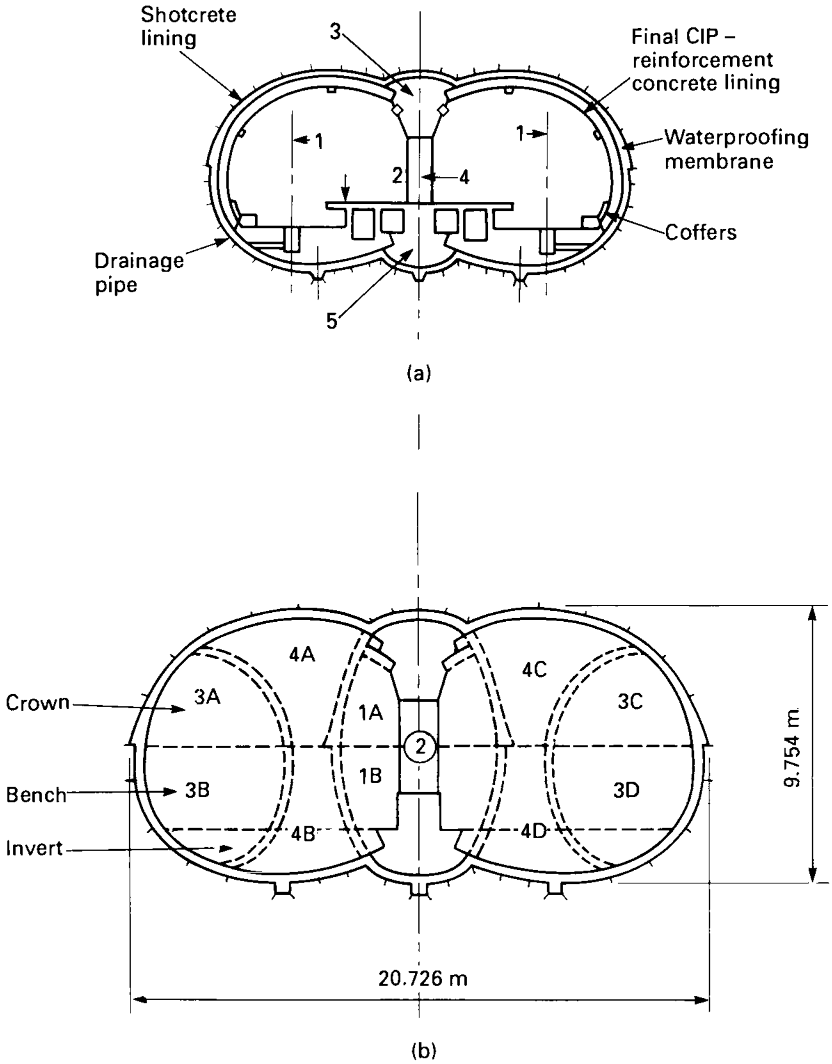

Figure 20.11 Station with twin tunnels, exterior supporting system. Construction by NATM (Washington metro project). (*a*) Cross section of the station. 1, Track centreline; 2, platform level; 3, RC beam (top); 4, column at 10.16 m; 5, RC beam (bottom). (*b*) Construction sequence (NATM)

Three-bore tunnels

The Moszkva Square Station on the East–West Line of Budapest Underground was built at the end of the 1960s for large passenger traffic (Figure 20.12). The central tunnel is shorter than the outer ones including a 120 m platform, and there are four passages at each side (3 m width) between the RC pillars. The external sidewalls and the inner supporting system had first been built in the rather hard and dry clay. Thereafter the top and invert arches were constructed.

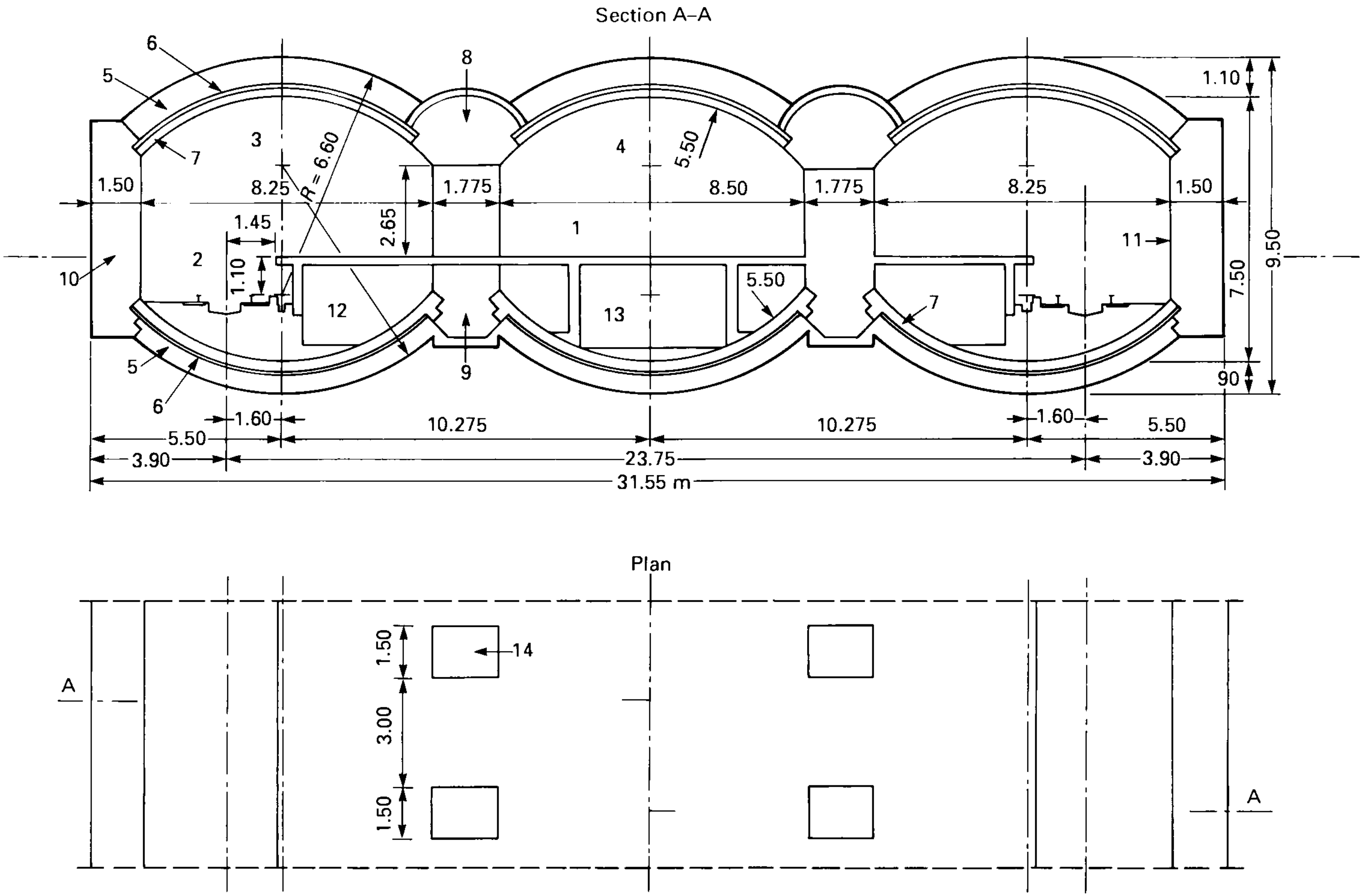

Figure 20.12 Station with three intersecting tunnels, exterior supporting system (Budapest, Moszkva Square station). 1, Platform level; 2, track centreline; 3, tunnel centreline; 4, station centreline; 5, concrete arch; 6, steel plate waterproofing; 7, RC lining (against water pressure); 8, RC beam (top); 9, RC beam (bottom); 10, RC sidewall; 11, steel plate waterproofing (anchored into the sidewall); 12, ventilation duct; 13, service rooms; 14, RC pillar

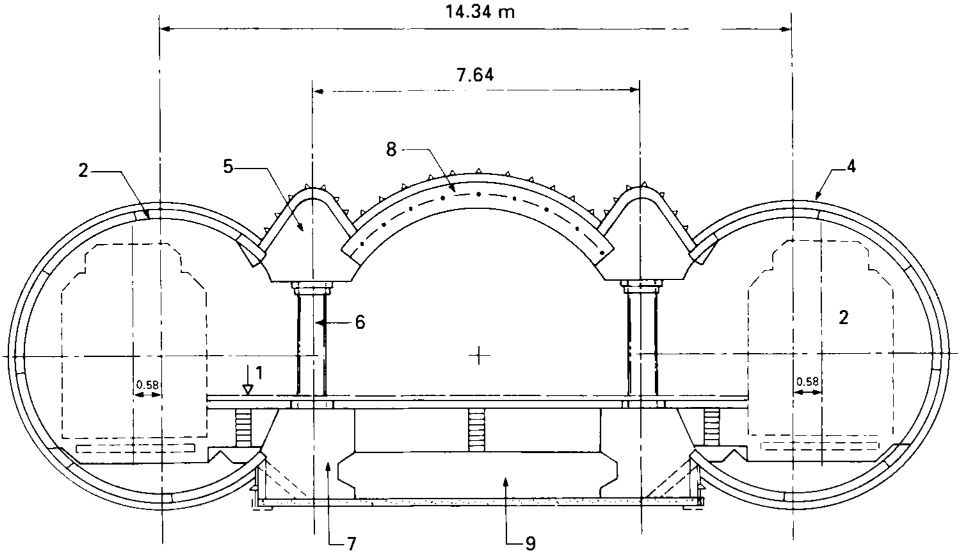

Figure 20.13 Station with three intersecting tunnels of different sizes, exterior supporting system (Munich). 1, Platform level; 2, track centreline; 3, CI segments (bolted); 4, grouted zone; 5, RC beam (top); 6, steel pipe column; 7, RC beam (bottom); 8, concrete arch; 9, RC bottom slab

After the five-bore tunnel system had been developed in Budapest, a three-bore tunnel station was designed for the Munich Underground (Figure 20.13). The outer running tunnels of 6.6 m outside diameter were shield driven as closed rings. Inside, a temporary steel supporting structure was built, enabling the outbreak for constructing the top and bottom RC beams inside drifts and placing the steel pipe columns at 4 m crs. Following these, the central arch and finally the RC bottom slab were constructed. The cross section had to be built along the entire platform length (in Munich, 120 m).

Five-bore tunnels

In the Budapest Underground three types of station have been developed in relation to tunnelling along the entire line. One of these consists of large-diameter tunnels constructed independently of the running tunnels. The inside diameter of 6.3 m enables the shields for constructing the running tunnels to be hauled through the already constructed outer station tunnels.

The second type is of a smaller inside diameter. The outer tunnels are constructed as sections of the running tunnels with cast-iron segments, or with precast RC segments, and outbreaking from these all the other parts of the station have been built.

Figures 20.14 and 20.15 show cross sections of the passenger concourse. The length of this is generally at least 20 m and it is placed in the inner third of the total platform length of 120 m.

In the central tunnel at one end of the hall is the stretching chamber of the escalators, built with greater clearance. There are service rooms at the other end on the interior part of the platform and outside it are the platforms (Figure 20.15). For a second station access the stretching chamber of the escalators enclosed in a

Cross section

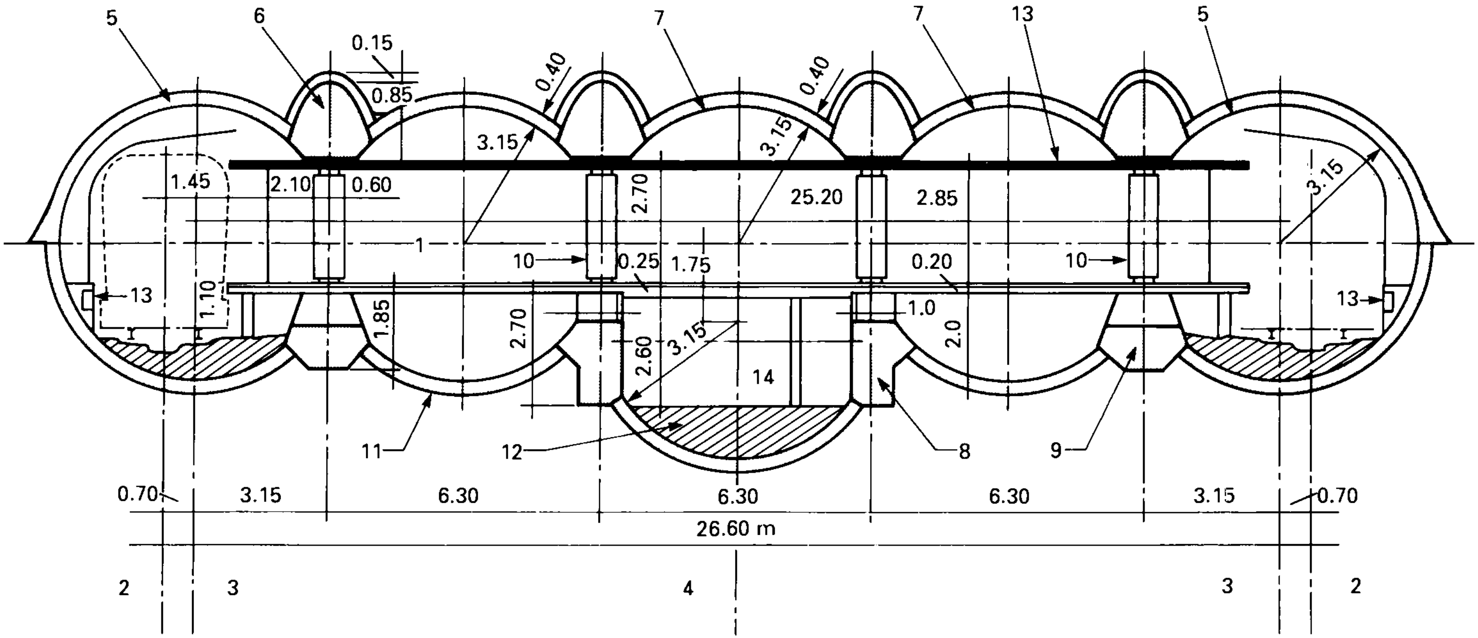

Figure 20.14 Station with five intersecting tunnels of large diameter, exterior supporting system (Budapest). 1, Platform level; 2, track centreline; 3, tunnel centreline; 4, station centreline; 5, concrete arch, steelplate waterproofing on the inside with shotcrete anti-corrosive protection; 6, RC beam (top); 7, concrete inner arch, steelplate waterproofing on the inside with shotcrete anti-corrosive protection; 8, RC beam (bottom); 9, RC beam (bottom); 10, steel pipe column at 4 m; 11, concrete arch, steelplate waterproofing on the inside with shotcrete anti-corrosive protection; 12, filling concrete; 13, suspended ceiling ('luxaflex'); 14, service rooms

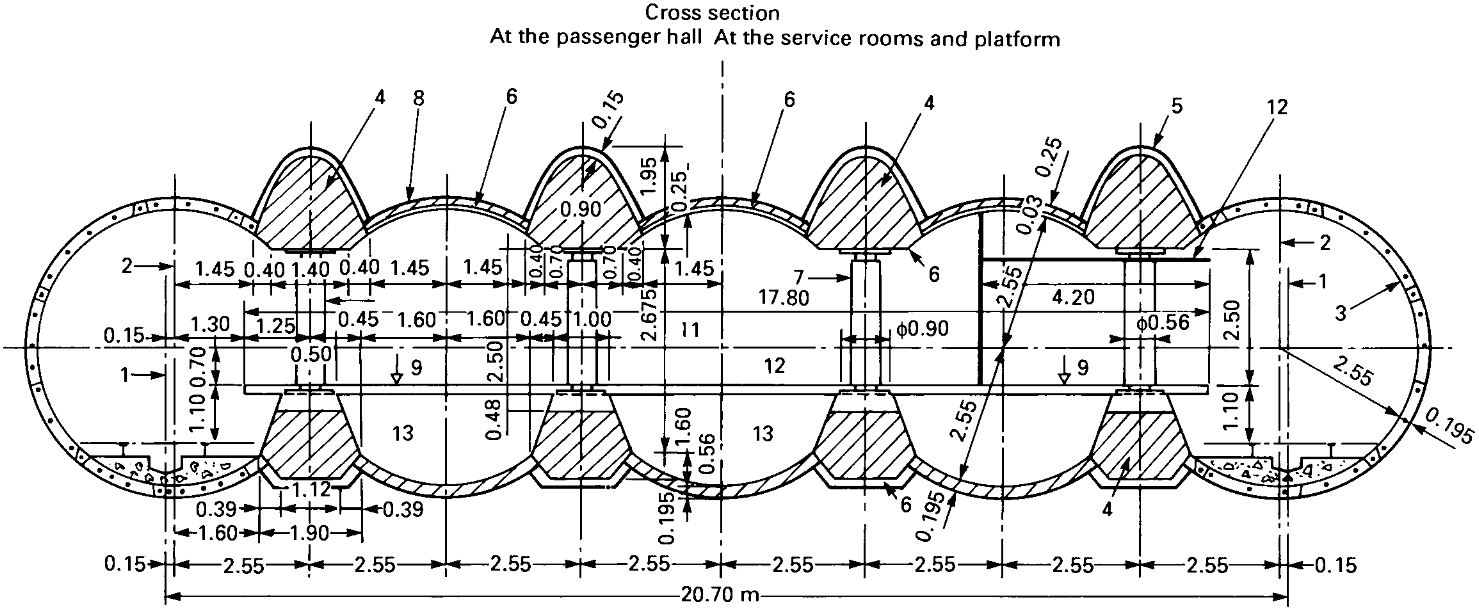

Figure 20.15 Station with five intersecting tunnels of small diameter, exterior supporting system (Budapest). 1, Track centreline; 2, running tunnel centreline; 3, CI segment (bolted); 4, RC beam; 5, concrete lining of the drift; 6, steelplate waterproofing; 7, steel pipe column (φ0.56 m) at 4 m; 8, concrete arch; 9, platform level; 10, passenger hall; 11, service rooms; 12, 'luxaflex' ceiling; 13, ventilation duct

second escalator tunnel is placed there. The flexibility of small-diameter tunnels allows the placing of three or even four escalators.

