6 Health and safety, and risk management in tunnelling

The health and safety of personnel carrying out the construction work as well as the general public is of paramount importance. This chapter introduces this important subject and the topic of risk management.

6.1 The health and safety hazards of tunnel construction

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6.1.1 Introduction

Tunnelling is increasingly recognized as an environmentally friendly way of providing road and rail capacity in an increasingly congested world. However, not all tunnels are of large diameter and the many small diameter water, sewage and cable tunnels which are built every year by the utility providers should not be forgotten. Many hazards are common to soft ground and hard rock tunnels.

The risk to health and safety is not confined to those directly undertaking tunnel excavation as members of the public can also be affected. Over the past few years there have been a number of spectacular tunnel collapses around the world which have resulted in both workers and members of the public being killed. With mechanized tunnelling, the risk to the workforce from ground collapse has largely been removed except for those entering the cutterhead for inspection and maintenance purposes. When a collapse does occur with a shield driven tunnel in an urban area, it is probably those on the surface, likely to be the public, who are at greatest risk.

Tunnels are high value assets both in terms of their intrinsic worth and their value within the national infrastructure. Massive social and disruption costs can arise when a tunnel ceases to be available for operational use such as after a major fire. In these circumstances when the tunnel is no longer fit for operational use, it is almost inevitable that attempts will be made to recover and repair the tunnel and this has safety-related implications for those

involved in the recovery operations. The two major fires in the Channel Tunnel of 1996 and 2008 illustrate this. Consequently the use of appropriate measures to protect the tunnel lining from fire is one aspect that should be considered at the design and construction stage.

6.1.2 Hazards in tunnelling

All the health and safety hazards of normal civil engineering construction can be found in tunnelling along with a few which are specific to tunnelling. In most cases the risks arising from these hazards present more severe consequences in tunnelling. This increase in severity is due to a number of factors including:

- The degree of uncertainty in the nature and variability of the ground through which the tunnel is being driven.
- The confined space of the tunnel environment particularly in small utility tunnels.
- A safety culture at all levels in the workforce which has until recently been poorly developed.
- A lack of commitment from all parties to the project in addressing occupational health and safety.
- Failure by the industry, to learn from the experiences and mistakes of others.
- Work in compressed air.

Comprehensive guidance on the hazards of tunnelling and on mitigating the risks arising from these hazards can be found in the current version of British Standard 6164 'Code of practice for safety in tunnelling in the construction industry' (BSI 2001b).

6.1.3 Techniques for risk management

The management of health and safety risk is no different to the management of other project risks. The hazards which arise in tunnelling should be identified from experience and by reference to relevant technical publications (Lamont 2006). It is rarely necessary to use formal hazard identification techniques, such as hazard and operability studies. Once the hazards have been identified, the risks which could arise should be assessed in terms of their likelihood of occurrence during the life of the project and their consequence. Risk assessment techniques used in tunnelling extend from the use of simple likelihood/consequence matrices to numerical quantified risk assessment techniques.

The use of sophisticated numerical techniques is only possible where appropriate input data exist and this information is not readily available

in tunnelling. It is particularly important to consider the risk of low frequency but high consequence events such as collapse and plan for such events, which in practice happen more frequently than many realize. Risk management is described in more detail in section 6.2.

6.1.4 Legislation, accidents and ill health statistics

Occupational health and safety in tunnelling is normally subject to the same legislation as surface construction. The principal statute in the UK is the Health and Safety at Work Act 1974, which sets out generic goal setting requirements applicable to all work activity. This is supported by the Management of Health and Safety at Work Regulations and industry specific regulations such as the Construction (Design and Management) Regulations. Hazard specific regulations relevant to tunnelling include the Work in Compressed Air Regulations. A comprehensive description of health and safety legislation relevant to construction can be found in Appleby and Lamont (2009). In the author's experience, the UK tunnelling industry has taken occupational health and safety very seriously in recent years and the fatal and major accident rates have reduced significantly. However, although accident statistics are frequently used to measure the effectiveness of the safety management system, they are a poor measure as they are negative indicators, i.e. indicators of failure in the safety system. In addition they are subject to significant error, due to under-reporting of incidents and ill health. In general, few regulatory authorities or contractors publish detailed statistics on tunnelling accidents and ill health. One exception to this is with decompression illness which arises from work in compressed air. Engineers have had an interest in the causes of and cures for decompression illness since compressed air work was first undertaken in the mid-nineteenth century and many have published their thoughts and experiences. The Proceedings of the Institution of Civil Engineers contain a comprehensive selection of relevant papers which were summarized by Lamont (2007).

Most countries have some form of labour inspectorate which is ultimately responsible for occupational health and safety. Some countries also regulate health and safety through statutory social insurance organizations. These link premiums to performance and often provide training along with compensation and rehabilitation services for injured workers. In recent years, the major global re-insurance companies have taken a direct interest in tunnel construction works particularly seeking to reduce their exposure to claims by raising standards of ground risk management in the industry. Initially they worked with the British Tunnelling Society to produce a code of practice for the UK but this document has now been extended to be applicable internationally (ITIG 2006).

6.1.5 Role of the client, designer and contractors

Clients and designers, as well as contractors, have a contribution to make in ensuring good health and safety performance in tunnelling, and their responsibilities are set down in legislation and guidance in many countries. In the UK the Construction (Design and Management) Regulations lay down a statutory framework within which clients, designers and contractors must acquire and share safety related information overseen by a 'CDM coordinator'. Designers have specific duties to consider the health and safety of those affected by the building of a project and those working in the completed structure. The regulations require a 'principal contractor' to be appointed to oversee cooperation and coordination in matters relating to health and safety, between all contractors working on a project.

The client can set a framework for project procurement covering both design and construction, which should include requirements for the health and safety strategy for the project. In addition, the client can ensure that those whom they, directly or indirectly, employ to design and construct the tunnel works, in turn make available adequate resources, including finance and time, in order to address health and safety issues.

Designers can strongly influence health and safety, although in the past they have often shown little inclination to do so. The fundamental safety-critical aspects of tunnel design are diameter, alignment, shaft size/positions and portal location and once these have been fixed, the rest of the design process becomes more one of detail. Examples of how a designer can influence health and safety include choosing an alignment which facilitates the use of a TBM by avoiding a rock/soil interface or routing a tunnel away from, rather than through contaminated land. Where this is not possible, the designer should consider the impact on those building the tunnel and pass on relevant information to the contractor along with advice on risk mitigation measures.

In small utility tunnels the lack of working space contributes directly to the health and safety risk to the workforce, hence the primary factor in determining the minimum tunnel diameter may be the need to provide adequate working space to build the tunnel safely. Designers can use the specification to eliminate techniques or materials which are hazardous and should consider 'buildability' when designing openings in the tunnel and changes of cross section.

It is good practice to include requirements for fire fighting, atmospheric monitoring systems, communication and ventilation systems in the contract documents and to consider how to integrate what is required during tunnel construction with what the client requires in the finished tunnel. It can be argued that during construction these are the contractor's responsibility, as they are part of the temporary works: however, the client pays for them anyway through the contractor's overheads.

Any contractual arrangement which brings together design and construction expertise such as partnering, joint ventures or early contractor involvement can help mitigate risk in the tunnel during construction through sharing experience and improving buildability.

6.1.6 Ground risk

This has the potential to affect the most people in the event of a tunnel collapse. Those affected by a collapse include the client who suffers financial loss, those building the tunnel who are at risk of death or injury and the public who may also be at risk of death or injury. Spectacular tunnel collapses, such as that at Heathrow in 1994 in the UK (HSE 2000), do occur with disappointing regularity, hence engineers should always consider them in their risk assessments and plan their emergency measures accordingly. When they occur, the consequences will most likely be so great that there will be political repercussions in addition to the disruption to the works.

For all tunnel projects, adequate site investigation is essential. The designer must know the geology and hydrogeology in order to adequately address all the risks from the ground. The most comprehensive site investigation possible is required to identify ground parameters, discontinuities, water, gas and contamination (see Chapter 2).

Designers should liaise closely with contractors to ensure the stability of the tunnel under construction. This liaison must go beyond just the stability of the permanent works to include the stability of the tunnel at all stages of construction. In rock tunnels, the stability of the ground through which the tunnel is being driven has to be considered along with the stability of the ground around the tunnel intrados.

Often a primary sprayed concrete lining is classed as temporary works and considered to be the contractor's responsibility, however it is also the primary means by which the tunnel is supported during construction which makes it of fundamental importance for the safety of all those in the tunnel (and also for safeguarding the client's asset). The sequencing of the excavation process, particularly in complex tunnel layouts, can be crucial in ensuring safety. Designers should always ensure that the construction sequence which they envisaged in their design is adhered to.

Contractors should have a proper appreciation of the engineering principles behind the design and ensure they adhere to the design and specification and do not sacrifice quality of materials and workmanship to achieve cost savings and productivity. Quality assurance schemes have a place in tunnelling, but are no substitute for good engineering practice and supervision. It is important to learn from the mistakes of others.

6.1.7 Excavation and lining methods

The method of tunnel excavation can influence safety. In soft ground, most tunnels are now driven by shielded TBMs or a combination of the New Austrian Tunnelling Method (NATM) or sprayed concrete lining (SCL) techniques. In rock, TBMs or drill and blast techniques are normally used (see Chapter 5).

NATM/SCL is an observational method and as such requires considerable engineering input if proper management of the ground risk is to be achieved. Designs must be developed for both the most probable and the most unlikely conditions along with a number of incremental steps in between. Action or trigger limits in terms of relative values, e.g. differential movement between measuring stations, absolute values, e.g. total deflection, and rates of change must be determined. Throughout construction monitoring must be carried out and results compared against the alarm or trigger limits (see section 7.3).

It is very important that contingency plans, whose impact has been predicted in advance, should be in place during construction. It should be possible to put these into effect sufficiently quickly when alarm/trigger limits are exceeded to allow an effective recovery of the situation. In addition emergency plans should be in place which can be put into action when the worst happens and the contingency plans have not been effective.

6.1.8 Tunnel boring machines

Tunnel boring machines have become highly sophisticated but complex machines, and there are many hazards associated with their operation. One of the most hazardous areas of a TBM is the segment build area within which the erector operates. Here, heavy segments are handled whilst visibility for the erector operator can be poor. Miners are expected to place packing between segments as well as bolting up the segments to secure them in position. The risk of serious personal injury is always present.

The power consumption of large TBMs can be between 5 and 10 MW and supply voltages of 11 kV are becoming common. High standards of electrical safety are necessary if electrical accidents are to be prevented. The problems of working in a wet metallic environment, potentially explosive atmospheres, possible oxygen enrichment and compressed air, all add to the complexity of the electrical engineering problems in tunnelling.

TBMs for rock tunnelling are similar in many respects to those for soft ground tunnelling. In addition, self propelled machines such as roadheaders and specialized drill rigs for tunnelling often referred to in the industry as 'jumbos' can be used depending on rock strength. Specially adapted excavators can also be used for certain applications. One result of increased mechanization has been the marked reduction in hand tunnelling and its associated hazards of manual handling, noise, vibration and heat strain.

There are a number of European standards relating to the mechanical and electrical hazards of tunnelling machinery, which meet the requirements of European Directives on machine safety. For example, BSI (2005 and 1997) cover the safety of shielded and unshielded tunnel boring machines respectively. BSI (2002b) sets out requirements for the safety of roadheaders whilst BSI (2002c) relates to airlocks. Manufacturers supplying machinery into the European Community normally certify their machines to meet these requirements. The standards address a wide range of topics such as access to the cutterhead, handling of heavy components, rotation/stability, walkways and access openings, visibility, control systems, hydraulic and electrical systems and fire protection. These standards apply equally over the wide range of machines which are currently manufactured. This range extends from microtunnelling machines of under 1 m diameter to the largest TBMs currently being made of over 15 m diameter. Hence the requirements have to be somewhat general in nature. A separate standard covers airlocks and bulkheads whilst explosion protection of tunnelling machinery is covered by yet another standard.

6.1.9 Tunnel transport

Tunnelling often requires the transport of large numbers of men and considerable quantities of materials over long distances. A railway system is often used in bored tunnels, however wheeled or occasionally tracked plants are used in other tunnels. Much of the plant is of a specialized nature because the restricted space in the tunnel prevents conventional construction plant and vehicles from turning or slewing.

A tunnel is a confined space in which visibility is often poor due to lack of lighting. Consequently, there is a high risk of collision between men and machines which has resulted in a number of fatal and serious injury accidents in recent years. The provision of vehicle and pedestrian routes which are adequately separated and lit, the maintenance of vehicle lights in a serviceable condition and the provision of high-visibility clothing are all important means of mitigating these risks.

Increasingly other methods of removing excavated material from the tunnel are being used. These include slurry systems and conveyors and both give major safety benefits by significantly reducing the number of transport movements required in the tunnel. An added benefit is that neither method utilizes diesel engines, which generate contaminants for the tunnel atmosphere.

6.1.10 Tunnel atmosphere and ventilation

The quality of the tunnel atmosphere is very important and contaminants in the tunnel atmosphere affect everyone working in it. The most common atmospheric hazards and contaminants are oxygen deficiency and the

presence of harmful gases such as carbon monoxide, the oxides of nitrogen and carbon dioxide, and potentially explosive gases such as methane, and radon, which is radioactive. Other atmospheric contaminants include dusts containing silica. None of the atmospheric contaminants can reliably be detected without the use of monitoring equipment. In all cases the risks arising from them should be mitigated by ventilation. Waste heat from plant and equipment also builds up in the tunnel atmosphere and has to be controlled by ventilation.

Frequently the tunnel ventilation systems fail to function effectively due to poor design or maintenance. Ducting can be wrongly positioned and thus fail to supply fresh air to the miners or it may not pass the required quantity of air if it has been blocked or joints and leaks in it have not been sealed.

6.1.11 Explosives

In rock tunnelling, extensive use is made of drill and blast techniques. Specialized tunnel drilling equipment capable of drilling a number of holes simultaneously often under computer control is used. The main hazards are dust, noise and vibration and the risks associated with storing and using explosives. The main risks from using explosives include premature detonation and atmospheric contamination from the dust and blast fume released by the blast.

6.1.12 Fire, flood rescue and escape

Among the most significant safety hazards of tunnelling, to which the workforce is exposed, are fire and smoke. In particular it is the rapid spread of smoke through the tunnel system, rather than radiant heat generated by a fire, which can lead to fatalities. As recent fires in the Channel Tunnel have shown, the tunnel lining can also be severely damaged by fire. In most tunnels under construction, the main sources of fuel for a fire are the large quantities of plastic, rubber and other flammable materials found on plant, and equipment, along with the significant quantities of hydraulic fluid and possibly diesel fuel kept underground. Reduced flammability hydraulic fluids are available and should be used in all underground plant along with flame retardant grease around the TBM. All hydraulic systems should be well engineered.

Equally important is the need for effective fixed onboard fire suppression systems on all plant and equipment. These should be supplemented by handheld extinguishers and a fire main with hydrants and hose reels in the complex tunnels. Fixed systems have the advantage of allowing everyone to evacuate the tunnel and not requiring someone to remain in a position of danger to fight a fire. Good housekeeping is another vital precaution in minimizing the build up of flammable rubbish, which typically in tunnelling includes timber, plastic bottles, paper, discarded hoses and cables.

In all tunnels there should be an underground alarm system as well as one or more communication systems. In large or complex tunnels these should be linked into the main tunnel control systems along with a comprehensive fire detection system.

It is normal practice to issue oxygen self-rescuers to everyone going underground. These should be worn on the belt in order to be readily accessible in an emergency.

In every tunnel there should be adequate arrangements for escape and rescue. These can either be based on a team made up from the contractor's own work force or from the local fire and rescue service. Sometimes it is in the contractor's interest to provide the local emergency rescue services with specialized equipment such as long duration breathing apparatus. To facilitate escape and rescue, a clear and well signed walkway should run throughout the length of the TBM and from its outbye end to a place of safety. This place of safety can be on the surface or in a so called 'rescue chamber' underground. In very long tunnels a dedicated emergency train may be required.

6.1.13 Occupational health

Occupational health is seldom allocated the priority it should be, given the number of days lost to ill health. The over-riding principle of occupational health to which all industry should subscribe is that 'no one should arrive home from work less healthy than when they left home to go to work'. The reasons for occupational health provision are two-fold:

- to address ill health due to work;
- to ensure fitness for work.

Most of the occupational health hazards of construction resulting in ill health are also present in tunnelling. They include dermatitis from the use of cementitious materials, serious respiratory problems from exposure to dust, hand-arm vibration syndrome, noise induced hearing loss and severe musculoskeletal injury.

As an example of ensuring fitness for work, no one can work in compressed air without first undergoing a medical examination to ensure their fitness for such work. Thereafter routine medical checks are required at intervals of 3 months or 28 days depending on the pressure to which they are exposed. Fitness for work can also be a safety issue, e.g. checking the eyesight of plant operators or locomotive drivers to ensure their vision is adequate.

People suffering from ill health are often no longer able to work and therefore many cases of occupational ill health go unrecorded ('healthy worker effect'). In an industry where peripatetic workers make up a

significant proportion of the workforce, even more cases of ill health than for general construction may be going unreported. As tunnelling workers often have no access to regular health care when living away from home, the provision of occupational health facilities and even basic general health facilities becomes even more important.

6.1.14 Welfare and first aid

The provision of basic welfare in tunnels under construction is improving. Space for basic toilet and washing facilities is limited in small tunnels, but in larger tunnels there is enough space for toilet and washing facilities on the TBM or in the tunnel. A system for cleaning and maintaining the toilets is essential. The poorer the toilet facilities, the greater the need for hand cleaning facilities. Research has shown that significant reductions in the number of cases of minor ill health can be made by providing basic welfare facilities. In addition, messing facilities, with a supply of cold potable water are also required. A means of boiling water and heating food as part of the TBM equipment aids welfare and reduces the risk from improvised electrical installations. First aid provisions must be available to meet the requirements of the project in terms of shift working and remote working.

6.1.15 Work in compressed air

Compressed air working was first introduced in the mid-nineteenth century and has been a useful groundwater control technique ever since. The main occupational illnesses arising from it in tunnelling are decompression sickness and aseptic bone necrosis – collectively referred to as 'Decompression illness'. The normal symptoms of decompression illness are joint pain of varying severity or occasionally neurological symptoms. It is readily treatable by recompression and slow decompression back to atmospheric pressure. Osteonecrosis, which results in the breakdown of joint surfaces and causes disability, is only treatable by surgical replacement of the affected joints.

The era when large soft ground tunnels below the water table, such as the Dartford and Clyde Tunnels, were hand dug by miners working in compressed air has passed. Although such working practices have virtually ceased there is still a small legacy of bone necrosis cases from earlier exposure. TBMs, such as slurry machines and earth pressure balance machines, require the application of compressed air within the cutterhead to facilitate face inspection and cutterhead maintenance. Whilst the number of exposures has probably been cut by over 95% compared to hand digging the tunnel, some work under pressure is still required. There is also a trend for tunnels to be dug ever deeper in more challenging geological conditions, thus increasing the working pressures.

With air-only decompression, the incidence of decompression illness can exceed 2% for some pressure/time combinations. This is unacceptable for the twenty-first century. Many countries including the UK, have now adopted routine oxygen decompression to reduce the incidence of decompression illness. Overall, however, the compressed air tunnelling industry lags far behind the diving industry in its hyperbaric engineering practices. In a very small number of tunnels, saturation techniques as in offshore diving, requiring the use of mixed gas breathing, have been used and working pressures of over 10 bar are being required.

Not only does hyperbaric working present a health hazard, it also presents a safety hazard as fire risk increases directly with increasing atmospheric pressure. Oxygen leakage during decompression raises the oxygen concentration and results in an even higher risk. Work at pressure also leads to increased risk from heat strain and can also exacerbate exposure to contaminants.

The Health and Safety Executive provides extensive guidance on compressed air working in its guidance document HSE (1996b). Further information can also be found in Lamont (2007).

6.1.16 Education, training and competence

The traditional image of tunnel workers, and one which they have been reluctant to change, is of a hard working, hard living, macho culture. Concern for one's safety and health has not been a priority. It is vital if health and safety standards are to be improved, for more resource to be put into raising the general competence of all those in the industry as well as their competence in health and safety matters. There are training initiatives in some countries for tunnel operatives and first line supervisors, and the number of universities offering postgraduate courses in tunnelling is growing. Competent supervision of tunnelling works is vital and front line supervisors play a key role in fostering greater awareness of health and safety issues amongst the workforce. Large tunnelling projects may need to set up their own training facilities, for example CrossRail in the UK set up its own 'Tunnelling Academy' to train the large number of workers required for this project.

All new employees in the industry should undergo comprehensive induction training. Site-specific training, even for experienced employees who are new to a site, is also necessary. Engineers and managers now undertake training in health and safety matters as part of their professional education and continuing professional development. This training extends beyond what is required for personal safety to what is required to ensure the safety of those affected by their professional activities. Many national and international tunnelling organizations provide training materials and courses.

6.1.17 Concluding remarks

There is still scope for improvement in standards of health and safety in tunnel construction. Experienced practitioners should share knowledge, guidance and good practice with those entering the industry. Good standards of health and safety require the commitment of resources in terms of time and money but a productive workforce can only be sustained if working conditions are healthy and safe. Respect for people through respect for their health and safety must be our goal in the twenty-first century.

6.2 Risk management in tunnelling projects

6.2.1 Introduction

This section provides a brief introduction to the concept of risk management in civil engineering and particular tunnelling projects. This can only be regarded as an overview and the reader is encouraged to consult more detailed reference materials such as Eskesen *et al.* (2004), Clayton (2001), and also the Code of Practice for Risk Management (ITIG 2006). A Technical Guidance Note on Geotechnical Risk Management for Tunnel Works has also been produced by the Geotechnical Engineering Office of the Hong Kong Government (TGN25 2005).

The concept of risks and risk management is not new in construction, and back in 1993 Sir Michael Latham stated when reporting on construction procurement methods for the UK Government, 'No construction project is risk free. Risk can be managed, minimized, shared, transferred, or accepted. It cannot be ignored.' With respect to tunnelling operations, the late Sir Alan Muir Wood stated 'Uncertainty is a feature that is unavoidable in tunnelling. But it can be understood and controlled so that it does not cause damaging risk.'

Therefore the management of risks within a tunnelling project is vital to ensure a successful project. As Sir Alan Muir Wood mentioned, uncertainty in tunnelling is unavoidable. One aspect of uncertainty in tunnelling can be attributed to the ground, which, as described in Chapter 2, is characterized from laboratory and field tests conducted on only a very small proportion of the total ground affected by the tunnelling operation. Generally, the ground parameters are given a range of values and hence the risk management is important, taking into account the best and worst case scenarios as well as values in between. However, it is not only uncertainty, but the hazards that are involved in the overall tunnelling project which need to be considered in any analysis.

In order to evaluate risk, an understanding of the difference between hazard and risk is important, as well as other useful definitions given by Eskesen *et al.* (2004):

- *Hazard* – A situation or condition that has the potential for human injury, damage to property, damage to environment, economic loss or delay to project completion.
- *Risk* – A combination of the frequency of occurrence of a defined hazard and the consequences of the occurrence.
- *Risk acceptance criteria* – A qualitative or quantitative expression defining the maximum risk level that is acceptable or tolerable for a given system.
- *Risk analysis* – A structured process which identifies both the probability and extent of adverse consequences arising from a given activity. Risk analysis includes identification of hazards and descriptions of risks, which may be qualitative or quantitative.
- *Risk assessment* – Integrated analysis of risks inherent to a system or a project and their significance in an appropriate context, i.e. risk analysis plus risk evaluation.
- *Risk elimination* – Action to prevent risk from occurring.
- *Risk evaluation* – Comparison of the results of a risk analysis with risk acceptance criteria or other decision criteria.
- *Risk mitigation measure* – Action to reduce risk by reducing consequences or frequency of occurrence.

The risk management strategy for a project must consider all aspects, from the design life, durability and repair and maintenance of a structure. It is important to realize that the opportunity to minimize risks is highest during the early feasibility stage of a project and this opportunity decreases rapidly once the project moves into the design and construction stages, as illustrated in Figure 6.1. The cost of change also increases substantially the further one gets into the project.

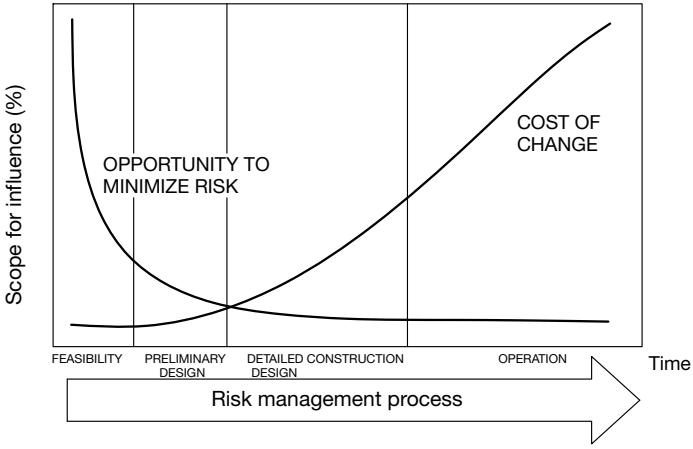


Figure 6.1 Risk management and impact versus time (after Caiden 2008, used with permission)

Although risk management is often regarded as negative, it should also be seen as providing an opportunity for doing things better. Hence, the following elements should form part of a risk management procedure:

- provision of an auditable framework to effectively identify, analyse, evaluate and treat risks on projects;
- ensuring the correct people with the most subject knowledge are involved;
- keeping budgetary and programme creep under control by pre-empting problems;
- ensuring insurability;
- provision of the necessary checks and balances to satisfy financiers or funding agents.

The process of risk management involves a number of steps which can be illustrated as shown in Figure 6.2. Figure 6.2 indicates that risk management is not a linear process and several aspects have to be considered. The key steps that need to be processed are: identification of the risks; assessment of the risks; and addressing the risks. It is important to realize that the risks include political, financial, legal, regulatory, contractual, technical and operational, i.e. are not just restricted to the actual construction operation. When assessing the risks and trying to understand these, it is essential to identify: the potential hazards/impacts; potential consequences; likelihood of occurrence; data/information sources; interested and affected parties; uncertainty, variability and unknowns. Once the risk analysis has been carried out, the risks need to be addressed and either accepted, avoided,

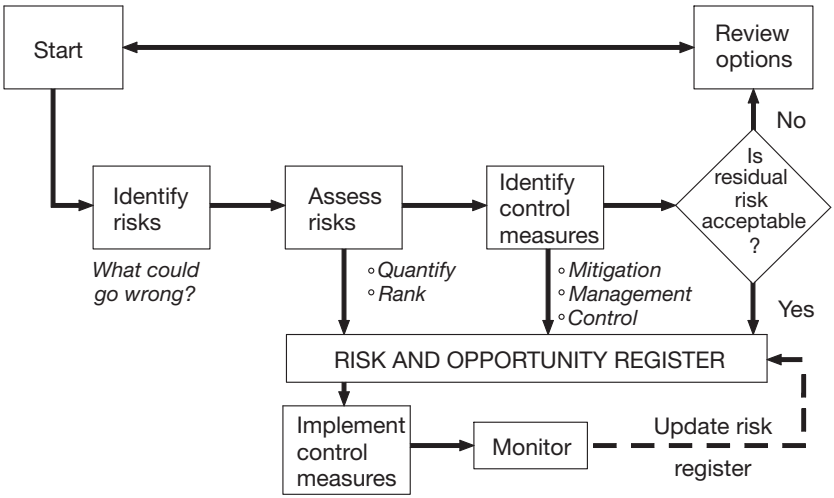


Figure 6.2 Risk and opportunity management flowchart (after Caiden 2008, used with permission)

mitigated or transferred. Further, it is critical that the risks are monitored during all stages of the project including for example cost estimates, labour issues, site and weather. Ultimately, it is important to react to any risks that may occur.

6.2.2 Risk identification

The process of risk identification may rely upon: (a) a review of worldwide operational experience of similar projects with written submissions from partner companies, (b) the study of generic guidance on hazards associated with the type of work being undertaken, and (c) discussions with qualified and experienced staff from the project team and other organizations around the world. It is important to identify the potential hazards in a structured process (Eskesen *et al.* 2004).

The identification and classification of the risks is best carried out through brainstorming sessions with risk screening teams consisting of multidisciplinary, technically and practically experienced experts guided by experienced risk analysts. The aim should be to identify all conceivable hazardous events threatening the project including those risks of low frequency but high possible consequence.

In section 6.2.1 it was mentioned that uncertainty in a project is one significant contributor to risk (although not the only one), with the uncertainty related to the ground characteristics being one of the key contributing factors. However, examples of other common areas of uncertainty affected by any civil engineering project include political and economic environment; planning, regulatory and approvals procedures; environmental and sustainability requirements; construction and buildability issues; safety; project delivery and implementation requirements.