By enlarging the interior diameter to 5.5 m and the distance of column rows to 6.1 m, and keeping the same shield-driven outer tunnels (as with the smaller-diameter tunnel) four escalators can be placed inside the station.

There is a third variant, Station Nyugati pályaudvar, with a medium diameter. With this type, due to eccentricities the inside radii of the internal arches and the outer tunnels are different. The columns of welded steel structure had to be restrained to the top and bottom beams.

The station's ventilation tunnel crosses beneath the central area of the station length, and is vertically connected to the internal station tunnels. All these stations have been built in moderate clay and silt layers under compressed air at a depth of 18–25 m.

For almost 30 years (1950–1980) of deep-level underground construction in Budapest the following types of stations have been constructed on the East–West and North–South lines:

1. Five three-bore tunnel stations in the 1950s and 1960s (Figures 20.4 and 20.12);
2. Nine five-bore tunnel stations in the 1960s and 1970s, four with large diameters and one with a medium diameter;
3. One six-bore tunnel station in the 1970s.

Three of the three-bore tunnel stations were built in free air and all the others under compressed air (1.0–1.8 bar).

Large-diameter tunnels The cross section of a five-bore tunnel (Figure 20.14) consists principally of five intersecting rings. The outer rings and the internal arches are supported in the vertical line of the intersecting points. The supporting systems are composed of the bottom RC beams cantilevered from the columns in two directions to half the distance between the columns constructed in drifts of limited length, and of the two-support top RC beams. The steel columns are provided with a bottom and top plate for pressure distribution.

The bottom invert of the central tunnel is lowered, compared to those adjacent to it, for providing clearance for service rooms below the platform. The spaces below the platform of the second and fourth tunnels are for ventilation.

This type can also be used as an interchange station. In this case, in an inclined lowering or raising of the central tunnel two escalators moving in two direction can be placed in the intermediate part, or three escalators at the end, and connected by a cross-passage tunnel below the station and leading to the other station (as was carried out in Budapest at Deák Square Station). Other possibilities are provided by multiplying the number of tunnels. Therefore with this system a cross-platform interchange can be built at the same level.

Small-diameter tunnels The cross section of this type of station is shown in Figure 20.15 and the static model for calculation of the statically indeterminate structure is shown in Figure 20.16. The inner forces M, V and H and the elastic reaction r on the elastic embedded structure due to the vertical overburden q and horizontal load e were calculated by computer. The outer shield-driven tunnels consist of seven cast-iron segments, two cast-iron key elements and two special elements (Figures 20.17 and 20.18(a)), enabling easier construction of the outer supporting system. The special elements are made of a welded steelplate frame, the inner part of which

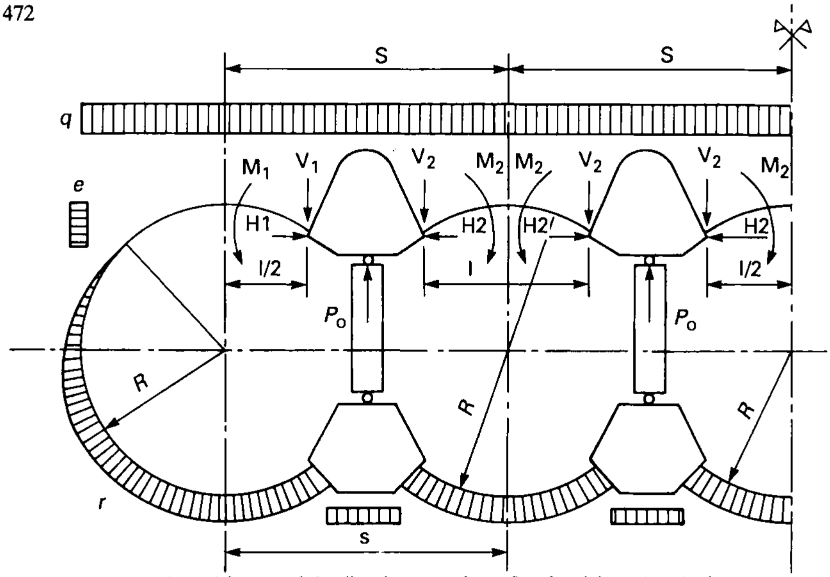

Figure 20.16 Loads and forces of the five-bore station. Q = load (overburden); e = earth pressure at rest; r = ground reaction due to elastic bedding

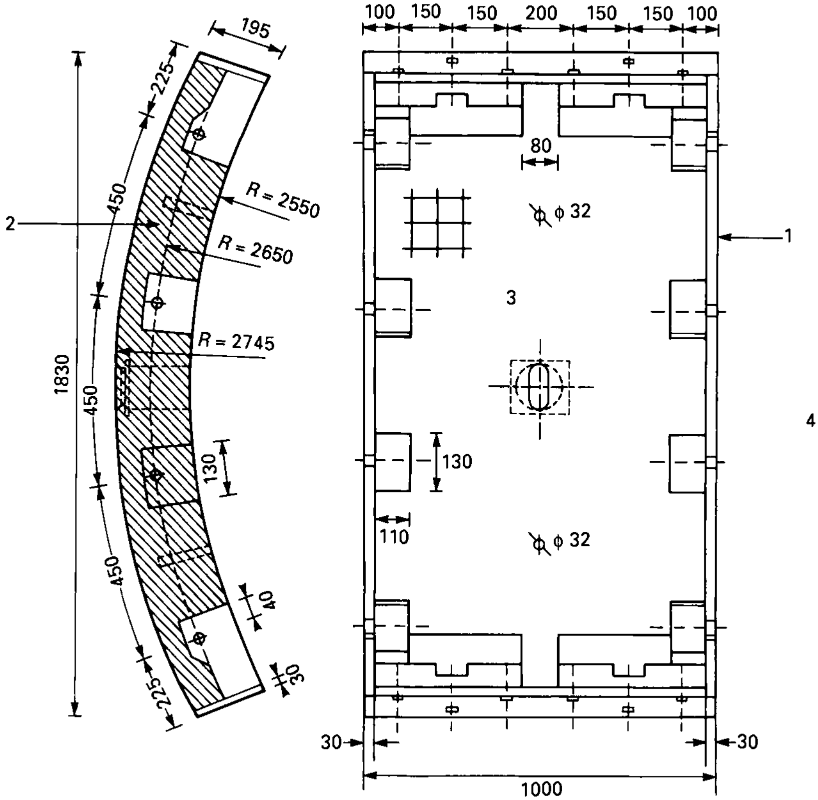

Figure 20.17 Station with five intersecting tunnels of small diameter. Special welded steelplate – RC segment for the construction of the supporting system. 1, Welded steelplate frame; 2, RC filling; 3, 6 mm φ bars at 100 mm crs two-ways; 4, dimensions (mm)

is filled with reinforced concrete[9]. By using these special elements, temporary supports of the running tunnel during construction of the beams and columns could be omitted. During construction outside the column, the ring remained closed and was temporarily propped horizontally against the sheet pile. The frames of the special elements were left in the RC beams.

Before construction of the beams, the RC filling of these elements had been broken out. Mucking and concreting was carried out through the holes (Figure 20.18).

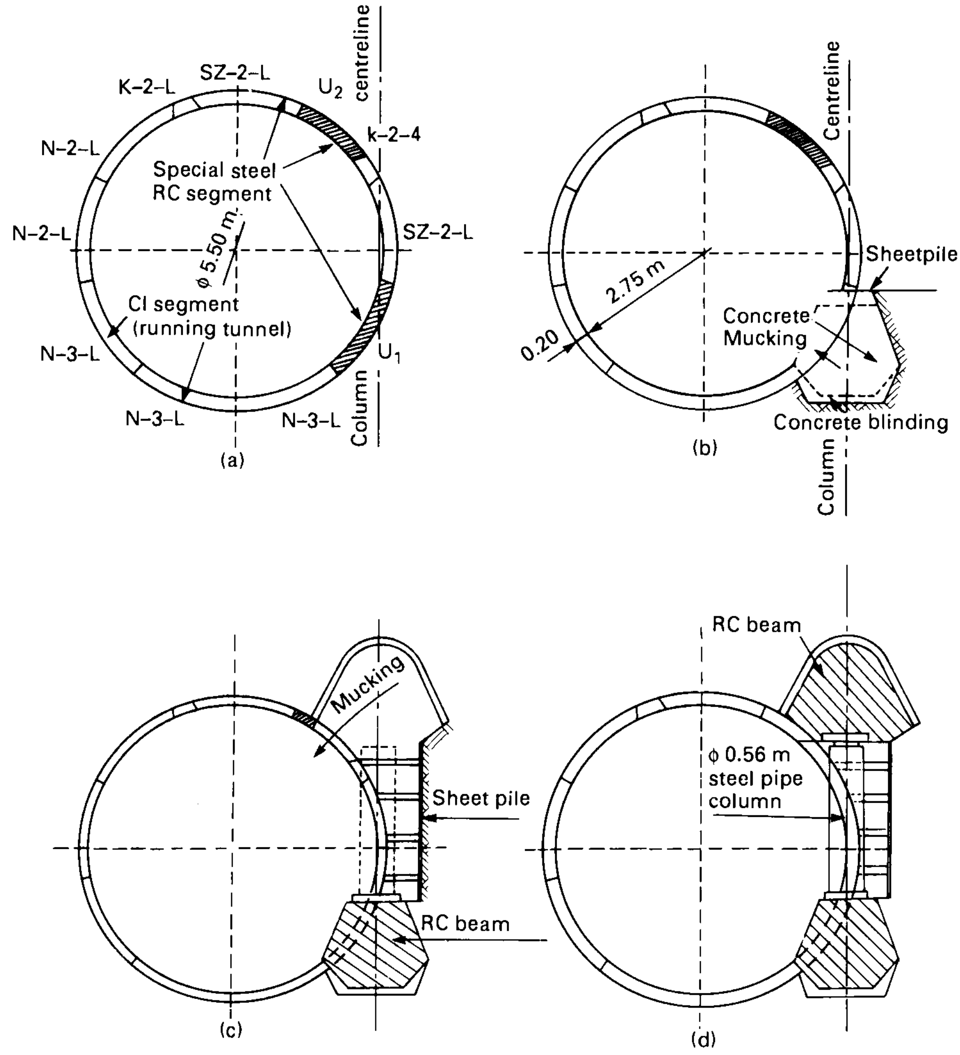

Figure 20.18 Station with five intersecting tunnels of small diameter. Construction sequence of the supporting system. (*a*) Construction of the running tunnel along the station; (*b*) construction of the bottom RC beam along a section of 4 m length; (*c*) between two columns; (*d*) construction of the top RC beam between two columns

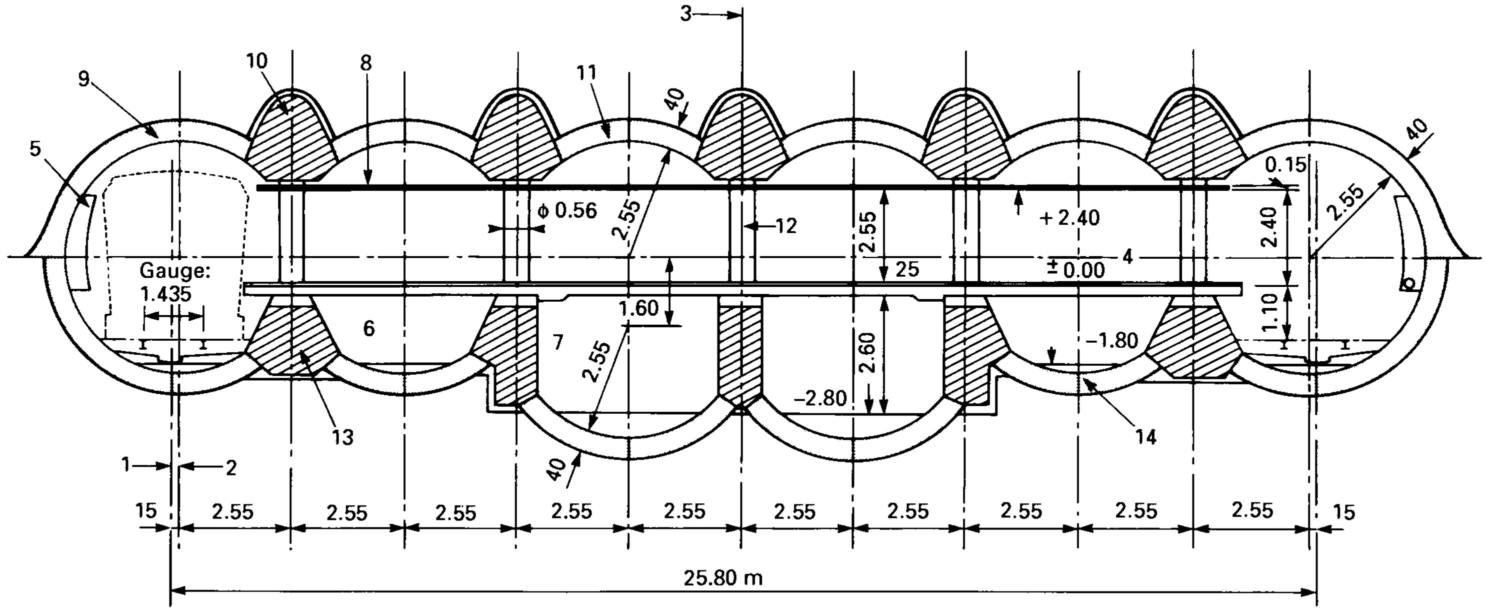

Figure 20.19 Station with six intersecting tunnels of small diameter, exterior supporting system (Kálvin Square, Budapest). 1, Track centreline; 2, tunnel centreline; 3, station centreline; 4, platform level; 5, cables behind wall covering; 6, ventilation duct; 7, service rooms; 8, suspended ceiling; 9, concrete arch, steelplate waterproofing on the inside with shotcrete anti-corrosive protection; 10, RC beam (top); 11, concrete arch, steelplate waterproofing on the inside with shotcrete anti-corrosive protection; 12, steel pipe column, with inside concrete filling; 13, RC beam (bottom); 14, concrete arch, steelplate waterproofing on the inside with shotcrete anti-corrosive protection

For construction of the intermediate upper arches a steel structure in the shape of a shield was used. This slid on rails let into the upper beams and was pressed forward by hydraulic jacks bearing against the vertical formwork of the arch which had previously been concreted.

Six-bore tunnels

A station for interchange and large passenger traffic had to be built in poor soil conditions (sandy clay and clay–sand layers) at Kálvin Square on the North–South Line of the Budapest Underground in the 1970s. As soil strengthening was not economical, small areas had to be excavated. As a further development of the earlier station systems, a six-bore tunnel system was built (Figure 20.19). In running-tunnel construction, the shields excavated the adjacent running tunnels in the direction of this station. The outer tunnels had been constructed in concrete by mining, as had the entire station.

The length of the passenger concourse is 32 m and that of the section comprising the stretching chambers in the two central tunnels for four escalators is 28 m. This means that the length of the six-tunnel cross section is about 60 m. The other parts of the station were constructed as twin tunnels. The inclined escalator tunnel connecting the station with the passenger underpass of an angle of 30° with four escalators was also constructed as a twin tunnel.

For minimizing surface settlements, construction of the station was carried out in three longitudinal sections. A transport drift was led to each section from the outside ventilation tunnel, transverse to the station. Thus simultaneous parallel construction of the supporting system rows in drifts could be avoided. Only each second row was built at the same time, and only in shorter parts of its total length. Therefore surface settlements could be minimized to an allowable extent.

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Chapter 21

Station facilities

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Contents

21.1 Introduction

Other chapters in this book have described the basic configurations of underground stations and the alternative structural forms and methods of construction. This chapter briefly describes the internal planning of the facilities, equipment and architectural finishes that must be provided within a station. These influence the overall dimensions of a station and also the detailed design of the main structure. In many cases they also introduce a requirement for subsidiary structures within the main one.

The effects of these facilities are considered in the context of a typical two-level, cut and cover station, since this form is more frequently found on new underground railways. However, the implications on other configurations and structural forms are fundamentally similar.

21.2 Passenger handling

The primary function of a station is to provide for the safe and efficient movement of fare-paying passengers between the public thoroughfares at ground level and the trains. The sequence of events in this movement are as follows:

1. Entry
2. Ticket purchase
3. Revenue control
4. Descent to platform
5. Train journey
6. Ascent from platform
7. Revenue control
8. Exit.

Discussion of passenger handling is concerned with events 2, 3, 4, 6 and 7.

21.2.1 Ticket purchase and revenue control

Facilities for ticket purchase and the revenue-control barriers are normally provided in a passenger concourse above the station platforms (Figure 21.1). These and the space required to accommodate them are very dependent on the type of fare structure and the ticketing system adopted.

The simplest fare structure (and that requiring the least facilities) is the so-called 'flat-fare' system, i.e. all passengers pay the same fare, irrespective of the length of journey. The equipment and facilities comprise a simple entry barrier operated by a coin or token. Tokens are dispensed by machines or from a booking office which also serves as an information counter. The exit barrier comprises turnstiles free-wheeling in the exit direction but preventing illegal entry. However, a flat-fare system inhibits short-distance travellers, who effectively subsidize the longer-distance ones, and it will seldom provide the optimum revenue for a new underground railway.

Most railways, both old and new, adopt a fare structure where the amount paid bears some relationship to the distance travelled. A fully distance-related fare structure is seldom used since fare collection becomes too complicated, and most urban railways limit the number of different levels of fare to ten or less, each level of fare representing a distance band.

Figure 21.1 Ticket machines, booking/enquiry office, entrance gates

A zoned fare system is adopted on many urban rail networks. Under this, one fare is charged for all travel within any one zone, with a higher fare being incurred for crossing a zone boundary.