Once the risk has been identified, it is important to register this risk properly. This is particularly important during the construction process. The *Risk Register* provides current details on identified risks and opportunities. Without a formalized mechanism for registering risks, it would not be possible to keep a track record of any risks that have occurred and mitigate the risks. For the mitigation of the risk it is important to nominate a person(s) responsible for the specific risk treatments and associated timeframes for implementation.

6.2.3 Analyzing risks

Risks can be analysed by *qualitative* and *quantitative* methods. Qualitative based analysis is generally a word-based process and involves setting priorities and is used as a decision-making tool. Qualitative analysis is an essential pre-requisite to quantitative analysis.

Quantitative analysis is number based and often involves probabilistic analysis techniques such as those developed in the 1990s (for example for the Adler tunnel, Einstein *et al.* 1994). It can provide an aggregate view of

risks and is focussed on targets and contingencies. This can be conducted using Monte Carlo type simulations, which involve fitting probability distributions to the risks according to surrounding conditions of the variable and then running a large number of analyses taking random variables from each of these probability distributions to assess the overall risks (Rubinstein and Kroese 2007). There is also scheduling and estimating software available based on methods such as fault, event or decision tree analysis or multirisk analysis. However, all these quantitative analysis methods rely on input parameters and hence the expertise of the person doing the analysis.

6.2.4 Evaluating risks

When evaluating the risk, it is critical to look at the identified risks and determine the frequency that these may occur. Table 6.1 shows an example of what type of risk frequency can be utilized.

Once the frequency of a risk has been identified, the consequence of such a risk needs to be determined, which could range from insignificant to catastrophic (Table 6.2). It is now important to develop a matrix to determine a risk rating (for example from low to extreme). This can be achieved qualitatively by combining the likelihood of an event occurring with the resulting consequences. An example of such a matrix is shown in Table 6.2. It should be noted that Table 6.2 is only an example and the likelihood-consequence matrix has to be developed for each project.

6.2.5 Risk monitoring and reviewing

After identification of the risks, their likelihood of occurring and the consequence of them occurring, it is vital to determine various ways of treating these risks. It is always desirable to avoid the risk. Options here include changing the project plan/scope of the works to eliminate the risk. However, in the event that this is not possible, ways have to be considered to mitigate

Table 6.1 Example of risk evaluation (after Caiden 2008, used with permission)

Descriptor	*Description of frequency*
Rare	May occur only in exceptional circumstances – can be assumed not to occur during the period of the project (or life of the facility)
Unlikely	Event is unlikely to occur, but it is possible during the period of the project (or life of the facility)
Possible	Event could occur during the period of the project (or the life of the facility)
Likely	Event likely to occur once or more during the period of the project (or life of the facility)
Frequent/ almost certain	Event occurs many times during the period of the project (or the life of the facility)

Table 6.2 Example of qualitative risk rating (after Caiden 2008, used with permission)

		Consequence				
		Insignificant	*Minor*	*Moderate*	*Major*	*Catastrophic*
Probability (likelihood)	Rare	Low	Low	Low	Medium	Medium
	Unlikely	Low	Low	Medium	Medium	High
	Possible	Low	Medium	Medium	High	High
	Likely	Medium	Medium	High	High	High
	Frequent	Medium	High	High	Very high	Extreme

the risk with either a reduction in probability of occurrence or reduction in consequence if something happens. If this is not possible, then the option of risk transfer or sharing needs to be considered. Finally, if all else fails, the risk needs to be accepted, but a contingency needs to be created. This option involves agreeing to accept the consequences of a risk. It is important that accepted risks are considered carefully and the consequences of these risks are prepared for in advance and appropriate contingency or fallback plans developed. It is also important that these risks are continually monitored carefully and reviewed (after Caiden 2008).

EXAMPLE

The risk assessment carried out as part of the Piccadilly Extension at T5, London UK included: confined working space, hot works, use of electrical equipment, working at height, plant operations, manual handling and lifting, dust, noise, tunnel construction, COSH survey, shotcreting overhead and others (details of this case history are provided in section 8.2). Each of these were analysed with respect to the hazard they posed, for example the hazards related to tunnel construction in this case were:

- excavation;
- face collapse;
- vault collapse;
- unforeseen ground conditions;
- application of sprayed concrete;
- failure of sprayed concrete due to insufficient strength;
- excessive deformations causing failure of the lining.

Each of these hazards were analysed, identifying the level of risk (high or medium), and the persons at risk, which included employees, the public, subcontractors and others. Control measures were established to address these key issues. Examples of the control measures for the excavation hazards included applying an initial layer of sprayed concrete to the face, using an inclined face where possible and monitoring ground conditions.

Table 6.3 Example headings for risk assessment table

No.	Operation	Hazard	Persons at risk				Risk (H/M)	Key issues addressed (control measures)	Residual risk (H/M/L)
			Employees	The public	Sub-contractors	Others			

Control measures for the application of sprayed concrete included having quality control measures in place, systematic testing to assess strength gain, monitoring of the structure and establishment of trigger levels. Based on this analysis and the control measures, the residual risks could be determined, which were assessed as high (H), medium (M) or low (L). Assuming all the control measures were implemented correctly and all the key issues addressed, the residual risk should be low in all cases. The headings used in the risk assessment table are shown in Table 6.3.

7 Ground movements and monitoring

This chapter introduces the topics of how to estimate ground movements caused by the construction of tunnels in soft ground, the importance of these ground movements with respect to their effects on adjacent structures and how these movements are monitored during the construction process. In addition, the important topic of how assessment of the stability of the tunnel during the construction for open face tunnels, such as NATM, via in-tunnel monitoring is introduced.

7.1 Ground deformation in soft ground

When tunnelling in hard ground (rock), ground movements are not normally a problem, except in squeezing ground conditions, and ground movements propagating up to the ground surface as a result of the excavation are unlikely unless the cover depth of the tunnel is relatively small, i.e. in portal areas, or where the groundwater in the overlying soft ground may be affected. In soft ground, however, displacements can occur due to a number of reasons and these are shown for a shield tunnel on Figure 7.1. These components are (after Mair and Taylor 1997):

1 deformation of the ground towards the face due to stress relief;
2 radial ground movements due to the passage of the shield, possibly due to an overcutting edge (bead) used to help steering, or whilst trying to maintain alignment of the shield (pitching and yawing angles);
3 tail void due to the difference in diameter of the tail of the shield and the installed lining, and hence the tendency for ground to move into this gap;
4 distortion of the tunnel lining as it starts to take the ground loading;
5 time dependent consolidation in fine grained soils.

Component 1 is particularly important with open face tunnelling methods. However, if TBMs with pressurized faces, such as EPB and slurry TBMs, are used this component can be negligible if good face control is achieved. It should be noted that over-pressurization at the face can lead

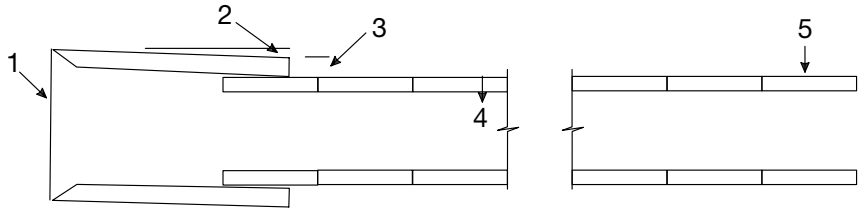


Figure 7.1 Primary components of ground movements associated with shield tunnelling (after Mair and Taylor 1997, from Cording 1991, used with permission from Professor E.J. Cording)

to outward movements and heave at the ground surface. 'Ravelling' or re-compaction due to local loss of ground at the face can also contribute to this component. *Component 2* can result if there is difficulty keeping the tunnelling shield on the correct alignment, or if there is a need to tilt the shield up slightly to prevent it from diving into the ground. *Component 3* can be minimized by immediate grouting of the void. *Component 4* is usually small compared to the other components once the lining ring is completed. *Component 5* can be important for soft clays, and results from the fact that the construction process changes the stress regime locally around the tunnel. This causes changes in the water pressure within the pores between the soil particles. As these excess pore water pressures equilibriate over time the ground will change volume and consolidate (see section 3.2 for more information on stresses around tunnels). It should be noted that when tunnels are constructed with no shield, for example NATM, components 1, 4 and 5 are still applicable.

These components can result in displacements reaching the ground surface, which can be particularly significant in urban areas, where they can influence overlying or adjacent structures such as buildings, other tunnels and services. In contrast, if there are no ground-structure-interaction effects, these ground movements are termed 'greenfield' movements.

It is important to estimate these ground movements so that tunnelling techniques can be optimized in order to control the movements of overlying or adjacent structures. In addition, other measures can be implemented to control these movements, for example the use of compensation grouting. These ground movements can be estimated using numerical methods, as described in section 3.6, or semi-empirical methods as described below.

7.1.1 Surface settlement profiles

Although these days enormous advances in computer based numerical methods for calculating ground displacements are being made, there are still some advantages of using simple empirical based methods in soft ground (Devriendt 2006):

- these methods allow a rapid initial appraisal of ground displacements and can use established risk assessment criteria;
- they provide a conservative risk assessment of the potential damage to structures;
- for 'flexible' structures such as long masonry walls at the ground surface, interaction effects may be minimal and hence semi-empirical approaches based on assuming greenfield conditions can give realistic results.

A number of reviews have been conducted of this subject, for example Mair and Taylor (1997), BTS/ICE (2004) and ITA/AITES (2007). However, a brief overview of the subject is provided in this section.

Schmidt (1969) and Peck (1969b) established, via case history data, that the ground surface settlement 'trough' above tunnels, i.e. normal (or '*transverse*') to the direction of the tunnel, can be described by an inverted normal probability (or 'Gaussian') curve (equation 7.1 and Figure 7.2).

$$S(y) = S_{max} \exp\left(-\frac{y^2}{2\,i^2}\right) \tag{7.1}$$

where S(y) is the vertical settlement at point y, S_{max} is the maximum settlement directly above the tunnel centreline, y is the transverse horizontal distance from the tunnel centreline of the trough, and i is the trough width parameter, which represents the point of inflection on the transverse profile, equivalent to one standard deviation in a normal probability distribution. This has subsequently been confirmed by numerous authors from other case history data, for example O'Reilly and New (1982) and Attewell *et al.* (1986).

By integrating equation 7.1, the volume of the surface settlement trough (per metre length of tunnel), V_S, can be approximated by equation 7.2.

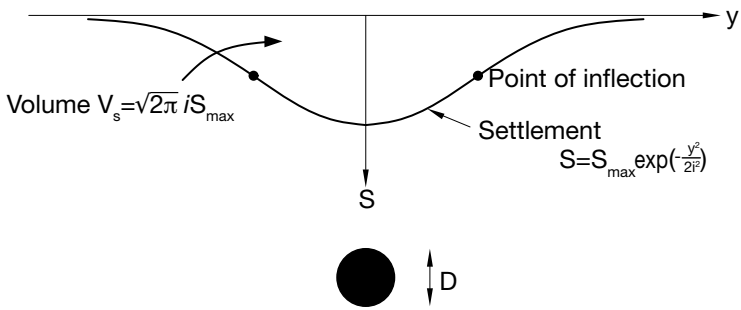


Figure 7.2 Gaussian curve for representing the transverse settlements above a tunnel in soft ground (after Dimmock and Mair 2007a, used with permission from Thomas Telford Ltd and Professor R.J. Mair)

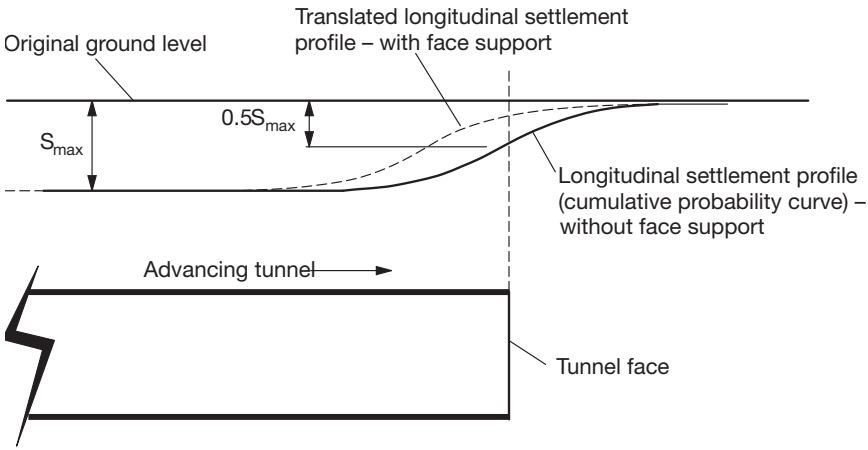


Figure 7.3 Longitudinal settlement profile above tunnels in soft ground, showing the difference in distribution for open face tunnelling and where there is significant face support (after Mair and Taylor 1997)

Equation 7.2 can be rearranged to calculate the maximum vertical settlement, S_{max}, directly above the tunnel.

$$V_S = \sqrt{2\pi}\, i\, S_{max} \tag{7.2}$$

The geometry of the settlement trough is uniquely defined by selecting values for the volume, V_S, and the trough width parameter, i. The choice of these values is discussed later in sections 7.1.1.1 and 7.1.1.2.

In the *longitudinal* direction to the tunnel construction, it has been found that the vertical displacements can be estimated, following examination of a number of tunnel construction case histories in clays (Attewell *et al.* 1986, Attewell and Woodman 1982), by a 'cumulative probability curve' as illustrated in Figure 7.3. For tunnels constructed in stiff clays without face support, the surface settlement directly above the tunnel face corresponds to $0.5S_{max}$. For tunnels in soft clays with face support, for example in EPB or slurry shield machines, the surface settlement directly above the tunnel face is much less than $0.5S_{max}$. In these cases, the major source of the ground movement is further back from the face and this leads effectively to a translation of the cumulative curve (Ng *et al.* 2004).

The *transverse* and *longitudinal* ground displacement profiles can be combined to represent the full three-dimensional surface displacement as shown in Figure 7.4.

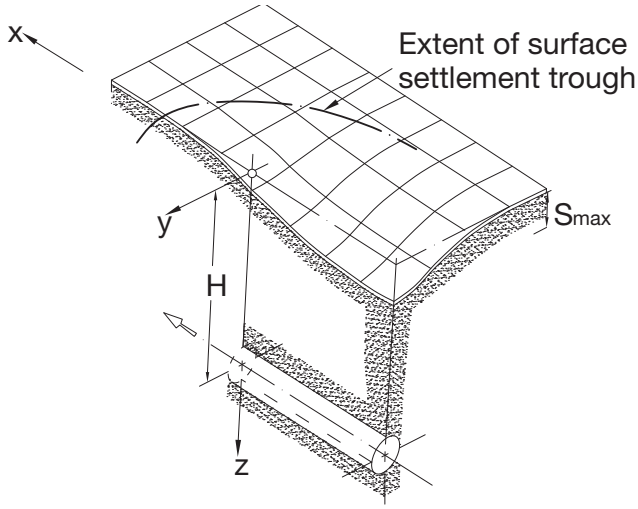


Figure 7.4 Three-dimensional representation of the surface settlement as a tunnel is constructed in soft ground (Attewell 1995, after Yeates 1985)

7.1.1.1 Estimating the trough width parameter, i

There are a number of empirically derived methods for estimating the trough width parameter, i. It has been shown by various researchers on the basis of case history data, for example O'Reilly and New (1982), that the trough width parameter at the ground surface is an approximately linear function of the depth of the tunnel, H, and is largely independent of the tunnel construction method and tunnel diameter (except for very shallow tunnels where the tunnel depth to diameter ratio is less than one). Therefore, the relationship shown in equation 7.3 can be used.

$$i = KH \tag{7.3}$$

where H is the depth from the ground surface to the tunnel axis level, and K can be estimated as shown in Table 7.1

Table 7.1 Typical K values

Soil type	*K*
Stiff fissured clay	0.4–0.5
Glacial deposits	0.5–0.6
Soft silty clay	0.6–0.7
Granular soils above the water table	0.2–0.3

In the urban environment there are often situations where tunnels are constructed close to existing subsurface structures and hence there is a need to estimate the settlements below the ground surface. Mair *et al.* (1993) analysed subsurface data from various tunnel projects in stiff and soft clay, together with centrifuge model test data in soft ground. They showed that subsurface settlement profiles can also be reasonably approximated in the form of a Gaussian curve in the same way as surface settlement profiles. For subsurface regions up to one diameter away from the tunnel, at depth z below the ground surface and above a tunnel at depth H, the trough width parameter can be expressed as shown in equation 7.4.

$$i = K(H - z) \tag{7.4}$$

where z is the depth from the ground surface to the level being considered and K is given by equation 7.5. Equation 7.5 yields shallower, wider and more realistic subsurface settlement troughs at depth compared to those obtained when K is kept constant.

$$K = \frac{0.175 + 0.325\,(1 - z/H)}{(1 - z/H)} \tag{7.5}$$

It should be noted that only one method of estimating the subsurface trough width parameter is presented here and other methods exist.

In the previous equations presented in this section, the assumption is that the ground is homogeneous. However, tunnels are often constructed in layered ground comprising fine and coarse grained soils. Selby (1988) and New and O'Reilly (1991) suggest that the trough width parameter for the surface settlement trough could be estimated from the trough width factor K for each layer and the relative thicknesses of each layer. Hence, for a two layered ground, i would be calculated as shown in equation 7.6.

$$i = K_1 z_1 + K_2 z_2\ (+\ldots) \tag{7.6}$$

where K_1 is the trough width factor for soil type 1 of thickness z_1 and K_2 is the trough width factor for soil type 2 for thickness z_2. Field evidence suggests that for sands overlain by clays wider surface settlement profiles are obtained than if tunnels were in sand alone (Ata 1996 and Atahan *et al.* 1996). However, there is less evidence for coarse grained soils overlying fine grained soils. For example, Grant and Taylor (1996) used centrifuge physical model tests to investigate the ground displacements when soft clay is overlain by a sand. In this case they found that the trough was wider than would be expected if there was only soft clay and does not reflect the narrowing predicted by equation 7.6. This is probably due to the overlying sand layer being significantly stiffer than the soft clay (Mair and Taylor 1997).

7.1.1.2 Volume loss

The volume of the surface settlement trough, V_S, must be estimated, together with the trough width parameter, in order to determine the magnitude of the settlements. This trough volume derives from the various short-term components for why the ground movements develop around tunnels during construction, as listed in Figure 7.1, components 1 to 4. These occur mainly in the region close to the tunnel and the term 'volume loss', V_t, (sometimes referred to as ground loss), is used to describe the accumulation of these components, i.e. the volume of 'lost' ground that can propagate up to the ground surface causing the surface settlement trough.

The choice of the volume loss value is fundamental to all the methods of estimating tunnelling displacements. Volume loss can be defined as the ratio of the estimated volume 'losses' (V_t) over the excavated volume of the tunnel (V_o). It is usually defined in the two-dimensional sense as a percentage of the excavated face area, i.e. volume per metre length of tunnel (equation 7.7). If the tunnel is circular then $V_o = (\pi D^2/4)$, where D is the tunnel diameter.

$$V_l(\%) = V_t/V_o \cdot 100\% \tag{7.7}$$

When tunnelling in drained conditions, for example in coarse grained soils such as dense sands, the volume of the surface settlement trough V_S is less than V_t because of volume changes that occur within this type of ground as it moves (Cording and Hansmire 1975). However, when tunnelling in a fine grained soil, such as clay, the ground movements usually occur undrained (constant volume) and hence $V_S = V_t$.

The selection of the volume loss value (V_l) is based on engineering judgement and experience from previous projects in similar ground, or where similar tunnelling techniques were used. Various authors suggest possible values in different ground conditions and/or for different tunnelling methods (O'Reilly and New 1982, Ng *et al.* 2004). Mair and Taylor (1997) concluded the following based on projects conducted at that time:

- volume losses in stiff clays such as London clay using open-face tunnelling are generally between 1% and 2%;
- recent project in London clay using sprayed concrete linings can produce volume losses between 0.5% and 1.5%;
- EPB and slurry machines can achieve a high degree of settlement control, particularly in sands with volume losses as low as 0.5%. In soft clays, short term volume losses of only 1–2% have been reported;
- in mixed face conditions volume losses may be higher for EPB and slurry machines.

As shown on many recent tunnelling projects involving TBMs, if good control of the face pressures can be achieved, volumes losses of less than 0.5% are achievable. On the Channel Tunnel Rail Link (CTRL) project through London in the UK, volume losses of less than 0.5% were commonly recorded using EPBMs (ITA/AITES 2007). Keeping the volume loss small means smaller ground displacements (refer back to equation 7.2) and hence less impact on overlying and adjacent structures (see section 7.2).

Dimmock and Mair (2007b), after work by Macklin (1999), proposed a relationship between volume loss and load factor (LF) for tunnels constructed in overconsolidated clay. This relationship, shown in equation 7.8, was derived empirically from field monitoring data. It is recommended that for design purposes a range of values should be considered for these parameters.

$$V_l(\%) = 0.23\ e^{4.8(LF)}\ (\text{for LF} \geq 0.2) \tag{7.8}$$

where LF is defined as the ratio of the stability ratio (N) over the stability ratio at collapse (N_c) (stability numbers are described in section 3.3).

7.1.2 Horizontal displacements

From the point of view of damage to structures and services it is not only important to determine vertical displacements within the ground, but also the horizontal movements (Burland *et al.* 2001b). In the transverse direction to the tunnel construction, the surface (and subsurface) horizontal displacements can be estimated by various assumptions. The simplest is to assume that the ground movements are radial, i.e. directed to the tunnel axis (equation 7.9 and illustrated in Figure 7.5a).

$$S_h = S_v\, y/H \tag{7.9}$$

where S_v is the vertical ground displacement, S_h is the horizontal ground displacement and y is the transverse horizontal distance from the tunnel centreline.

Alternative methods have been proposed. For example, Mair *et al.* (1993) demonstrated that in London Clay the radial movement assumption can over-estimate the horizontal movements, particularly those close to the tunnel, and proposed the relationship shown in equation 7.10. This assumes that the displacements are directed to a point 0.175H/0.325 below the tunnel axis (Figure 7.5b).

$$S_h = S_v\, y/(1 + 0.175/0.325)\, H \tag{7.10}$$

New and Bowers (1994) developed the idea of the cumulative probability distribution model to provide a full array of equations for the prediction

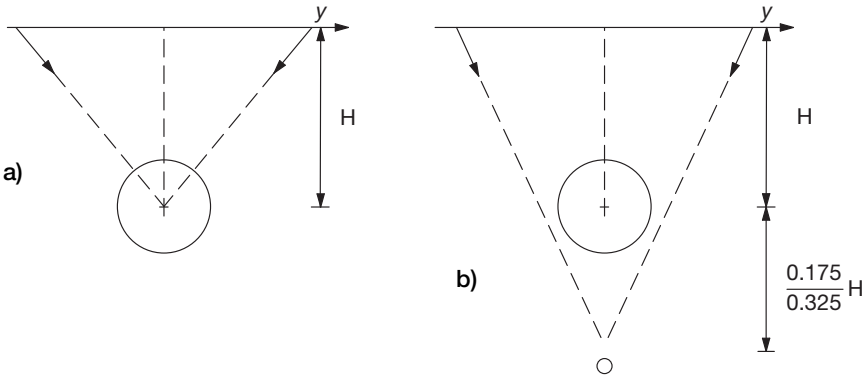


Figure 7.5 Direction of the ground displacement vectors above tunnels in clay (after Mair and Taylor 1997), a) vectors directed towards axis (Attewell 1978, O'Reilly and New 1982), b) vectors directed towards point O (Taylor 1995b)

of ground movements in three dimensions. This approach has been shown to give significantly improved predictions in the vicinity of the tunnel. (BTS/ICE 2004)

7.1.3 Long-term settlements

Long-term settlement is a phenomenon predominantly associated with fine grained soils and is associated with component 5 in Figure 7.1. As described previously, it is the result of the equilibrium of excess pore water pressures within the soil over time and the associated volume changes that occur. Other contributory factors to the total long-term settlements could be (after Devriendt 2006):

- the tunnel acting as a drain (depends on the permeability of the tunnel lining relative to the ground);
- time dependent distortion of the tunnel lining;
- time dependent dissipation of excess pore water pressures due to grouting behind the lining or due to mitigation measures such as compensation grouting;
- creep and secondary consolidation processes in soils;
- time dependent closure of the grouted annular gap due to: bleeding and curing (hardening and shrinkage) of the grout, insufficient grout or loss of grout.

As well as the maximum settlement increasing in the long-term, these effects cause the settlement trough to widen (Burland *et al.* 2001a). For soils such as soft clays and silts the component of volume loss due to initial

'undrained' movements may be small compared to the time dependent processes. The maximum settlement may reach between two to four times the short-term value and the trough width parameter at the surface from 1 to 2.5 times the short-term value. Fang *et al.* (1993) proposed a hyperbolic time settlement relationship to describe the increase in maximum settlement with time for earth pressure balance tunnelling machines commonly used in these soils. Alternatively, when tunnelling with traditional compressed air methods, a number of authors have suggested that long-term settlements in soft soils increase linearly per logarithm of time (Devriendt 2006). Work in this area is continuing, for example Wongsaroj *et al.* (2007) investigated the long-term ground movements in London resulting from the construction of the Jubilee Line Extension, which opened in 1999.

7.1.4 Multiple tunnels

The previous work described in this chapter has related to single tunnels. However, in many situations rather than constructing a single large tunnel, twin tunnels are constructed. This has to be taken into account when determining the ground movements generated by the tunnel constructions. New and O'Reilly (1991) proposed equations for the prediction of cumulative displacements for parallel tunnels with a given separation based on the principle of superposition. However, Attewell *et al.* (1986) discuss the 'interference volume' effect where the volume loss is commonly greater when a second tunnel is excavated adjacent to the completed tunnel (asymmetric effect in the final settlement trough will occur for two side-by-side tunnels of the same cross section and depth) (Devriendt 2006).

Addenbrooke and Potts (2001) considered this phenomenon numerically and found it to be due to the accumulation of shear strain adjacent to the first tunnel. This results in a lower stiffness and hence greater displacements where subsequent tunnels are constructed close to the first tunnel (Devriendt 2006). This has also been investigated by Cooper *et al.* (2002), Hunt (2005) and Chapman *et al.* (2007), with observations from case history data, numerical analyses and small-scale physical modelling, respectively. These authors propose a method for estimating the ground movements (both surface and subsurface) above closely spaced twin side-by-side tunnel constructions in soft ground. This method should be considered if the clear separation between the tunnels is approximately three diameters or less.

7.2 Effects of tunnelling on surface and subsurface structures

One of the most important aspects of tunnelling in soft ground is assessing the effect of any ground disturbance caused by the tunnelling operations on surface or subsurface structures. This is particularly important in urban areas. It is important to recognize that the ground movements, caused by

the tunnelling, and the structure interact. The impact of these movements on the structure depends on the size, shape and material of the structure, as well as its position relative to the tunnel. It is the stiffness of the structure that is crucial when assessing the effect of these ground displacements, as a stiffer structure can reduce the effects. Old masonry structures tend to follow the ground displacements closely, as do structures on pad footings. Conversely, structures constructed of reinforced concrete will experience smaller horizontal displacements than the ground, due to their higher longitudinal stiffness, i.e. they do not stretch as much. These structures also experience reduced distortions due to their higher flexural stiffness, i.e. they do not bend as much. Stiff structures exhibit a high level of shear resistance, i.e. relative movements within the structure are small and tend to be subject to tilt rather than distortion. The response depends on such factors as the height of the structure, the number of openings and the design of the structure, for example concrete walls or beam and column construction (ITA/AITES 2007).

In addition, the location of the structure in relation to the ground deformations influences the movements it experiences. For a Gaussian shaped settlement trough, the structure will experience extension and hogging over the convex parts of the settlement trough and compression and sagging over the concave parts. Figures 7.6a and b show the typical response of short buildings. As the tunnel approaches, the short building rides the forward settlement wave with little significant sagging or hogging deformation (Figure 7.6a). Furthermore, short buildings experience tilt as a rigid body, but little significant sagging or hogging deformation across a transverse settlement profile (Figure 7.6b). Figures 7.6c to e show the response of a stiff long building as the tunnel advances, i.e. it experiences progressive deformation and differential settlements. Figures 7.6f and g show the potential sagging and hogging of a long building across a transverse settlement trough when it is directly above the centreline of the tunnel and also offset from the centreline. It should be noted that structures are more sensitive to differential movements, and hence it is important to assess the position of the structure in relation to the ground deformations caused by the tunnel.

Cracking is often used as an indication of distress in structures. Several researchers have investigated cracks and developed classification methods to assess structural damage, for example the reader is referred to ITA/AITES (2007), Burland and Wroth (1975) and Boscardin and Cording (1989). Further information is provided in section 7.2.2.