Both distance-related and zonal fare systems necessitate the issue of a ticket, which identifies the station or zone of origin, and a means of checking the ticket at the destination to confirm that the correct fare has been paid. Tickets can be issued manually at a booking office or by machine. Most systems now use machines to reduce labour costs, but even when these are used, at least one booking-office counter is provided as an information point and to give change for operating the ticket machines. The number of machines, and therefore the space required, depends on the passenger throughput of the station, the level of complexity of the fare structure and on the type of machine. A machine dispensing a single ticket value has a higher rate of issue than one where the user has to make a selection, and one or more machines will be required for each value. If many different fare values are required there may not be any saving if multi-fare machines are used. However, the number of machines can be significantly reduced if the fare structure encourages the use of season, multi-journey or stored-value tickets.

On entry or exit from the paid area, tickets can be inspected either manually or by the use of automatic ticket barriers. Manual ticket checks require less space in a station and, where labour costs are low, may well provide the optimum solution. However, new railway systems and many long-established urban networks are installing automatic gates operated by magnetically coded tickets. These automatic fare collection (AFC) systems are preferred because of lower labour costs and the virtual elimination of fraud. They provide excellent statistical data on travel patterns and a total audit of revenue collection. It is difficult to envisage that a manually controlled ticket barrier would provide the optimum solution for any new

Figure 21.2 Exit gates, excess fare office, top of escalators

railway for the whole of its useful life, and so all new stations should be able to accommodate the greater space requirements of AFC.

The number of ticket gates required is also dependent on passenger throughput. Typicaly, a single gate is 2 m long × 300 mm wide with a 500 mm wide gangway and will handle 30–35 passengers per minute. The number of entry gates should be sufficient to handle the normal peak flow but not provide more capacity than is available on the stairs and escalators leading to the platforms, since congestion should be especially avoided in the paid area of a station for reasons of safety. Also for safety reasons, exit gate capacity should exceed the capacity of the escalators and stairs from platform level. Spare gates should be provided to allow for equipment malfunction, but total numbers can sometimes be reduced by reversing gates between the peak hours, where tidal flow conditions prevail. The number of gates or other exits from the paid area must also provide sufficient capacity to cater for emergency conditions. Within sight of each group of exit gates it is necessary to provide an office for the payment of excess fares (Figure 21.2).

All the ticket-issuing machines and gates of an AFC system must be connected to a central processor, and so cabling is required to each machine. In theory, ducts could be provided in the concrete structure but this would restrict layout flexibility, which will undoubtedly be needed over the life of the station, to cater for changing travel patterns and new technology. Thus, in practice, it is necessary to provide a non-structural screed on the concourse, of about 100 mm thickness, within which cable ducts can be provided initially and rerouted in the future. This screed is additional to any floor finish.

21.2.2 Escalators and stairs

Underground railways make extensive use of escalators for a combination of reasons. They are provided for passenger comfort and convenience, particularly for upward movements, and also for reasons of capacity. An escalator takes up more space than a stair of equivalent theoretical one-way capacity, but in practice it is impossible to ensure one-way travel on a stair since passengers will not obey the signs, and a single passenger going in the wrong direction considerably reduces the capacity. Escalators, by controlling the direction of passenger movements at selected points, enable a much greater degree of movement control in the station as a whole, so that, with careful planning, points of conflict between inward and outward flows can be largely avoided.

In a cut and cover station with an island platform the escalators and stairs are usually located between two rows of columns in the middle of the island platform. Where escalators are located in pairs, the clear space between the two rows of columns should be not less than 4.2 m. This allows 2.2 m between the centrelines of the two escalators and a minimum of 1 m from the escalator centreline to the column face. For design purposes, additional allowance must be made for construction tolerances. Below the working level of the escalator at least 1.5 m of space is required on each side of the escalator centreline for maintenance, particularly at each end of the escalator truss. The motor driving the escalator is mounted at the top end of the escalator, either under the truss itself or forward of the top end of the truss. In either case, ancillary supporting structures will be required. At the bottom end of the escalator the maintenance pit may have to be recessed into the main structural slab, and this will introduce additional drainage requirements.

The escalators and stairs also influence the longitudinal spacing of the station columns since they require large holes in both the concourse floor and the platform structure.

21.2.3 Platform structure

Unlike many railways in the world, all underground railways have high-level platforms which are designed to be at, or slightly below, the lowest level of the car floor, to facilitate rapid embarkation of passengers. The platform is therefore an ancillary structure within the main station. For a typical island platform the platform slab will be supported on four longitudinal walls, two along the lines of columns and two inset 400–500 mm from the platform edge so that this edge is cantilevered. The space below the platform therefore consists of three long rectangular ducts with the central duct being partially obstructed by the lower end of any escalators.

21.2.4 Headroom in passenger areas

The headroom in public areas, below any suspended ceiling or ventilation ducting, is normally about 3 m. This provides sufficient clearance in the large expanses of space at both concourse and platform levels to avoid a sense of claustrophobia. It also enables signs to be hung below the ceiling level at sufficient height to be visible from a reasonable distance.

21.3 Environmental control

21.3.1 Removal of excess heat

Much heat is generated within an underground station. Trains give off heat, particularly under braking, as do the passengers, both on the trains and in the station. There are also many items of equipment, from lighting to escalators, which generate heat. In order to maintain an acceptable environment within the stations, excess heat must be removed. In temperate climates simple ventilation is normally sufficient to maintain acceptable levels of temperature and humidity within the stations, but in hot and humid ones consideration must be given to the provision of air conditioning. (This subject is covered more fully in Chapter 22.)

21.3.2 Ducting

The requirements for ventilation ducting in the public areas of an underground station are substantial, due to the large volume of the space at concourse and platform levels and because of the length of the station and therefore of the ducts. At concourse level supply air ducts will be required and extract ducts may also be needed. Ducts will be up to 800 mm diameter and a space of 700–900 mm is normally provided at ceiling level to accommodate these and any other services.

Much of the heat at platform level is generated below the floor of the train, and it is common practice to use the ducts below the platform adjacent to the track to extract this. However, supply ducts, and sometimes additional extract ducts, are also required and these are provided at high level; again a space of about 900 mm is needed. Traditionally, overhead ventilation ducts are made of galvanized iron sheets which have a limited life compared to that of the railway. Replacement of the high-level ducts is difficult and dangerous to undertake in close proximity to the track, and this would normally have to be carried out at night, when trains are not operating. This is a particular problem where overhead traction supply is adopted. To avoid the problems of replacement, the overhead ducts at platform level are normally cast in concrete. It is often found convenient to provide a grid of cast-in fixings in the structural soffits for the suspension of ducts and cable trays.

21.3.3 Ventilation plant rooms

Ventilation plant rooms will be required at both ends of a typical cut and cover station. These will normally be provided at concourse level and will require up to 300 m^2 of space to house air-handling units, supply and extract fans, control panels and, where air conditioning is required, chillers. For a large, air-conditioned station the air-handling units may be up to 6 m × 4 m × 2.5 m high, and space is required around each unit for maintenance. While large, most items of plant are not especially heavy, and a general floor loading of 10 kN/m^2 is normally sufficient. However, chiller units are heavy, and local strengthening may be required. Small plinths will be needed to support the equipment, some of which has particular requirements for drainage which must be incorporated into the floor screed of the plant room.

While a large proportion of the air in a station may be recirculated, a proportion of fresh air is always required, and twin ventilation shafts are required at each plant room to provide fresh air and to dispose of warm contaminated air such as that extracted from the underplatform ducts.

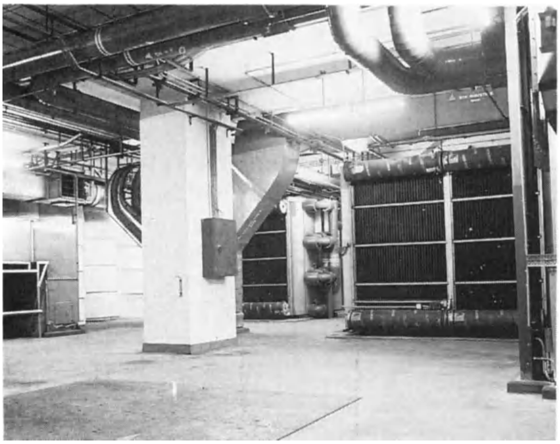

Figure 21.3 Ventilation plant room

Typically, a ventilation plant room acts as a plenum chamber within which fresh and recirculating air are mixed and from which air is supplied to the station (Figure 21.3). There will inevitably be a differential between the air pressures inside and outside this plenum, so the walls must be designed to withstand the resultant forces. Large-sized ducts pass through the walls, and often the floor, of the plenum and, where they do, dampers are provided to control the air flow. The design of the main structure and of the ancillary walls must accommodate these openings.

21.3.4 Smoke extraction

Smoke constitutes the greatest danger in the event of a fire in an underground railway, and the environmental control system must be designed to control the spread of smoke and to extract it from the station. The objective of the smoke-extract system is to create an air flow that removes smoke from the area of a fire and maintains a flow of smoke-free air in the escape routes from the station. This is usually achieved by turning off all the ventilation at concourse level and running all the systems at the lowest (i.e. the platform) level in extract mode. This results in fresh air being drawn into the concourse through all the station entrances and from the concourse to the platform down the escalators and stairs, thus keeping the escape routes fed with fresh air.

The smoke drawn into these smoke-extract ducts will be hot as well as contaminated. To ensure that the extract ducts do not become the means by which the fire is spread to other parts of the station and to maintain the security of the extract systems, some sections of the ductwork and the extract fans will need to be constructed with a suitable fire-resistance period (FRP).

3.5 Tunnel ventilation and draught relief

A train travelling in a single-track tunnel pushes a considerable body of air in front of it. This can create an unacceptable draught on the platforms and also interfere with the controlled air-flow patterns in the station. It is often necessary, therefore, to provide draught relief at each end of a station. Sometimes this is achieved by interconnecting the approach and departure tracks at the end of the station.

However, it may also be necessary to provide large draught-relief shafts at each end of a station to enable air to be vented from the tunnels directly to ground level. Such shafts, which may be up to 25 m^2 in area, may also be required for smoke-extract purposes in the event of a fire in a tunnel, and they may be combined with the extract ventilation shaft of the station.

21.4 Other station plant

The ventilation plant is normally the largest occupier of space in a station, but many other items of equipment, necessary for the safety and operation of the railway, must also be housed in it.

21.4.1 Power supplies

Station lighting and many of the smaller pieces of equipment in a station, such as the AFC equipment, operate at normal domestic voltages. It would be possible to arrange for electrical supply at these voltages at each station through a city's normal supply system. However, for reasons of economy and security, an urban railway system will normally take all its electricity supplies at high voltage from a few selected locations and will provide its own distribution system for supplying electricity to each station. Because of the distances involved, such a distribution system is normally at 11 kV or more, and so transformers are required at each station to step down the voltage to the normal domestic level. For security of supply, two transformers are provided at each station, each capable of supplying a half to three-quarters of the required load. In the event of failure of one supply, non-essential loads are shed and all essential ones are fed from the continuing supply. These transformers and their associated high-voltage switchgear are housed in separate secure rooms at platform level, so that there is easy access to the high-voltage distribution cables located in the tunnels (Figure 21.4).

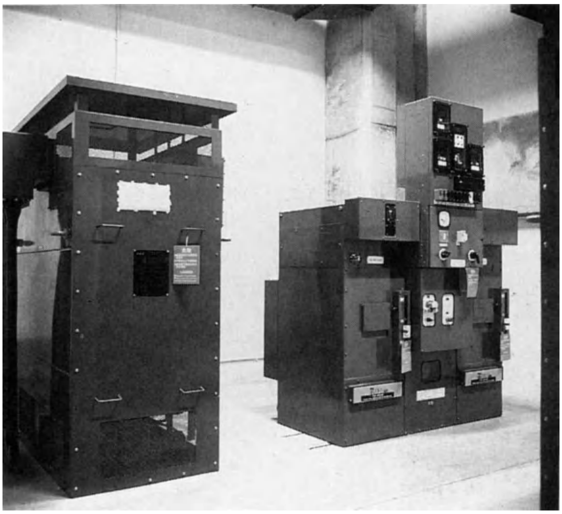

Figure 21.4 High-voltage switchgear room

The output from these station transformers is connected to two MV distribution panels from which electricity is supplied to the various items of station plant. These distribution panels should also be housed in secure rooms within the station (Figure 21.5).

The power supply to the trains is likely to be at 750–1500 V DC. At this voltage, and depending on the level of power demand, it is necessary to provide an in-feed supply to the track at a spacing of as little as 3 km. The power supply system will involve a main high-voltage distribution network at 22–33 kV AC with rectifier transformer substations at each track feeder point. It is often convenient to attach these substations to a passenger station, through they can be entirely separate. The transformers are very heavy items of plant and are therefore normally housed at track level for both structural reasons and for ease of future replacement by rail when required. They may also be oil-filled, which leads to a requirement for sumps to be provided to contain and collect any oil spillage in the event of a failure of a transformer. The rectifiers and switchgear may be located at either track or concourse levels. The switchgear at these high voltages are also large items of equipment, and may require greater headroom than is normally available at the concourse level of an underground station. The required headroom may be achieved by lowering the concourse level, if clearance requirements at platform level permit, or by raising the roof of the station locally. In extreme cases of malfunction, transformers and swtichgear may explode, and it is necessary for the equipment to be secured in blast- and fireproof compartments.

The electrical supply to certain items of plant is vital to the safety of the railway and its passengers. A minimum lighting level is required to enable the railway to be evacuated. Communications must be maintained so that the situation can be controlled. The train signalling system must continue to function for safety reasons. It is necessary, therefore, to provide for back-up electrical supplies in the event of failure of all the normal supply systems. Batteries are normally provided to secure these supplies and, depending on the detailed design of the system, it may be necessary to house them in a separate secure room at each station.

Figure 21.5 Medium-voltage switchgear room

21.4.2 Traction supply and corrosion control

Most underground railway systems use DC current supply. The supply to the trains may be through a third, electrified, rail or through an overhead contact wire. Third-rail systems can be accommodated in smaller tunnels and have less visual impact where tracks are above ground. For these reasons they are often used for urban rail systems. However, such systems are intrinsically less safe for staff and passengers than an overhead collection system. Voltages are normally limited to 750 V compared to 1500 V for overhead electrification, which leads to higher power distribution costs, and there are operational disadvantages. For overhead collection, a clearance of 450–500 mm is required above the rolling stock.

In both overhead and third-rail systems, current return to the substations is through the running rails. All possible steps must be taken to prevent current leaking from the running rails into the structure. Leakage cannot be totally eliminated and can cause corrosion of steel reinforcement. In undertaking the detailed design of station structures the potential for corrosion must be assessed and steps taken to eliminate it. In the extreme, it may be necessary to bond all the reinforcement to prevent corrosion (see also Chapter 6).

21.4.3 Signalling

Equipment associated with the safe operation of the trains is required at each station. This is normally housed at track level for ease of connection to the signalling cables located in the tunnels and occupies a space of about 25 m^2. The equipment is lightweight and therefore has no direct implications on the design of the main station structure. However, its security is paramount to the safety of the railway, and therefore it must always be housed in a secure compartment with a carefully controlled dust-free environment.

21.4.4 Communications

A modern railway system will be provided with a variety of communications systems, including telephone, radio, public address and closed-circuit television (CCTV). All these systems will require equipment to be housed in each station. This equipment is lightweight but again it must be securely housed and requires a space of about 25 m^2.

21.4.5 Lighting

The provision of lighting does not impose any demands on the structure, though space for the fittings (about 200 mm depth) must be allowed in the determination of the overall internal structural height at platform and concourse levels. The design of the lighting must be by an experienced lighting engineer, in association with the architect, to provide a bright but glare-free and uniform level of illumination of public areas. Signs, using pictograms where possible, must be illuminated and coloured to stand out. Emergency-use-only signs must be operated from the control room.

21.4.6 Drainage

There are sources of water in a station and there will also be some leakage of water through the walls of any underground structure. It is necessary, therefore, to provide one or more sumps in each station to which this water can be drained and subsequently pumped up into the city's drainage system. These sumps affect the detailed design of the station structure.

21.4.7 Cables

The main high-voltage power supply cables are normally located in the running tunnels of the underground railway and will pass through the stations at platform level. They are normally positioned adjacent to the track below the cantilevered edge of the platform.

Main signal and communications cables also pass through each station and may be housed in the central under platform duct or on the wall of the station remote from the platform.

Internal electrical cabling within the station is normally carried at high level on cable trays and can be accommodated within the space provided for the ventilation ducts. Power supply to the escalators may be carried in the central under platform duct.

21.4.8 Fire

Fire is a major consideration in the design of underground stations. The design must embody measures to avoid sources of ignition and fuel, to detect a fire at the earliest possible time, to prevent its spread and to extinguish it. The avoidance of ignition is achieved by careful specification and selection of equipment and by good maintenance. Fuel sources are eliminated or minimized by careful choice of materials, especially of architectural finishes, and by maintaining high standards of cleanliness. Detection is by the installation of smoke or heat detectors throughout the public and staff areas and plant rooms.

Spread of fire is contained by compartmentation, wherever practicable. Different types of equipment are housed in separate rooms with FRPs varying from 2 to 4 hours, depending on the degree of risk. In some cases the fire rating is geared to the degree of risk inside the compartment, as in an electrical substation. In other cases, such as signal or communications rooms, the internal risk is small, and the creation of a fire barrier is more concerned with ensuring that an external fire does not affect the equipment in the room.

The staff areas are also fire rated with FRPs from 1 to 2 hours. Major cable ducts, such as the central underplatform duct, must also be broken down into compartments not exceeding about 25 m in length. All rooms in the station must be provided with a fire-protected escape route. In general, the public areas of stations cannot be compartmented.

All compartments with electrical equipment normally have automatic BCF gas extinguishing systems. Others may be protected by sprinklers. Where gas systems are used, a secure fire-rated compartment must be provided for the gas cylinders and control equipment. Automatic systems are not normally adopted in public areas, where manual hose-reel installations are preferred. Hose-reels and hydrants are provided to allow manual firefighting in all areas of a station, including the plant rooms. Many new railways also have a separate fire-secure access from street level to the lowest level of a station for firemen to be able to reach the seat of a fire more easily and quickly without obstructing the escape routes being used for passenger evacuation.