7.2.1 Effect of tunnelling on existing tunnels, buried utilities and piled foundations

In addition to buildings, it is important not to forget about structures that lie beneath the ground surface, for example existing tunnels and buried

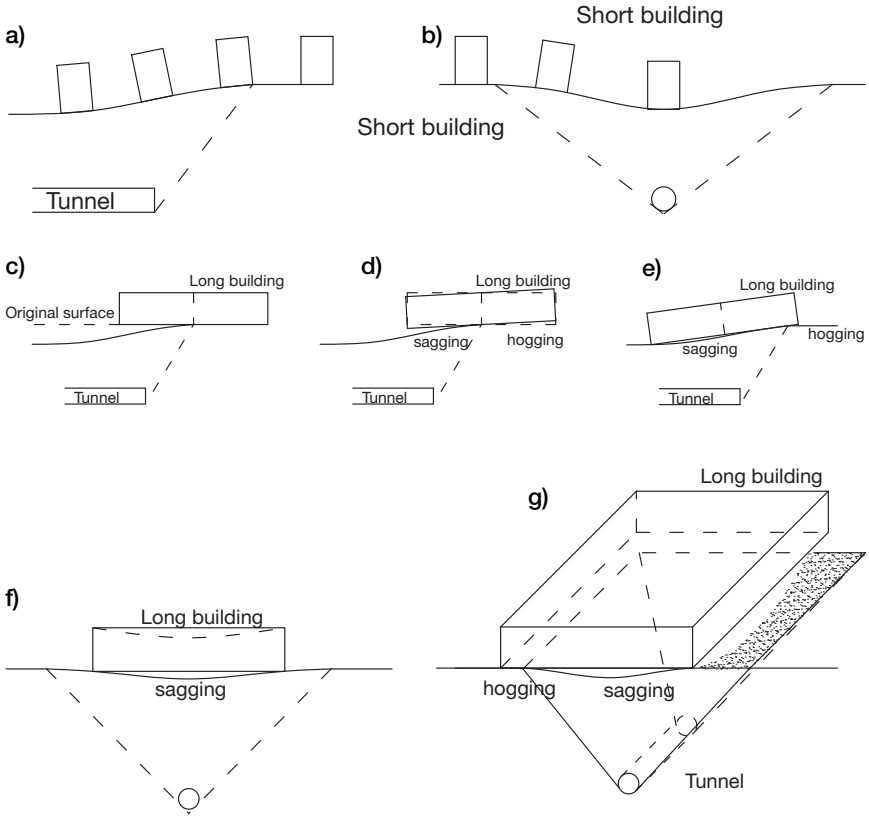


Figure 7.6 Some idealized modes of behaviour for short buildings and long buildings due to a tunnel construction below (Attewell 1995, after Attewell *et al.* 1986)

utilities. Pipelines in particular can be vulnerable to ground displacements caused by tunnelling activities, particularly the older, more brittle, cast iron gas and water pipes.

Attewell *et al.* (1986) provide a comprehensive investigation of pipeline response to tunnelling displacements. They state that in the vicinity of a buried cast iron pipeline the greenfield ground movement is modified, since the pipe stiffness is typically 1000 to 3000 times the soil stiffness. The resistance of a pipeline to system disturbance depends on the longitudinal flexural rigidity (including the flexural rigidity of the pipeline joints), longitudinal bending strength (including reduction in strength caused by corrosion or service holes) and the pipe diameter. The true risk in any particular case will largely depend on existing local stress levels due to previous disturbance, stress due to internal pressure, stress due to external loading such as traffic, and seasonal effects associated with ground temperature and moisture changes. Attewell *et al.*

(1986) suggest that consideration should be given to the effects of tunnelling on cast iron pipes (grey iron) when the movement is expected to exceed 10 mm. For more flexible pipes, consideration should be given if the movements are expected to exceed 50 mm. The direction of the tunnelling relative to the pipeline is also an important factor. A pipeline parallel to the tunnelling operation will expose the whole pipeline to the maximum movement effect. Any point of weakness is found by the wave of bending caused by the tunnel. Conversely, only a small part of a transverse pipeline is exposed to the maximum bending, with this maximum being in the sagging mode. They state that the main factors affecting the behaviour of a pipeline are the magnitude and distribution of the ground movement, the soil-pipe stiffness (including the effect of joints) and the yield stress of the soil. In order to assess pipelines subject to ground deformation, the designer needs to have information on the load-deformation characteristics of the soil around the pipe, the pipe itself and the pipe joints (Attewell *et al.* 1986 provide estimates of these values for typical cases). They show that even though the methods are essentially all based on linear elastic assumptions, they agree well with field observations and are quite adequate for practical applications. Bracegirdle *et al.* (1996) provide further guidance on the assessment of risk of damage to cast iron pipes. These authors propose a methodology for evaluating potential damage to cast iron pipes induced by tunnelling in soft ground.

More recently Klar *et al.* (2005) and Marshall and Klar (2008) looked at the commonly used approach of assuming the pipeline to be a simple beam by comparing two different theories; the Euler–Bernoulli simple beam theory, and a more accurate representation using shell element theory (FLAC3D® 3-D finite difference analysis). It was found that, in general, steel and concrete pipes are well represented using beam theory (due to the high relative pipe-soil material stiffness for steel pipes, and large wall thickness to diameter ratio for concrete pipes). For polyethylene pipes (which have a low value of relative pipe-soil material stiffness and can have both large and small values of wall thickness to diameter ratios) predictions using the beam theory deviate significantly from the shell element predictions. Therefore, it appears that the shell element formulation is better suited for the analysis of polyethylene (or similar) pipelines. Although it should be noted that these analyses were linear elastic and did not take into account possible 'sliding' between the ground and pipe (possibly important for the smooth wall polyethylene pipes).

In terms of the effects of tunnelling on existing tunnels, an extensive investigation was conducted by Cooper (2002) based on a number of case histories in London, UK, but particularly focusing on one associated with the Heathrow Express Tunnel to Terminal 4 in 1995 (Cooper *et al.* 2002). The research looked at the extensive monitoring data taken inside the London Underground Piccadilly Line tunnels (unbolted segmental concrete lined tunnel) as the new tunnels were constructed underneath. It showed

how the tunnels behaved in terms of potential disturbance as the tunnelling passed underneath. It also showed that in this case, the empirical settlement prediction method, described in section 7.1, could be used to estimate the settlements, rotations and deformations of the existing tunnel lining.

There are other examples in the literature of monitoring existing tunnels during new tunnelling works. For example, Moss and Bowers (2006) describe the approach adopted on the Channel Tunnel Rail Link (CTRL) in the UK with respect to monitoring the effects on London Underground lines in London. This staged approach is similar to that described in section 7.2.2. The overall philosophy, though, was to minimize the ground movements at source, i.e. using high specification EPB tunnelling machines, and to use a risk-based engineering assessment of the effect of the tunnelling works on the existing tunnels. There have also been studies to investigate the behaviour of existing tunnels due to adjacent excavations, for example, Chang *et al.* (2001).

EFFECTS OF TUNNELLING ON PILED FOUNDATIONS

There have been a number of investigations into the effects of tunnelling on piled foundations. These include studies using small-scale physical modelling (for example Lee and Bassett 2007), centrifuge modelling (for example Jacobsz 2002, Jacobsz *et al.* 2004), three-dimensional numerical modelling (for example Loganathan *et al.* 2001, Mroueh and Shahrour 2002, Lee and Ng 2005), and also full-scale field monitoring (for example Selemetas *et al.* 2006, Pang *et al.* 2006). There has also been work conducted into analytical analysis methods for pile groups affected by tunnel construction, for example Huang *et al.* (2009).

The findings from this research have indicated that there are 'zones' of influence that affect the piles in different ways depending on their relative position to the tunnel centreline. Figure 7.7 shows the findings of Selemetas

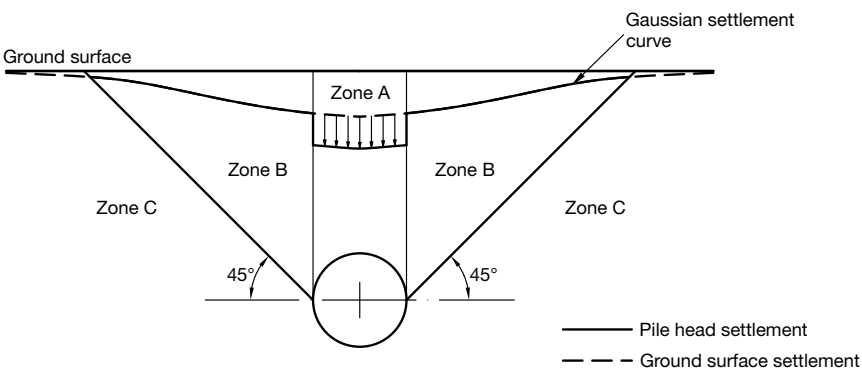


Figure 7.7 Zones of influence of pile settlement due to earth pressure balance shield tunnelling in London Clay (after Selemetas *et al.* 2006)

et al. (2006), although they stress that the boundaries of Zone B are simplified and probably are a function of the shearing resistance of the soil and the volume loss during tunnelling and are therefore not constant (see also Jacobsz *et al.* 2004, and Lee and Bassett 2007). Piles in Zone A were found to settle more, those in Zone B settled by the same amount and those in Zone C settled less than the ground surface. In addition, piles in Zone A experienced a considerable reduction in their base loads during the tunnelling and this was accompanied by differential pile settlement. Piles in Zone B and C only experience small changes in their base loads and actually showed a net gradual increase with time due to the ground-induced negative shaft friction (Selemetas *et al.* 2006).

7.2.2 Design methodology

This section describes a methodology recommended by ITA/AITES (2007) for studying the effect of underground works on existing structures. This methodology is broken down into six stages and a brief summary is provided below.

PHASE 1: INVESTIGATION OF EXISTING STRUCTURES

This phase involves surveying and data collection on the nature, configuration and condition of buildings and utilities together with topographic measurements and technical expert reviews. This is essential to assess the baseline or zero condition of each structure prior to the start of construction works.

PHASE 2: INFORMATION SUMMARY

This involves a 'typological' classification of the structures according to, for example, nature, function, value, size, design, age and current condition.

PHASE 3: SELECTION OF DAMAGE CRITERIA

This is aimed at converting the objectives required in terms of damage limitation into criteria to be used by the designer. This could be, for example, a system based on strains, where the strains are based on the initial reference condition and evaluation criteria are used with respect to additional strains induced during the construction.

PHASE 4: MODELLING

This phase is intended to correlate the building displacements induced by ground movements to its structural deformations. Mair *et al.* (1996) suggested a three stage assessment process, as follows.

Preliminary assessment Based on the empirical predictive methods described in section 7.1.2 (i.e. assuming greenfield settlements), an assessment is made on the level of ground movement and hence the effect on the surface structures. This is often done using contours of settlement and it is considered that any settlements of less than 10 mm cause negligible risk of damage to the structure. In addition, it should also be checked that no structure experiences a slope, due to differential settlements, in excess of 1 in 500. Table 7.2 shows the damage risk assessment and suggested actions.

Second stage assessment The ground deformations will comprise both sagging and hogging zones and so, depending on the relative position of the structure to the point of inflection of the settlement trough, there will be zones of tensile and compressive strains. The second stage assumes that the structure is weightless and fully flexible, i.e. follows the greenfield displacements exactly (refer to Burland 1995, Franzius *et al.* 2006, Burland *et al.* 1977 and Boscardin and Cording 1989). This is usually conservative as the stiffness of the structure will reduce the actual movements. An example of how the potential damage can be classified according to the maximum tensile strain as shown in Table 7.3.

Detailed evaluation This stage would normally be undertaken for structures where a 'moderate' or greater level of damage has been predicted in stage 2. It considers the existing condition of the structure, the tunnelling sequence, three-dimensional aspects, characteristics of the structure and the soil-structure interaction effects. Protective measures (for example compensation grouting, section 4.2.8) would then be considered for structures remaining in the 'moderate' or higher damage categories. It is possible at this stage to use numerical methods and the method proposed by Potts and Addenbrooke (1997), updated by Franzius *et al.* (2006).

PHASE 5: DETERMINATION OF THE ALLOWABLE DISPLACEMENT THRESHOLDS

ITA/AITES (2007) states that the purpose of this phase is to determine the contractual threshold requirements that will have to be met during construction. The summary, prepared as part of Phase 2, is essential to allow contractual criteria to be developed that meet the needs, in terms of protection, of the structures.

Threshold values must never be taken as constant. They should be essentially treated as alarm indicators and continuously reviewed based on the actual building response due to the tunnelling works. However, it is important that 'alarm' thresholds and 'stopping' thresholds exist for each project. A system based on amber and red trigger values is often used (see section 7.3.2 on trigger values).

Table 7.2 Typical values of maximum building slope and settlement for damage risk assessment, and suggested action for various risk categories (after Rankin 1988, used with permission of the British Geological Society)

Risk category	*Maximum slope of building*	*Maximum settlement of building (mm)*	*Description of risk*	*Description of action required*
1	Less than 1/500	Less than 10	*Negligible*: superficial damage unlikely.	No action. Except for buildings identified as particularly sensitive for which an individual assessment should be made.
2	1/500–1/200	10 to 50	*Slight*: possible superficial damage which is unlikely to have structural significance.	Crack survey and schedule of defects, so that any resulting damage can be fairly assessed and compensated. Identify any buildings and pipelines that may be particularly vulnerable to structural damage and assess separately.
3	1/200 to 1/50	50 to 75	*Moderate*: expected superficial damage and possible structural damage to buildings. Possible damage to relatively rigid pipelines.	Crack survey: a schedule of defects and a structural assessment. Predict extent of structural damage. Assess safety risk. Choose whether to accept damage and repair, take precautions to control damage or, in extreme cases, demolish. Buried pipelines at risk: identify vulnerable services and decide whether to repair, replace with a type less likely to suffer damage, or divert.
4	Greater than 1/50	Greater than 75	*High*: expected structural damage to buildings. Expected damage to rigid pipelines. Possible damage to other pipelines.	

Note: The above criteria relate to *near surface* foundations or pipelines.

Table 7.3 Classification of building damage (from Devriendt 2006, after Burland *et al.* 1977 and Boscardin and Cording 1989)

Damage category	*Description of typical damage (ease of repair in italics)*[a]	*Approx. crack width (mm)*[b]	*Limiting tensile strain (%)*
0 Negligible	Hairline cracks of less than about 0.1 mm are classed as negligible	< 0.1 mm	0–0.05
1 Very slight	*Fine cracks which can easily be treated during normal decoration.* Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.	1 mm	0.05–0.0075
2 Slight	*Cracks easily filled. Redecoration probably required.* Several slight fractures showing inside of the building. Cracks are visible externally and some re-pointing may be required externally to ensure weather tightness. Doors and windows may stick slightly.	5 mm	0.075–0.15
3 Moderate	*The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Re-pointing of external brickwork and possibly a small amount of brickwork to be replaced.* Doors and windows sticking. Service pipes may fracture. Weather tightness often impaired.	5 to 15 mm, or a number of cracks > 3 mm	0.15–0.3
4 Severe	*Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.* Windows and door frames distorted, floors sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing on beams. Service pipes disrupted.	15–25 mm	> 0.3
5 Very severe	*This requires a major repair job involving partial or complete rebuilding.* Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually > 25 mm, but depends on number of cracks	> 0.3

Notes:
(a) In assessing the degree of damage account must be taken of its location in the building or structure.
(b) Crack width is only one aspect of damage and should not be used on its own as a direct measure.

PHASE 6: BACK ANALYSIS AND CALIBRATION OF MODELS WITH OBSERVED DATA

It is essential to check the displacement estimates (Phase 4) by monitoring the construction works and the effects on the surrounding ground and structures. Validating the design assumptions should be a routine part of all construction management planning.

A practical example of how the buildings along the route of a new metro in Amsterdam (North/South Metroline) were assessed for potential damage prior to the tunnelling works, and also how the monitoring during construction was linked to the tunnelling operations, i.e. the TBM operations, is described in van Hasselt *et al.* (1999). The so called 'Interactive Boring Control System' (IBCS) meant that the TBMs were not controlled solely on the basis of tunnel and machine data, but also by using the settlement monitoring data as an additional criterion. The machine parameters were used to make virtual predictions of the subsequent excavation section (based on, amongst other factors, the face control pressures and tail void grouting) and if this prediction showed the likelihood of unacceptable damage to the overlying structures, the machine parameters were adjusted.

7.3 Monitoring

7.3.1 Challenges and purpose

The challenge for tunnel engineers is to achieve the completion of a tunnelling project without the general public realizing that the tunnelling operations are taking place beneath/around them. In order to achieve this, there must be no inconvenience to people going about their daily lives in terms of disruption, noise and dust (this is not always possible when using cut-and-cover construction) and no noticeable effect on structures, e.g. buildings, other tunnels or utilities in the vicinity of the tunnel construction (see section 7.2). At the current time no tunnelling operation can completely eliminate disruption or ground movements (although with modern tunnelling machines and good quality control very small movements can be achieved) and so monitoring of affected structures is usually required.

In tunnelling using NATM methods monitoring inside the tunnel as construction proceeds is an integral part of the construction process, and this is described further in section 7.3.4.

Monitoring can be done on a number of levels depending on the purpose. It is therefore important to ask the questions 'what is the purpose of the monitoring?', and hence 'what information is required?'. This needs to be clearly understood prior to designing the monitoring regime. It is also beneficial to keep the instrumentation regime as simple as possible for the required purpose. This does not mean that the instruments should not be state-of-the-art, but their arrangement should not be overcomplicated. It should be borne in mind that the data from the instruments must be

analysed regularly and it is important that the data inform this process in the most effective way. It is also important that there is suitable redundancy of instrumentation in critical or inaccessible areas to insure against failures.

BTS/ICE (2004) states that instrumentation is typically installed to:

- obtain 'baseline' ground characteristics;
- provide construction control;
- verify design parameters;
- measure performance of the lining during, and after, construction;
- monitor environmental conditions (for example settlement, air quality and effects on the groundwater regime);
- carry out research to enhance future designs;
- monitor mitigation measures, for example compensation grouting and ground freezing.

It is important to understand the quality of the data received from the instrumentation. For this reason some important definitions related to monitoring and instrumentation are given below (after Dunnicliff and Green 1993):

- *Conformance*: the presence of the measuring instrument should not alter the value of the parameter being measured. The degree by which the parameter is altered by the instrument is known as its *conformance*.
- *Accuracy*: this is the closeness of a measurement to the true value of the quantity measured. Accuracy is synonymous with *degree of correctness*. The accuracy of an instrument is evaluated during calibration to a known standard value. It is customary to express accuracy as a ± *number*.
- *Precision*: this is the closeness of each of a number of similar measurements to the arithmetic mean. Precision is synonymous with *reproducibility and repeatability*. The number of significant figures associated with the measurement indicates precision. For example, ±1.00 indicates a higher precision than ±1.0.

The difference between accuracy and precision is illustrated in Figure 7.8.

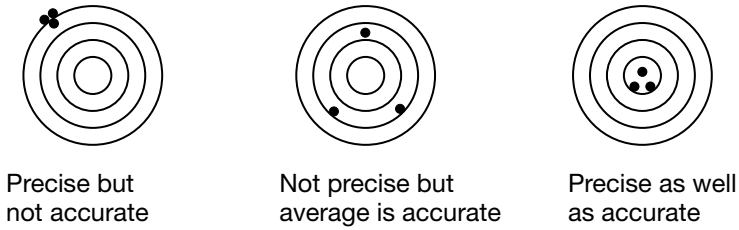


Figure 7.8 Accuracy and precision (after Dunnicliff and Green 1993)

- *Resolution*: this is the smallest division on the instrument readout scale.
- *Error*: this is the deviation between the measured value and the true value, i.e. it is mathematically equivalent to accuracy. Errors can occur due to human carelessness, fatigue or inexperience, or can be due to improper calibration, poor installation procedures or environmental conditions such as heat, humidity or vibration.

The data from the instrumentation and monitoring programme need to be appropriately managed to ensure that it is in a suitable form to be clearly understood. It should be regularly reviewed by qualified people, experts in monitoring so that unexpected trends can be identified easily and appropriate actions taken. It should be routine on any project that comparison is made between the predicted and the observed values in order to understand the behaviour of the structure and the ground being monitored.

7.3.2 Trigger values

It is common practice to establish 'trigger values' for key measurement parameters associated with a project, for example displacement. If these values are exceeded during the tunnelling project, then certain actions need to be clearly defined. Two trigger values are normally established (BTS/ICE 2004):

- *amber or warning value*: this could be a pre-determined value or a rate of change in a parameter that is considered to indicate a problem;
- *red or action value*: this could be where threshold values for safe operation are exceeded. This should initiate an immediate check of the instrument function and a visual inspection, and initiation of pre-determined action, for example temporary cessation of the tunnel work.

The determination of trigger levels is often specific to the project. A widely used approach is, for example amber trigger value = the calculated displacements exceeded by 50%: red trigger value = the calculated displacements exceeded by 100%.

Even if the trigger values are not exceeded, the monitoring data should be carefully checked to highlight any unexpected trends so that these can be acted on appropriately before a problem develops. For this reason there is often the green trigger value established (the green or early warning value), for example green trigger value = the calculated displacements are reached.

It should be noted, however, that trigger values are more easily applied when monitoring existing structures in the vicinity of the tunnelling operation, where specific limits must not be exceeded in order to avoid damage to the structure. It is more difficult to apply trigger values to in-tunnel monitoring data because significant estimations have to be made regarding the behaviour of the ground and the tunnel lining. Therefore, the calculated

displacements are only estimates based on engineering judgement. This is discussed in more detail in the next section on the observational method as well as in the section on in-tunnel monitoring for NATM (section 7.3.4).

7.3.3 Observational method

Instrumentation and monitoring forms an integral part of the observational method. This method is very important in civil engineering and particularly ground engineering projects, such as tunnelling, and it is carried out, whether formally or informally, on all projects. The ground cannot be characterized exactly and even if a good site investigation has been conducted, it is possible that the predicted behaviour is not observed in practice. The observational method allows deviations to be identified early (these can be both positive and negative) and appropriate actions taken.

Nicholson *et al.* (1999) carried out a comprehensive review of this approach and they define the method as 'a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction'.

Peck (1969a) considered that the complete application of the observational method embodies the following aspects:

- sufficient exploration to establish at least the general nature, pattern and properties of the ground, but not necessarily in detail;
- assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role;
- establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions;
- selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis;
- calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions;
- selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis;
- measurement of quantities to be observed and evaluation of actual conditions;
- modification of design to suit actual conditions.

In terms of applying the observational method to uncertainties in the ground, the principal objective is to apply sufficient resources to prevent uncertainty to unacceptable levels of risk. In this respect there are three types of uncertainties (after Nicholson *et al.* 1999), as follows.

GEOLOGICAL UNCERTAINTY

On projects where there are complex geological and hydrological conditions, there may be unexpected variations in the ground conditions between boreholes. A conceptual model of the geological conditions will have been developed at the design stage. Based on this model, modifiable design solutions are developed for a range of conditions. The actual ground conditions are determined during the works and the appropriate design solution selected to suit these actual conditions.

PARAMETER UNCERTAINTY

Uncertainties exist in the knowledge of the ground characteristics and the modelling of its behaviour, and hence it is not possible to accurately determine ground parameters. The observational method involves developing flexible and robust designs for a range of parameters. Monitoring results are reviewed during construction and the design modified as appropriate.

GROUND TREATMENT UNCERTAINTY

There are a number of ground treatments available to improve specific properties of the ground. The use of these techniques is often based on a performance specification identified by the designer. The effectiveness of the technique is monitored and reviewed during the treatment and modifications implemented where necessary to meet the specification.

There is also a fourth area of potential uncertainty.

SUPPORT UNCERTAINTY

The time dependent behaviour of sprayed concrete (i.e. the development of strength, ultimate strain, creep) is difficult to simulate accurately, and hence estimate, during the design process. In addition, the load transmission in the joints of segmental linings is difficult to estimate.

For tunnelling in soft ground using a sprayed concrete lining, the ground treatment, for example compensation grouting, can also be an uncertainty with respect to influencing the stresses on the tunnel support. The geological uncertainty could apply to the heading excavation and result in the need for face logging and possibly horizontal boreholes to log ahead of the face. It is important to have sufficient time to support the excavation, but there is potential uncertainty in the associated ground parameters. Convergence monitoring within the tunnel can help to provide data to review parameter uncertainty.

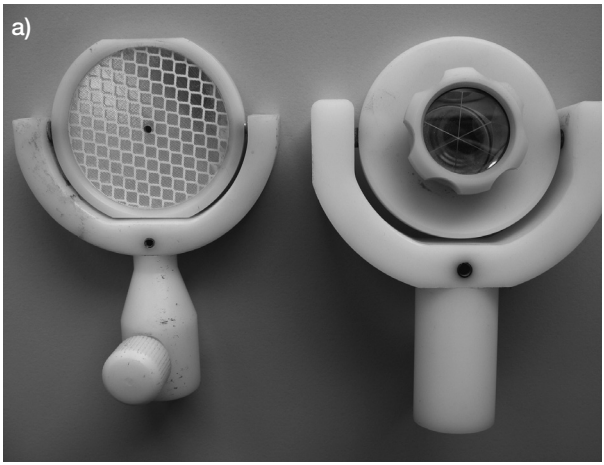
NATM tunnelling is a prime example of where the observational method is an integral part of the philosophy for this method and hence the construction process. The monitoring associated with NATM is described in the next section. Further information on the application of the observational method to tunnelling can be found in Powderham (1994 and 2002).

7.3.4 In-tunnel monitoring during New Austrian Tunnelling Method tunnelling operations

7.3.4.1 Measurements

Monitoring cross sections or measuring profiles are installed in the tunnel while it is being constructed. The distance between monitoring cross sections can vary from approximately 3 m to more than 50 m, and is dependent on the ground conditions, the ground-lining interaction, and the sensitivity of service structures around the tunnel and buildings at the surface. A common distance in inner city tunnels is 10 m. Each monitoring cross section contains a number of monitoring points at which targets are positioned for the laser theodolite. Two commonly used targets are shown in Figure 7.9a. The left-hand side of Figure 7.9a shows a Bireflex target. The name derives from the reflective mirror foil on both sides ('bi'-reflex) of the target and it has a diameter of 50 mm or 60 mm. The actual target is a small hole in the middle. The reflective foil helps the laser of the theodolite to find this hole. The right-hand side of Figure 7.9a shows a prism target. In this target, three prisms form a circle of 25 mm in diameter. The prisms guide the laser by the intensity of the reflexion to the centre. The targets are screwed on top of short bolts (15 to 30 cm), which have previously been sprayed firmly into the sprayed concrete lining (Figure 7.9b). Prism targets are five times more expensive than Bireflex targets. They are, however, more accurate and they can be used for automated monitoring.

Typical monitoring arrays are shown in Figure 7.10. For a typical tunnel construction, in which the excavation is divided into crown, bench and



b)

Figure 7.9 a) Monitoring targets: Bireflex target (LHS), prism target (RHS), b) Bolt on which the monitoring targets are mounted

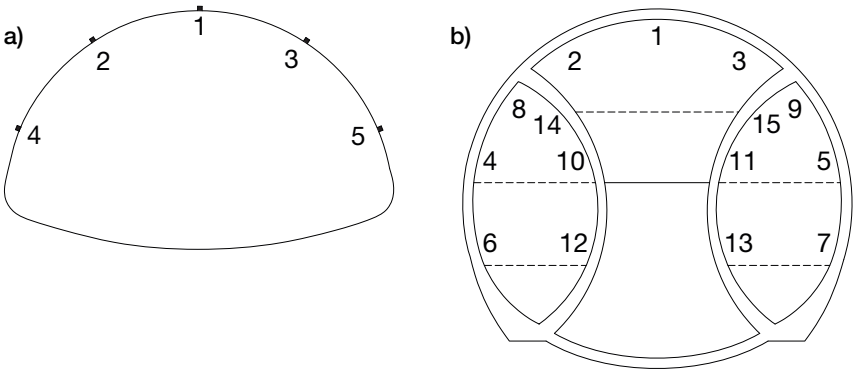


Figure 7.10 Examples of a measuring profile a) for a crown and, b) a double track tunnel excavated with side wall drifts

invert, five monitoring points are installed in the crown (Figure 7.10a), and one or two points on each side in the bench (not shown). The monitoring points are numbered so as to distinguish them from each other. It is common to number the highest monitoring point in the middle of the crown as number '1'. Usually numbering continues on the left-hand side with even numbers, and on the right-hand side with odd numbers (line of vision in the direction of excavation). The invert is not usually monitored because it is covered with backfill that provides a track for the construction equipment. For information on monitoring the invert see section 8.1 on the case history of the Eggetunnel.

Readings are taken by a laser theodolite, which digitally stores the three-dimensional coordinates of each target: vertical, horizontal and longitudinal. After collecting the readings, these data are transferred to a computer for further processing. From the change in coordinates, displacements in all three directions can be calculated and the deformation behaviour of the structure can be derived (this is discussed later in this section).

It is important to have a few stable reference points at the beginning of the measurements, which are not affected by the tunnel construction. The accuracy of the laser theodolite measurement should be ±1 mm or better for ideal measuring conditions. The accuracy can go down to ±3 mm or worse in difficult conditions, e.g. dust or large temperature differences inside the tunnel. This can happen for example, if the monitoring is done from a large cross section into a small one with a significantly higher temperature due to the hydrating of the sprayed concrete or if the ventilation is shut down for maintenance.

The digital technique of using a laser theodolite has widely replaced measuring with a tape extensometer. The latter is a special steel tape, usually of 20 m or 30 m in length. With a tape extensometer distances instead of coordinates are measured. This means that immediate information on

convergence (distances becoming shorter) or divergence (distances becoming longer) is available. Although the resolution can be as good as 0.5 mm, in practice reasonable accuracy can be as low as approximately ±10 mm. This is especially true when measuring longer distances, as the tape extensometer sags due to its own weight. It has to be tensioned to mitigate this effect. Although the force necessary to tighten the tape must stay in a defined range (lots of tapes have been ripped apart), the sag and the tightening reduces the accuracy of a tape extensometer significantly. Since digital processing of data is not common with tape extensometers and the readings are time consuming, tape extensometers are only used in situations where targets are difficult or impossible to focus in on with a theodolite, e.g. in the bottom of a shaft.

7.3.4.2 General development of displacements

The ground usually reacts to the approaching tunnel excavation before the heading actually reaches a particular point (see section 7.1.1). In addition, the influence of the tunnel excavation does not stop immediately, but slowly fades out as the face moves forward. In general, the weaker and softer the ground is, the earlier the displacements will start and the longer they will last. This is particularly true when the ground has time dependent behaviour such as creeping (this is often observed in clay). As a rule of thumb the influence of the tunnelling excavation is about ±2D before and behind the monitoring cross section where D is the tunnel diameter. Figure 7.11 shows the idealized displacement curve of one monitoring point reacting to an approaching and disappearing tunnel heading.

Figure 7.11 indicates the tunnel face moving along a number of chainages and the related displacements (Stärk 2009).

- Tunnel face is at Chainage 1: the face is far away from the monitoring cross section so no displacements can be determined at the monitoring cross section. The primary stress condition is undisturbed.
- Tunnel face is at Chainage 2: the face is closer to the monitoring cross section and the first small displacements occur.
- Tunnel face is at Chainage 3 and 4: the face is even closer to the monitoring cross section and the displacements have increased significantly.
- Tunnel face is between Chainage 4 and 5: the tunnel excavation passes the monitoring cross section. The displacements increase further, but are not yet measurable from inside the tunnel.
- Tunnel face is at Chainage 5: the measuring bolts in the monitoring cross section are installed. Thereafter the first measurement ('base reading') is taken.
- Tunnel face is between Chainage 6 and 7: further readings are taken. The increase in the displacements between the measurements is slowly reducing, which means that the influence of the excavation is fading out.

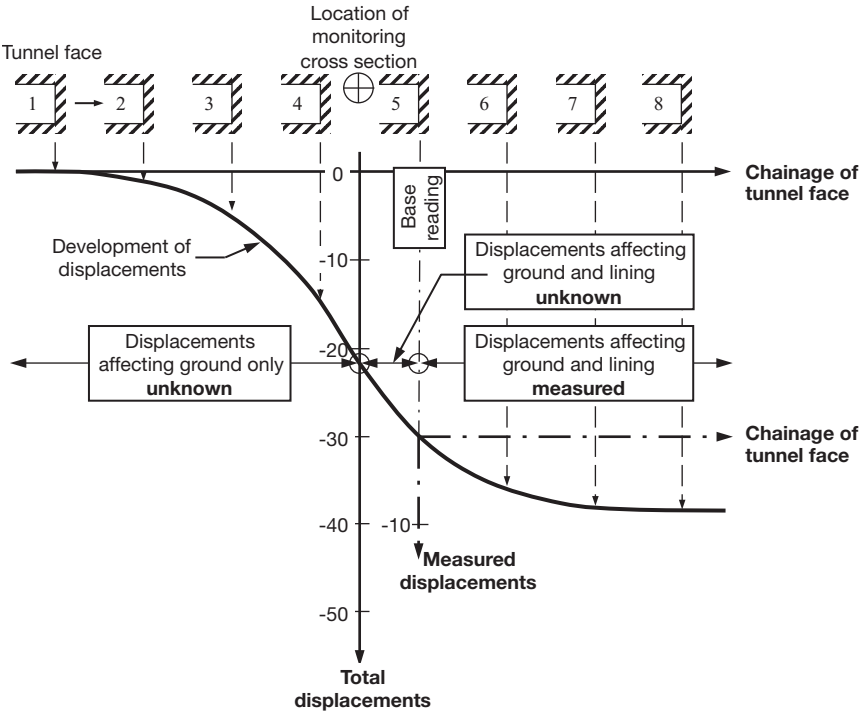


Figure 7.11 Development of the displacements for one monitoring point in relation to the position of the tunnel face. The scaling and magnitude of the displacements are arbitrary and only by way of example

- Tunnel face is at Chainage 8: the increase in the deformations has stopped. At this distance the excavation has no more influence on the displacements.

It should be noted that the monitoring cross section cannot be installed until the tunnel construction has reached Chainage 5. The displacement data are therefore only available from inside the tunnel after the base reading at Chainage 5 has been taken. The curve prior to the base reading, from Chainage 1 to Chainage 5, can be measured using external instruments, such as vertical extensometers, which can be installed from the ground surface (see section 7.3.5 for a description of vertical extensometers). However, in general this section of the displacement curve remains unknown. This means that only a fraction of the total displacement can actually be measured. Depending on the geological conditions, the displacements affecting only the ground range typically from approximately 30% to 70% of the total displacements (Chainage 1 to the monitoring cross section). The better the ground, the better its ability for stress redistribution and the larger the displacements affecting only the ground (maximum 100% if no support is

required). The displacements from the monitoring cross section to Chainage 8 affect both the ground and the tunnel lining. These displacements load the lining and it is therefore important to determine these with respect to the stability of the tunnel.

There is a period when there is a lack of monitoring information. This period is between the face passing the monitoring cross section and the actual base reading. Therefore the base reading must be taken as quickly as possible and should be positioned as close as possible to the tunnel face. The monitoring cross section is usually installed in the face advance currently under construction and the base reading is taken before the excavation of the next advance commences (the base reading is taken within six hours or less). If the base reading is done in this way, the previously mentioned lack of information is negligible with respect to the loading of the sprayed concrete; because the 'young' sprayed concrete cannot take much load. It is therefore very important for the interpretation of the measuring results that there is an indication of the place and time of the base reading in relation to the location of the face.

7.3.4.3 Interpretation of the measurements: displacements

The basic graph derived from the measurements is displacement versus distance to 'face – monitoring cross section', or displacement versus time. By default, a distance-dependent graph is used to control the effect of the tunnelling progress on the already excavated section. However, if the advance rate is constant and the excavation process is uniform, a time-dependent graph can be used. In the case of a hiatus in the excavation, for example the Christmas shutdown, one also has to switch to a time-dependent graph.

The displacements increase quickly immediately after the base reading. The influence of the tunnel construction then decreases, and finally the displacements do not increase any further. A new stable equilibrium between the support and the ground has then been established and the displacements must remain constant. This statement is very important with respect to settlement control and stability. Figure 7.12 shows, by way of example, the vertical displacements of the crown.

CRITICAL TRENDS OF THE MEASURING CURVE

The displacement graph is also important to identify adverse situations, for example when the displacements do not remain constant or increase again after a period of stability. It is important that the measurements are not stopped too early as settlements of the crown can occur after a hiatus. Possible adverse causes for increasing displacements are listed below:

- modification of the ground behaviour due to ingress of water (in the joints, fissures etc.). This can result in a reduction of the internal friction;

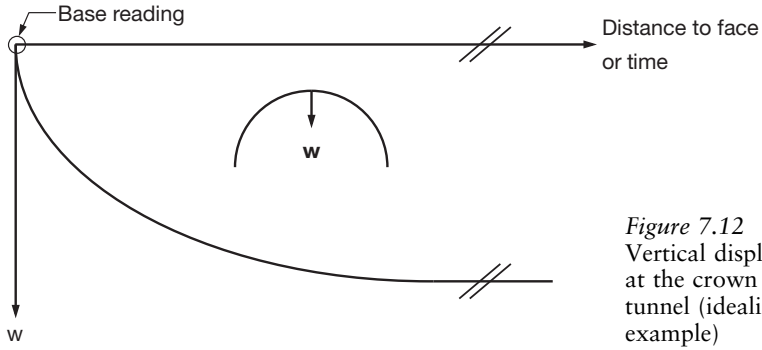


Figure 7.12 Vertical displacement at the crown of a tunnel (idealized example)

- deformation behaviour of the ground is heavily time dependent;
- sudden failure of the sprayed concrete lining;
- the horizontal shape of the measuring curve only 'pretends' that the settlements have stopped due to the intervals between measurements.

The design and scheduling of the excavation sequence can also create additional new displacements after an apparent halt, for example:

- restart of an excavation after a longer pause (e.g. the Christmas shutdown);
- the time delayed excavation of successive headings (e.g. bench excavation follows crown) creates a new stress redistribution and thus leads to further displacements of the leading heading.

The bedding of the footing of the crown is being removed during the excavation of the bench so that the sprayed concrete has to arch over the excavated area. The area of this longitudinal support is thus likely to be affected by further displacements in addition to the stress redistribution due to the excavation of the ground (Figure 7.13).

Figure 7.14 shows by way of example the vertical displacements of the monitoring cross section at Chainage 650 of the leading side wall drift in Section W of the Lainzer Tunnel LT31, Vienna (for detailed information

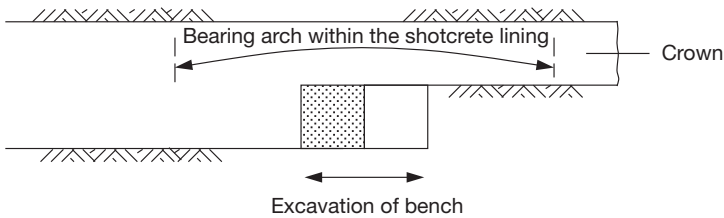


Figure 7.13 Longitudinal support within the lining during bench excavation

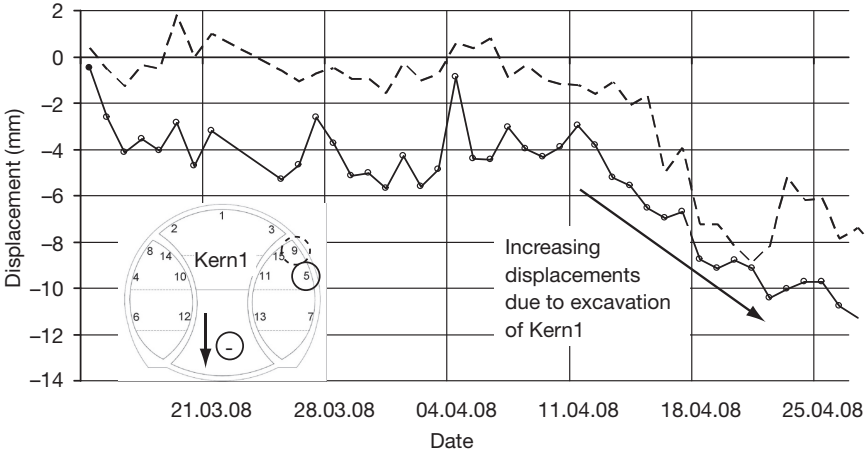


Figure 7.14 Displacement versus time graph showing the development of new displacements in the right-hand side wall drift following excavation of Kern1 (LT31, Vienna, Section W, Chainage 650)

on this project see section 8.3 of the case histories). For reasons of clarity only two points (5, 9) are displayed in the figure. The displacements remained constant for approximately four weeks, until mid April, when new displacements appeared, nearly 2.5 times as large as the old ones. The reason for this was the excavation of the remaining crown (Kern1) following the leading side wall drift after a scheduled delay of four weeks.

7.3.4.4 Interpretation of the measurements: comparative observation

The task of the interpretation of the measurements is not only to see whether a new stress state in the combined system ground-lining has been established, but also to derive a statement with respect to the stability and to check the measured displacements against triggers. At first, one would compare the measured displacements with those calculated in the design. In section 8.2, the London Heathrow T5 PiccEx Junction case history, an example is given in which the comparison of calculation and measurement successfully led to an improvement in the safety of a neighbouring tunnel. However, in general some difficulties do arise. During the design a tunnel is divided into sections, in which similar geological conditions are anticipated. Calculations are prepared for each section, the results of which include expected displacement in the vertical and horizontal directions. During excavation a number of monitoring cross sections are installed within the respective sections. However, each monitoring cross section will record different displacement values. The reason for this is obvious: the *in situ* displacements depend on the geological conditions, on the type and quantity of the support, the time the support is installed and on the

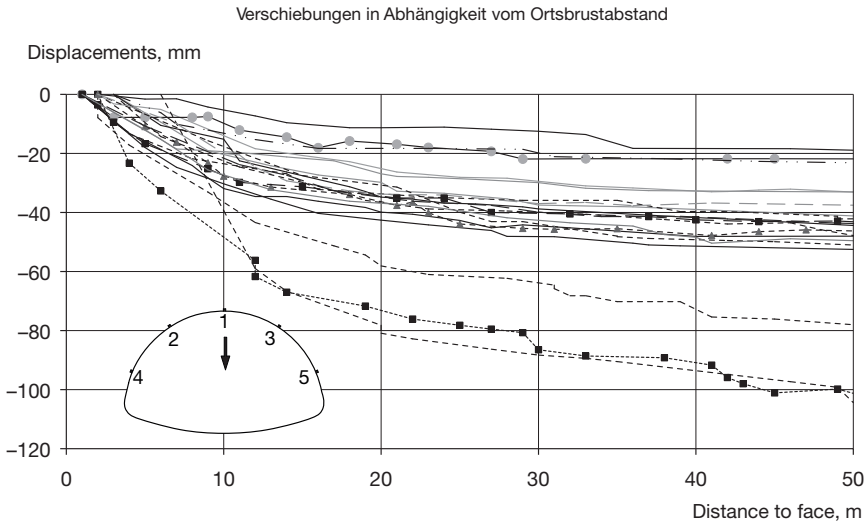


Figure 7.15 Vertical displacements for point 1 at different cross sections in the Eggetunnel for Chainage 482–684 in similar geological conditions

excavation sequence. The combination of these factors is never the same, and as a consequence there will be different displacements in each monitoring cross section. Due to the time and expense involved in computing these displacements, and the limited accuracy of the input data, the technique of parameter variation can only cover a small spectrum of the possible permutations. Quick *et al.* (2001) give an example of such a parametric study using three-dimensional finite element method calculations to simulate actual displacements measured. In their presentation they come to the conclusion that, because of the wide scatter of measured displacements, a comparison of the measured and the theoretical displacements makes little sense, as it is not known which of the monitoring cross sections is actually represented by the calculations.

This result reflects the authors' own experience. Figure 7.15 shows, by way of example, some monitoring results of the Eggetunnel (for further details on this project see section 8.1). This shows the vertical displacements of monitoring point No. 1 (crown) at different cross sections between Chainage 482 and 684. In this section similar geological conditions were predicted and encountered. Nevertheless the displacements for the different cross sections range from 20 mm to more than 100 mm; in other words, the displacements scatter by more than 500% for just one monitoring point. Monitoring results always come with a range and never with a single value. This makes it impossible to derive a statement with respect to stability. In addition, all the other monitoring points have to be taken into account as well.

In conclusion, therefore, a comparison of calculated and measured displacements is often not possible because tunnelling conditions are too variable to be modelled adequately in the calculations. In addition, it must not be forgotten that the measured displacements are the true displacements, and not the calculated values. If the measured displacements cannot be checked against calculated triggers, the monitoring cross sections have to be checked against each other (comparative observation). The aim is to filter a 'normal' range of displacements and to identify adverse trends well in advance, i.e. before any critical situation arises. This can be done by adopting a monitoring cross section as a reference section. A precondition for this is of course that no complications arise in the reference section. Alternatively it could be done by looking at average displacements over a couple of monitoring cross sections.

Figure 7.15 indicates that there is obviously an accumulation of curves around 40 mm. One could possibly refer to this as the 'normal' behaviour of the displacements. This would cut the scatter to a mere 40 mm ± 10 mm. However, the deviation outside 'normal' behaviour must not be ignored.

The average displacement can be set as a trigger. But one must be aware that it takes some time and a couple of monitoring cross sections to establish average values. In consequence, triggers must be adjusted during excavation (further information on trigger values can be found in section 7.3.2).

7.3.4.5 Interpretation of the measurements: deformation

Another important reason for monitoring is to aid the estimation of the residual bearing capacity of the lining. When new displacements occur, it is necessary to know if these additional displacements are likely to overstress the lining. If the calculated and measured displacements differ considerably, or if the displacements scatter widely as shown in Figure 7.15, it is also necessary to check the bearing capacity of the lining against collapse. Concrete often has cracks, and so does sprayed concrete. However, it is essential to investigate the cracks in the sprayed concrete because of the potentially severe consequences in this case (an example is given in section 8.3.6). There are many reasons for determining the bearing capacity of the lining, especially in a shallow tunnel in soft ground. To do this, deformations are needed. Deformations result from different displacements within a monitoring cross section (vertical, horizontal and longitudinal) and they are an indication of the stress state of the lining. It is possible to differentiate four distinct and different stress states depending on the observed displacements (Figures 7.16 to 7.20) (Rokahr *et al.* 2002). The dashed line in these figures represents the deformed lining. The displacements shown are not to scale and are exaggerated to make the deformation visible. In these figures, 'w' is the vertical displacement of the crown, 'v' is the vertical displacement of the footing, and 'h' is the horizontal displacement.

Figure 7.16 Rigid body displacement If the vertical displacement of the roof is equal to the vertical displacement of the footing, then the lining is just subject to a rigid body displacement; it is not deformed and therefore not stressed.

Figure 7.17 Roof settlement Roof settlement with none or negligible settlement of the footing is a deformation which is often encountered. The deformation generates normal forces and bending moments in the lining.

Figure 7.18 Divergence Roof settlement accompanied by an outwards movement of the footing is called divergence (the distance lengthens). This deformation is common in a ground with a low horizontal earth pressure. The deformation generates similar stress as the roof settlement shown in Figure 7.17, but with higher bending moments developing.

Figure 7.19 Convergence A combination of roof settlement and an inward movement of the footing is called convergence (the distance shortens). In the schematic shown, the vertical displacement is equal to the horizontal displacement. The original circular lining is not deformed but shortened. The radius of the circle becomes smaller. The lining is stressed by normal forces only, and no bending moments develop.

It should be noted that the deformations do not necessarily remain circular; theoretically an unlimited number of possible deformation shapes

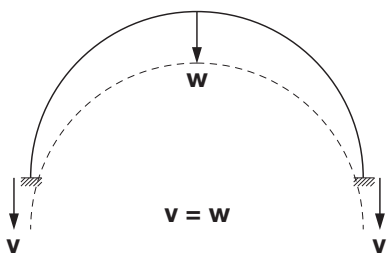


Figure 7.16 Rigid body displacement

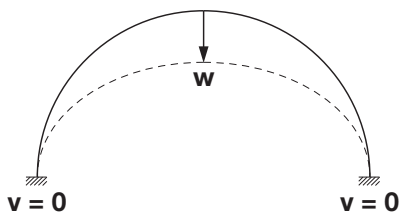


Figure 7.17 Roof settlement

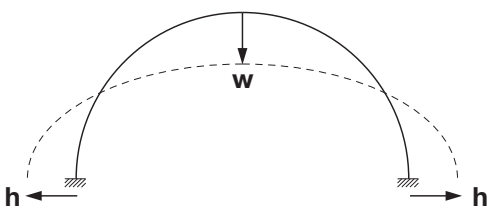


Figure 7.18 Divergence in combination with roof settlement

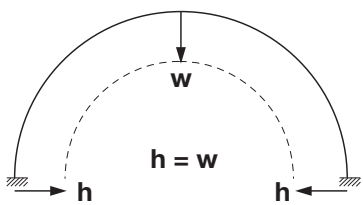


Figure 7.19 Convergence in combination with roof settlement

exist. In Figure 7.20 two possible shapes are shown, with the displacements of the roof and the footings being the same as shown in Figure 7.19. Both generate totally different stress states in the lining. To be sure of the actual deflected shape, the number of monitoring points in each monitoring cross section must be sufficient to derive an unambiguous shape of the tunnel lining from the displacements. In the example shown in Figure 7.20, representing a crown, monitoring is also required at the shoulders of the tunnel to confirm the deformed shape. Hence, at least five measuring points per monitoring cross section are necessary (see also Figure 7.10). The displacements in all three directions must be measured, i.e. horizontally, vertically, and in the longitudinal direction (x, y, z).

In order to get a better idea of the deflected shape, it is often useful to plot the displacements as vectors on the cross section, as shown in Figure 7.22a. It should be noted that depending on the scale used, it is possible to get a completely different impression from the displacements or deformations (as with all graphs!). Figure 7.22b shows the same displacements plotted with a different scale. Whereas the displacements in Figure 7.22a seem to be negligible, the ones in Figure 7.22b look worrying. Therefore, it is always a good idea to have a look at the actual values too.

It has to be emphasized, however, that it is generally difficult to make exact statements regarding the loading of a lining by looking at measurement curves

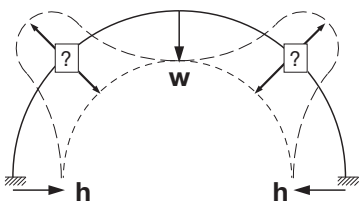


Figure 7.20 Shoulder displacements

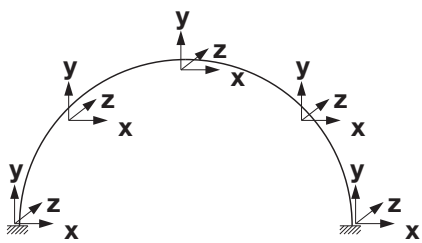


Figure 7.21 Minimum required monitoring points to create a deflected shape

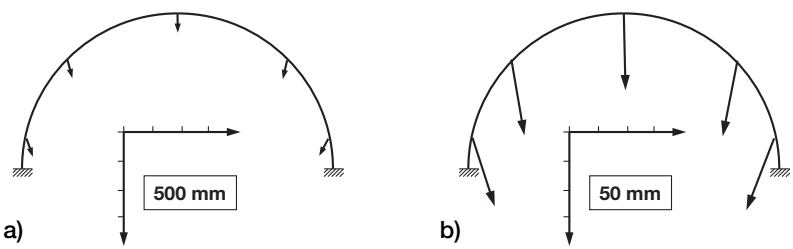


Figure 7.22 a) and b) Vertical and horizontal displacements plotted as x-y-vectors using different scales (Rokahr *et al.* 2002)

or deflected shapes, especially with regard to the level of the lining capacity used, and conversely how much capacity is still left. It is therefore necessary to determine a stress-intensity-index.

7.3.4.6 Interpretation of the measurements: stress-intensity-index

The stress-intensity-index, η, is based on the measured displacements and is defined as the ratio of the existing stress and the permissible stress in the tunnel lining at a certain point in time. If the stress-intensity-index is 100%, the ultimate strength of the sprayed concrete has been reached (Zachow and Vavrovsky 1995, Stärk *et al.* 2001, Rokahr and Zachow 1997).

Unfortunately it is not possible to transfer the deformation directly into a stress state of the sprayed concrete lining. The sprayed concrete is applied to the tunnel wall when it is still soft, and in this state the sprayed concrete is already loaded by the stress redistribution in the ground and by further excavation. The young sprayed concrete is still a long way from the standardized 28-day-values for compressive stress, compressive strain, and Young's modulus. However, the sprayed concrete gains strength, i.e. its material parameters and stress-strain behaviour change rapidly during the first few days as a function of time. Furthermore, the stress will generate a significant time- and stress-dependent creep strain (or relaxation) in the sprayed concrete lining. A determination of the actual stress state must take into account the time, importantly the time of the actual reading, the age of the sprayed concrete at the time of this reading, and the deformation history before this reading.

Figure 7.23a gives a simple example to help clarify this. It is assumed in this case that there is no displacement other than the roof settlement. Therefore the vertical axis displays the relative settlement between the roof and the footing or the deformation, respectively. After comparing both of the measuring curves one is tempted to declare the sprayed concrete lining belonging to curve 2 as less loaded than the sprayed concrete lining belonging to curve 1. However, exactly the opposite is the case. Despite the larger total settlement, the sprayed concrete lining 1 is less loaded than lining 2 and although both deformation curves are geometrically similar, the stress-intensity is quite different (Figure 7.23b). The reason for this surprising statement is the creep ability of the young sprayed concrete. Creep and relaxation in the first 8 to 12 days avoids a large proportion of the load.

Sprayed concrete lining 1 starts at the first day with, in this case, a high deformation of approximately 15 mm. This results in a high stress-intensity-index of more than 50%. Since creeping of the sprayed concrete depends on several factors, such as the sprayed concrete mix, its age and also on the stress-intensity, the high stress generates significant creeping. In addition, the increase in strength is the highest during the first day. Hence, the stress-intensity reduces on the second day. During subsequent days the

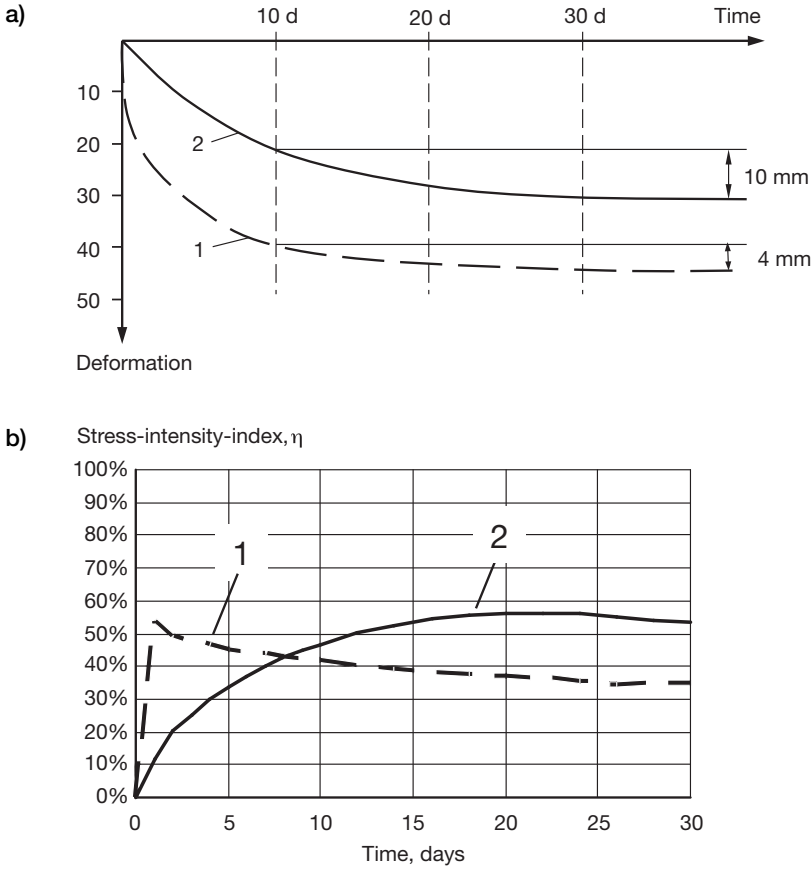


Figure 7.23 a) Example deformations of the crown depending on time (d = time in days), b) corresponding stress-intensity-index

deformation still increases, but with ongoing creeping and increasing strength the stress-intensity reduces steadily down to approximately 35%.

Sprayed concrete lining 2 starts with a small value of approximately 2 mm deformation, which generates neither a high stress-intensity nor a high creep rate. During subsequent days the deformation increases in a more regular way (compared to curve 1). As a result the stress-intensity grows slowly. The creep ability reduces with time and the gain of strength also slows down. A large proportion of the deformation affects sprayed concrete lining 2 when the creep ability is already reduced. To highlight this, the deformation after 10 days of both sprayed concrete linings is marked on the figure. In this example, lining 1 shows only 4 mm of settlement between the 10th and 30th day, but lining 2 shows 10 mm.

Thus, in order to determine the level of loading of a sprayed concrete lining in tunnelling, the age at which the sprayed concrete is deformed is

significant. The later the deformation occurs the more they are likely to generate a high stress level. As a rule of thumb one can state that creeping after 28 days is negligible.

In order to get reliable results when calculating the stress-intensity-index, the following points must be taken into account:

- the stress-intensity-index must be based upon the actual monitoring results;
- calculation must be done with every displacement reading, usually on a daily basis;
- the actual contour of the sprayed concrete lining (which is given by the position of the monitoring points);
- the actual absolute location of the monitoring points relative to the tunnel axis;
- the time-dependent development of the ultimate strain of the sprayed concrete:
- a non-linear, time-dependent material law of the sprayed concrete to cope with creep and relaxation;
- since everything is time dependent, the deformation history is necessary.