21.4.9 Maintenance

The means and access routes used to deliver and install equipment at the time of initial construction may be unavailable or unsuitable for later maintenance and replacement. The requirements for such maintenance and replacement can have a significant effect on the layout of plant rooms and on the structural provisions, and must be considered and incorporated into the structural design of stations. These are likely to include a large hatch in the concourse floor, with lifting equipment above, in each plant area to enable equipment to be delivered and removed by rail as well as lifting points to aid the installation of heavy items such as escalator trusses. The width of doors and access corridors is also influenced by maintenance requirements.

2.5 Operations

There are a number of operational functions which necessitate the provision of rooms and other facilities within a station.

21.5.1 Control room

A control room is provided in each station of a new underground railway. This should be located as close to a station entrance as practicable while remaining in a position to be able to directly monitor the station concourse. The room is the focal point of all the information systems in the station, and will house such facilities as the fire-monitoring panel, CCTV monitors, microphone and control panel for the public-address system, telephone and radio links to all parts of the station and to the line controller, and also telephone links to the emergency services. The controller should also be able to monitor the status of all the ventilation plant and have the facilities for remote control of the ventilation system, for the remote stopping of each escalator, for releasing all ticket gates to provide quick evacuation of the station in an emergency and for the control of signs indicating emergency exit routes. This room should have a 2 hour fire rating and, where practicable, independent ventilation to ground level so that it can be used by the fire authorities to control firefighting operations when necessary.

21.5.2 Staff facilities

Changing rooms, toilets and rest rooms must be provided for staff. These rooms should have a fire rating of half an hour.

21.5.3 Cleaning

Regular cleaning is essential to the safety of the station, since the accumulation of rubbish is a major contributor to fire in an underground railway. Rooms are required for the storage of cleaning equipment and materials and also for rubbish pending its removal from the station. These rooms should be fire rated and equipped with automatic fire-extinguishing systems; either gas extinguishers or sprinklers may be used.

The cleaning of floors often involves the use of large machines. On some railways a lift is provided to enable these to be moved from one level to another.

Consideration must be given at the design stage to the removal of rubbish from a station. Some railways arrange for all rubbish to be removed by rail. This has the disadvantage that removal can only take place at night and leads to a greater accumulation of rubbish within the station with a correspondingly increased storage requirement and fire risk. It is preferable to provide suitable facilities for the storage of rubbish at ground level, close to a station entrance, to which it can be removed throughout the day and from which it can be collected by the city's cleansing authorities.

21.5.4 Cash handling

An underground railway will collect a large part of its revenue each day in coin. At a busy station the daily coin may weigh more than a tonne, and so its security and transport to a bank becomes a significant matter. For security reasons, it is customary to transport cash receipts by train to a central sorting depot at the end of each day. Special railcars are provided for this purpose, but the transfer of these receipts from the concourse, where they are collected, to the platform will normally require the provision of a lift. Also, secure storage space is required at platform level pending the arrival of the train.

21.5.5 Miscellaneous storage

There are other needs for storage in a station for items such as tickets and maintenance. Typically, each store is restricted to a particular use and needs to be secure. Depending on the material being stored, the room may need to be fire rated and equipped with an automatic fire-extinguishing system.

21.5.6 System maintenance

Most maintenance of system-wide equipment is managed from the main depot of the railway. However, there is usually a need for some ancillary maintenance bases away from the main depots, and these are conveniently located at a station.

21.6 Architectural finishes

Architectural finishes do not have much effect on the structure of a station, but in some cases they may influence the overall dimensions.

26.1 Wall-cladding systems

The external structural walls of an underground station are seldom watertight since it is not economic to make them so. Wherever public or staff areas immediately abut an external wall, it is necessary to install a cladding system, since finishes cannot be applied directly to the station structure. Such cladding systems generally occupy about 200 mm of space, and this must be taken into account when sizing the structure.

21.6.2 Advertising

The wall of a station facing a platform is a significant potential source of advertising revenue to the railway since passengers waiting for a train constitute an excellent captive audience. To obtain this revenue it is necessary to allow sufficient space for the installation of advertising panels. Internally illuminated panels will generate the most revenue, and for these a space of about 300 mm is needed.

Chapter 22

Influence of air-treatment systems

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Contents

22.1 The requirement for ventilation – overall considerations

A constant and secure supply of fresh air is needed in an underground railway system for the following purposes:

1. To meet the physiological needs of passengers and staff;
2. To maintain a safe and pleasant environment, preventing odours from building up and keeping pollutant levels below the appropriate design levels;
3. To keep air temperatures and humidities to safe and acceptable levels;
4. To remove smoke in the event of fire.

Most of the underground railway systems designed and constructed up to about 1960 relied heavily on the trains themselves to produce air velocities that were sufficiently high in all parts of the system to keep temperatures and air quality within reasonable bounds. It is only in recent years that public and operator demands for better control of underground conditions and higher levels of fire safety, together with the advent of high-performance trains liberating large amounts of energy into the system, have brought a more systematic approach to the design of underground railway ventilation systems. Designers have been demanding more and larger underground plant rooms and tunnels in which to accommodate ventilation equipment and associated systems, and better-sized and located air transfer tunnels and relief shafts with which to help control train-induced airflows.

If a decision has been taken to provide platform-edge doors at each station the effect on the ventilation system and the requirements for relief shafts will be considerable. Where doors and walls are fitted the provision for draught relief may be reduced but the requirement for tunnel ventilation may be measured, since almost all the train heat will have to be abstracted through the tunnels and little through the station system. The remainder of this chapter deals with the situation where no such doors are provided, but noting that the types of problems to be overcome are similar in the two radically different station layouts.

It is sound and common practice to treat the stations, the tunnels and the passenger compartments of the trains independently for the purposes of ventilation design. Designers have considerable freedom to select different temperature and air quality criteria for each. To a certain extent, they may also be able to influence the design tactics or the type of engineering solution adopted to meet the station and tunnel air system design criteria. Solutions to the train interior problem may also vary widely, and although not altering the station and tunnel solutions in principle, may affect considerably the ventilation requirements. For example, if the trains have powerful air conditioning more heat will be discharged into the stations and tunnels.

22.2 Train-induced airflows – the 'piston effect'

During normal train operation, very considerable air movement is generated by the movement of the trains themselves. Train-induced airflows may have a very significant effect on the conditions in both tunnels and stations. If differing environmental criteria are to be adopted for various parts of the system – sometimes even a single area such as a station is thus divided – the ventilation system designer must consider the effect of train-induced airflows on them all.

The magnitude of the train-induced airflows depends on both the speed of the trains and on the ratio of their cross-sectional area to that of the tunnel (the 'blockage ratio'). For a blockage ratio of about 0.6 and a train speed of 80 km/h, a piston-generated flow rate of about 150 m^3/s may be established. In general, the air in front of a train is travelling at around half the train's speed as the train enters a station.

In temperate climates, train-induced air movements on this scale are usually sufficient to ventilate the whole underground system. During normal operation, they meet the physiological needs of passengers and staff and keep the tunnels cool; clearly they cannot control or move smoke, since the piston effect dies when the trains stop moving.

The beneficial influence of the piston effect can be enhanced within the tunnels: in a twin-tunnel system, for example, the provision of cross passages to connect the tunnels improves air movement; and the exchange of air with the outside is augmented wherever an extra shaft to the surface is provided for any reason. The piston effect supplements any station ventilation system that there may be, but it has the disadvantages that large air velocities can be created in the stations, which can be a nuisance to passengers, and that large-scale, uncontrolled air exchange between the stations and the tunnels can take place.

A moving train propels air into a station space and draws air out behind it, in considerable quantity. This can cause unwelcome draughts in some areas. One of the most obvious manifestations of the problem is the high air velocities often experienced in escalator shafts. A limit of 5 m/s is normally laid down in modern systems; but in older systems, velocities of over 8 m/s have been measured. This is sufficient to severely disturb passengers' clothing and to make old and infirm people feel unsteady.

To reduce these velocities, extra shafts, normally referred to as draught-relief shafts, have to be provided. Draught-relief shafts are expensive and difficult to construct, but underground railways cannot nowadays be built without them. Not only do they contribute to passenger comfort, they also play a vital role in the preservation of life by removing smoke during a fire. Perhaps the ventilation system designer's greatest single problem is how to devise an effective underground draught-relief system without incurring the civil engineer's wrath by specifying large and otherwise empty spaces.

22.3 Arrangement of draught-relief shafts

Draught-relief shafts may be installed either in a station or in the adjoining tunnel, on the approach or the exit side of a station. Some of the many variations are shown in Figures 22.1 and 22.2. Although calculations have shown that all these arrangements are satisfactory, the best arrangement is to connect the draught-relief shafts to the system immediately outside the station headwall. This does not in itself significantly reduce air velocities in the escalator shafts, but it does prevent draughts from occurring at the ends of platforms; and by reducing the exchange of air between the tunnels and the stations, control of the station environment is improved.

Although a preference has been expressed, the actual point at which draught-relief shafts are connected to the tunnels at station level does not greatly influence the aerodynamics of the system, and the decision is usually left almost entirely to the civil engineer. Apart from the actual construction of the shaft, it can be very difficult to find a suitable site for it on the surface, especially in city-centre areas where extensive development has already taken place. A number of

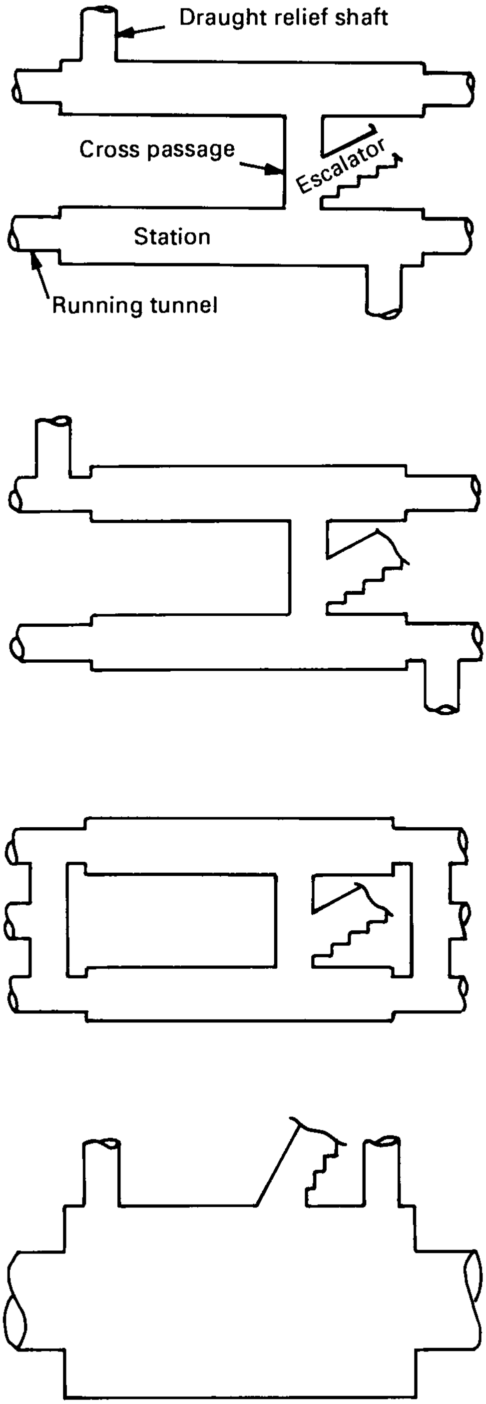

Draught relief shaft
Cross passage
Escalator
Station
Running tunnel

Figure 22.1

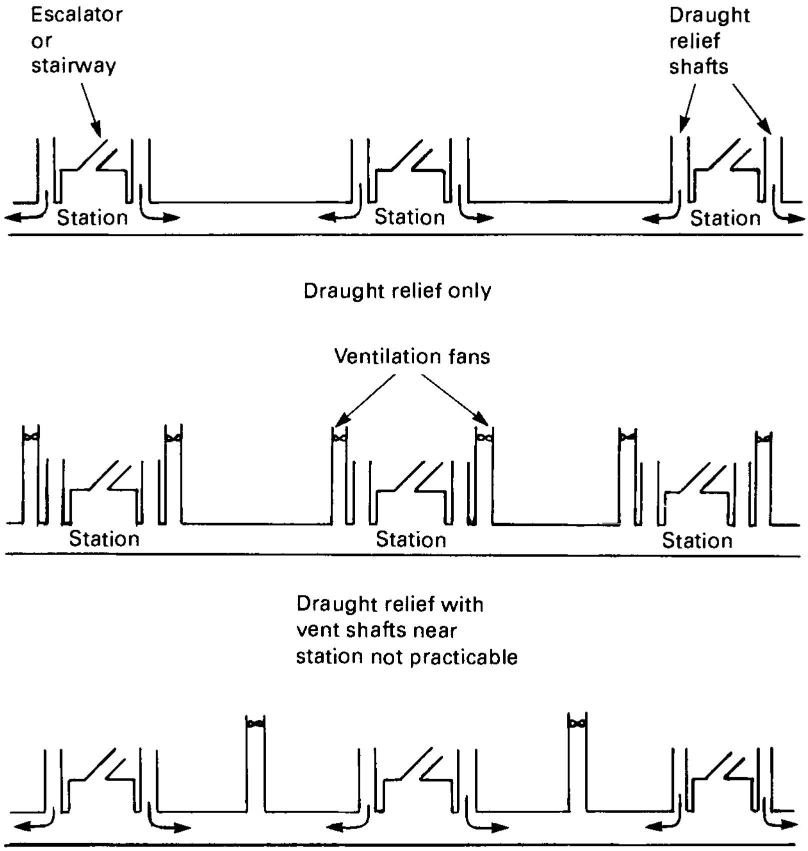


Figure 22.2

ingenious solutions to this problem have been achieved over the years; Figure 22.3, for example, shows how a false facade has been used to disguise the top of a relief shaft (Eldon Square, Newcastle). In some cases shafts have been constructed with up to 120 m of horizontal heading in order to reach a place where a site was available on the surface.

There is a trend worldwide for a property developer to join forces with a transport authority in developing the land above an underground railway station. Such a developer may find himself responsible for incorporating a draught-relief shaft somewhere in the surface works. There may well be strong resistance from the developer to incorporating such shafts; but the need for them is fundamental, and they have been incorporated into buildings in some railway systems, with passenger entrances, ventilation inlets and exhaust shaft openings at surface level. This is particularly true if the inlet and exhaust shafts are provided with mechanical ventilation. Nevertheless the correct siting and sizing of shafts is so vital that priority must be given to the satisfactory location of the openings both to prevent noise and polluted air annoying the building's occupants and to give good ventilation performance within the railway. Inadequate vertical and horizontal separation raises the undesirable possibility of smoke being recirculated between inlet and discharge shafts and between the shafts and escalator entrances.

Figure 22.3 Draught-relief shaft, Eldon Square, Newcastle

Escalator shafts are often driven through water-bearing ground and constructed from circular cast-iron tunnel sections. Such shafts are often provided with false ceilings; but if the upper part of the shaft were simply given a suitable aesthetic finish and otherwise left largely clear, the area available for air movement would be greatly increased and air velocities reduced.

Many old stations have only one stairway or escalator connection. This means that the cross-sectional area of the airway from platform level is small and that draught velocities are necessarily high. If the increasing attention to means of escape results in the provision of an additional stairway or escalator at the other end of the platform this will greatly assist draught relief.

22.4 Air treatment in stations

The measures adopted to control the airflows induced in the tunnels by the trains, and perhaps to take advantage of them, have a direct influence on some of the station design parameters. The need for draught-relief shafts at station ends, connected to the surface, and/or cross passages between tunnels, will be fixed. Escalator shafts must be at least large enough to handle the required passenger flows safely and to keep air velocities down to an acceptable level. Over and above these considerations will be the need to include ventilation systems and, in hot climates, mechanical cooling systems in the station area itself, to ensure that the environment in all public and plant areas is maintained within acceptable limits.

Modern high-performance rolling stock consumes large quantities of electric power, which is supplied by underground traction substations. Much of the electrical energy supplied is released within the underground environment and reappears as waste heat, emitted directly by the power equipment and indirectly by

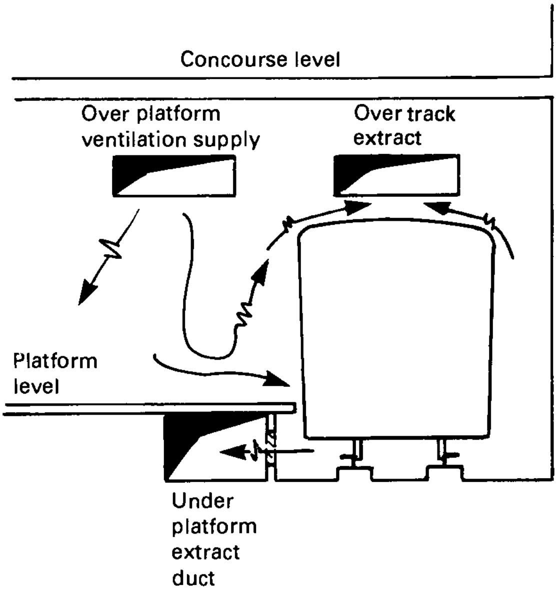


Figure 22.4

the train brakes in stations. Removal of the waste heat requires large amounts of air and necessitates the allocation of space to air ducts and fan equipment.

A typical arrangement of a ventilation system for dealing with the heat emitted by trains is illustrated in Figure 22.4. While the underplatform extract duct is easily accommodated within the depth of the platform, and does not contribute to the overall length of the station, the overplatform and overtrack ducts are directly responsible for increasing the depth, perhaps by as much as 750 mm in a cut and cover station. The need for such high-level ducts is a direct result of the environmentally satisfactory platform conditions demanded by passengers and of the consequent need to extract the waste heat emitted by the trains as close to its source as possible. To allow the waste heat to be removed via escalator shafts and stairways to an upper level or to the surface would greatly reduce the comfort of the passengers in the station area.

The arrangement shown in Figure 22.4 has a basically vertical bias. Figure 22.5 illustrates how a horizontally biased alternative can be developed to enable the civil engineer to minimize the depth of excavation, and hence the cost of construction, without preventing the ventilation system designer from successfully capturing the waste heat and providing a thermally acceptable environment.