The use of the stress-intensity-index is common on NATM/SCL tunnels in soft ground. Rokahr and Zachow (2009) give details of two different methods currently in use.

7.3.4.7 Measuring frequency and duration

As long as the displacements change significantly, a measurement is taken at least every day. After they have stabilized, the monitoring frequency can be reduced stepwise to every other day, twice a week, weekly and finally monthly. If the total cross section is excavated by a staggered multi-face heading, the monitoring frequency should be increased again to daily measurements well in advance of the following heading. If adverse trends are detected or displacements begin to increase again, readings should be taken at least once a day, or even potentially twice a day.

Generally, measurements must be taken until the displacements have become stable. The measurements should be extended until the face is at least a distance of 4D from the monitoring cross section. An exactly defined period cannot be given as the time over which one has to measure is mainly dependent on the ground type and the advance rate. Possible stoppages during excavation can increase the period of the measurements.

7.3.4.8 Contingency measures

When constructing a tunnel, it is in the nature of the project that there are gaps in the knowledge of the main construction material, i.e. the ground

(see Chapter 2). The geological model for the ground is only an estimation with the assessment of its bearing capacity resting largely on experience, and 'even the most complex calculation is still only an approximation of reality' (Thomas 2009b). Therefore, during the excavation, it is possible that the interaction between the ground and the support does not behave according to the estimations and calculations, but potentially diverges from this in a negative way. This presents an exceptional circumstance, for which one has to be prepared with contingency measures.

Contingency measures can become necessary if the support is not sufficient to ensure quality, usability or stability. This can manifest itself by larger than expected surface settlements, larger than expected in-tunnel displacements, an unstable face, overloading of the ground, overloading of the lining or other performance indicators, which are not in accordance with the design or expectation. Special attention should be paid to the overloading of the lining because of the potentially severe consequences for the overall stability of the tunnel. There are visual signs (cracks) and acoustic signs (the concrete sounds dull and hollow) that give an indication of an overloading of the lining. However, not every crack in the sprayed concrete lining results in a collapse or is the result of overloading of the system. But every crack should be taken seriously due to its potential origin and the resulting serious consequences.

Prior to taking further measures, it is normal to observe the cracks: are they getting longer, do they open up? The end of the crack should be marked with a colour and the date. It is then easy to check if the crack is getting longer. If it is, then a new mark has to be made. Measuring the width of the crack (to see whether the crack opens) is difficult due to the rough surface of the sprayed concrete. It is possible to place gypsum patches over the crack. Gypsum hardens quickly and is very brittle and hence it will break immediately if the crack opens further. A better option is to install a Tell Tale™ over the crack (Figure 7.24). These consist of two overlying transparent plates which can move relative to each other. A scale on the plates shows exactly if the crack under the Tell Tale™ is changing and by how much.

The sprayed concrete around the crack should be hit with a hammer or similar. If sprayed concrete layers have separated from each other, then the hammer blow sounds dull and hollow. This is a clear indication of a weakened lining. Taking cores from the sprayed concrete lining just over the crack provides good information on whether the crack is due to bending or shear forces, or just due to shrinkage of the sprayed concrete (which is not a matter of concern with respect to stability). Cores will also show if the sprayed concrete is seriously damaged or still intact.

If overstressing of the lining is suspected, then an intensified monitoring regime must be implemented: the measuring intervals should be shortened and the monitoring data from the particular area must be analysed very thoroughly. It is not only overstressing of the lining and a potential collapse

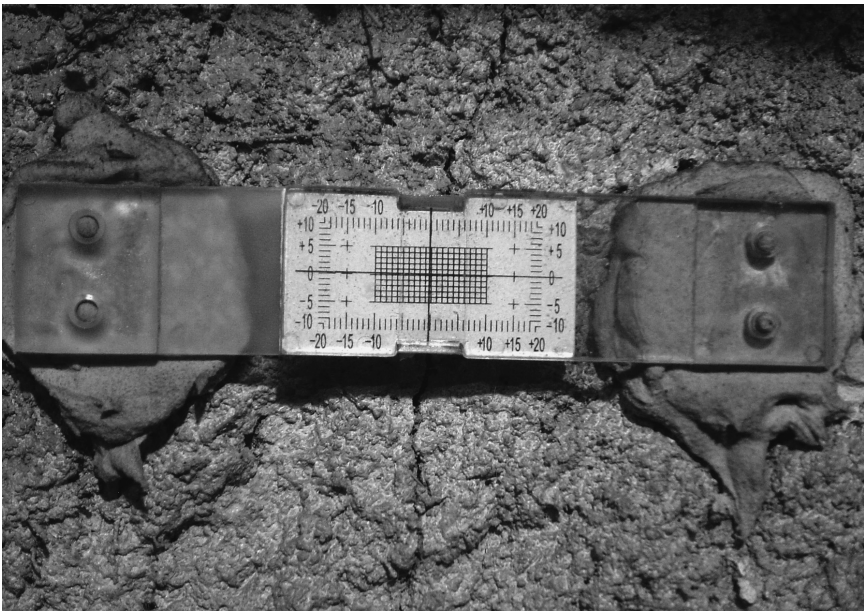


Figure 7.24 Tell Tale™ placed over a crack in a sprayed concrete lining

which make contingency measures necessary. Larger than expected displacements, at the ground surface or in the tunnel, can cause potential harm to third party structures or can reduce the usability of the tunnel.

A list of common contingency measures is given below. This does not claim to include all measures available:

ELEPHANT'S FEET (SETTLEMENT REDUCTION)

Elephant's feet are enlargements, which look similar to the foot of an elephant, of the sprayed concrete lining where it bears onto the ground at the sides of the tunnel, i.e. they enlarge the foundation or footing. They reduce the ground pressure under the footing and thus help to reduce the likely settlement. In order to avoid the lining protruding into the internal tunnel space, the elephant's feet are usually constructed towards the extrados, i.e. the outside of the lining.

TEMPORARY INVERT (SETTLEMENT AND CONVERGENCE REDUCTION)

A temporary invert makes the sprayed concrete lining into a closed ring. A ring closure of the lining stabilizes the whole system. The temporary invert reduces the ground pressure under the side footings and thus helps to reduce the likely settlement. In addition, it reduces convergence since it stiffens the lining horizontally. This is usually used in addition to elephant's feet.

FOOTING PILES (SETTLEMENT REDUCTION)

Footing piles are steel rods located in the footing of the lining. Boreholes are drilled vertically from the elephant's feet down into the ground. Usually two piles are placed on either side with each advance. The piles distribute the load deeper into the ground and thus reduce the bearing pressure under the footing. The steel rods have a diameter of approximately 30 mm to 70 mm, and the borehole is grouted to guarantee friction between the ground and the piles.

FOREPOLING (PROVIDES OVERHEAD PROTECTION OF AN UNSUPPORTED HEADING)

Steel rods are placed around the circumference of the roof at a spacing of 20 cm to 40 cm in the direction of the excavation. Steel rods are rammed or bored into the ground: the girders act as an abutment inside the tunnel, and the ground acts as an abutment in front of the face. Thus they protect the work area from any ground falling from the roof of the tunnel until the excavation of the advance has been completed and the support installed. The common lengths of rod are 3 or 4 m, with a diameter of approximately 22 mm to 32 mm. They are placed with every advance. Forepoling needs to have an overlap of one or two advance lengths. Therefore the steel rods are slightly inclined by a few degrees. (See section 4.2.5 for further details.)

SHEET PILING (PROVIDES OVERHEAD PROTECTION OF AN UNSUPPORTED HEADING)

This method is similar to forepoling, but uses steel sheets instead of rods. The steel sheets are rammed into place and are positioned so that they touch. Sheets are used in coarse soil. (See section 4.2.5 for further details.)

FACE SUPPORT (HELPS TO PREVENT FAILURE OF THE FACE AND REDUCES GROUND LOSS)

The face is sprayed with between 3 cm to 20 cm of sprayed concrete, mesh or steel fibre reinforced if the thickness exceeds 10 cm. The sprayed concrete can be accompanied by face anchors (horizontal anchors in the direction of the excavation). Face anchors usually overlap by several times the advance length. (See section 4.2.4 for further details.)

SUPPORT CORE (HELPS TO PREVENT FAILURE OF THE FACE AND REDUCES GROUND LOSS)

The tunnel advance is only excavated around the circumference; the centre of the face is not excavated and acts as an abutment for the face. A support core can be sealed with sprayed concrete and can be bolted against the face with face anchors.

ADVANCE LENGTH

Shorter advances can help prevent overstressing of the ground and reduce settlement. With shorter advances, the disturbance of the ground is reduced,

as is the time necessary for excavation and completion of the support. Thus, the load is transferred from the ground to the support more quickly. The installed support potentially needs to be increased to cope with the additional load.

DIVIDED FACE

Dividing the face into smaller cross sections has the same effect as using shorter advance lengths. In addition, this also increases the face stability.

ADVANCE RATE

Limiting the number of advances per day has the same effect as shorter advance lengths. The stress redistribution due to the excavation acts on an older and therefore stiffer lining, which takes a larger part of the load, and thus reduces the stress in the ground.

ANCHORING

Systematic anchoring avoids or reduces the loosening and the weakening of the ground due to the deformation following the excavation. The anchors 'nail' the ground together and thus help it to maintain its bearing capacity. Anchors are only active if there is a rigid bond between the anchor and the ground, which is achieved for the most common anchor types by using injected mortar. (See section 4.2.4 for further details.)

LINING

A thicker and stiffer lining reduces settlement and helps to reduce overstressing of the lining. The stress redistribution due to the excavation acts on a thicker and, therefore, stiffer lining, which takes a larger part of the load, and thus reduces the stress on the ground. As the lining is much stiffer than the (soft) ground, this reduces the settlement. Furthermore, if the lining itself shows symptoms of overstressing, it needs to be strengthened by thickening. In this case, the excavation must become larger to avoid the lining encroaching into the final tunnel profile.

SEALING THE GROUND

A 'flash' coat of sprayed concrete protects against any ground and rocks falling off the unsupported heading. It also hinders groundwater flowing into the tunnel. Flowing water can wash fine particles out of the ground causing the latter to lose cohesion and destabilize. Water must therefore be controlled.

GROUTING

This increases the bearing capacity of the ground and reduces or stops water inflow. (See section 4.2.3 for further details.)

In the case of overstressing of the lining such that a collapse would be unavoidable if nothing were done, there are still usually some options to strengthen the support long before the tunnel has to be evacuated. Some of the most suitable contingency measures are:

STOP EXCAVATION

Any additional load has to be avoided.

POST ANCHORING

For this measure, additional anchors are installed in the already completed tunnel support. If grouted anchors are used it takes approximately a day until the mortar around the anchor hardens and for the method to become effective.

POST SHOTCRETING

In order to strengthen the sprayed concrete lining, an additional layer of sprayed concrete can be sprayed in areas where the lining is already overstressed. This measure can be carried out quickly because the sprayed concrete is available immediately on site. However, it also takes at least 12 hours for the sprayed concrete lining to become reasonably load bearing. It has to be considered that at a later stage it is essential to remove the additional sprayed concrete lining as it encroaches into the final tunnel profile. In many cases it is often necessary to renew the overloaded sprayed concrete so that the additional sprayed concrete lining is removed anyway. Post shotcreting all of the lining of an overstressed section can be very time consuming. In order to gain time it is possible to spray a couple of sprayed concrete ribs placed around the circumference (see section 4.3.2).

BACKFILL

Placing backfill against the face can re-stabilize it. The additional load from the backfill can stop further cracking of a broken invert.

TREE TRUNKS

As an immediate measure against a threatening collapse, this option is well suited for cross sections of up to 6 m in height (for example, in the crown heading, Figure 7.25). For larger heights, tree trunks are not suitable from a practical and structural point of view (danger of flexural bending increases). Tree trunks can easily be adjusted to the actual geometry (by using a saw). If fixed well with wooden wedges and sprayed concrete on either side, tree trunks can immediately take loads and help to avoid a collapse. It is compulsory to have a supply of tree trunks on a tunnel site in order to be able to react immediately if required. If trees have to be placed, it is often necessary to renew the destroyed sprayed concrete lining

Figure 7.25 Emergency support measures using tree trunks during the tunnel excavation (Eggetunnel, crown heading) (courtesy of Professor Dr-Ing. habil. Reinhard B. Rokahr, photograph by Ulrich Mertens DFA DGPh, Atelier für Kunst und Fotografie, Hamburg)

in sections. The tree trunks themselves can be removed relatively quickly if necessary care is taken. The tree trunks should be positioned in such a way that the excavation plant can still pass. Of course, steel beams or other similar elements can also be used, but they are not as easy to cut to the required length.

7.3.5 Instrumentation for in-tunnel and ground monitoring

In addition to laser theodolites, steel tapes and crack monitoring mentioned in the previous section for in-tunnel monitoring during construction, pressure cells can be used to determine the stresses in the tunnel lining. Pressure (or 'stress') cells can be installed between the lining and the ground (total pressure cells, tangential pressure cells), or cast into the lining (radial pressure cells) and can use either liquid pressure or vibrating wire transducers. They need careful installation and experience to interpret the results due to the complexities of the sprayed concrete behaviour, which causes much debate as to their reliability. However, research by Jones (2007) has suggested various procedures to reduce the potential errors when using pressure cells.

Furthermore, it is important to monitor the ground around the tunnel during construction in order to assess its behaviour. There are a number of common instruments available and these are briefly described below (see section 7.3.6 for references).

BOREHOLE MAGNET EXTENSOMETER (RELATIVE VERTICAL MOVEMENT)

These devices consist of a series of circular magnets fixed at certain levels within the borehole to either a rigid or telescopic access pipe. A probe is inserted to record the level of each magnet. The rigid plastic tube and 'spider' magnets can cope with small vertical compressions of up to 1%.

BOREHOLE ROD OR INVAR TAPE EXTENSOMETERS (RELATIVE VERTICAL MOVEMENT)

These can consist of simple rods of different lengths anchored at different levels within the borehole. More sophisticated methods involving linear variable differential transformers (LVDTs) are also available. Wire based extensometers, although more difficult to install than rod extensometers, are useful over long distances.

SATELLITE GEODESY (RELATIVE VERTICAL MOVEMENT)

This method is useful for monitoring relative movements over large areas of the ground surface.

CONVERGENCE GAUGES – (LATERAL DISPLACEMENT)

Gauges, consisting of tape, wire and rods with a deformation indicator, can be used to measure horizontal displacements between permanent anchor points, for example ground surface settlement points.

BOREHOLE INCLINOMETER PROBES (CHANGE IN INCLINATION)

When horizontal deformation measurements are required within the ground, a permanently installed vertical casing is used. A probe containing a gravity-sensing transducer is inserted into the casing. The guide casing usually has tracking grooves for controlling the orientation of the probe. An alternative system uses borehole electrolevels.

HORIZONTAL BOREHOLE DEFLECTOMETER (CHANGE IN INCLINATION)

These rely on angle transducers instead of tilt transducers (as used in inclinometer systems) and this means that they can be used in inclined or horizontal boreholes (as well as vertical) as the sensors are not reliant on gravity.

'PUSH IN' TOTAL PRESSURE CELLS (CHANGES IN EARTH PRESSURE)

These can be either diaphragm or hydraulic cells. Diaphragm cells consist of a circular membrane with strain gauges attached. The membrane deflects under pressure and the strains measured can be related to the change in pressure. The hydraulic cells consist of two membranes sealed around the edge, with the gap between them filled with liquid. The pressure acting on the cell is measured via the pressure of the liquid in the cell. They can be used in combination with piezometers to obtain effective stress values.

STANDPIPE PIEZOMETER (CHANGE IN GROUND WATER PRESSURE)

Used to monitor groundwater pressure at a particular elevation. The main disadvantage of these piezometers is the problem of assessing 'real time' fluctuations in piezometric head due to manual reading and time lags.

PNEUMATIC PIEZOMETER (PORE PRESSURES ARE BALANCED BY APPLIED PNEUMATIC PRESSURES/CHANGE IN WATER PRESSURE)

Uses the pressure of gas on a flexible diaphragm to measure the external pore water pressure, i.e. the external gas pressure is increased until it balances the water pressure. These instruments have a short time lag.

VIBRATING WIRE PIEZOMETER (CHANGE IN WATER PRESSURE)

Uses a vibrating wire strain gauge attached to a diaphragm. As the pore water pressure changes, the diaphragm deflects and registers a change in strain. This can be related to the magnitude of the pressure. These instruments have a short time lag and are easy to read.

STRAIN GAUGED BOREHOLE EXTENSOMETERS INSTALLED FROM WITHIN A TUNNEL (GROUND DEFORMATIONS)

These can be directly measured and have multiple extensometers in one borehole. Using vibrating wire strain gauges they can be automatically data-logged. The longest/deepest extensometer is assumed to be beyond the disturbed zone, otherwise relative movements are underestimated.

Example of an instrumentation layout An example of an instrumentation layout used to monitor the ground behaviour is shown in Figure 7.26. This monitoring arrangement was used as part of a research project conducted by Imperial College on the London Underground Jubilee Line Extension project in the UK, and illustrates the type of instruments that can be used. Further details of this and the other monitoring conducted on this project can be found in Burland *et al.* (2001a and b).

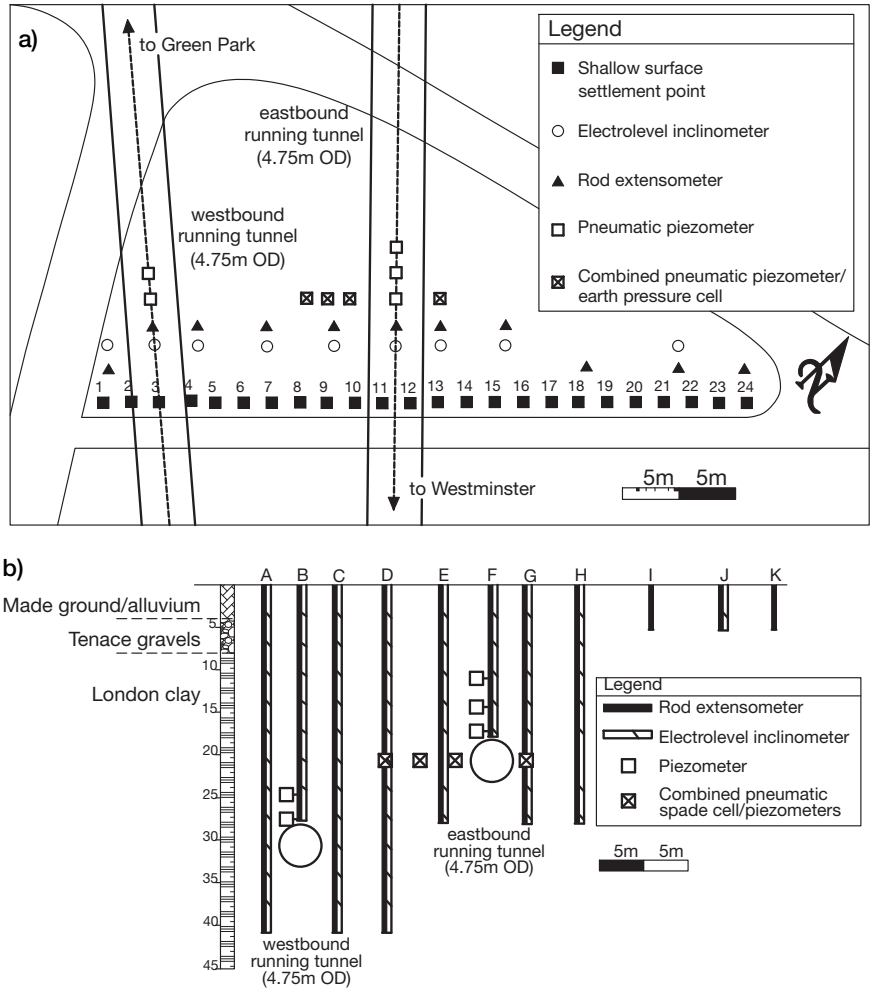


Figure 7.26 An example of the instrumentation used to monitor the ground behaviour resulting from a bored tunnel as part of the Jubilee Line Extension project on the London Underground, UK, a) site plan, and b) instrumented cross section (after Standing *et al.* 1996)

7.3.6 Instrumentation for monitoring of existing structures

A few of the more commonly used instrumentation for monitoring existing structures (both surface and subsurface) are briefly described in this section. There is considerable literature available on this topic, for example Dunnicliff and Green (1993), Clayton *et al.* (2000), BTS/ICE (2004) and Kavvadas (2005). A detailed list of commonly used instrumentation for tunnelling projects is provided by BTS/ICE (2004), together with their

respective range, resolution and accuracy. (These references also apply to section 7.3.5.)

AUTOMATED TOTAL STATIONS (RELATIVE MOVEMENT)

These have been used in recent years in conjunction with optical targets attached to existing structures. However, traditional survey techniques (theodolites, total stations and levels) are still commonly used. A network of automated total stations was utilized on the redevelopment of King's Cross Station, London, UK and the associated tunnelled connections to existing infrastructure as reported by Beth and Obre (2005). In this case, a network of automated total stations was used to monitor both above ground structures and also within operating station tunnels during the works. Each total station was used to observe a group of reflective optical prisms located on the structures as well as reference prisms outside the zone of influence of the construction works where possible. This system has also been used in Hong Kong and Amsterdam, Netherlands (van Hasselt *et al.* 1999, van der Poel *et al.* 2006). Although automated total stations can also be used within existing tunnels to monitor displacements, in some metro systems the running tunnels are too small to have such a system in the crown of the tunnel (for example London Underground running tunnels), and hence the system is confined to larger diameter station tunnels.

PRECISE LIQUID LEVEL SETTLEMENT GAUGES (RELATIVE VERTICAL MOVEMENT)

These are instruments that incorporate a liquid-filled tube or pipe for the determination of relative elevation. Relative elevation is determined either from the equivalence of the liquid level in a manometer or from the pressure transmitted by the liquid. These have been used in existing metro tunnels to monitor 'rotations' when the tunnels are affected by new construction works. The tube is passed from one side of the tunnel under the track and up the other side.

PLUMB-LINES (CHANGE IN INCLINATION)

These can be used for monitoring the tilt of structures by measuring the horizontal distance between two points at different elevations. Direct plumb-lines consist of a weight suspended from the highest possible elevation of a structure and measure the horizontal movement of the suspension point relative to a point at the base, about which the weight moves. Inverted plumb-lines have a similar operation, but the plumb-line wire is fixed at both ends and movement is observed at an intermediate elevation. A digitized plumb-line was used as part of the monitoring of the Big Ben Clock Tower in London, UK during the construction of the Jubilee Line Extension project. The plumb-line was suspended from 55 m above

the ground, to provide an accurate and precise tilt measurement. The movement of the base of the plumb-line was sensed on a digitizing tablet (placed just below the plumb) and the measurements were processed in real-time. This was important as it provided continuous feedback to the compensation grouting (section 4.2.8) that was being used as a protective measure during these construction works (Kavvadas 2005).

TELL TALE™ AND CALLIPER PINS/MICROMETER (DEMEC™ GAUGES) (CRACK OR JOINT MOVEMENT)

These are manual methods for monitoring structural damage, such as cracks. The distance between measuring studs attached to the structure are measured accurately using a Demec™ gauge (basically a distance measurement device). These devices can also be used to monitor tunnel lining as detailed in section 7.3.4.8, Figure 7.24.

VIBRATING WIRE STRAIN GAUGES (STRAIN IN STRUCTURAL MEMBER OR LINING)

These are the main way of monitoring individual structural members and are accurate, robust and reliable.

FIBRE OPTICS (STRAIN IN STRUCTURAL MEMBER OR LINING)

These are based on the ability of glass fibres to carry light from a source. These fibres can be embedded in concrete or attached directly to a structure, but have to incorporate a light source and optical analyser at one end. Systems can be based on Fibre Bragg Gratings, which use discrete optical strain gauges positioned along the optical fibre, i.e. discrete strain monitoring (Metje *et al.* 2008), or on pulsed light systems which allow axial deflection and bending to be monitored at all points along a fibre, i.e. continuous strain monitoring (Vorster *et al.* 2006 and Mohammad *et al.* 2007).

TAPE EXTENSOMETERS ACROSS FIXED CHORD (TUNNEL LINING DIAMETRICAL DISTORTION)

This is a relatively simple manual method, but obviously needs access to the tunnel to take readings. This limits monitoring within live metro lines to 'engineering hours', i.e. when the trains are not running. It can also disrupt construction processes if used in new tunnels. (see section 7.3.4.1)

BASSETT CONVERGENCE SYSTEM (TUNNEL LINING DIAMETRICAL DISTORTION)

The Bassett Convergence system (Bassett *et al.* 1999) is based on a series of rods and electrolevels, which are attached around the inner circumference of a tunnel. It can be used whilst the tunnel is operational, but does need

a suitable clearance between the vehicles and the tunnel wall. Electrolevels are tiltmeters that contain an electrolytic level (a sealed glass vial similar to that used on a conventional builders level, however this contains a conductive liquid and uses contacts within the vial to register changes in resistance as the vial tilts). Electrolevels can be used in 'beam' arrangements for measuring relative movements within tunnels and also on structures affected by tunnelling activities.

Example of monitoring existing tunnels An example of the monitoring arrangement that can be used in existing tunnels to assess the effects of adjacent construction activities, including new tunnels, is shown in Figure 7.27. The tape extensometers are used for monitoring diametrical distortion and levelling either side of the track can be used for relative vertical displacements of the tunnel. In addition, the levelling can be used to obtain a measure of rotation of the tunnel by taking the difference in the levels either side of the tack. Relative movements can also be obtained from electrolevel strings running along the tunnel (Cooper 2002). Automated total stations can also be used to monitor existing tunnels if there is space inside the tunnel, as described earlier in this section.

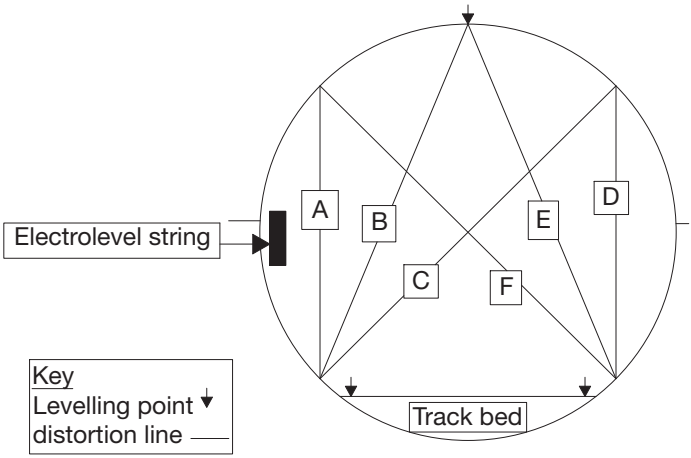


Figure 7.27 Example of monitoring within an existing tunnel in order to assess the effects of adjacent construction activities (after Cooper 2002)

8 Case studies

This chapter focuses on three different case studies. Each case study describes distinctive aspects of the tunnelling works and how some of the information described in previous chapters is applied in practice.

8.1 Eggetunnel, Germany

Unexpected invert failures of sprayed concrete linings are not unusual, despite a rigorous monitoring regime within the observational method. This case study gives an example of an invert failure, explains the failure mechanism and how to detect an invert failure by means of monitoring.

8.1.1 Project overview

The railroad line connecting Kassel and Paderborn, Germany, of which the Eggetunnel is a part, has been upgraded to allow higher speeds and to increase the route capacity. The Eggetunnel crosses the so called Egge Mountains between the towns of Willebadessen and Neuenheerse, both located in the federal state of North-Rhine-Westphalia. The tunnel became necessary because the existing railroad was affected by landslides which caused damage and delays. It was found to be safer, more efficient and less costly to build a tunnel right through the mountains rather than to stabilize the unstable slopes. The two-track railroad tunnel, length 2880 m and width 14.5 m, was constructed between 1998 and 2000, and was opened in 2003.

Construction commenced from both ends of the tunnel. From the northern portal, where hard rock formations were dominant (sand and limestone), the excavation was achieved using drill and blast, and from the south portal, where soft rock (clay) was present, excavator and side wall drifts were used. Temporary support was provided by a sprayed concrete lining, lattice girders and wire mesh reinforcement throughout the tunnel.

Some parts of the sprayed concrete-supported invert of the soft rock section experienced heavy cracking and failure, and despite a rigorous displacement monitoring regime this remained undiscovered for a long time and created some critical situations when finally detected.

The authors have come across this displacement pattern with a number of tunnels that have suffered from invert failures. However, only in rare occasions have invert failures been recorded sufficiently or even published. This is possibly because people are afraid to discuss their potential mistakes, or invert failure has not been considered a serious enough stability concern to be worth publishing. Due to the lack of knowledge, the latter is a popular fallacy among tunnel builders. Invert failure could cause a serious tunnel collapse, and to the authors' knowledge, a few have already occurred (John *et al.* 1987, Golser and Burger 2001). Even in some cases of heading collapses where the causes were not known unequivocally, a broken invert may be suspected of being one of the causes or even the main cause of the failure. The collapse of three tunnels at Heathrow airport during construction in October 1994, for example, can be put into this category (Rokahr and Mussger 2001).

The Eggetunnel, otherwise built and monitored perfectly, did not collapse; the invert failure was detected early enough to put contingency measures into effect. It has been chosen here by way of an example because, in this particular case, the invert failure – once detected – was well recorded, which should help to answer the question, why do invert failures often remain undetected for too long?

8.1.2 Invert failure of the total cross section in the Eggetunnel

It had been known from the top heading that the clay developed a significant long-term settlement due to the excavation. Therefore precautions were taken to protect the sprayed concrete invert of the full cross section. The thickness of the sprayed concrete was increased to 40 cm (which is about the maximum thickness for a regular sprayed concrete lining); a deep invert vault was built so that the shape of the tunnel was close to circular; in addition an invert monitoring device was developed and implemented as described below (see section 8.1.3).

The difficult clay zone extended over a distance of 250 m. In addition to the regular displacement monitoring it was agreed with all the parties involved to monitor the sprayed concrete invert in this area. The invert was covered with an approximately 3 m thick layer of backfill, which provided a track for the plant. In general the monitoring should not disturb or hinder the ongoing excavation. These demands were fulfilled by the so called 'Eggemouse' (see Figure 8.1a): During excavation a tube was placed every 15 m diagonally into the invert and was sprayed in place. The ends of each tube extended out of the sprayed concrete. A steel cylinder, attached to a cable, was pulled through every tube once a day from one end to the other (Figure 8.1b). The steel cylinder, the 'Eggemouse', was 70 mm long and 20 mm in diameter. In the case of a broken invert, the steel cylinder would not have passed through the tube and would have got stuck. Since this first use, the Eggemouse has been used in many other tunnels.

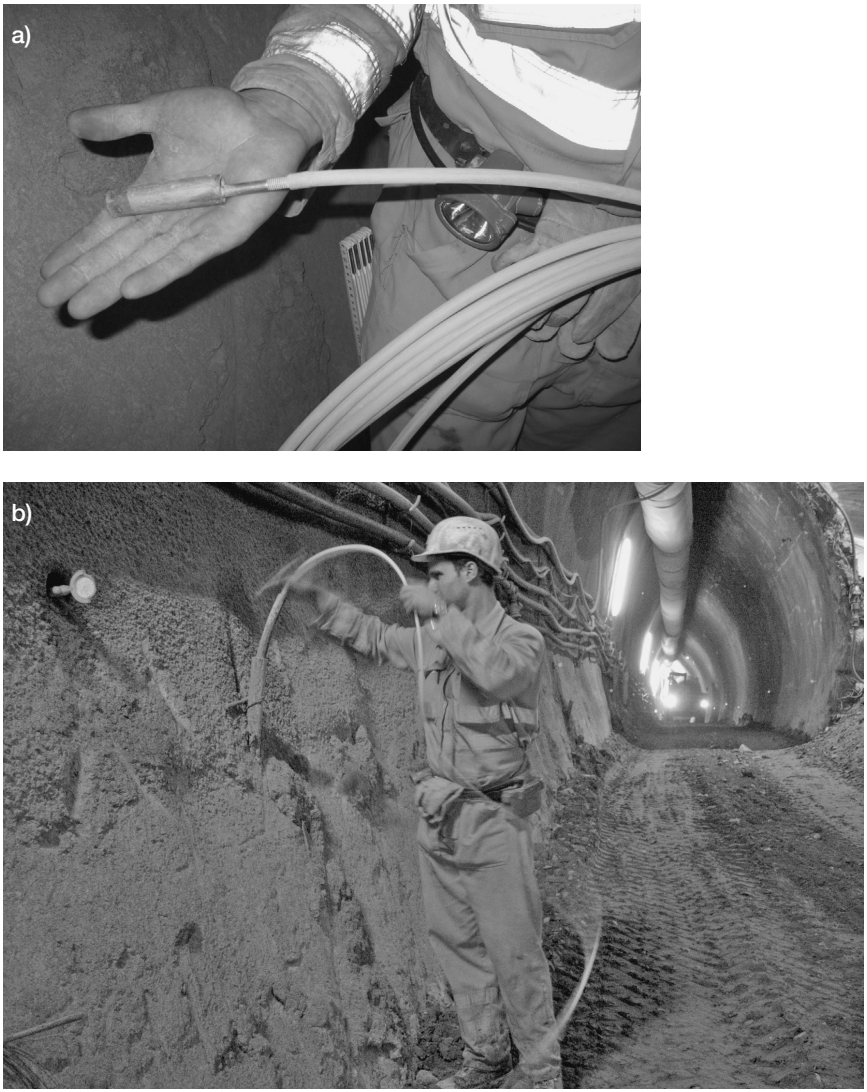


Figure 8.1 a) 'Eggemouse' and b) invert control with the 'Eggemouse' at the Lainzer Tunnel LT31, Vienna, Austria

After months of stable monitoring results, the monitoring was finally abandoned. As it turned out this was premature. After completion of the excavation, the backfill was removed to allow construction of the inner lining. While removing the backfill, the miners were surprised by a longitudinal crack, nearly 100 m long, in the middle of the invert. The displacement monitoring had not given even the smallest indication of any disturbance in the sprayed concrete lining. However, the edges of the crack began pushing

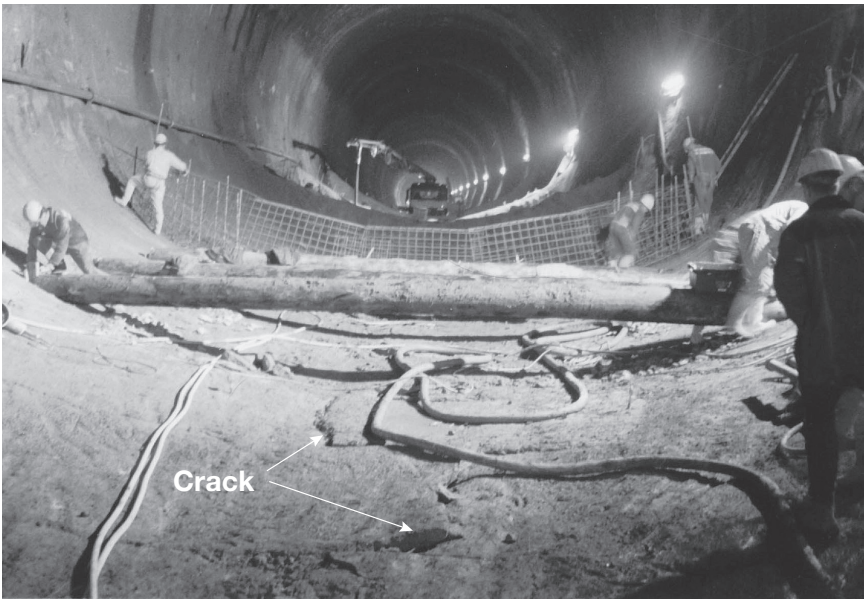


Figure 8.2 Contingency measures in the form of tree trunks at the completed cross section due to cracking after removing the backfill

over each other, with the damage to the invert getting worse and a collapse to the tunnel seemed to be a possible scenario. The movement had to be stopped immediately. But how could this be done? Replacing the backfill would have taken too long, as would stiffening the invert using massive sprayed concrete ribs. Anchoring back the invert would not have been effective enough since the displacements were mostly horizontal. The tunnel was about 13 m high, i.e. too high for setting up tree trunks. The only option was to place the tree trunks horizontally: 50 logs, each 7 to 8 m long, stopped the increasing displacements and gave enough time to reconstruct the invert with a thicker, 60 cm, sprayed concrete lining (Figure 8.2).

The invert must have failed over a longer period of time, and only the weight of the backfill provided the necessary force to maintain equilibrium; once removed, the displacements started again. But how could it happen that despite a rigorous monitoring regime, experienced staff and vigilance against any failure, massive cracks in the invert could develop without detection? This will be investigated in the following sections.

8.1.3 Sprayed concrete invert – its purpose and monitoring

In soft rock tunnels, due to low strength of the ground a sprayed concrete invert is constructed to avoid shear failure under the footings. A quick ring closure ensures the bearing capacity of both the ground and the lining. 'Quick' in this case means that the invert has to be constructed close to

the leading face; a practical distance with respect to buildability is 1 to 4 m in the top heading (as part of a larger tunnel) and 4 to 10 m in a complete cross section. After construction, the invert is covered with muck and backfill. This protects the invert against damage and provides a track for the heavy plant while the excavation continues. Depending on the shape of the tunnel and the space required for manoeuvring the plant, the thickness of this cover is approximately 1 to 3 m. However, this makes the invert invisible and it is impossible either to inspect its integrity or to install any of the common optical monitoring systems.

These days, the state-of-the-art in monitoring the invert is by doing it indirectly and involves interpreting the displacement measurements of the vault (i.e. crown or crown and bench). However, the wide-spread experience on many construction sites is that a broken invert can only be detected in a very progressive state of damage – if at all. At least the Eggemouse can tell if the invert is broken, which is of paramount importance. However, it is still not known exactly how to recognize the beginning of an invert failure by interpreting vault monitoring data. Any conclusions drawn from interpreting the monitoring data of the vault with respect to the integrity of the invert cannot be proven as the invert is not visible. The behaviour of the invert is therefore open to speculation.

Based on these experiences some essential questions kept recurring:

1 Is it generally possible to get early signs of the reduced bearing capacity of the invert by interpreting the displacement measurements of the crown?
2 Also, is it possible to assess the residual bearing capacity of the broken sprayed concrete lining?

In order to answer these questions a comprehensive research project was undertaken at Hanover University, Germany (Stärk 2002). At the beginning of the research, measured data from tunnels with broken inverts were analysed. However, the data, even at the Eggetunnel, gave no indication of any problem with the invert, and since the moment of the collapse of the invert was never known, this approach was not successful. Calculations were also not helpful, as the theoretical model could not be verified due to the lack of measured data in the invert. The research was therefore advanced by using model tests (Figure 8.3). Two different shapes were used: a full cross section with a deep invert vault, and a crown section with a temporary invert. The model tunnels were made from gypsum and embedded in clay.

The load was applied by horizontal and vertical hydraulic jacks, independently controlled to achieve different coefficients of lateral earth pressure. Depending on the geometry, 12 or 14 monitoring points were distributed over the complete cross section including, of course, the invert. The location of the monitoring points in the crown and bench corresponded to the traditional monitoring positions in a real tunnel.

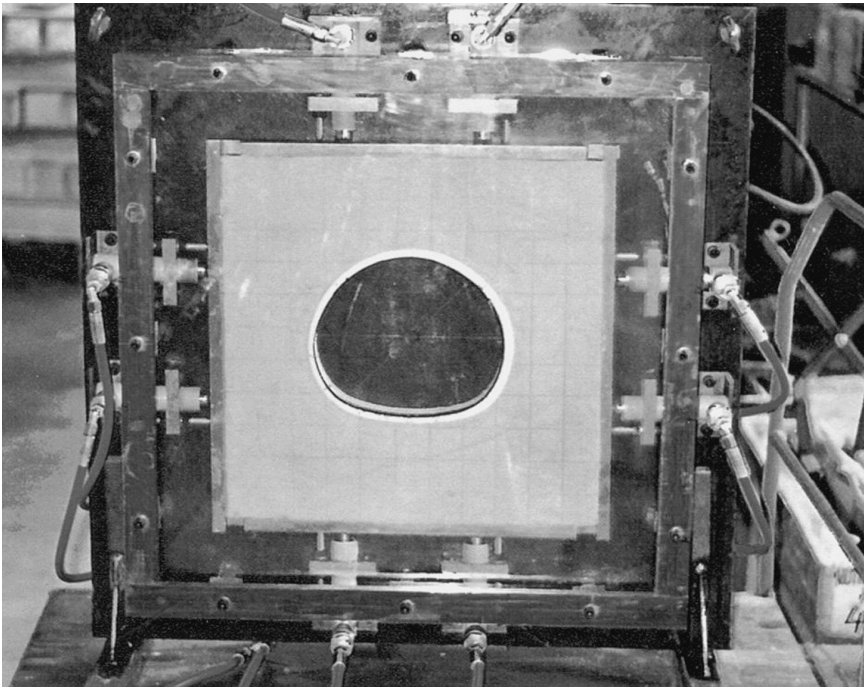


Figure 8.3 Model testing

Figure 8.4 shows, by way of example, the vertical displacements in the crown (monitoring points 1 to 5) versus the applied pressure of the jacks. As expected the displacements increased, in this case linearly, with increasing pressure. It seemed likely that the displacements in the crown would be affected by the cracking of the invert. However, nothing happened throughout the test in the crown even though five cracks developed in the invert. Looking at Figure 8.6a, this was quite a surprising result. However, in fact this reflects the typical behaviour of the crown perfectly.

According to widespread opinion, the sign of collapse of the invert is horizontal convergence of the footing (i.e. points 4 and 5, Figure 8.4). However, this could not be confirmed as inverts have collapsed in situations where convergence has occurred, where divergence has occurred, and where no horizontal displacements have happened at all.

The only way of achieving reliable information on what was happening was to monitor the invert itself, and especially to determine the stress-intensity-index of the lining. The stress-intensity-index is based on the monitored displacements, and according to Rokahr and Zachow (1997) is defined as the ratio of the existing stress and the permissible stress at a certain point in time. If the stress-intensity-index is 100%, the ultimate strength of the lining has been reached. With the stress-intensity-index the

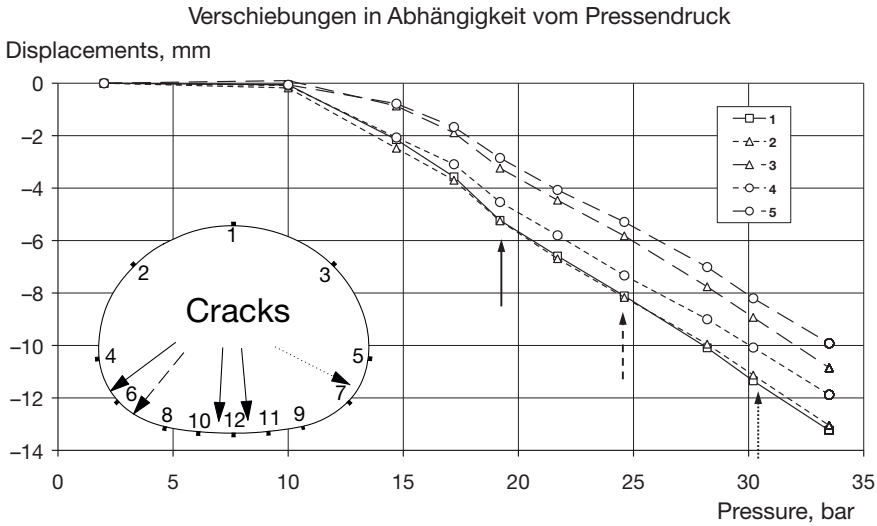


Figure 8.4 Vertical displacements at the crown and corresponding cracks

actual stress state can be determined in real time, so that there are no uncertainties about the residual bearing capacity due to increasing displacements. Figure 8.5 shows the development of the stress-intensity-index in two adjacent locations on the lining, at the crown and at the invert. The stress-intensity-index in the crown (points 1–3–5) initially reaches an untypically high value of approximately 40%. The remaining curve has typical values

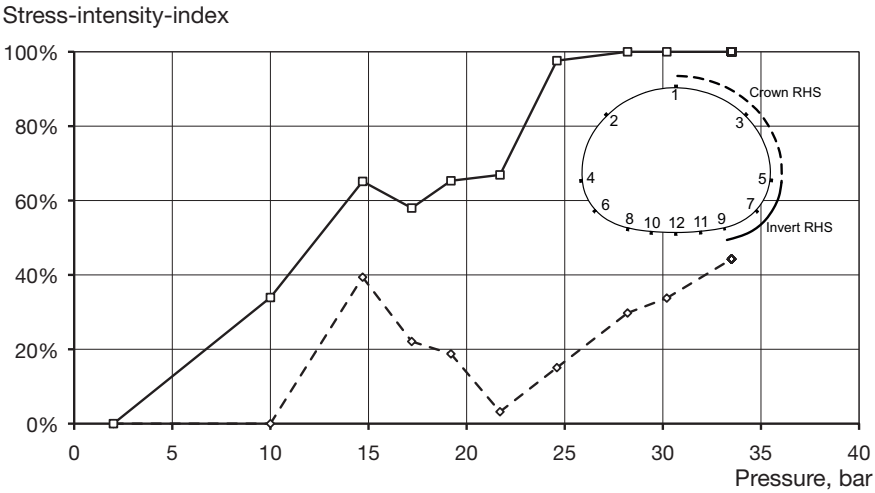


Figure 8.5 Stress-intensity-index at the crown (monitoring points 1–3–5), and invert (5–7–9)

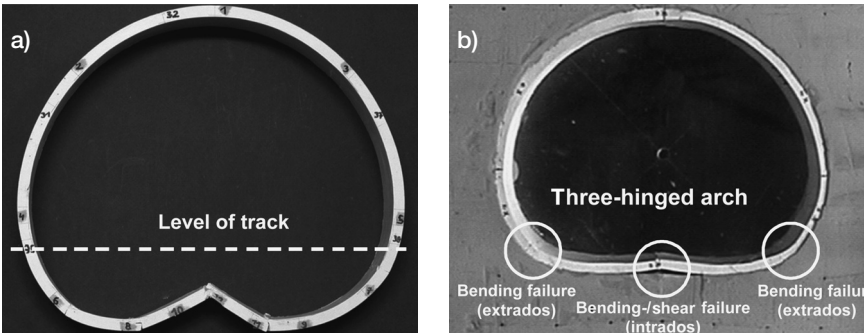


Figure 8.6 a) Invert collapse at the end of the test, and b) typical development of the cracks at the invert

of around 20% or less. Only at the end of the test, following the fifth and last crack in the invert, does the stress-intensity-index again reach 40% and peaks at an absolutely uncritical 45% as the invert finally collapses.

At the same time, the increase in load in the invert is clearly visible. The stress-intensity-index (points 5–7–9) rises from the start and indicates with a value of 100%, corresponding to the crack at point 7 (30 bar), that the load capacity of this part of the lining has been reached.

Generally, it was not predictable whether the model tunnels were going to collapse immediately after the first crack in the invert had developed or if the pressure could still be increased as shown in the test above.

Nevertheless, the invert failure mechanism can be described as follows. The vault, i.e. above the backfill, showed vertical displacements, mostly without bending. The stress remained low. The invert, i.e. below the backfill, did not follow the vertical displacements, but remained more or less in position, which is the purpose of an invert support. With ongoing displacements in the crown the load in the invert increased rapidly, leading to cracks in the middle of the invert and around the footing, a system similar to a three-hinged arch developed. At this point the invert lost its capacity to be a wide abutment for the vault (Figure 8.6b). All the cracks occurred below the backfill and would not have been detected *in situ*. Furthermore, the bending failure around the footing occurred at the extrados and therefore they would have remained undetectable even after removing the backfill for inspection purposes. The cracks in the middle of the invert were not necessarily accompanied by heaving, as visible in Figure 8.6a. Levelling of the invert would therefore only be of limited success.

From the test results the following conclusions can be drawn with respect to tunnels in soft rock:

1 Monitoring the vault only gave very late hints on a collapse of the invert, if at all.

2 Therefore, the invert should be monitored. As long as displacement monitoring of the invert is not possible, and the stress-intensity-index cannot be determined, the 'Eggemouse' is a proven and effective tool.
3 The remaining bearing capacity of the system after cracking of the invert is generally unpredictable. Just one crack in the invert can be followed by the collapse of the system. If one has knowledge about an invert failure it is recommended to repair this immediately.
4 The invert does not follow the vertical displacements of the crown causing a quicker development of the stress in the invert. This must be considered in the design. Even a temporary invert must be designed to be as robust as the vault.

8.2 London Heathrow T5, UK: construction of the Piccadilly Line Extension Junction

For complicated geometries or short tunnels, there is no alternative to using excavator and sprayed concrete support. On the London Heathrow T5 project a new tunnelling method called LaserShell™ was utilized, setting benchmarks in safety of construction and quality of SCL tunnels. This section describes arguably the most difficult and interesting part of T5's tunnelling work.

8.2.1 Project overview

London Heathrow airport has been expanded by adding a new Terminal 5 (T5), which opened in 2008 and was constructed away from the central terminal area. This had to be connected to the existing terminal buildings and to downtown London by means of a total of seven tunnels. The running tunnels were constructed by TBMs, but all the connecting constructions, such as headshunts, shafts, emergency exits, ventilation openings and cross passages were constructed using sprayed concrete lining (SCL). In total, more than 40 SCL structures with an overall length of more than 1100 m had to be built (Hilar *et al.* 2005, Williams *et al.* 2004). The tunnelling work was successfully completed in 2006. This case history focuses on the Piccadilly Line Extension (PiccEx) Junction.

London Underground provides public transport to London Heathrow via the Piccadilly Line, which had to be extended to serve the new T5. To connect the T5 Piccadilly Line Extension with the existing Piccadilly Line, the so called PiccEx Junction had been constructed in the middle of the heart of one of the busiest airports in the world (Figure 8.7).

8.2.2 The 'Box'

A 'box' was excavated, approximately 20 m deep, 50 m long and 20 m wide, to allow access to the existing tunnel systems (Figure 8.8).

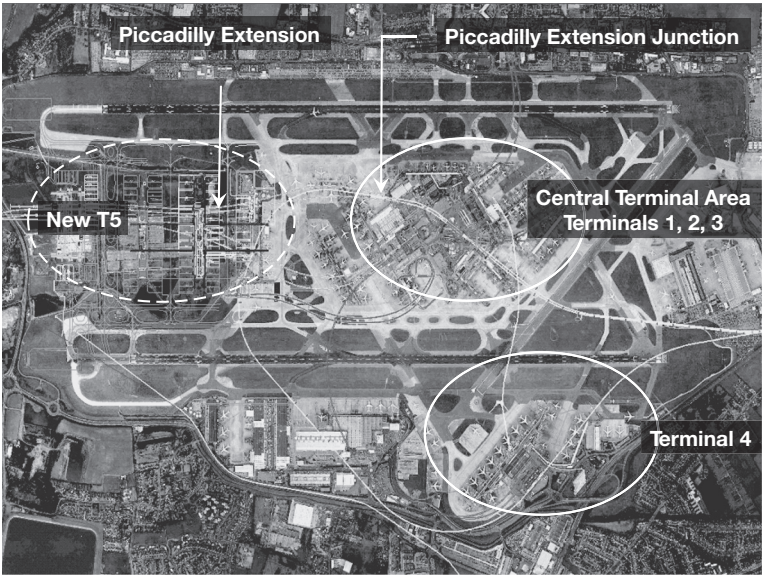


Figure 8.7 Overview of London Heathrow airport

Figure 8.8 The 'Box'

The box structure consisted of 1.2 m thick reinforced 'diaphragm' retaining walls. Half way down it was stiffened by a gallery-like intermediate slab. Two concrete pillars in the middle of the box provided additional support. All machinery, gear, equipment and material had to be lifted down the box opening by means of a crawler crane. For the first few weeks this also included the sprayed concrete supply until a permanent sprayed concrete pipe could be installed to the bottom of the box.

At the openings for the tunnels, one metre thick and heavily reinforced headwalls were cast to support the portal.

8.2.3 Construction of the sprayed concrete lining tunnels

In order to enable trains to reach the new T5, a turnout from the existing Piccadilly Line had to be built (Figure 8.9). East of the box the existing Piccadilly running tunnels needed an enlargement to provide enough space for the necessary switches for the Piccadilly Line Extension turnout, resulting in Eastbound and Westbound Turnout tunnels.

On the West side of the box the connection to the previously built Piccadilly Extension had to be completed, creating a need for Eastbound and Westbound Stub tunnels. Table 8.1 gives a brief overview of all four SCL tunnels at the PiccEx Junction.

8.2.4 Ground conditions

The only ground encountered during these works was London Clay, which appeared homogeneous, with almost no water seepage. A layer of scattered clay stones of up to 300 mm in diameter followed all four headings,

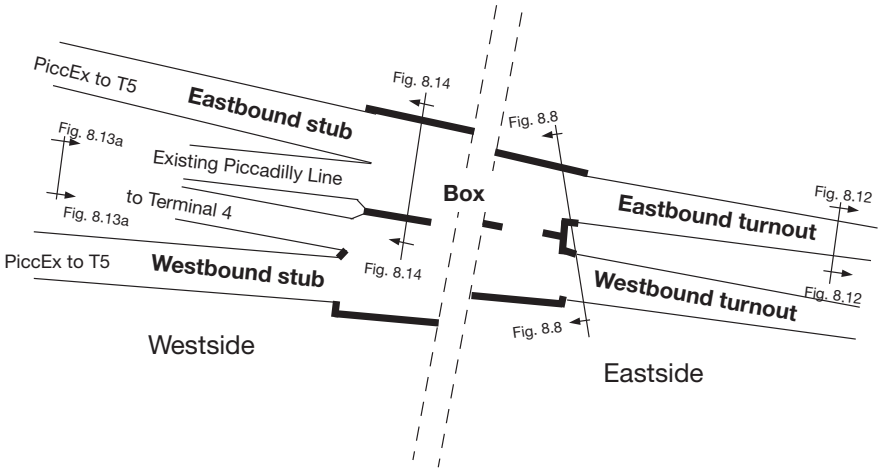


Figure 8.9 Overview of the PiccEx Junction

Table 8.1 Tunnels at the PiccEx Junction (in order of construction)

Tunnel	*Length (m)*	*Excavation diameter (m)*
Eastbound turnout	52.9	6.755–4.850
Eastbound stub	26.7	5.500/4.500
Westbound turnout	52.9	6.755–4.850
Westbound stub	30.1	5.500/4.500

approximately at tunnel axis. Occasional 'greasy backs' were not a problem since the excavation followed the principles of the LaserShell™ method (see section 8.2.5). Greasy backs are boulders likely to fall from the face. This is a particular problem in clay as water seepage in fissures reduces the friction holding the boulder in place. Greasy backs are not usually visible and can fall off the face without warning making them dangerous, which is one of the reasons why the Health and Safety Executive in the UK forbids anyone from entering the unsupported vault and face.

8.2.5 The LaserShell™ method

Until now, it has been common when using sprayed concrete support, to enter the unsupported heading to install girders and steel mesh. This is a fundamental aspect of the support system in soft ground. Girders provide immediate support and are required to fix the first layer of mesh. In addition, the girders are used to control the profile. Both mesh and girders can only be installed manually and therefore it is necessary to enter the heading, which, during this construction phase, is unsupported or only sealed with a thin layer of sprayed concrete. However, due to British health and safety regulations, no one is allowed to enter an unsupported, or even a partially supported, heading, which implies no installation of girders and hence no tunnel construction.

Thus, a tunnelling method had to be developed which could satisfy the British health and safety regulations, which does not need any girders and steel mesh, and which is applicable in London Clay. The LaserShell™ method was therefore born (Eddie and Neumann 2003 and 2004).

The most obvious feature of the LaserShell™ method is the inclined and domed face (Figures 8.10 and 8.11 Stage 1). The inclined face guarantees that nobody stands under any exposed ground at any time and only under the already hardened sprayed concrete of the previous advance. In addition, the inclination is contributing to a more stable face compared to a vertical one. As a permanent support steel fibre reinforced sprayed concrete was used with a 75 mm initial layer, and a minimum of 200 mm structural layer (Figures 8.11 Stages 2 to 5). A 50 mm finishing layer without steel

Figure 8.10 LaserShell™, inclined face

fibres covered up the steel fibres of the previously sprayed structural layer and provided a smooth finish comparable to a shuttered concrete surface. There was no inner lining.

Another less obvious feature of this method is that the structural layer is sprayed circumferential in one go, including the invert. This reduces the joints to an absolute minimum, i.e. just the radial joints between heading advances remain, with the added bonus of increased quality.

8.2.6 TunnelBeamer™

Another issue under the rigid British health and safety regulations is profile control. Since nobody is allowed to enter the unsupported vault, the profile can only be controlled from a distance, thus the TunnelBeamer™ became of paramount importance. The TunnelBeamer™, a laser theodolite, was connected to a laptop. The latter contained all relevant information with respect to geometry, i.e. the chainage of each advance and shape of its face (inclined and domed), and, furthermore, the geometry of the unsupported heading, as well as the geometry of every layer of sprayed concrete. During excavation of the profile of the heading and during the spraying process, the thickness of each layer of sprayed concrete was monitored continuously and easily in real time. All as-built profiles were stored and could be used for quality control purposes and later evidence as required.