Rather than concentrate the fans and cooling equipment serving the air supply and extract functions together at one end of the station, it is better either to site it all at the midpoint or to provide separate supply and extract plant rooms at each end. Duct systems can then be arranged so that each serves approximately half the station length, so reducing duct cross-sectional areas or depths by up to 50 per cent and reducing excavation costs.

Each plantroom may be arranged on several levels, serving different levels in the station, such as an upper concourse and one or more lower platforms (see Figure 22.6). The plantrooms typically incorporate duplicate sets of equipment to serve

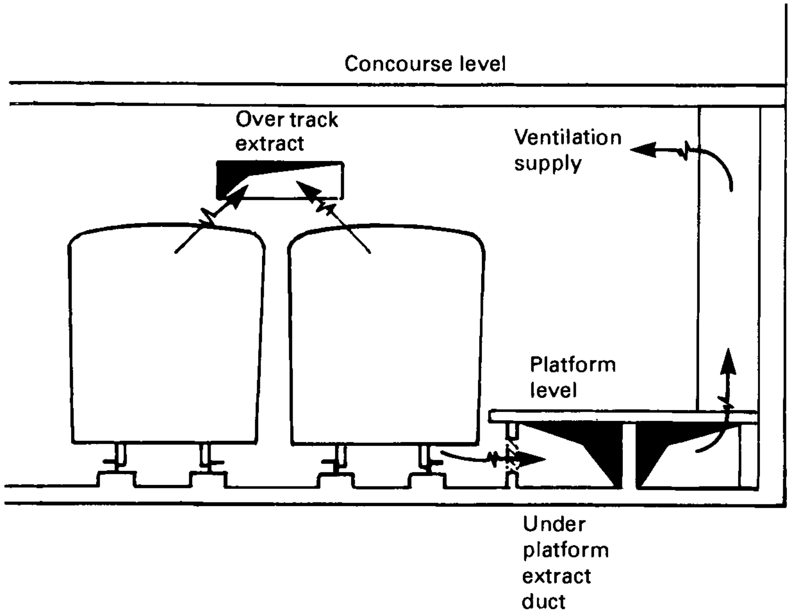


Figure 22.5

each side of the station as well as each level, and may also house some parts of the tunnel ventilation system.

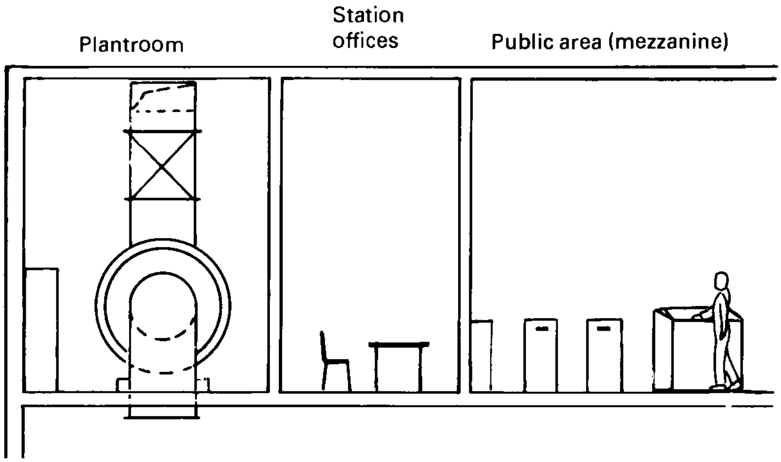
Key elements of the ventilation system for the public areas include:

1. Fresh-air delivery fans, supplying air to platform and concourse areas;
2. Exhaust-air return fans, extracting air from platform and concourse areas, for discharge either to atmosphere or for return to public areas after filtering and, perhaps, cooling;
3. Filter systems for the removal of atmospheric dust, clothing fibres and brake dust;
4. Acoustic silencers and duct attenuators for noise suppression;
5. Damper systems for ducted air control and volume regulation.

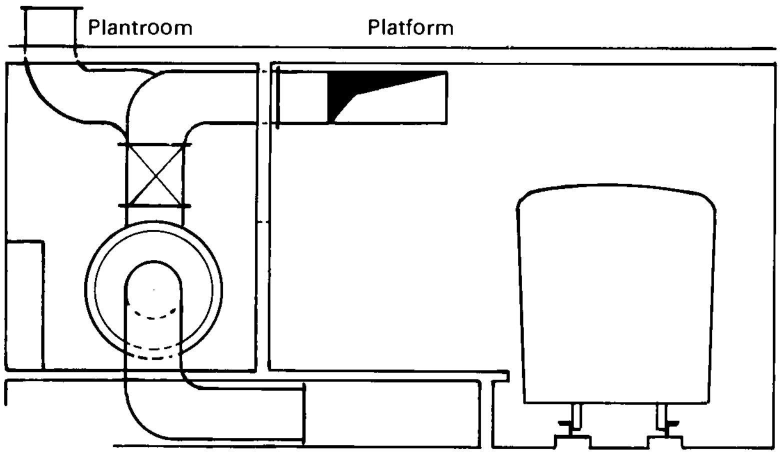
Additional independent ventilation systems are provided for other, non-public, station areas. They may involve either ventilation supply and extract only or full air conditioning. Basic ventilation is adequate for electrical equipment rooms and unmanned mechanical plant areas such as plantrooms; air conditioning may be necessary for signalling relay rooms, which require close control of temperature, and for administrative offices and ticket offices.

22.5 Mechanical ventilation of tunnels

Mechanical ventilation becomes necessary when the airflows induced by the piston effect of the moving trains are no longer sufficient to guarantee effective ventilation and cooling. In temperate climates this may only happen at certain times of year, and then only at times of peak levels of traffic. Although the thermal mass of the



Elevation on concourse



Elevation on platform level

Figure 22.6

system tends to damp out thermal peaks, long spells of warm weather may produce problems underground. If it is considered likely that long periods of mechanical ventilation may be needed, careful consideration must be given to the interaction between the potentially conflicting needs for draught relief and mechanical ventilation.

For draught-relief purposes, the shafts are usually designed to limit air velocities in occupied station areas to an acceptable level. This implies a cross-sectional area of around $20\,m^2$ for a typical individual shaft. If the shaft must also accommodate powered ventilation air (at a flow rate, say, of $100\,m^3/s$, and a velocity of $10\,m/s$), the area must be increased to around $30\,m^2$.

Clearly, the two separate air movement functions cannot take place simultaneously in a single shaft unless the shaft is divided. It is very likely also that access stairs for maintenance will be required in the shaft. Generally, deep shafts are sunk with a circular cross section and lined with cast-iron tunnel linings, and are not easily divided vertically, especially if stairs are fitted.

If ventilation shafts were constructed close to the draught-relief shaft, recirculation would occur unless complicated damper arrangements were provided. It would therefore be necessary in these circumstances to provide draught-relief shafts at the stations and mid-tunnel ventilation shafts between the stations.

22.6 Ventilation systems for emergency conditions

In the event of breakdown, power failure or similar incident, trains may come to rest in the running tunnels. In hot climates especially, or if the trains are fitted with air conditioning, the air around the trains can increase in temperature until it becomes necessary to activate the tunnel ventilation system to move the tunnel air and thereby prevent a further local temperature increase. The fans must be supplied from a separate power supply independent of the traction supply, so that a single fault cannot cause the failure of both the trains and the fans, leading to a catastrophic rise in temperature in the interior of the trains.

Although fortunately they are very rare, fires do occur in underground railway systems. Fires are always accompanied by smoke, and, in general, smoke is regarded as much the greatest hazard in these circumstances. The control of smoke therefore warrants a heading of its own.

22.7 Mechanical ventilation for smoke removal

In an unventilated closed space such as a tunnel, smoke spreads in both directions along the roof, gradually increasing in depth to fill the tunnel. If the tunnel is ventilated, the spread of smoke can be controlled. By inducing an air velocity in the tunnel that is greater than the speed with which the smoke would otherwise be spreading, it is possible to ensure that the smoke travels in one direction only. A smoke-free path is left in the other direction, which can be used by the passengers as an escape route and by firefighters as a means of access to the fire.

The subject of mechanical ventilation has been discussed above. If the need for tunnel cooling has been separately established, the arrangement in which ventilation shafts with fans are placed at mid-tunnel is entirely suitable. If, however, the need for ventilation has not been established separately, it is possible to mount the ventilation fans in the draught-relief shafts. Two major factors must be considered when proposing this arrangement. First, the fans must not be operated while trains are running, or there will be no draught relief. Second, the installation must not encroach on the free cross-sectional area of the shaft. Various arrangements, all of which incorporate motorized dampers, can be used to meet these conditions. Figure 22.7 shows a typical example. The elevation shows the dampers set for the draught-relief mode; the plan shows them in the ventilation mode.

It is beyond the scope of this chapter to discuss the factors that determine the size of a fire or the rate at which smoke spreads, but clearly they include the amount of combustible material present. By suitable choice of construction materials, rolling

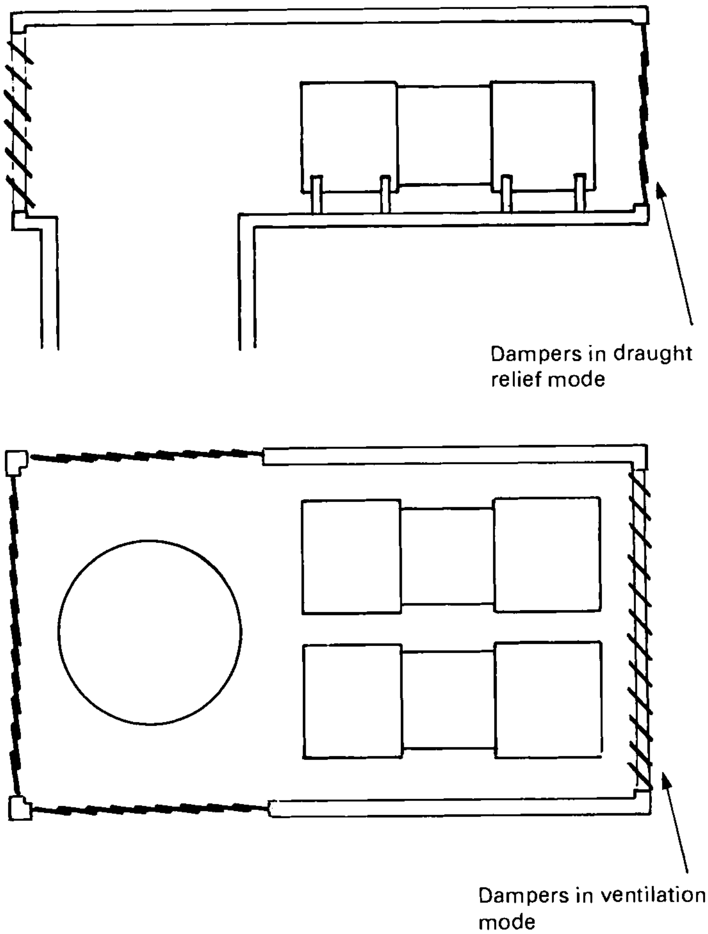


Figure 22.7 Combined draught and ventilation shaft

stock manufacturers are able to make a major contribution to fire safety. Passengers, operators and legislators are likely to demand increasingly close attention to this factor.

22.8 Passenger evacuation

In the past it has been customary, when an underground railway train has had to be evacuated, for the passengers to walk along the tunnels and escape to safety via the nearest station. In the early 1980s, however, the United States National Fire Protection Association (NFPA) published a code of practice, NFPA 130, in which it is recommended that escape shafts should be provided every 750 m if the stations are widely separated. A similar proposal is emerging in the United Kingdom as a result of the close examination of procedures that is following the Fennell Report on the fire at King's Cross on the London Underground in November 1987.

In central urban areas, inter-section distances are often well below 1 km, and escape shafts are unnecessary by the NFPA criterion. After their initial success, however, many systems are now being extended into suburban areas, with considerably greater inter-station distances. In a recent project, two successive inter-station distances were 3 km and 2 km. Adoption of the NFPA 130 recommendation required the provision of three and two escape shafts, respectively, in these sections. This requirement had to be reconciled with the smoke-removal function discussed above. If two adjacent shafts were not provided with fans it would not be possible to remove smoke from the intervening tunnel in either direction at will. A number of alternatives were considered, and the arrangement shown in Figure 22.8 was chosen as meeting all the requirements discussed.

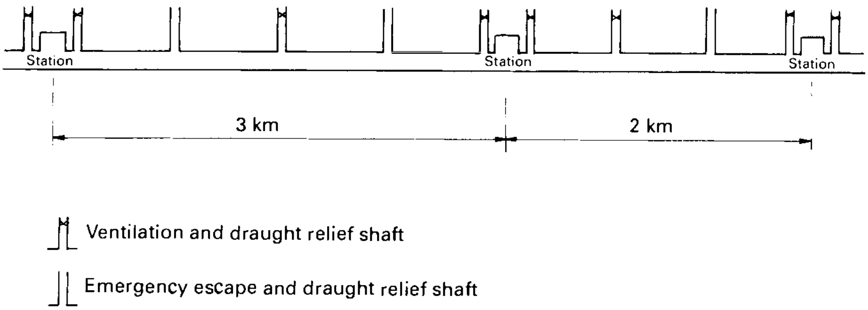


Figure 22.8

22.9 Provision of space underground and optimization of civil construction costs

No two underground railways are ever likely to give rise to the same civil construction problems: simple variations in ground conditions through to more complex considerations of hump-alignment profiles and sub-surface infrastructure and building foundations will always require different solutions. The simple choice between cut and cover and bored construction is one of the most obvious differences in construction techniques; but how deep to cut or bore is strongly influenced by the particular unique environment in which the railway is located. The different solutions adopted for each of these parameters will affect each other and influence the design of the ventilation system.

Similarly, no single solution is available to say that a tunnel ventilation system must be of a particular form or that a station air-conditioning plant must be arranged in a particular way. The fixed parameters of a system usually relate to the maintenance of a required level of safety for passengers; this may be specified as a required number of air changes per hour or the ventilation system may have to be able to operate in a fully reversible mode for a minimum length of time at a specified temperature.

A well-designed ventilation system does not always have to add to the station's underground volume or the tunnel size. Small local fan plantrooms may conveniently be located beneath stairways and escalators. Main plantrooms at station ends can usefully concentrate plant at an upper level and so reduce the overall length of a station. Duct systems may be constructed with one or more of their sides formed from the edge of the station's structural box. Such measures may directly contribute to a reduction in the station's underground volume and hence help to minimize construction costs. Conversely, a ventilation system that has been squeezed into too small a space will usually cost more to install, maintain and operate; and, in an emergency, it may not operate as effectively as it should.

Whatever the requirements, the key to a safe and successful underground railway is that the civil engineer should recognize the life-preservation and passenger-safety roles of the ventilation system, and that the ventilation system designer should be prepared to be flexible in designing that system.

22.10 Summary

The ventilation and air-treatment systems are the most expensive non-revenue-earning areas of an underground railway. Overreacting to this by infringing minimum requirements undoubtedly leads to less than satisfactory performance, both in normal operating conditions and in an emergency, and demonstrates a failure to recognize the essential safety aspect of the system.

Chapter 23

Civil defence requirements

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Contents

23.1 Introduction

23.1.1 Objectives

The use of underground railways in time of war has posed questions for engineers for over 50 years. This chapter will provide guidance on the provision of civil defence facilities in underground railway systems.

In the modern world the threat of warfare comes in various guises, and it is necessary first to consider the nature of weapons available to a possible aggressor. A description of the types of weapon and their likely effects is included in Section 23.2. Section 23.3 describes the form which protection against them could take. The extent of any facilities provided will depend on consideration of their costs relative to their benefits, as well as the feasibility of their provision.

23.1.2 History

Being below street level, underground railway stations have, in the past, been used as improvised shelters against the effects of conventional high-explosive attack [1]. In London, such stations were mostly of bored construction, and lie at depths generally greater than stations where cut and cover construction methods are used.

During the blitz of World War II Londoners used the underground railway system particularly for overnight sheltering when squadrons of bombers flew each night. Being there for periods of 8 to 12 hours, they often made their own improvised provisions for food, etc.

Of the hundreds of thousands of people who sheltered in the London Transport stations some were killed and injured by enemy action. Although there were incidents of people suffering from direct blast entering the bores, the cause recorded most often was that of physical collapse of the station structure or tunnel lining, direct entry of soil, etc. However, it must be remembered that structural collapse will have required more effort and coordination from the rescue services, and hence there was a greater likelihood of these incidents being documented.

Damage to the London Underground was recorded. These records, together with other data, provide an interesting background which would allow the deduction of how segmentally lined bored tunnels respond to explosion.

23.1.3 Underground railways in wartime

In view of the far-reaching influence on the life of a city of any rapid transit system it is important that the option of making provisions for protection against attack be considered and their extent decided. Where a city needs public sheltering the stations of an underground railway could make a significant contribution to those requirements when they are satisfactorily planned and prepared.

One of the strategic purposes of an air raid is to disrupt the smooth functioning of community life. It is therefore preferable that a railway should not cease to function during an air raid alert. The removal of a railway as a means of travel would not appear to be compatible with the aim of keeping the principal functions of the city active during an emergency. The additional load on surface traffic would cause congestion and confusion. While a degree of protection may be sought, any sheltering requirements should, as far as possible, be designed not to compromise the normal operation of the railway.

When civil defence facilities are discussed it is important to distinguish between use of the underground railway for:

1. Sheltering (that is, the use of the available underground space for the protection of people against enemy attack); and
2. Protection of the railway system (that is, securing the transportation system and the value of the investment against the damage and disruptive effects of enemy action, and protecting the travelling passenger).

Authorities considering the provision of facilities should bear this distinction in mind. The protective measures for each will not all be identical.

23.1.4 Underground railways as railways

Previous chapters in this book have established that a new underground railway will have stations arranged as close as possible to the surface with short approaches to allow ease of access for passengers. In an ideal situation this leads to cut and cover construction being the appropriate technique. However, to avoid disruption of the city at ground level, boring can be the preferred technique for tunnels for the running lines between stations. For successful boring the tunnels must be sufficiently deep.