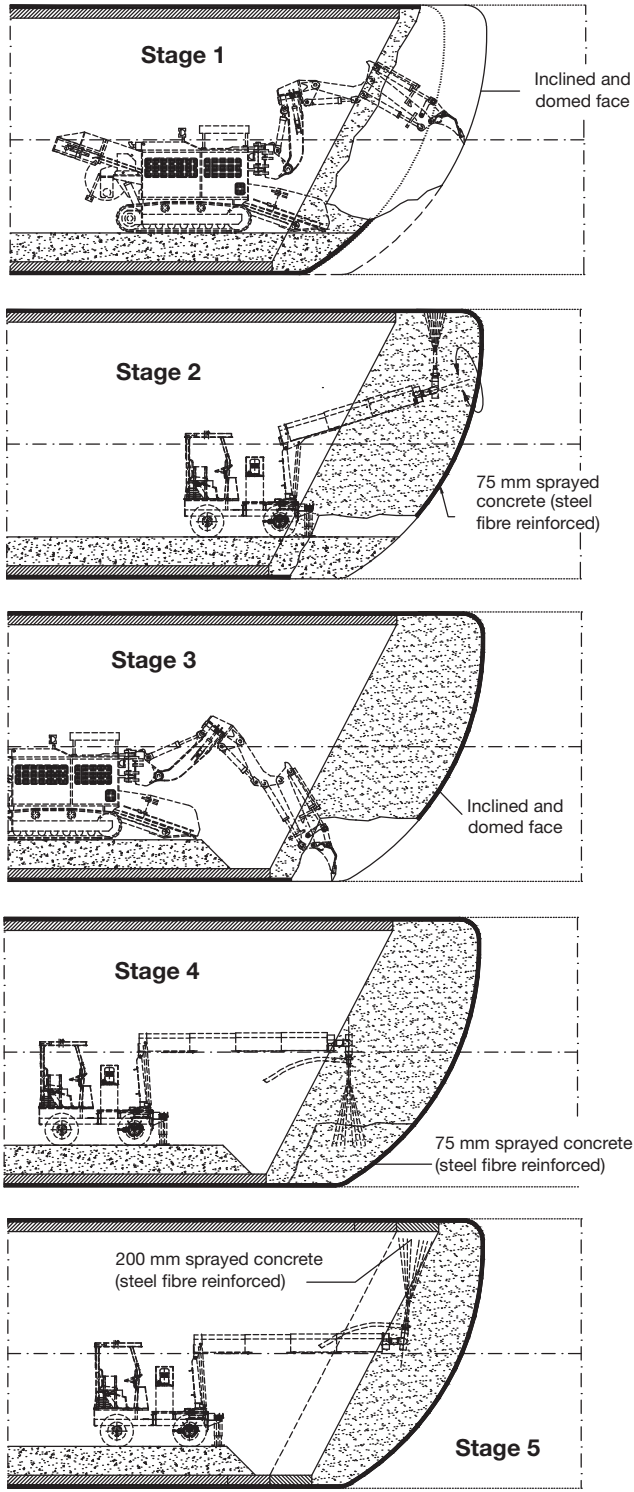


Figure 8.11
Stage 1: excavation of the crown and bench;

Stage 2: initial layer of sprayed concrete at the crown and bench;

Stage 3: excavation and trimming of the invert;

Stage 4: initial layer of sprayed concrete at the invert;

Stage 5: structural lining (permanent support)

(courtesy of ALPINE BeMo Tunnelling GmbH Innsbruck)

8.2.7 Monitoring

8.2.7.1 Existing Piccadilly Tunnel Eastside

On the Eastside of the PiccEx Junction the existing Piccadilly Line tunnels had to be enlarged over a length of 53 m (see Figure 8.9 for the location). The first half of the existing Piccadilly Eastbound was plugged with foam concrete for stability reasons during construction of the enlargement, with the second half remaining unplugged during excavation. This gave the unique opportunity to monitor the unplugged area ahead of the SCL-face once the plugged area had been left behind (Figure 8.12). The information obtained from monitoring the existing tunnel helped to determine the amount of stress re-distribution ahead of the current SCL heading and thus to assess the displacements already acting on the sprayed concrete lining before the base reading could be done.

It was possible to take readings up until the approaching heading was within just one width of a ring, i.e. 600 mm. It was discovered that there was no evidence of any deformation ahead of the face. Two conclusions could be drawn from this surprising result:

1 It confirmed previous observations that displacements in advance of the heading are small or negligible. Although it should be noted that this is not a generic statement for any tunnel construction in London Clay, and only applies to the particular conditions at the PiccEx Junction.
2 Under normal circumstances there is a delay between applying the sprayed concrete, installing the monitoring array, and taking the base reading. During this period the freshly applied sprayed concrete is already loaded and displaced by the stress re-distribution around the heading. Due to this inevitable delay in taking the base reading, this pre-displacement cannot be read and this information is lost. With the knowledge of the results discussed above, it was clear that this unreadable pre-displacement must have been very small. Therefore, the ordinary monitoring provided an almost complete picture of the total displacement and hence of the load acting upon the sprayed concrete lining. Again, it should be pointed out that this is not a general statement for any tunnel construction in London Clay, and only applies to these particular conditions at the PiccEx Junction.

8.2.7.2 Existing Piccadilly Tunnel Westside

Before this particular part of the monitoring is described, some general information on the existing Piccadilly Line tunnels should be given. The lining of the existing Piccadilly Eastbound tunnel was made of spheroidal graphite (cast) iron segments (SGI-segments). The joints between the segments were filled with plywood, or similar, to cope with curves and changes in gradient.

Figure 8.12 View into the unplugged existing Piccadilly Line Eastbound Turnout tunnel

No gaskets had been used to prevent water from infiltrating through the joints. The London Clay at T5 in general is considered to be practically impermeable, however water did somehow find its way down to the tunnels and through the joints. As a result plenty of stalactites were growing on the lining and rusting of the iron segments was also visible (see Figure 8.13a, left-hand side and Figure 8.13b for the detail).

The distance between the existing Piccadilly tunnels and the SCL Stub tunnels was very small (Figure 8.9, Figure 8.14). The remaining ground pillar between the tunnels varied from 0.4 m at the box to approximately 3 m at the end.

The integrity of the existing structures during the SCL work had to be ensured, therefore monitoring of the existing Piccadilly Line tunnels was essential. Of particular interest was the performance of the existing Piccadilly Line Eastbound Tunnel during the excavation of the SCL Eastbound Stub Tunnel, which was the first of the stub tunnels to be constructed.

The existing Piccadilly Line Eastbound was approximately 20 m long (before it merged with the existing Piccadilly Line Westbound, Figures 8.9 and 8.13a), of which the first 8 m, measured from the box, were plugged with foam concrete to enhance the stability during the excavation of the adjacent SCL Eastbound Stub. In the remaining unplugged part four monitoring arrays were installed. Access was possible through the neighbouring Piccadilly Line Westbound Tunnel.

The existing Piccadilly Line Eastbound Tunnel experienced 3 to 5 mm vertical displacement in the crown, and 5 to 7 mm horizontal displacement at axis level directed towards the SCL Eastbound Stub, indicating a clear ovalization. All displacements were limited to the crown and the side of the lining where the SCL Eastbound Stub passed by (Figure 8.15).

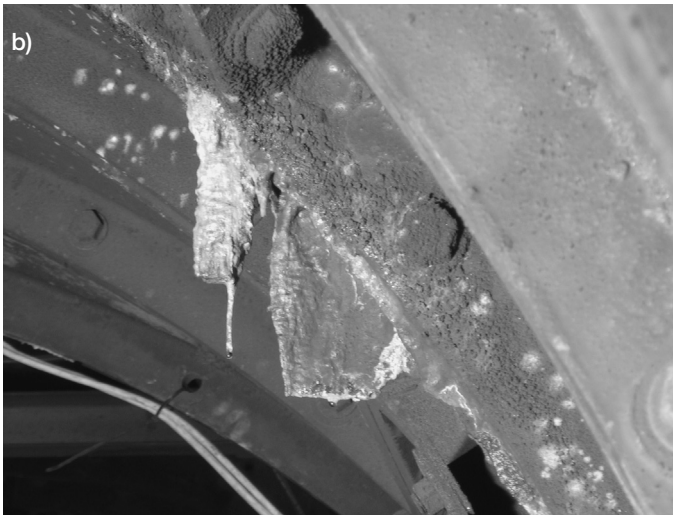


Figure 8.13 a) Existing Piccadilly Line tunnels, SGI-Segments (LHS), Concrete Segments (RHS), b) existing Piccadilly Line Eastbound Tunnel, stalactites on the SGI-Segments

Figure 8.14 (LHS) existing Piccadilly Line Tunnel, (RHS) SCL Eastbound Stub Tunnel

Significant displacements could only be observed approximately 2 m ahead of the current SCL face and continued 4 to 5 m after the SCL face had passed the relevant monitoring array (equivalent to half a tunnel diameter in advance and one tunnel diameter after passing the monitoring array, or in terms of time equivalent to approximately three days altogether).

Although the overall displacements in the existing Piccadilly Line Eastbound Tunnel were 2 mm smaller than anticipated in the design (Jäger and Stärk 2007), the gradient appeared steeper, creating a larger longitudinal 'bending' of the SGI lining towards the SCL Stub Tunnel. There was no concern for the stability of the SGI lining as all the segments were bolted together to form rigid and robust rings. However, this information was important with respect to the stability of the adjacent tunnel, the existing Piccadilly Line Westbound Tunnel, during the upcoming excavation of the SCL Westbound Stub. As shown in Figure 8.13a the existing Piccadilly Line Westbound Tunnel was supported by expanded lining. Precast concrete segments form rings which were not bolted, but were held in place only by radial forces generated by the keystone and ground pressure, and longitudinal forces generated by the TBM rams during tunnel construction. This technique normally works perfectly well. However, for whatever reason in this case

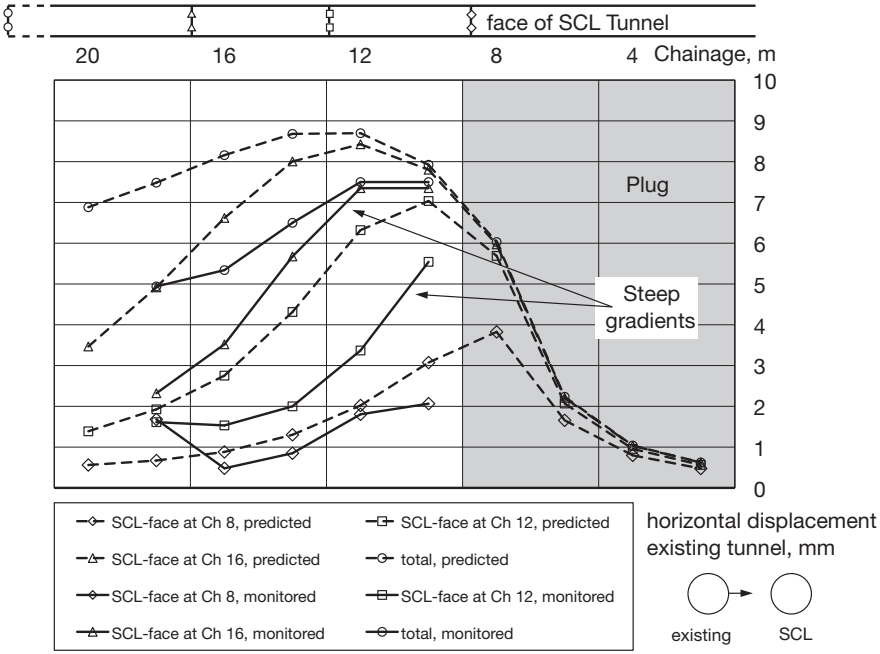


Figure 8.15 Predicted and measured horizontal displacements in the existing Piccadilly Line Eastbound Tunnel

the precast concrete segments did not form an even surface, but showed irregular steps between the joints of the precast concrete segments of up to approximately 60 mm (Figure 8.16).

Knowing the future development of the displacements as shown in Figure 8.15, it was quite clear that the concrete segments would experience additional forces during construction of the SCL Westbound Stub causing the joints to open wider. The question was how this would affect the stability of the concrete segments? In fact, nobody wanted to find out and it was decided to extend the foam concrete plug in the existing Piccadilly Line Westbound Tunnel to the full length (instead of only 8 m). Thus any additional movement of the expanded lining and any further opening of the joints were prevented. This example shows that the overall magnitude of relatively small displacements is not important, but it is their effect on the ground and structures.

The civil engineering and tunnelling works at PiccEx Junction were completed successfully in 2006. The Laser Shell™ tunnelling method is now commonly used in the UK. (Note: LaserShell™ and TunnelBeamer™ are registered trademarks by Beton- und Monierbau and Morgan=Est. The TunnelBeamer™ has been patented by Beton- und Monierbau and Morgan=Est.)

Figure 8.16 Existing Piccadilly Line Westbound Tunnel showing steps of up to 60 mm between concrete segments

8.3 Lainzer Tunnel LT31, Vienna, Austria

The Lainzer Tunnel Lot 31 (LT31) is a large railway tunnel in shallow soft ground beneath a densely built up urban area. This section describes the construction of 3 km of side wall drift, and highlights fundamental design issues as well as essential aspects of the complex monitoring regime.

8.3.1 Project overview

Today's rail traffic runs overground through Vienna, and rail traffic is quite heavy. The noisy freight trains are especially likely to disturb people's sleep at night in this densely built up urban area. This will change when most of the trains start running through the Lainzer Tunnel, which will be opened at the end of 2012 by the Federal Austrian Railroad.

The Lainzer Tunnel is 12.3 km long. Due to its length and changing geological conditions, the project has been divided into different 'lots'. LT31 forms, together with the neighbouring LT33, the core of the Lainzer Tunnel, i.e. the 6.5 km long Connection Tunnel. A significant change from soft ground to hard rock divides the Connection Tunnel into two sections of nearly the same length, which separated LT31 (soft ground) from LT33

Table 8.2 Overview of headings

Shaft	*Section*	*Length*	*Excavation method*	*Geology*	*Direction of excavation*
Lainzer Str.	W_{new}	595 m	Crown/Bench/Inv.	Hard rock	LT33
	W	790 m	Side wall drift	Soft ground	LT33
	P	596 m	Side wall drift	Soft ground	Klimtgasse
Klimtgasse	M	593 m	Side wall drift	Soft ground	Lainzer Str.
	S	1051 m	Side wall drift	Soft ground	LT44

(hard rock). In order to ensure a coordinated date for the opening of the Lainzer Tunnel in connection with the new railway line from Vienna to St. Pölten, the lot boundary was moved for the benefit of LT31 by 595 m. In addition to the original 3.05 km long soft ground section, LT31 was extended to include a 595 m long hard rock section. The soft ground was excavated completely by means of side wall drifts, which was possibly the longest side wall drift in the world at that time. The remaining 595 m in hard rock were excavated conventionally with crown/bench/invert using a roof pipe umbrella and drill and blast, respectively. This case history will focus on the construction and monitoring of the side wall drift section.

Tunnelling started in October 2006 from two 30 m deep mucking and delivery shafts, 'Lainzer Straße' and 'Klimtgasse', in two directions, each resulting in four headings excavated simultaneously. Section 'S' connected LT31 to LT44 in the East, section 'W' and its hard rock extension 'W_{new}' connected to LT33 in the West, and sections 'M' and 'P' met in the middle. Table 8.2 gives an overview of this.

The breakthrough between sections P and M was in September 2008, with the breakthrough to LT44 in December 2008, and the excavation of section W_{new} completed in May 2009. Figure 8.17 depicts a bird's eye view of LT31 with direction of sight to the West.

The Lainzer Tunnel is designed with emergency exits approximately every 500 m, seven of which are within LT31 (see Table 8.3 for details). Both of the existing mucking and delivery shafts were to be converted into emergency exits, and the remaining five had to be newly constructed. At each emergency exit a shaft provides a vertical access down to the main tunnel level. Galleries connect the shafts with the main tunnel. The shafts have a diameter of 9.4 m and a depth ranging from 20 m to 55 m. The connecting galleries have cross sections of 25 m^2 to 30 m^2 and they are 20 m to 258 m long. On the opposite side of the five new emergency exits 8 m long transformer niches had to be constructed. Furthermore, at the lowest level of the tunnel an 8 m deep sump was excavated to collect water flowing into the main tunnel during operation. This water will be pumped out through the nearest emergency exit, Jagdschlossgasse.

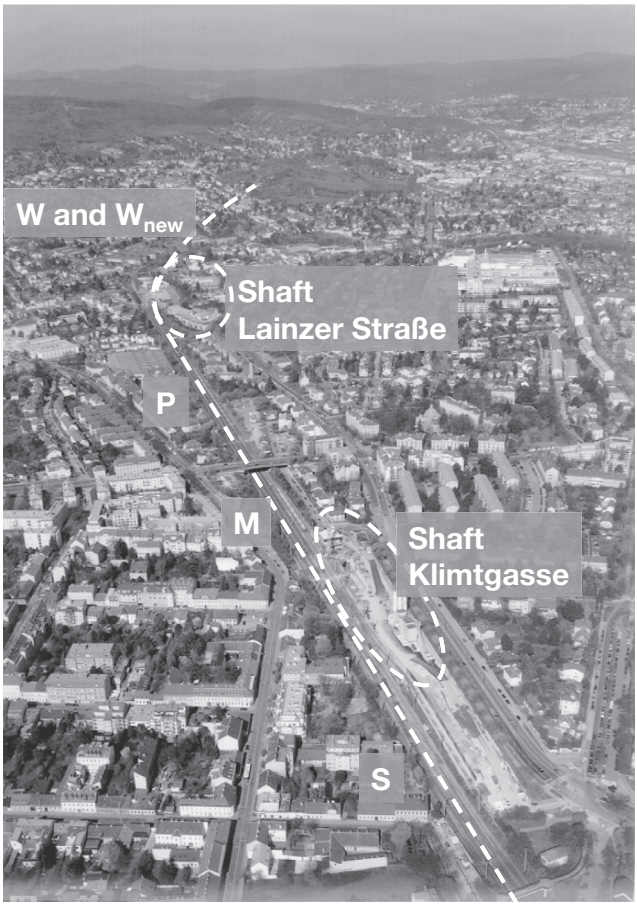


Figure 8.17 Project overview of LT31

Table 8.3 Overview of emergency exits

Emergency exit	*Section*	*Shaft depth*	*Gallery length*	*Geology*
Veittingergasse	W_{new}	55 m	258 m	Hard rock
Jagdschlossgasse	W	35 m	67 m	Soft ground
Lainzer Straße	–	30 m	Conversion of shaft	
Himmelbaurgasse	P	30 m	27 m	Soft ground
Schönbachstraße	M	32 m	32 m	Soft ground
Klimtgasse	–	30 m	Conversion of shaft	
Schlöglgasse	S	23 m	50 m	Soft ground
5 transformer niches	Same as exit	–	8 m	Same as exit
Sump Waldvogelstraße	W	8 m	11 m	Soft ground

8.3.2 Geology

The soft ground (sections W, P, M, S) was dominated by alternating layers of silt/clay, sand, gravel, and wide graded sediments. Except for the silt/clay layers the ground is permeable with the water table above the tunnel roof. Groundwater lowering was necessary by means of wells from the ground surface well ahead of the leading excavation face to avoid stability problems at the face. The groundwater layer was often enclosed by silt/clay layers resulting in confined groundwater aquifers. Rigid conglomerates and layers of hard sandstone, up to 3 m thick, were embedded between the soft ground layers. The hard rock formations 'Flysch' (section W_{new}) were of varying quality from extremely poor to fair. At the transition from soft ground to hard rock a 300 m long section of lower ground quality was stabilized by roof pipe umbrellas. The overburden extended from 6 to 26 m in the soft ground, and from 26 to 66 m in the hard rock formation.

8.3.3 Starting construction from the shafts

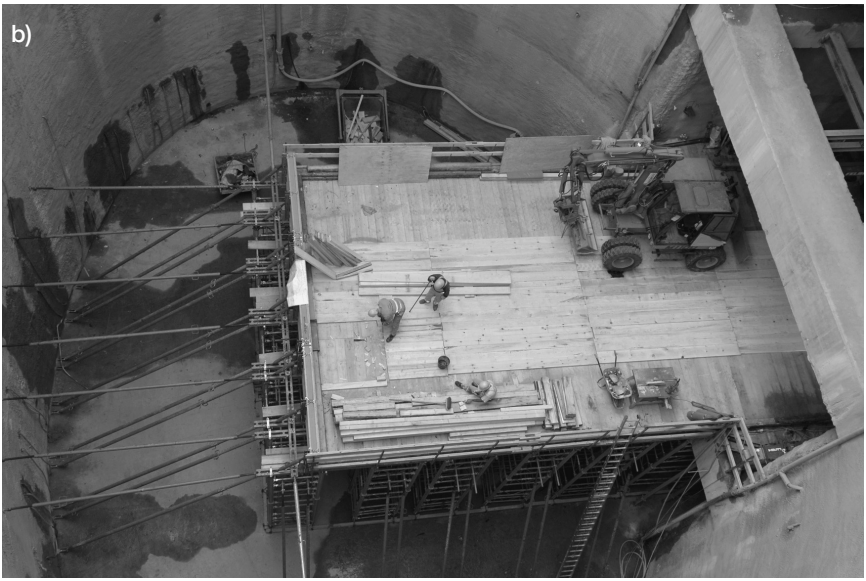
Both shafts, 'Lainzer Straße' and 'Klimtgasse', consisted of an open oval section supported by sprayed concrete and a rectangular section with the live railway on top. The open oval section was used for mucking and delivery, while the excavation started from the rectangular section. The rectangular section was supported by a bored pile wall and four levels of bracing, the lowest of which ran right through the tunnel profile (Figures 8.18a and 8.21). The bracing was not allowed to be dismantled all at once, but only in sections with the advancing excavation. Since a tunnel construction starts with the top heading this was a problem, because it was not known exactly how the excavator could reach the top heading while the bracing was still in place. It was decided to set up a platform above the lowermost bracing. This kind of platform was a challenge because there was no reference on how to design a platform for dynamic loads of heavy excavators. Shoring towers used for formwork seemed to be the most robust support for the platform. The shoring towers were rigidly attached to the walls. On top of the shoring towers solid web girders were laid close together, which were covered by a two layer crisscross nailed up planking (Figure 8.18b). The planking protected the girders against damage (surface wear and tear) and acted statically like a rigid disc, helping to distribute the load equally onto the girders and towers.

From the platform, the crown sections were excavated to a length of 5 m to both sides of the rectangular shaft, and then the platform was dismantled (Figure 8.19). The excavation of the bench and invert of the leading site wall drift could also start after the bracing was partly dismantled. Figure 8.20 shows the tunnel after the total cross section was constructed over a length of approximately 30 m, giving space in the shaft area to manoeuvre the plant.

8.3.4 Side wall drift section: excavation sequence and cross section

Approximately two-thirds of the 3.05 km side wall drift section was excavated beneath the existing railroad, which was still subjected to the regular heavy rail traffic; the rest was located under buildings and streets. Therefore the need for a robust excavation method with low subsidence

Figure 8.18
a) Shaft 'Klimtgasse'. Excavation platform with planking nearly finished,
b) excavation platform. Shoring towers in place, web girders are laid close together and covered in planking



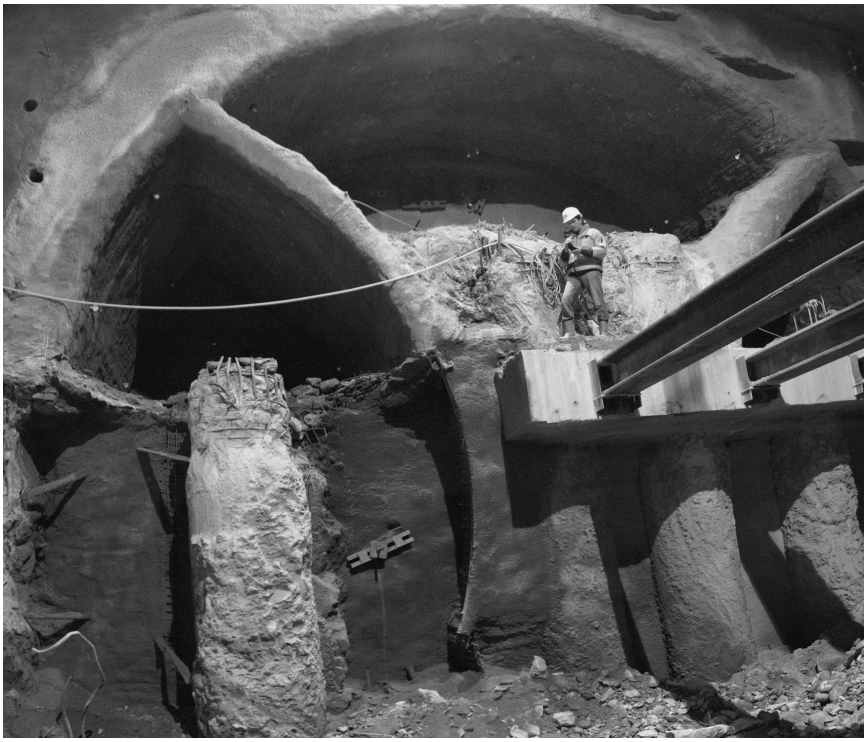


Figure 8.19 Crown excavated, and enlargement of leading side wall drift started. Remaining bracing still in place

Figure 8.20 30 m of total cross section (on the RHS, the open oval section, the remainder of the lower most bracing can be seen still in place)

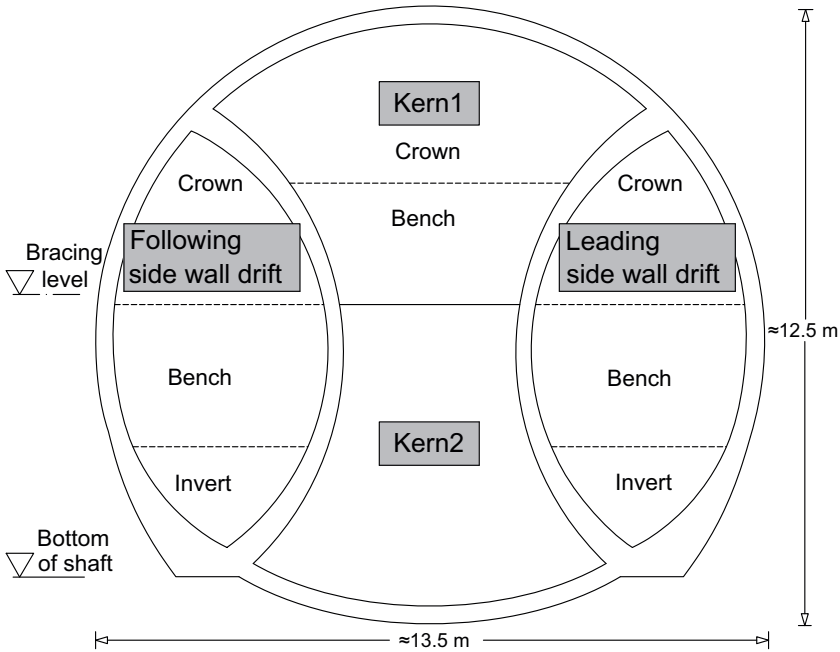


Figure 8.21 Cross section of the tunnel, including the two side wall drifts

was important. Figure 8.21 shows the cross section of the area with side wall drift excavation.

There were several reasons for choosing this design:

1 The soft ground has limited bearing capacity. With respect to railroad, buildings and service structures above the tunnel a stiff support was a paramount issue to control ground loss at the face, to avoid face or roof instability, and finally to minimize settlement at the ground surface. This could be achieved by dividing the tunnel into smaller headings and limiting the cross section of each heading.
2 Most disturbance of the original stress level in the ground had been expected when excavating the remaining top heading. To mitigate this effect, the roof of the side wall drifts were positioned so as to reduce the span of the remaining top heading.
3 Big footings at the invert of the side wall drifts, similar to elephants' feet, helped to avoid settlement and any inward orientated movement during excavation of the remaining top heading.
4 With respect to buildability, quality of construction joints and safety of construction, the external walls of the side wall drifts were integrated into the permanent sprayed concrete lining as much as possible, resulting in a high and slender shape.

5 The shape of the side wall drifts provided enough space for plant to excavate the remaining top heading and bench.
6 The high and slender side wall drifts were more sensitive to high horizontal loads (rather than more circular side wall drifts). This had to be considered in the design process and resulted in a sprayed concrete thickness of 30 cm for the internal wall and 35 cm of sprayed concrete for the circumferential outer walls, all girder supported and rebar reinforced.
7 The side wall drifts, approximately 9 m high, were divided into crown, bench and invert with a short distance for ring closure of at most 10 m. The remaining top heading was also divided into crown and bench sections.
8 The crown had to be opened in up to four sub cross sections and the bench in up to two sub cross sections.
9 The crown and bench support was accompanied by forepoling and face anchors.

Figure 8.22 shows a plan view of the excavation sequence. The minimum distance of each heading had to be 10 days or 20 m, respectively. The first reason for this was to let the sprayed concrete gain enough strength to cover the additional load of the following headings. The second reason was to allow the displacements of each heading to come to a halt before the following heading passed the relevant area. This was necessary to fully control the ground surface settlements with respect to third party structures. In addition, the leading side wall drift allowed for dewatering of any residual groundwater making the excavation safer and easier for the following headings.