23.1.5 Underground railways as shelters

The use of an underground railway for sheltering while it continues to operate as a transit system poses some obvious problems. While the public circulation areas in the stations (that is, all areas at platform and concourse levels) provide large spaces for the congregation of people below ground they are not necessarily suitable for giving protection. Their openness and nearness to the surface for ease of access is contrary to the needs of resistance to weapons.

Where additional, non-travelling, people are seeking shelter in addition to normal travellers there is bound to be a degree of congestion. Station management will therefore require special procedures in times of sheltering. Railway operation during any time of sheltering must also be given consideration such as train-operating speeds, headways, method of control, etc.

Modern capabilities for carrying out an air raid dictate that the assumption should be made that an attack could occur without warning. For this reason, where the presence of the public seeking shelter is anticipated, careful design should avoid elaborate and time-consuming preparations to achieve the planned degree of protection.

Unless an underground structure can provide adequate safety, either naturally or by virtue of inbuilt modifications, the public should not be permitted to enter it. Where such safety does not result from railway considerations alone, an underground railway permitted to continue in operation during conditions of warfare should include additional protective measures.

23.2 Weapon characteristics and effects

23.2.1 General

Weapons of modern warfare fall into several classes and will be considered here under four headings:

1. Conventional;
2. Nuclear;
3. Biological; and
4. Chemical.

Conventional (high-explosive) weapons create damage and injury by their blast and shock effects, while biological and chemical weapons rely upon the effect of more insidious elements. A nuclear weapon involves both effects, although its blast and shock is of a magnitude totally different to those of conventional weapons. The last three are often grouped and referred to as NBC weapons.

23.2.2 Conventional weapons

The destructive power of a conventional weapon results from a combination of its casing strength to penetrate a target and its explosive content (charge weight) to cause damage and injury. While it is possible to have almost any combination of casing and charge weight, these weapons can be considered in four categories:

1. *Armour-piercing (AP) weapons*. These generally have the heaviest casings, with the charge weight being only 10–15% of the total weight. For greatest effect they would be fused with a slight delay to achieve maximum penetration before exploding. This category may also include weapons with lighter casings but containing 'shaped charges' which are fused instantaneously to achieve penetration by their special explosive effect.
2. *Semi-armour-piercing (SAP) weapons*. These have casings of sufficient strength to withstand impact on concrete slabs or framed buildings, etc. and hence achieve a degree of penetration. They have a typical charge weight of 20–30% of the total. A slight delay in their fuse will be required unless they employ a shaped charge for their penetrating effect.
3. *General-purpose (GP) weapons*. Such weapons have casings of sufficient strength to withstand impact on open ground or on buildings of average strength. Their charge weight is generally between 40% and 60% of the total weight and fusing can be varied to suit the particular target.
4. *Light-case (LC) weapons*. These rely for their destructive effect on the blast of a large charge. This is, typically, 70–80% of the total weight, which is not normally less than 1000 kg. They are fused to explode at the moment of impact.

All these weapons can take the form of bombs or missiles. The total weights of bombs are normally 100 kg and larger, the largest sizes being limited only by the capacity of aircraft that can carry them. Missiles start at lower weights. Compared with bombs, the explosive power of a missile is less in proportion to the total weight because the rocket motor and fuel have to be included. The US Department of the Army assembled extensive data on bombs from their own resources [2] and from the studies collected together by Dr D.G. Christopherson [3] following World War II. Conventional weapons themselves have not greatly changed since that time, advances being primarily in more accurate systems of delivery, more predictable detonation and in designs that achieve greater penetration.

While missiles can be launched from the ground or from aircraft against specific targets, general offensives using bombs dropped from aircraft are still carried out. However, sophisticated defence methods using electronic guidance systems will ensure that saturation bombing from high and medium altitudes, so extensively used during World War II, now has a limited use. Low-level flight with the bombs finally delivered by special techniques is now the norm. Attack from several miles' distance is also possible using artillery, but this may be seen more as a military situation, beyond the scope of civil defence. Any protection provided against an air attack will, however, also provide a good degree of protection against artillery.

23.2.3 Nuclear weapons

The size of a nuclear weapon is measured in terms of the weight of the conventional explosive, TNT, which would have the equivalent explosive effect. Weapons having a power of a few megatonnes are common in the strategies of the major powers while smaller weapons of (say) 20 kilotonnes could be more appropriate when considering the civil defence requirements of the smaller nations. Weapons may be fused to detonate on, or just above, the ground or in the air.

The effects of a nuclear explosion relative to civil defence are (approximately in the order in which they occur):

1. Heat and light flash (fireball);
2. Electromagnetic pulse;
3. Initial nuclear radiation (INR);
4. Ground shock;
5. Blast; and
6. Fallout.

These effects are described below.

During the few seconds of the nuclear fireball buildings within a few kilometres, depending on the size of weapon, will be set alight. People sheltered from the direct effect of the thermal radiation will not be burnt but there will still be a hazard from subsequent fires.

A high-altitude nuclear burst could seriously disturb the ionosphere and disrupt radio communications. Low-altitude bursts produce a pulse of electromagnetic radiation (similar to a lightning flash but with a pulse of a much faster rise time). Conductors, such as power lines and radio aerials, could receive this pulse and pass a damaging current into the equipment to which they are attached.

INR is defined as that nuclear radiation emitted within the arbitrary period of one minute of the burst. Most is emitted within a few seconds. It is particularly penetrating, and the dosage of radiation is likely to kill a high percentage of the exposed population within several kilometres. The neutron bomb is a weapon with a warhead developed to give enhanced INR. These have a greater capacity for killing personnel but with reduced blast and residual radiation. They have been developed primarily for battlefield use with sizes up to 2 kilotonnes.

Ground bursts from nuclear weapons cause large craters and serious ground shock. Air bursts give rise to three distinct damaging effects. These are the diffraction effect of the blast wave, the drag loading of the following wind and the crushing weight of the overpressure. Above-ground structures suffer from all three of these effects. As an example of the extent of damage above ground, all brick-built structures within 1 km of the point of burst are likely to be destroyed by even a 'small' nuclear weapon. For below-ground structures the first two effects do not occur and the overpressure is attenuated with depth. Where the depth is such that the earth cover is greater than half the minor span of the structural member the soil acts as an arch, dissipating some of the already attenuated overpressure.

Against this the positive phase of the overpressure below ground acts longer than in air by up to 50%. Damage to structures below ground will vary with the type of structure and character of the soil. Small self-contained structures would move bodily with the surrounding ground. Long flexible structures underground may be able to accommodate themselves to the ground movement and be undamaged if they are sufficiently distant (say, further than three craters radii) from the burst.

A detonation which results in the fireball reaching the ground is a 'ground burst' and will result in dangerous radioactive fallout. Fallout consists of soil particles which have been melted, partially vaporized and taken up into the fireball and onto which radioactivity from the bomb has been deposited. The coarser particles descend first while those smaller than 50 μm will travel in the wind for several days before coming to earth. The emission of radiation is measured in roentgen units. The absorption of radiation by the human body exposed to fallout is measured in 'rads' (Radiation Absorbed Dose). A person exposed to 600 roentgens of gamma radiation would have only a 50% chance of survival. The bone marrow, where the damage is caused, would have absorbed 450 rads. Survivors with lesser doses would, nevertheless, suffer from the greater probability of cancers in later years. Zones in which fallout is deposited are dependent on wind speed and rate of deposition. These zones would be roughly elliptical where the winds are steady and are defined by the dose rates which prevail. Figure 23.1 shows a typical trend of the dose rate in relation to time and to average distance from the point of burst. The actual amounts will be dependent on the size of weapon chosen. Care should be taken to ensure that no more than about 10 rads per day are absorbed. At this level of daily dose the human body can generally recover without any immediate consequences. This and the prevailing dose rate will define the period during which it would be unsafe to emerge from a shelter. There will be little, if any, fallout from

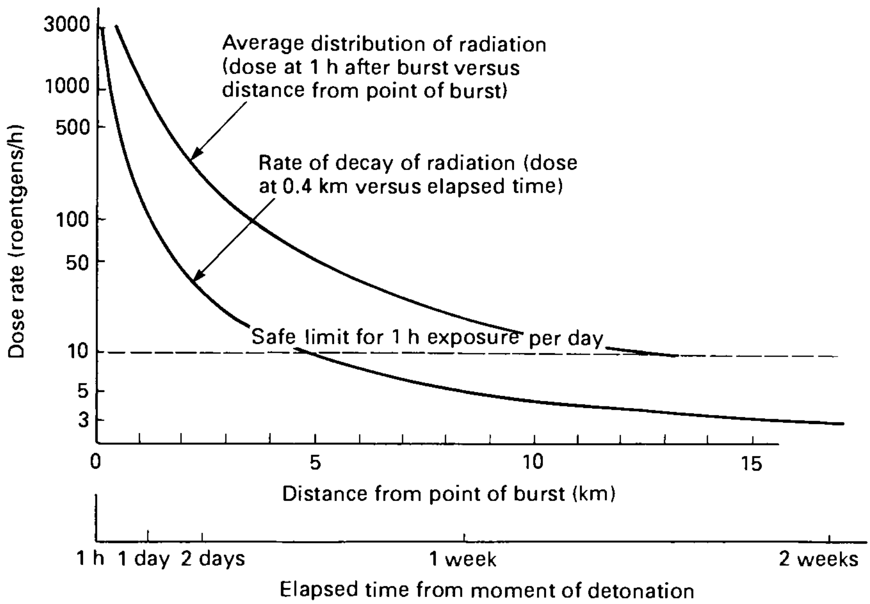


Figure 23.1 Radiation from fallout: typical distribution and decay. (The curves shown are for a 20-kilotonne weapon)

an air burst and most of what is produced will not be deposited for days or weeks. By this time its radioactivity will have decayed to a large extent and will have been spread sparsely in other parts of the world.

23.2.4 Biological weapons

An attacker wanting to induce disease in humans by disseminating living microorganisms must overcome a number of problems. While it is technically possible for such microorganisms to be introduced into the human body by injection (i.e. from the bite of a carrier insect) or by ingestion (i.e. by contaminating food or water), the most technically suitable method is by inhalation. To achieve this, the attacker must disseminate the microorganisms into air which will be breathed. The process of dissemination can be carried out easily using an aerosol spray, but such a device would need to be of large capacity to mount a serious attack. However, microorganisms are very susceptible to changes in temperature, humidity and atmospheric pollution. They also die very rapidly in the presence of ultraviolet light from the sun. An attack mounted at night will overcome some of these problems and a contaminated parcel of air will drift downwind and have time to spread during the hours of darkness.

In order to enter the body through inhalation and be retained in the lungs the actual size of the aerosol particles must be in the range of 0.5–5 μm. This imposes requirements on the type of spray generator.

23.2.5 Chemical weapons

A large number of chemical compounds have been classified as 'chemical agents', i.e. the toxic materials of chemical warfare. These can be divided into two groups with regard to their method of application, namely those which act principally as a vapour and those which are a hazard on contact. These two groups cover both the nerve agents and the vesicant (blistering) agents.

Some vapour agents have very high toxicity (some being fifty times more effective, weight for weight, than, say, hydrogen cyanide). On release from containment they evaporate rapidly and spread downwind. A few breaths of contaminated air can prove fatal. Some contact nerve agents are even more toxic than the vapour agents. One drop can be lethal when absorbed through the skin. The vesicant agents cause painful blisters which are difficult to heal and can result in death. Inhaled from a vapour, they produce blistering in the respiratory tract. Some chemical weapons are persistent, remaining a hazard for some weeks, while the danger from others will be over in an hour or two.

23.3 Protective measures generally

23.3.1 Against conventional weapons

There are two basic methods for providing a sheltered space with protection against the effects of high-explosive weapons:

1. By shielding the structure with an additional sacrificial layer in such a way that the weapons do not explode too close; and
2. By strengthening the shelter structure itself to resist the direct effects of the explosion.

Frequently a satisfactory degree of protection will be best achieved by a combination of both methods.

Shielding must be such that effects of the explosion do not exceed the capacity of the shelter structure to provide acceptable protection. Where a very high standard of protection is required a double skin can be utilized. An outer massive layer will bear the brunt of the attack, and within this will be an inner layer to protect the occupants or contents from the disintegration of the outer layer which can result from an intensive attack. The double-skin method is more appropriate to small structures.

Where a sheltering structure is deep underground the soil will provide shielding. However, weapons with delayed-action fuses can penetrate the soil and the surrounding soil can then intensify its effect (see 'Tamping' in Section 23.5.4). This must be taken into account. Where the structure is insufficiently deep for the protection afforded to be adequate by virtue of its depth alone then a 'burster slab' is a very effective shielding solution.

A burster slab is a slab supported fully and continuously by the ground at, or just below, the ground surface to form an impenetrable barrier to the weapon. The weapon, being exploded at the surface, will then have a large portion of its energy dissipated as blast into the air. Shock propagated downwards towards the protected structure through the burster slab can still be a hazard. Sufficient thickness of infill material (depending on the size of the weapon) is placed between the burster slab and the roof of the buried structure to provide cushioning against the reaction at the underside of the burster slab. Some alternative arrangements of burster slab appropriate to the protection of cut and cover underground railway stations are shown in Figure 23.2. Note the extensions of the burster slab outside the shielded structure to take account of the possible horizontal component of penetration as weapons enter the ground (see Section 23.6.5).

Where an unshielded structure is exposed to attack the structure must resist:

1. The penetration and shock due to impact; and
2. The destructive effect of explosion.

Both are significant for the design of structures above or near the surface of the ground. Impact will become progressively less important with increasing depth of the structural member under consideration. Explosion will become more critical at the depths at which tamping occurs. The strengthening of a structure to resist weapon damage is often called 'hardening'.

23.3.2 Against nuclear weapons

A shelter for protection against nuclear weapons needs to be designed to resist the several different effects. People inside shelters will be protected from the direct effects of the heat flux. It is important, however, that no flammable materials are used in exposed superstructures because these may catch fire. Another hazard could arise from lethal fumes produced from burning or smouldering debris which could be drawn into the ventilation system. It is not possible to filter out some toxic fumes such as carbon monoxide. The ventilation systems must, therefore, take account of the possibility of fires outside the shelter.

Electrical equipment needs protection from electromagnetic radiation. In the event of adequate warning of an imminent nuclear burst, equipment must be disconnected from any outside lengths of conductor. A well-earthed Faraday cage around vulnerable equipment will provide an additional precaution.

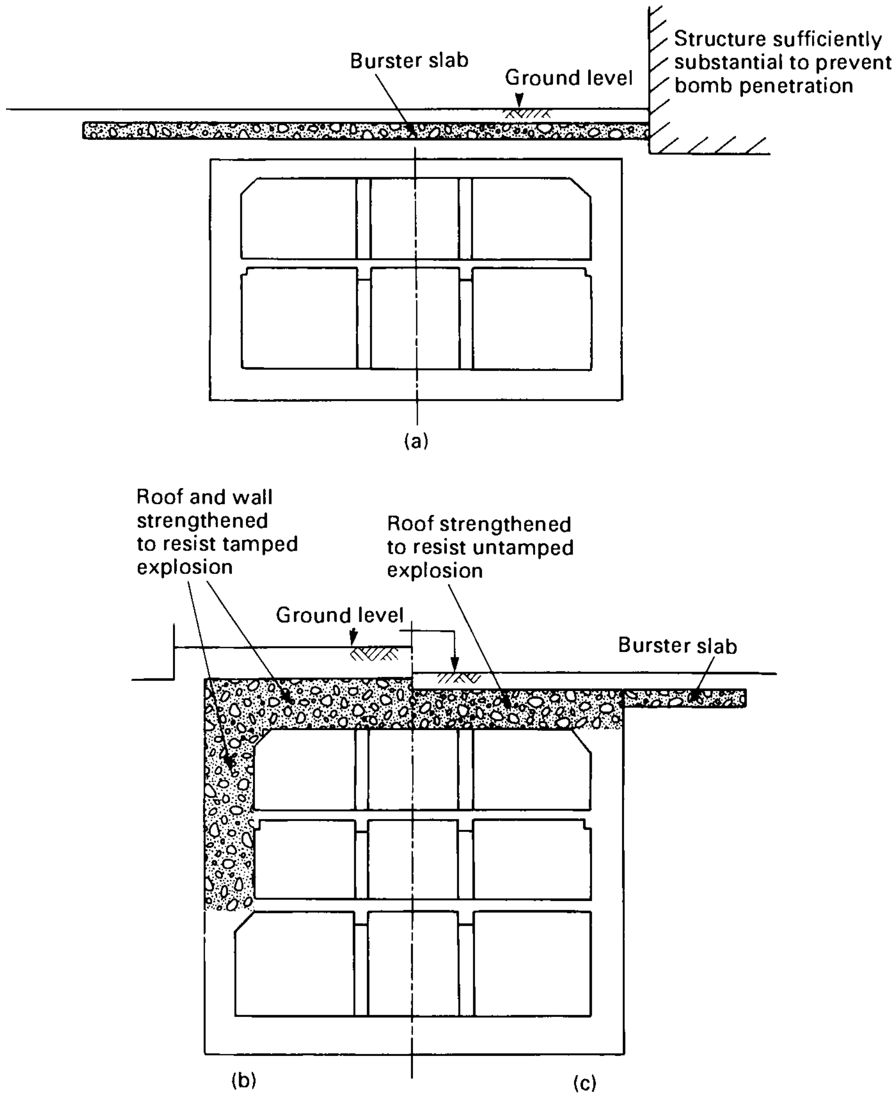


Figure 23.2 Structural protection of stations: some alternative typical arrangements. Structural members and/or burster slab which are designed to resist the effects of direct exposure to explosion and/or impact are shown shaded

INR is resisted by mass of material. Thick concrete in the walls of an underground shelter is therefore an economically effective protection. The intensity of radiation is approximately halved by each 56 mm of concrete or 84 mm of soil. It is found that, generally, the thicknesses of concrete required for a shelter's strength against a nuclear weapon burst at a certain stand-off distance is adequate to provide good protection against INR from that weapon. However, further weapon developments may alter this balance.

Protection against a direct hit from a nuclear weapon is impracticable. A nuclear shelter is normally constructed to have resistance to the explosion of a chosen weapon size at a chosen stand-off distance. These characteristics must be decided at the outset of the design process. The load to which the shelter will be subjected can then be defined and the structural strength designed by a normal engineering approach.

With weapon size and detonation distance established, all other protective features must be made compatible. The major difficulties arise from the need for apertures in the structure for air supply and extraction and for the exhaust from motors such as generators (which are required for power supply). Depending on the design overpressure, protection of apertures must be provided by blast valves, baffle walls or expansion chambers, or an appropriate combination.

Protection against fallout is achieved by effectively sealing the shelter against entry of the contaminated particles. There must be provision for the decontamination of clothing and equipment of people entering the shelter. Entry may be restricted through an 'airlock' where outer doors are only opened when inner doors are closed and inner doors are only opened when the airlock is 'clean'. However, since fallout is in the form of relatively coarse particles, an airlock is not a mandatory requirement. Air must be filtered before use for ventilation, although the filters in this case need not be particularly fine. The fallout dose rates outside a nuclear shelter will need to be measured at intervals either by robots or by personnel in protective clothing. As the dose rates are shown to reduce, a stage will be reached when shelterers will be able to leave the protection of the shelter for very brief periods. The permissible periods will become longer as the dose continues to decay. When the size and stand-off distance of weapon is defined in relation to the shelter to be provided, estimates can be made of the period of time people will be expected to remain in the shelter before it is safe to emerge. The size of shelter and its facilities in relation to the number of shelterers must then be established to be compatible with this period.

23.3.3 Against biological weapons

Protection against biological warfare is achieved by sealing the shelter effectively against entry of the assailing microorganisms for periods long enough to ensure that contamination in the outside air has completely decayed. Air taken from the outside into the sealed environment must have passed through a filter designed to retain particles in the range of 0.5–5 μm. Such filters restrict air flow significantly. The adequate filter area needed is likely to be large and they are easily blocked.

23.3.4 Against chemical weapons

Protection against chemical weapons, like that against biological weapons, needs a sealed environment. Removal of chemical vapours from air needed to be taken into the shelter is achieved by passing the air over activated charcoal. Such installations are large. Some form of detector should be available to provide information upon which to base advice of the time at which emergence from the shelter is safe and to establish the persistence of the agents used.

23.3.5 Which measures to adopt

There is no complete protection against weapons of war. Weapons and methods of attack are under constant development and variations in their sizes are almost limitless. Therefore, protective design is based upon prediction of the most likely forms of attack, the risk of their occurrence and the potential losses that would be sustained if the installation is unprotected or only partially so. This is equated to the cost of protection, which uses valuable resources that may be more effective elsewhere. The weapon that is to be resisted and the degree of damage which will be permitted must be defined as precisely as possible. Only then can a consistent system of protection be devised, taking all aspects into account.

Generally, a shelter against nuclear weapons is considered practical only at distances from the burst which exceed 1 or 2 km. When the distance and size of weapon has been decided the overpressures are established and structural design can be carried out effectively.

However, structural protection is far from being all that is necessary for an adequate nuclear shelter. The need for an effectively sealed environment, blast valves and communication systems to be maintained for a significant period of time render underground railways of doubtful effectiveness against nuclear attack. Biological and chemical weapons also require sealed environments. Because of the open tunnels, an underground railway cannot be both railway and NBC shelter at the same time. Biological and chemical weapon protection require the further investment of much money and space for the provision and maintenance of the filtration systems.

Protection against conventional weapons also requires investment in space and money. However, blast valves need to be effective at one instant only. With a realistic balance of threat against protection requirements it is possible to incorporate a degree of protection against conventional weapons into an operational underground railway.

23.3.6 Costs

The costs of introducing protective measures will depend entirely on the threat to be anticipated and the degree of protection to be provided. If bored tunnels need protection the increased cost can be very substantial indeed. At 1989 prices an extra £20–30 million per kilometre of railway in tunnel could be involved. Otherwise, the largest part of the extra cost is likely to lie with the additional measures and facilities required at the stations. An additional 15–25% of the total costs for unprotected stations could be required. This would allow the inclusion of protection against the possibility of attack from commonly available conventional weapons occurring directly at a station. Authorities budgeting for the provision of sheltering in a new underground railway system have, in the past, allowed 7–10% of the project's total cost.

23.4 Sheltering strategy and facilities

23.4.1 Introduction

The extent of sheltering facilities will be allied to the level of warfare to be resisted. In Section 23.3.5 it was reasoned that provision of NBC sheltering would compromise the operation of the railway system. The development of NBC sheltering is therefore outside the scope of this book. The remainder of this chapter will be devoted to civil defence against conventional weapons.

23.4.2 Selecting basic criteria

Railway stations offer capaciousness. Where large numbers of people are to be accommodated the environment in that accommodation is closely related to their physiological and psychological condition and, ultimately, their safety. Where the expected period of sheltering is limited, reliance can be placed on the duplication of plant and standby systems. During sheltering periods, access to the plant rooms for maintenance would then be unnecessary.

On the assumption that operation of the railway will not be stopped during an air raid alert, access into and through the stations to reach the railway needs to remain open. Protection of the open-entrance structures is therefore essential (see Section 23.7).

It may be necessary to identify areas requiring different 'grades' of protection. For instance, the main areas where people are expected to congregate must have walls and ceilings which do not eject spalled particles of concrete under the action of the selected attack weapons. Unmanned service rooms alongside might need only to be strong enough to stop the penetration of an impacting weapon to ensure that its subsequent explosion does not blow through the walls. When constructed, the areas which satisfy the requirements for the protection of people must be delineated, and should be clearly indicated by signs, etc.

There are various options to be considered where non-users of the railway are to use the stations for sheltering. For example, escalators must not be permitted to continue to feed over-congested areas. On the grounds of safety, ticket controls should be waived during periods of air raid warning and shelterers and travellers alike allowed free use of the railway facilities.

23.4.3 Influence of size

Public shelters are normally built to a size which accommodates about 50 people. Should such a shelter be seriously damaged by a weapon with effects greater than that assumed for design, the scale of disaster will then be limited to 50 people. Shelters required to accommodate greater numbers of people in one place are, ideally, of cellular construction. Any damage will then be contained and casualties limited to those in the damaged cell.

Underground railway stations are capable of accommodating large numbers of people, but cellular construction is not practicable. The consequences of a station being damaged by a weapon are, therefore, likely to be greater than for a purpose-made shelter. Underground stations may also present a greater area of exposure, i.e. a greater 'target' area, than smaller discrete shelters. A judgement must therefore be made of the risk of catastophe should large numbers of people be concentrated together with the possibility that a single hit by a large weapon could penetrate such a shelter. The level of protection and design parameters chosen should give consideration to these points.

23.4.4 Facilities related to period of sheltering

The start of a period of sheltering will presumably be defined by the initial air raid warning (usually by siren) either in advance of, or at the time of, the first wave of an attack. An 'all clear' signal will be given at the end. Periods of attack cannot be predetermined but an estimate of the approximate maximum duration of continuous periods of sheltering must be made for design and planning purposes.

Where sheltering periods are limited to 6 h or less only limited public facilities, i.e. toilets and first-aid, would be required. For periods exceeding 6 h there would need to be a significant increase in facilities. Provision of water storage with standpipes and washing arrangements, food stores and catering, and stores of bedding would be included. In addition, the restlessness of the shelterers, or their need for exercise, will increase the demand on the environmental control systems.

23.4.5 Number of shelterers

Studies carried out in the United States have established the optimum space requirements as 1.5 m^2 per person for periods of sheltering of 6 h. The areas made available for sheltering can therefore be related to anticipated numbers of people that can be satisfactorily accommodated. Large, modern two-level stations of a major railway system could hold up to 2000 people.

Floor space must be allocated so that some is still available for the travelling public. Consideration must also be given to the possibility of loaded trains being halted at the stations and unable to progress.

23.5 Characteristics of blast

23.5.1 General

The shock wave from a high-explosive detonation propagates outwards. When surrounded by uniform material, the shock front expands and its intensity reduces as the area affected increases. The shock wave itself consists of an almost immediate rise from atmospheric pressure to a peak positive pressure, which is followed by a decrease back to atmospheric pressure. The time taken for this first phase is between a few, and a few hundred, milliseconds. This is followed immediately by a negative phase, i.e. a decrease to sub-atmospheric pressure followed by a return to atmospheric, which lasts longer but has a very much smaller variation from atmospheric.

When the shock wave meets an interface between one material and another, (such as a shock wave in air meeting a wall) the wave is reflected back. The reflected pressure, which would be the pressure felt by the wall, is much higher than the incident pressure.

The peak pressure and the duration of the phase are the two principal parameters which define the effect that the blast will have on a structure. The integral of the pressure–time relationship is the impulse. Values for impulse, pressure and duration have been established for many years based upon measurements from testing. Therefore, although more modern presentations exist, acceptable formulae for the blast design parameters can be found in TM 5–855–1 [2].

A relationship between distance and the amount of explosive (the charge weight) giving the same explosive effect has been well established by the experimental work of many authorities. This relationship takes the form of a factor, Z, called the proximity factor:

$$Z = r/w^{1/3} \tag{23.1}$$

where r = distance from point of detonation (m) and W = equivalent charge weight of TNT (kg). The proximity factor allows the scaleless comparison of the effects of different weights of explosive charge. This, for instance, permits the results of small-scale tests to be related to full-size weapons. It is a factor essential to the design process where blast pressures and impulses need to be derived.

23.5.2 Human injury from blast

The human body is capable of sustaining some blast without injury but the level of resistance is not absolute. When the blast presssure is around 100 kN/m^2 only 5% of people would be expected to suffer any significant injury directly from the blast. At pressures higher than this greater numbers of people will suffer from burst eardrums. However, at this level of blast people standing up are likely to be thrown down by the pressure wave and may suffer injury as a result.

Blast can, of course, also be capable of tearing away architectural fixtures and fittings. Such items would become missiles in their own right, causing injury if not securely fixed to resist the blast.

23.5.3 Blast above ground/in air

The intensity of the effects from a particular amount of explosive increases with the closeness of the weapon. For blast in free air there are three zones of increasing intensity. They are referred to as:

1. Remote from,
2. Near to, or
3. In contact with

the point on the structure being considered. The proximity factor allows consistent definition of the extent of these zones. The reference documents provide different formulae for the parameters in each.

A detonation is considered to be 'remote' at locations outside the flame region. Measurements of remote blast pressures, and hence the design formulae, are more reliable here. The edge of the flame region may be taken to be at a distance given by $Z = 1.5\,\text{m/kg}^{1/3}$. For example, for a 64 kg charge weight of TNT the extent of the flame region would be

$$\begin{aligned} r(\text{limit of near blast}) &= Z \times w^{1/3} \\ &= 1.5 \times 64^{1/3} = 6\,\text{m} \end{aligned}$$

It is the intense heat from the explosion which affects and enhances the impulse within the flame region. Blast within this region, but excluding the contact case, is considered to be 'near'.

Where values of the proximity factor are less than about $0.4\,\text{m/kg}^{1/3}$ the load from the explosion, if resisted, would exceed the strength of concrete. All detonations within distances given by this proximity factor are therefore treated similarly as 'contact' explosions.

23.5.4 Blast below ground/tamped explosion

Where weapons are able to penetrate below ground level and are fused so that they can explode there, the confining effect of the surrounding soil produces a more powerful effect in the immediate vicinity of the weapon. An explosion so confined is termed a tamped explosion. Military tests have allowed pressure–time relationships to be established which are fairly reliable for design to resist remote tamped explosions. However, unlike explosions in air, the pressures transmitted vary according to the nature of the intervening soils. To take this into account the pressure formulae contain a soils constant. It is required to have soil and water-table survey data for the ground which will surround the structure so that the appropriate constant can be selected. Values of blast parameters for tamped explosions, and soil constants for various soil conditions, can also be found in the reference documents.

When a bomb explodes in earth the explosive gases displace the surrounding soil to form a cavity called the 'chamber of compression'. If the explosion is relatively near the surface the soil above this chamber will break up and be projected into the air, leaving behind a crater. Alternatively, if the pressure of the confined gases is not sufficient to lift the overburden, the intact chamber of compression left is called a 'camouflet'.

Clearly, there can be a situation where the cover of soil is insufficient for the full confining effect to occur and tamping is only partial. This can become very relevant where structures, such as railway constructions, are not purpose designed as shelters. While some European authorities have propounded design values for partial tamping they must be treated with care, as they relate to free-field pressures, i.e. pressure measurements made without the influence of boundaries or constraint. The introduction of an adjacent structural wall provides a constriction which is likely to increase the theoretically reduced pressures. When the design of underground structures is undertaken it is suggested that, in the absence of a careful study of the particular cases, full tamping is assumed at between 1 and 3 m, depending on the size of weapon.

As with blast above ground, the intensity of the effects from a particular amount of explosive detonated below ground increases at a greater rate as the weapon detonates closer to the structure. In this case, however, only two zones of increasing intensity of effect can be defined, namely 'remote' and 'contact'.

Normally, a tamped explosion is treated as in contact with the structure wherever the proximity is less than the radius of the chamber of compression of the camouflet. On average, this will be equivalent to a proximity factor, Z, of $0.53\,\mathrm{m/kg^{1/3}}$. Explosions at greater distances are treated as remote.

23.5.5 Blast in passages and tunnels

Blast released into a passage or tunnel cannot expand outwards. Compared with the reduction when expanding freely in air, it reduces with distance in a tunnel by only a small amount. However, in the context of underground railway entrances, etc. these reductions can be important.

The derivation of pressures in passageways is a very specialist activity. The document TM 5–1300 covers partially confined explosions but only for distances up to one effective diameter from the point of detonation [4]. Based upon experiments carried out in tunnels by the Norwegians [5], the relationship between parameters of a blast wave travelling along a tunnel has been prepared by the author and is given in Figure 23.3. This relationship will allow attenuation and decay of the blast in the straight sections of entrance passageways to be taken into account where appropriate.

At bends or other restrictions blast is attenuated by various amounts. Data on such attenuation, mainly from results gathered during military trenching and tunnelling during the two world wars [3], have been widely published but are now

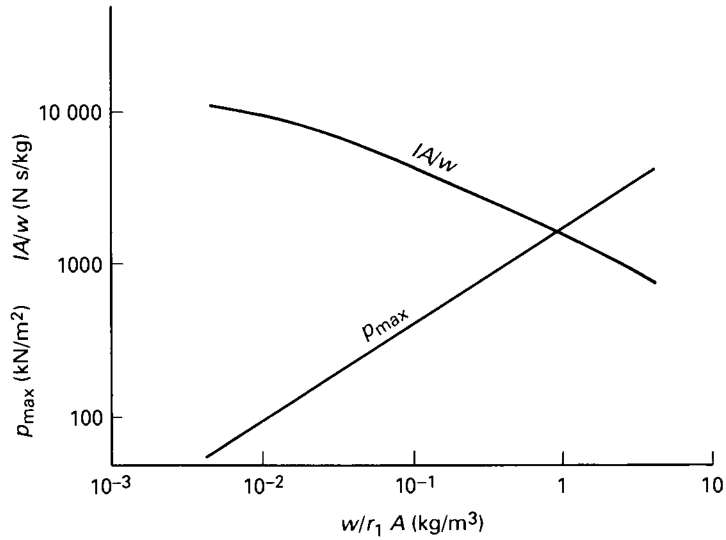


Figure 23.3 Propagation of blast wave in tunnels and passages: chart for derivation of pressures and impulses. p_{max} = peak positive blast pressure; I = positive blast impulse (Ns/m^2); w = charge weight (kg); r_1 = distance from point of detonation (m) measured along centreline of tunnel/passage; A = average cross-sectional area of tunnel/passage (m^2)

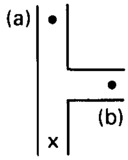
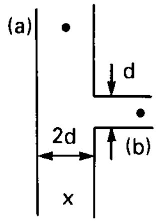
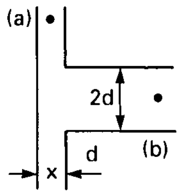
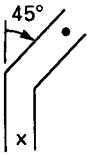
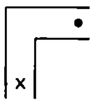
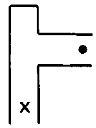
considered to give an overestimate of the decay in some cases and may give unsafe values for the resulting blast. Some typical attenuation values for configurations appropriate to station entrance design have been prepared by the author in the light of more up-to-date information and are given in Figure 23.4.

23.6 Designing the structural resistance

23.6.1 General requirements

Conventional reinforced concrete design and detailing resists static loads. Protective design must anticipate the action of loads from weapons which are beyond the limits of response considered as normal. Outside shapes and special detailing of reinforcement which assist in forcing weapons to ricochet should be considered. Special arrangements of reinforcement are sometimes used to prevent the disintegration of the concrete under deformations which are dynamic and larger than for normal service. One such arrangement, known as the 'Brunswick' method, uses vertical reinforcing links in a slab, while 'laced' reinforcement using diagonal links in both directions can be used where the enhanced resistance warrants the greater complexity. Some concrete mixes with larger aggregates are better than others at resisting penetration. This should be considered when preparing the specifications.

Ductility is to be sought after in the ideal shelter. Design and materials should provide the largest available ductilities. This will be achieved by attention to the balance of the amount of steel reinforcement in relation to the more brittle concrete. To ensure the reserves of strength necessary for relatively large ductile deflections, attention must be paid to the continuity of reinforcement. In addition, the specifications should exclude less ductile steels.

Layout				
p_{max}	1.0	(a) 0.8 (b) 0.4	(a) 0.9 (b) 0.4	(a) 0.6 (b) 0.4
I	1.0	(b) 0.5 (b) 0.5	(a) 0.5 (b) 0.4	(a) 0.45 (b) 0.4
Layout				
p_{max}	0.98	0.95	0.85	0.75
I	0.90	0.85	0.80	0.75

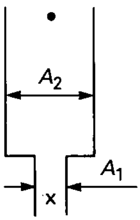
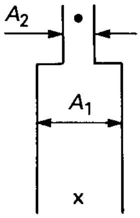
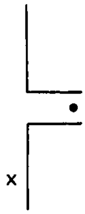
Layout			
p_{max}	$2A_1/A_2$ NB: $A_2 > 2A_1$	1.0	0.4
I	$\left(1-\frac{A_2-A_1}{A_1+A_2}\right)$	$\left(1-\frac{A_1-A_2}{A_1+A_2}\right)$	-

Figure 23.4 Factors for reduction of blast pressures and impulse at bends, expansions, contractions and other attenuating features in tunnels and passages. The table gives comparative factors for reduction of peak positive blast pressure (p_{max}) and positive blast impulse (I) from the point X to the point(s) ◉ indicated in the figures due to turns, expansions, contractions, etc. The tunnels/passages are assumed to continue in either direction with equal cross section except where noted otherwise. Any allowance for reduction due to 'roughness' over the length of tunnel/passage will be additional to reduction given by the above factors

The following sections outline the different types of damaging effect which can occur and the design necessary to combat them. Basic design formulae for a number of the aspects outlined can be found in TM 5–855–1 [2].

23.6.2 Detrimental effects to be protected against

A structure which has not been adequately designed against attack can suffer a number of effects detrimental to it and, therefore, its occupants can also suffer. The most important can be summarized under three headings:

1. *Spalling*, which may be defined as concrete pieces breaking away from the inside face of a shelter structure as a result of the reflection, at that face, of the shock wave of an explosion. The reflected shock wave here is a tensile stress. If these tensile stresses exceed the strength of the concrete it fractures and spalling results. Energy trapped in the particles causes them to be ejected, often with high velocity.
2. *Scabbing*, which may be defined as concrete pieces breaking away from the inside face of a shelter structure as a result of excessive tensile stresses caused by deformation of the wall under dynamic loads from the explosion.
3. *Bulging*, representing a degree of damage lying between the undamaged and the perforated states resulting generally from the intense local effects of a contact explosion.

The objective of protective design must be to reduce these effects to an acceptable level.

23.6.3 Impact/penetration into concrete

One of the characteristics of bombs, shells or missiles is their ability to penetrate. Their explosion would probably be in a confined space and be particularly devastating if they are not kept outside the protected zone at the time of their detonation. A significant part of achieving a satisfactory protective structure is, therefore, making it resistant to impact.

Formulae predicting the extent of impact penetration have been developed by various authorities. These rely on theory allied to observation and testing. The National Development and Research Committee (NDRC) based their formula upon numerous small-scale tests and it is generally accepted as giving the most reliable predictions. Nevertheless, the formula most appropriate to protective design is probably that of the US Army Corps of Engineers. This is widely accepted and gives results which are slightly conservative. It must be realized that penetration tests always show a wide scatter in the results.

A weapon striking a surface at an angle at or near to the perpendicular to the struck surface will achieve the greatest penetration. If the fuse detonates the explosive as the weapon is penetrating the slab the penetration is increased by the explosive effect. However, when the angle to the perpendicular (angle of obliquity) is 20° and more (depending on the speed of impact), ricochet can occur. In such cases the explosive effect is, to a large extent, released outwards away from the surface and not directed towards increasing penetration.

Design formulae give penetrations which assume that the slabs receiving the impact are infinitely thick. There is a critical thickness of slab below which the depth of penetration is influenced by the proximity of the face opposite the one being struck. Evidence of such an influence is when spall appears on this opposite face. Slabs having thicknesses less than this critical thickness are regarded as 'thin', and an appropriate allowance must be made for this influence.

23.6.4 Resistance to air blast

The face of a structure local to a contact explosion will be damaged. Assuming that the structual member has sufficient thickness remaining after the crater is formed in its surface, it will need to carry a peak design pressure applied over a restricted area very local to the explosion. The pressure will, however, be many times larger than the normal crushing strength of the concrete (which is possible because of the confined state of concrete in the structure under the transient effect of the blast).

When an explosion is anticipated at a distance from the structure being designed the values for pressure and duration for the appropriate proximity condition of either near or remote blast are calculated. These will allow a dynamic design of the structure to be carried out. It is possible to achieve structures which will resist the blast without their elastic limits being exceeded. However, such designs are not normally considered to be economic for protective design[6]. Substantial deflections which utilize the effectiveness of in-built ductility in resisting blast are often permitted.

The negative phase of the blast pressure needs to be considered in the design of lighter structures resisting remote explosions. This phase is generally negligible when a structure is being designed to resist a near or contact explosion.

23.6.5 Penetration into soils

The large number of factors make the penetration of bombs and missiles into soil extremely variable. Penetration is affected by the velocity at impact, angle of incidence and weight of the weapon, the shape and strength of its casing, and the nature and uniformity of the soil. Bombs and missiles frequently follow a path through the ground with a 'J' shape similar to that shown in Figure 23.5(a). The significance of the shape of the possible path is that weapons therefore have the capability of penetrating beneath structures to a certain extent.

Some of the most useful data concerning ground penetration of bombs were collected during World War II. Bomb-disposal authorities catalogued the depths at which unexploded bombs were recovered. These data are summarized in the reference documents for different soil types and bomb sizes. The weapons penetrating in this way will have delayed-action fuses. Figure 23.5(b) shows the range of possible maximum path lengths below ground for any weight of weapon. The wide scatter of recorded and experimental data for penetration makes its consideration more suited to an exercise in probability. The relationship between penetration and the permissible risk of damage to the structure from the weapon exploding below ground can be established.

23.6.6 Resistance to tamped explosion

The local effects of tamped contact explosions are particularly devastating. Methods for the design of slabs to resist such local effects are entirely empirical. Damage to the face of the concrete receiving the blast will be inevitable. It is necessary to provide certain minimum thicknesses of wall or slab such that spalling is limited to acceptable levels. Limiting thicknesses of concrete are given in TM5–855–1[2], and these values, used with care, will still give results which are correct for modern applications.

The empirical design criteria in the reference quoted for both contact and remote explosion are for rigid military structures. However, where a structure or its individual members can be assumed to deform, soil–structure interaction and the structure's dynamic response to the explosion can be taken into account. The blast pressure–time relationships will allow dynamic design. In some circumstances the large structures necessary for underground railways will be more economical when

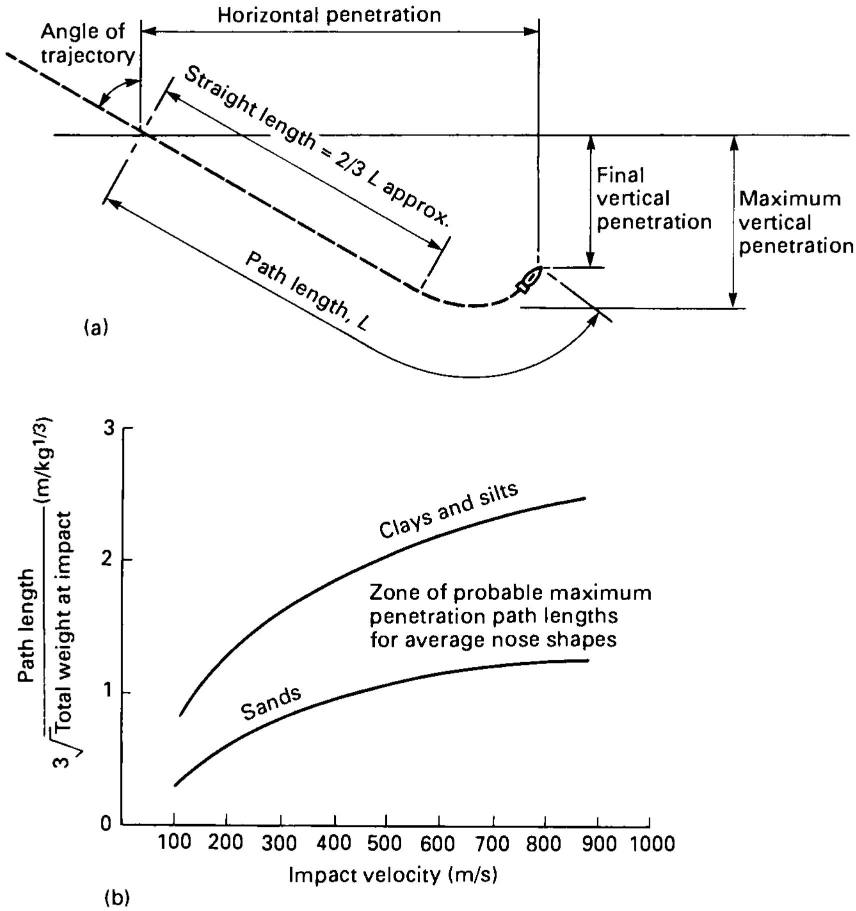


Figure 23.5 Penetration of weapons into soil. (*a*) Typical trajectory below ground level; (*b*) relation between striking velocity and penetration path length for projectiles or bombs of various weights

designed in this way. The amount of movement permitted during this interaction will be a function of the degree of damage which is deemed acceptable.

In seeking the optimum solution the protective value of any burster slab (see Section 23.3.1 and Figure 23.2) can be taken into account. The necessary thickness of wall to resist various proximities of the weapon can be designed (there will also be a solution for the normal service loading which will establish the minimum option). These can then be related to the horizontal penetration of the weapon under the burster slab to give the extent required of the associated burster slab. Figure 23.6 illustrates an example of such a relationship for a particular station wall design.

23.6.7 Elimination of spalling

Where the design thickness of a slab offers a degree of protection which has not eliminated the risk of slight spalling and such spalling is not permissible then

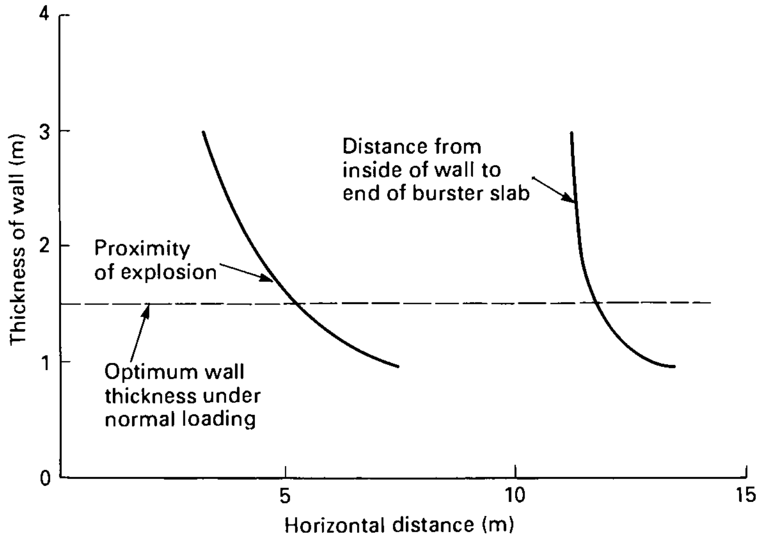


Figure 23.6 Typical relationship between extent of burster slab and thickness of station wall resisting tamped explosion within elastic limits. The relationship illustrated is for an explosion of 150 kg of TNT and a wall of concrete strength 30 N/mm^2 reinforced with compression and tension steel totalling 3.5%

additional measures are necessary. One solution is to provide anti-spall plates on the face furthest from the weapon. These steel plates, usually 3 mm thick, are deeply anchored into the concrete slab and are sufficient then to retain the spalled particles of concrete which would otherwise be ejected from the surface with a damaging high velocity.

An equivalent degree of protection can be achieved by increasing the thickness of the concrete by 15% above that which is designed to limit the damage to slight spalling. With this extra thickness the slab may be expected not to spall under the action of the particular weapon which provides the basis for the design parameters.

23.7 Entrances

23.7.1 General

One way of allowing access into a shelter while maintaining protection to persons already inside is to provide a holding space between two blast-resistant doors. During an air raid the outer door would only be opened to allow entry when the inner door was shut. The outer door would then be closed when arriving people were in the holding space. Only when the outer door was closed, and the safety of persons within the shelter thus ensured, would the inner door then be opened.

However, with the railway to remain in operation, access for the travelling public through two blast doors is likely to be unacceptable. An alternative treatment for the entrances must then be sought. The parameters are in conflict, namely a requirement for ease of passage of people combined with restricted access for blast.

23.7.2 Bomb-resistant entrance example

An example design of a bomb-resistant entrance is shown in Figure 23.7. The point of detonation of the weapon is kept sufficiently distant from the actual mouth of the station concourse by strengthening a length of the passage below ground so the weapon cannot penetrate it. The passage itself has bends and other attenuating features, but these, in this case, are insufficient to reduce the blast to acceptable levels before it reaches the main station concourse. Use of the data in Figures 23.3 and 23.4 show that the total reduction will be only about 75% for this example.

A blast-resistant door has therefore been introduced to protect the concourse area. The alternative passage introduced at the side of the blast door is sufficient to allow continued access but restrictive enough to attenuate blast to a safe level. A further 70% reduction of peak pressures will be achieved, leaving a resultant just 6% of the initial blast.

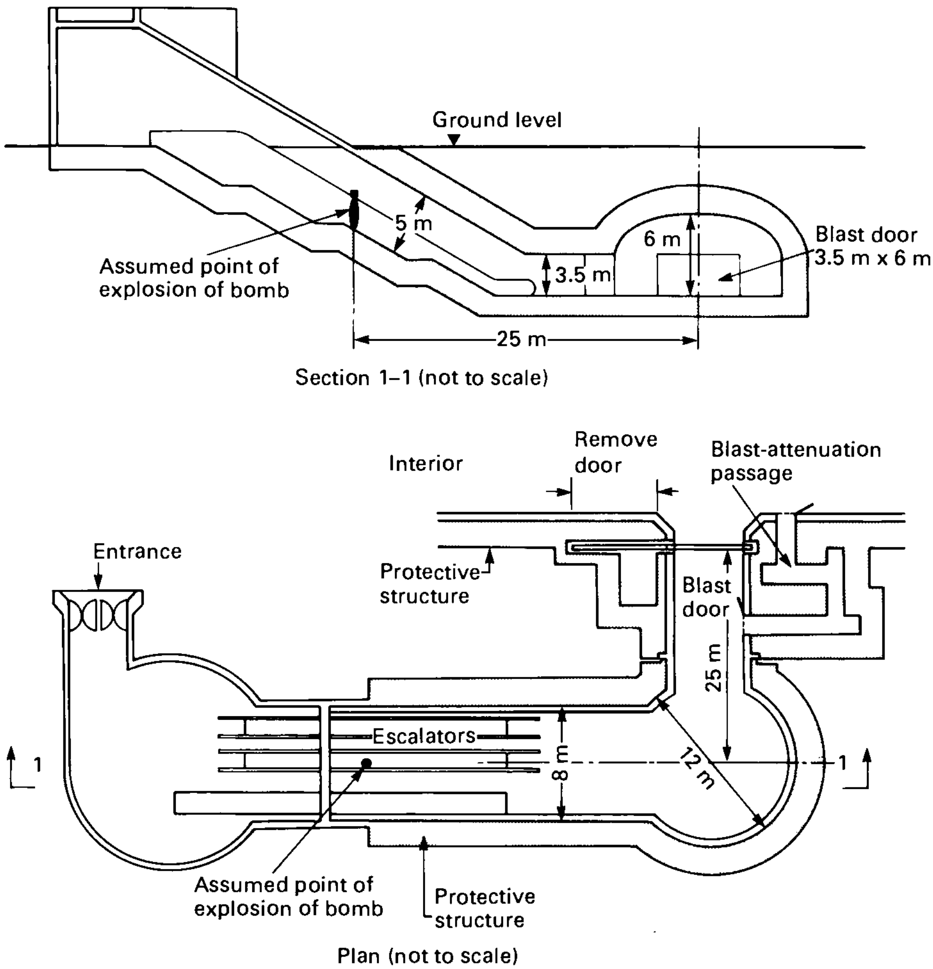


Figure 23.7 Example of arrangement of bomb-resistant entrance

The number of available entrances and emergency exits must be decided on the basis of the particular layout of the station and its anticipated usage. The blast-attenuating passages must be used one way only. Members of the public using them at the time of an air raid may already be in a state of stress and also in a hurry. Excessive risk of their progress being blocked or faltering must be avoided, as this can lead to frustration, lack of confidence, panic and, ultimately, injury.

23.8 Influence of other railway features

23.8.1 Tunnels under rivers

Many major cities are built alongside rivers and underground railways are required to pass beneath them. River crossings warrant particular attention because:

1. A difference in the soil strata is likely to be found in the vicinity of a river, often rendering the tunnels there more vulnerable to damage from weapons;
2. Any breach of a tunnel under a river will lead to a risk that not only soil slurry but the river water itself will find its way directly into the tunnel.

If river water were to flow into a breach in a tunnel the whole of an underground railway system would be at risk.

Additional safeguards may therefore be justified. Tunnels can be shielded by burster slabs (see Section 23.3.1) or retarder slabs, or can have their linings strengthened, though the latter may be very expensive. There may be a possibility of deepening the tunnels sufficiently.

Whatever protective option is adopted, there is always a risk of damage being caused by a weapon of a size which exceeds the design parameters. This can be taken into account in conjunction with the risk of the tunnels under a river being breached by accidental causes. Floodgates can be included in the tunnels at either end of a river crossing to limit the extent of such a catastrophe.

23.8.2 Depots and tunnel portals

It is normal for railway depots to be above ground. Where land is at a premium, developments may be constructed above, but the depot is still likely to be open to air. Any installations in the depot which need protection would be dealt with by normal protective design processes. It must not be forgotten, however, that the ends of tunnels will rise to ground level at depots and, perhaps also at other points on the railway.

The risk of explosion at tunnel portals will need to be taken into account where civil defence is included in a railway design. As discussed in Section 23.5.5, the attentuation of blast in tunnels is small. It is therefore possible that blast waves will propagate through the tunnels with their intensity remaining sufficient to cause damage and injury at, for instance, the next station.

Expansion chambers in the tunnels to provide attenuation of the blast wave can be considered. However, these are likely to pose additional problems in the protection of the chambers themselves. A suitable system of bomb-resistant canopies can provide protection at tunnel portals where considered necessary. These will have the effect of exploding the weapon sufficiently distant from the portal and making allowance for blast waves to dissipate freely into the air. Floodgates may also be required at low-lying portals and these can also provide a degree of blast resistance.

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