With ongoing excavation it could be proven that stress redistribution within the less cohesive/non-cohesive soil (gravel and sandy layers) was only 10 m. In combination with an achieved high compressive strength for the sprayed concrete (the required 28-days-values were already reached after seven days), the distance between heading faces could be reduced to five

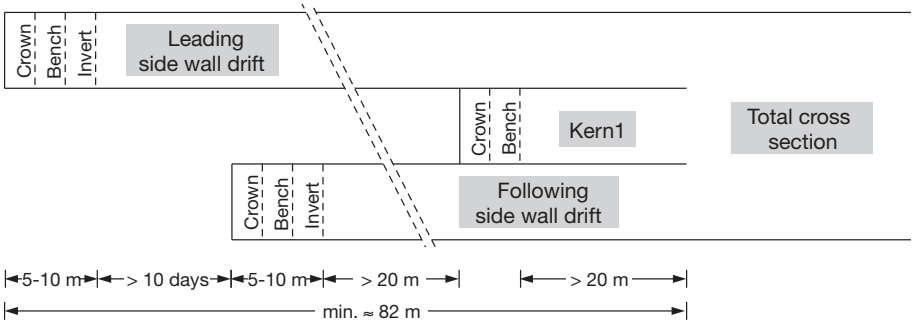


Figure 8.22 Plan view of the excavation sequence

days or 10 m, respectively. The stress redistribution in the cohesive silt/clay layers took longer due to creep effects; so they had to adhere to the original designed sequence.

Originally, a simultaneous excavation of the side wall drifts and Kern1 was not allowed for safety reasons. The risk assessment identified an overstressing of the inner walls of the side wall drifts during the excavation of Kern1 as a possible hazard. This could result in a collapse of the side wall drifts with the miners trapped at the face. The analysis of in-tunnel monitoring during construction, however, proved no adverse effects on the stability of the side walls. After a reasonable observation period of approximately six months the simultaneous excavation of both side wall drifts and Kern1 was permitted. As a precaution the in-tunnel monitoring was intensified and the face had to be opened in smaller sub-areas, respectively. Altogether the excavation was now much quicker. In-tunnel monitoring is usually used to identify adverse developments. However, in this case it helped to improve the performance, while keeping the safety to the same high level.

Figure 8.23 Construction of the invert at the total cross section (excavation Kern2 and demolition of inner walls of side wall drift)

A simultaneous excavation of the side wall drifts and Kern2 was still not allowed, but this was never the aim for buildability reasons. The excavation of Kern2 included the ring closure at the total cross section and the dismantling of the inner walls of the side wall drifts. During this process access to Kern1 was not possible and only with difficulty to the side wall drifts (see Figure 8.23). Therefore, these excavations had to be suspended. From a practical point of view, a compromise had to be found in such a way that on the one hand driving cycles into the side wall drifts would not increase too much, and on the other hand that the time consuming preparation for the excavation of Kern2 was kept to a minimum. It turned out that changing to Kern2 every 30 m to 60 m was the best option.

8.3.5 Monitoring of the sprayed concrete lining of the side wall drift section

During the design process a geotechnical safety management concept was established, which was a live document that was continuously revised during construction (Heissenberger *et al.* 2008). According to this concept the regular distance of monitoring cross sections was 10 m throughout LT31 and if necessary this was reduced to 5 m. Readings had to be taken 20 m ahead and 30 m behind the face on a daily basis. In consideration of the distance between faces, as shown in Figure 8.22, an area of 100 m to 140 m in each of the sections S, M, P, and W had to be monitored by means of displacement measurements. Additionally, measurements had to be taken during construction of all emergency shafts and galleries as well as the adjacent areas of the main tunnel. Since LT31 was situated almost completely under a live railway and other urban infrastructure a large number of surface surveying points had to be monitored. This section highlights some special features, however further information on the in-tunnel monitoring can be found in Moritz *et al.* (2008).

8.3.6 Cracks in the sprayed concrete lining

Cracks were detected for the first time in section W between Chainage (Ch.) 50 and 60 at the inner walls of both side wall drifts. The horizontal cracks occurred at the intrados approximately 0.5 m to 1.0 m above the intersection between the bench and invert (Figure 8.24). The ring closure of the total cross section, including dismantling of the inner walls of the side wall drifts, was completed up to Ch. 50, i.e. the cracks ran in the remaining inner walls in the direction of the face. The width of the cracks was up to several millimetres in some areas. From the experience of former projects (e.g. Eggetunnel, railway line Kassel-Dortmund) the development of cracks had been expected, but at the extrados in the area of the crown. The location and the extent of the cracks were therefore irritating and due to the sensitive urban area a detailed investigation was done. In order to

Figure 8.24 Crack in the inner wall of the side wall drift, section W, Chainage 52–60

check the integrity of the sprayed concrete lining, three cores were taken out of the inner walls of section W, Ch. 52 to 54, and these are shown in Figure 8.25. The cores had a diameter of 160 mm and a length of 37 cm to 42 cm. Only with specimen No. 1 was the sprayed concrete lining drilled through completely, i.e. the overall thickness of the inner walls was comfortingly greater than the required 30 cm. Specimens No. 1 and No. 2 were broken at the construction joint between the first and second layer of sprayed concrete; based on this fact the manufacturing of the second layer of sprayed concrete was improved immediately. Specimen No. 3 got jammed in the core-barrel and had to be drilled out, leading to it breaking in a couple of places. Nevertheless, the original crack in this specimen could still be identified. The straight nature and the opening of the cracks towards the intrados indicated flexural tension as the most possible cause of all the cracks.

The monitoring data confirmed the visual observations. During excavation of the side wall drifts the most significant displacement was a convergence between monitoring points 10 and 4, and 11 and 5 in the crown of the side wall drifts. During excavation of the following Kern1 the direction of the displacements changed in crown-points 10 and 11 resulting in a clear divergence developing between points 10 and 4, and 11 and 5

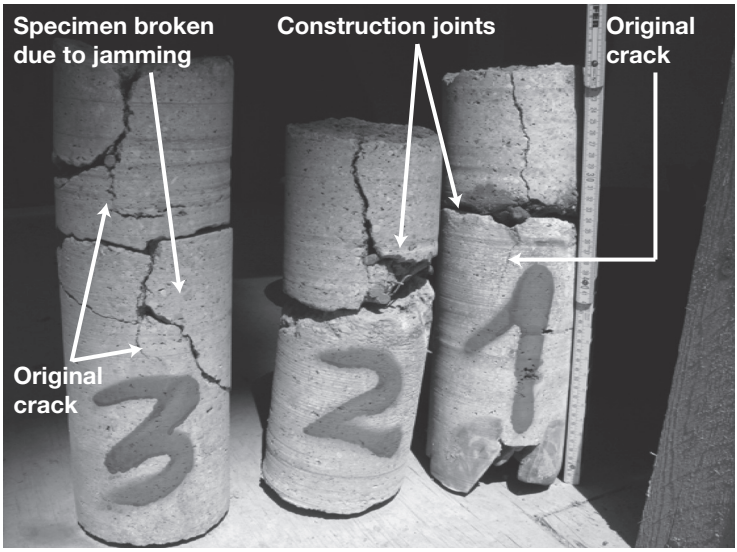


Figure 8.25 Cores, section W, Chainage 52–54

(Figure 8.26). The explanation is quite clear: with the excavation of Kern1 the bedding of the inner walls of the side wall drifts was taken away and the inner walls moved in the freshly excavated open space. This behaviour, although not in this magnitude, was well known from previous projects as mentioned above, with only the expected cracks at the extrados around

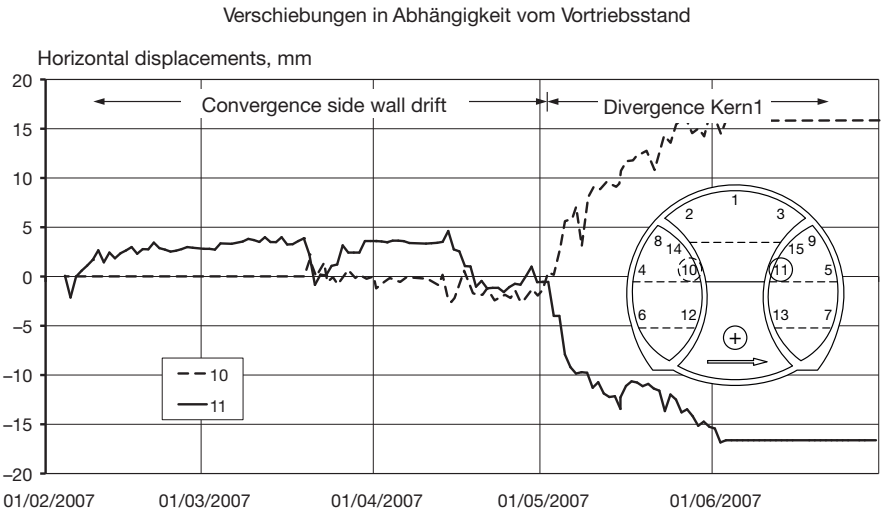


Figure 8.26 Change from convergence to divergence, section W, Chainage 60

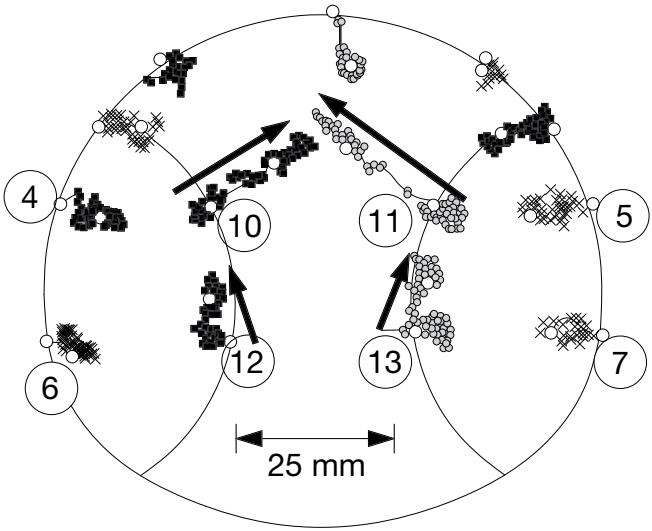


Figure 8.27 Displacements of side wall drift during excavation of Kern1, section W, Chainage 60

points 10 and 11 not being found. Instead, cracks at the intrados developed (as shown in Figure 8.24). The reason for this was that points 10 and 11 also showed a heave, which caused an unexpected heave of approximately the same amount in the bench-points 12 and 13. Figure 8.27 shows qualitatively how the points moved. The movement generated a negative

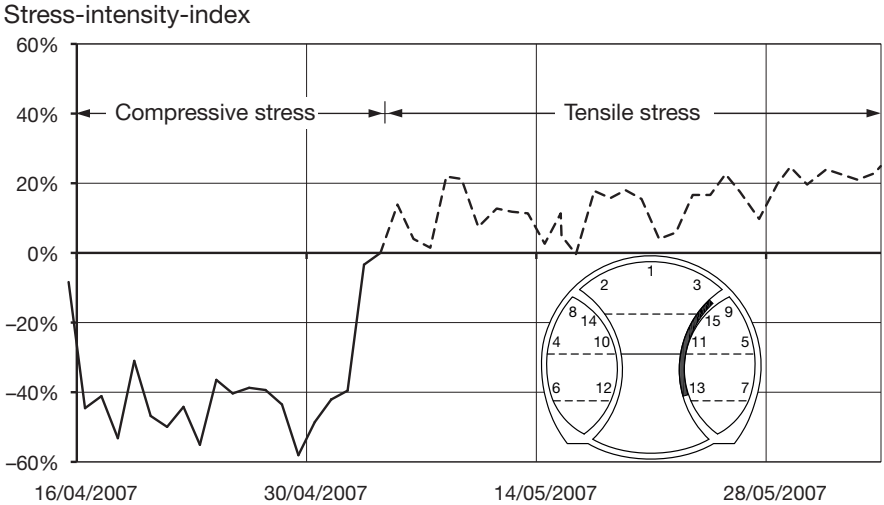


Figure 8.28 Stress-intensity-index, section W, Chainage 60

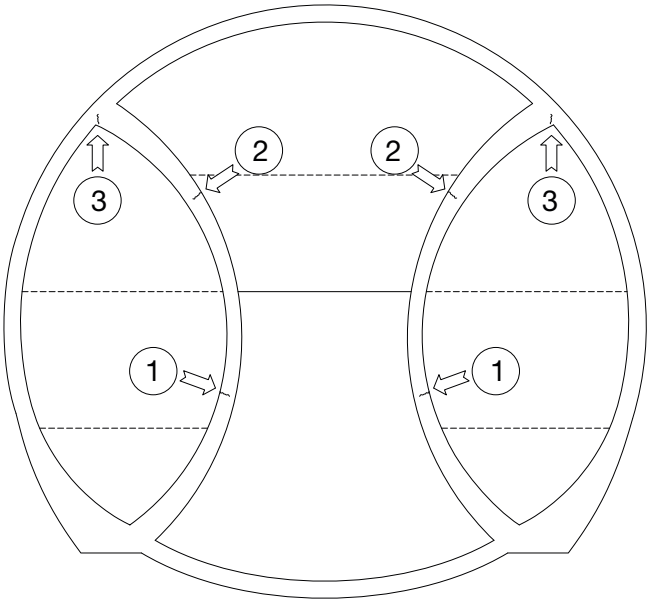


Figure 8.29 Crack pattern occurring in the walls of the side drifts

bending moment around points 10 and 11 (tensile stress at the extrados), and a positive bending moment/elongation around points 12 and 13. The sprayed concrete was, at this time, some weeks old and already hardened. Stress redistribution inside the sprayed concrete, e.g. due to creeping effects, was negligible. The movements therefore immediately caused an increase in stress inside the sprayed concrete, which was made visible by the cracks.

This was confirmed by the stress-intensity-index (see section 7.3.4.6) as shown in Figure 8.28 for the inner wall of the right-hand side wall drift at Chainage 60 (see marking in Figure 8.28 between points 13–11–15). With the face of Kern1 passing the monitoring cross section at Chainage 60, the stress changes from a compressive stress (negative sign) to a tensile stress (positive sign) and leads to the cracks. The stress-intensity-index shows a tensile stress of only about 20% after the development of the cracks. The safety factor against failure (η = 100%) was still around five. This gave the certainty that the tunnel was in a very stable situation. With the knowledge of the stress-intensity-index another conclusion could be made: Due to the cracks and the low stress level in the inner wall, most of the load had been redistributed into the outer walls of the side wall drifts. This effect was considered desirable with respect to the later demolition of the inner walls. Demolition was safer and easier with unloaded inner walls. Overall, the development of cracks in the sprayed concrete lining was beneficial in this case.

During further excavation, a uniform crack pattern developed at the same time in all four sections W, P, M, and S as shown in Figure 8.29. All the cracks ran towards the face over the complete length of the excavated Kern1. This confirmed the experience from earlier projects, although only the cracks at the extrados, marked with '2', had been expected. However, the other unexpected cracks at the intrados, marked with '1' and '3' were a logical consequence of the displacements according to Figures 8.26 and 8.27. Due to the rough surface of the sprayed concrete lots of soil stuck to it and the expected cracks '2' were very hard to detect even for experienced eyes, leading to the previously mentioned issue.

Appendix A

Further information on rock mass classification systems

A.1 Rock Mass Rating

Brief details of this rock mass classification system are provided in section 2.4.4.2, with further information provided in this appendix. Table A.1 shows the classification parameters used.

In section A of Table A.1, five parameters are grouped into five ranges of values. As these parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters. A higher rating indicates a better rock mass condition. The ratings for the strength of the intact rock, RQD and discontinuity spacing can be interpolated between the values indicated in the Table A.1 and these are shown in Figures A.1a to c, respectively. If either RQD or discontinuity data are lacking, then Figure A.1d can be used.

Once the ratings for the five parameters in section A of Table A.1 have been established, these are summed to provide the basic Rock Mass Rating (RMR) for the area of the rock mass being considered. The next stage is to include the sixth parameter, i.e. the orientation of the discontinuities, by adjusting the basic RMR according to section B of Table A.1. With regard to tunnelling projects, further information on this section can be found in section F of Table A.1. After adjustment for discontinuity orientation, the rock mass is classified using section C of Table A.1, which groups the final (adjusted) RMR into five rock mass classes. This value varies from 0 to 100. Subsequently, section D of Table A.1 provides practical meaning to each rock mass class as it relates this to specific engineering problems. Section E of Table A.1 provides guidelines for classifying the discontinuity conditions.

Davis (2006) states that one of the important aspects of this method is the way the various parameters are derived. It is advisable to choose a 'best estimate' and a 'worst credible' case and assess these for each parameter. Davis (2006) has the following advice on deriving the various parameters:

- *Uniaxial compressive strength of the rock material* – This can be obtained from laboratory UCS testing or point load strength testing of samples. Descriptions of the borehole logs can be used if no test data are available.

Table A.1 Rock Mass Rating system (after Bieniawski 1989)

	Parameter		*Range of values*						
A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4–10 MPa	2–4 MPa	1–2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial comp. strength	> 250 MPa	100–200 MPa	50–100 MPa	25–50 MPa	5–25 MPa	1–5 MPa	< 1 MPa
		Rating	15	12	7	4	2	1	0
2	Drill core quality	*RQD*	90–100%	75–90%	50–75%	25–50%		< 25%	
		Rating	20	17	13	8		3	
3	Spacing of discontinuities		> 2 m	0.6–2 m	200–600 mm	60–200 mm		< 60 mm	
		Rating	20	15	10	8	5		
4	Condition of discontinuities (see E)		Very rough surfaces; Not continuous; No separation; Unweathered wall rock	Slightly rough surfaces; Separation < 1 mm; Slightly weathered walls	Slightly rough surfaces; Separation < 1 mm; Highly weathered walls	Slickensided surfaces or Gouge <5mm thick or Separation 1–5 mm; Continuous	Soft gouge >5 mm thick or separation > 5 mm Continuous		
		Rating	30	25	20	10		0	
5	Ground-water	Inflow per 10 m tunnel length (l/m)	None	< 10	10–25	25–125		> 125	
		Joint water press/(Major principal σ)	0	< 0.1	0.1–0.2	0.2–0.5		> 0.5	
		General conditions	Completely dry	Damp	Wet	Dripping		Flowing	
		Rating	15	10	7	4		0	

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (see F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels and mines	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	

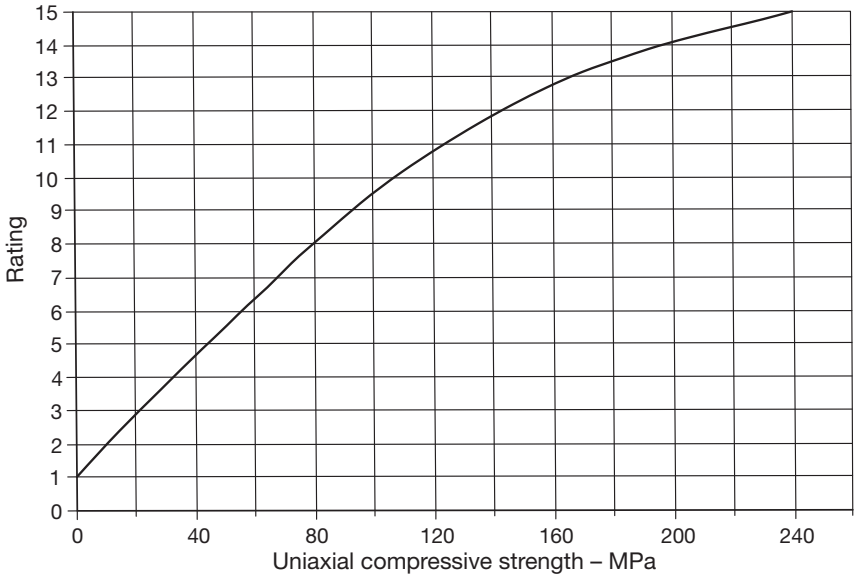
Table A.1 (continued)

Parameter	*Range of values*				
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
D. MEANING OF ROCK CLASSES					
Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300–400	200–300	100–200	< 100
Friction angle of rock mass (deg)	> 45	35–45	25–35	15–25	< 15
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS[a]					
Discontinuity length (persistence)	< 1 m	1–3 m	3–10 m	10–20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture) in (mm)	None	< 0.01	0.1–1.0	1–5	> 5
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gouge) in (mm)	None	Hard filling < 5	Hard filling >5	Soft filling < 5	Soft filling > 5
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	2	0

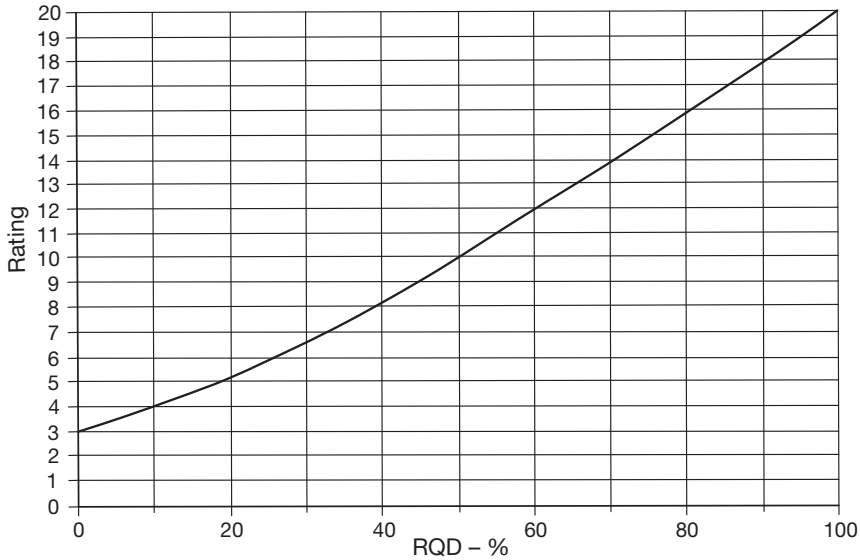
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING[b]

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip Dip 45°–90°	Drive with dip Dip 20°–45°	Dip 45°–90°	Dip 20° –45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip Dip 45°–90°	Drive against dip Dip 20°–45°	Dip 0°–20° – Irrespective of strike	
Fair	Unfavourable	Fair	

Notes: (a) Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A4 of this table directly. (b) Modified after Wickham *et al.* (1972).

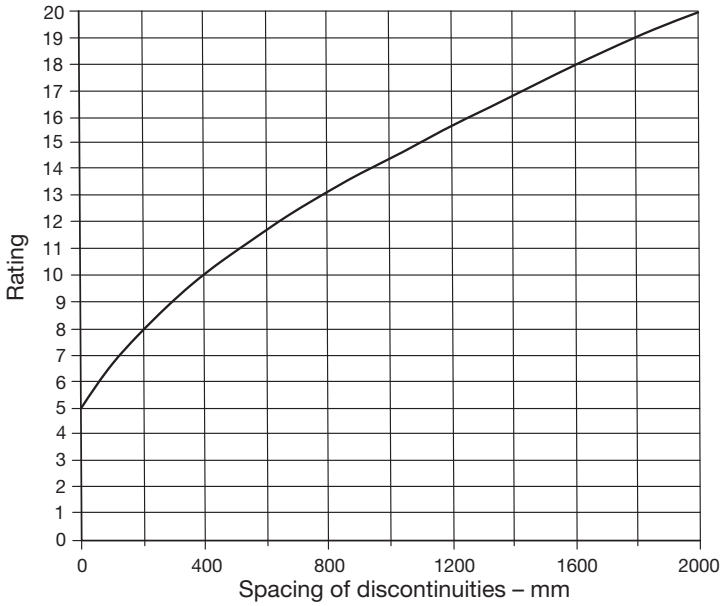


a) CHART A Ratings for strength of intact rock

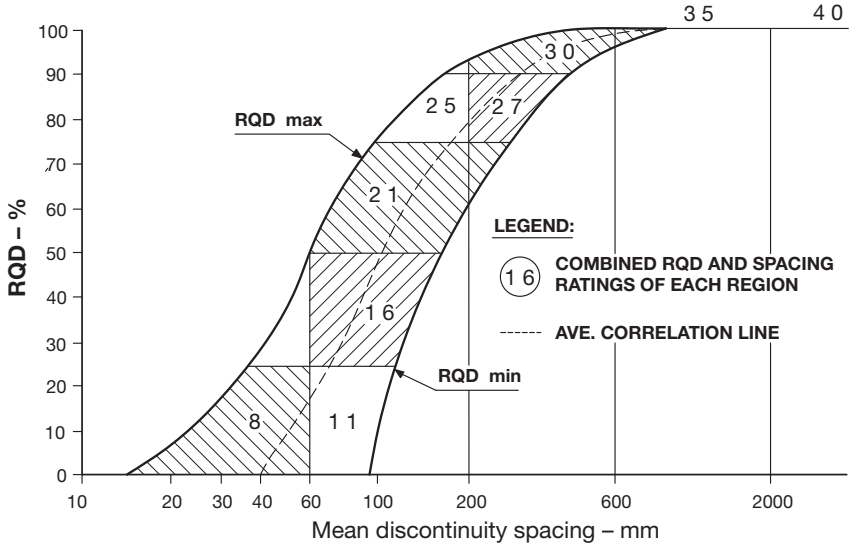


b) CHART B Ratings for RQD

Figure A.1 a) to d) Charts for various RMR ratings (after Bieniawski 1989)



c) CHART C Ratings for discontinuity spacing



d) CHART D Correlation between RQD and discontinuity spacing

Figure A.1 (continued)

- *Rock quality designation (RQD)* – For boreholes, a length weighted mean RQD should be calculated for each structural region. This means multiplying each run RQD by the length of the run, summing the results for the whole structural region and dividing the sum by the length of the structural region. For exposure, or face logging, an assessment can be made directly from scan lines, or using Figure A.1d.
- *Spacing of discontinuities* – For boreholes, Figure A.1d can be used, or can be assessed from the fracture index if this is recorded. For exposures or face logging, measurements can be made directly.
- *Condition of discontinuities* – Davis (2006) suggests five parameters to assess the condition of discontinuities. These are persistence (length of the discontinuity in exposure), aperture (discontinuity separation or openness), roughness, infilling and weathering. Persistence – can be obtained from exposures, but not from cores. Aperture – cannot be obtained from cores, although where infill is present, the aperture can be assumed to be the infilling thickness. With both of these, if no information can be obtained then judgement should be made on the significance on the design. Roughness – can be obtained from logged discontinuities for a structural region, where these are not available, but summary descriptions are available for each joint set, the summary term can be used to derive a rating.
- *Groundwater conditions* – This can be obtained from piezometer data along the tunnel alignment.
- *Orientation of discontinuities* – Possibly available before tunnel construction via an orientated core, downhole logging or exposure logging. The dip direction of discontinuities can be plotted on stereographic projections and related to the tunnel axis. Where no data are available, Davis (2006) suggests adopting a 'fair' rating as a best estimate.

A.2 Rock Mass Quality Rating (Q)

Brief details of this method of rock mass classification are provided in section 2.4.4.3, with this appendix providing further information. Tables A.2 to A.7 (Barton *et al.* 1974, Barton 2000 and 2002) provide the classification of individual parameters used to obtain the Rock Mass Quality Rating value, *Q,* for a rock mass.

Table A.2 Rock quality designation (after Barton 2002)

		RQD (%)
A	Very poor	0–25
B	Poor	25–50
C	Fair	50–75
D	Good	75–90
E	Excellent	90–100

Notes: (i) Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q. (ii) RQD intervals of 5, i.e.100, 95, 90, etc, are sufficiently accurate.

Table A.3 Joint set number (after Barton 2002)

		J_n
A	Massive, no or few joints	0.5–1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joints sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, 'sugar cube', etc	15
J	Crushed rock, earthlike	20

Notes: (i) For tunnel intersections, use ($3.0 \times J_n$). (ii) For portals use ($2.0 \times J_n$).

Table A.4 Joint roughness number (after Barton 2002)

		J_r
(a) Rock-wall contact, and (b) rock-wall contact before 10 cm shear		
A	Discontinuous joints	4.0
B	Rough or irregular, undulating	3.0
C	Smooth, undulating	2.0
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
(c) *No rock-wall contact when sheared*		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1.0

Notes: (i) Descriptions refer to small-scale features and intermediate-scale features, in that order. (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. (iii) J_r = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are orientated for minimum strength. (iv) J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ, where $\tau \approx \sigma_n \tan^{-1}(J_r/J_a)$.

Table A.5 Joint alteration number (after Barton 2002)

		ϕ_r *approx. (deg)*	J_a
(a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote	——	0.75
B	Unaltered joint walls, surface staining only	25–35	1.0
C	Slightly altered joint walls, non-softening, mineral coatings, sandy particles, clay-free disintegrated rock, etc	25–30	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20–25	3.0
E	Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays	8–16	4.0
(b) *Rock-wall contact before 10 cm shear (thin mineral fillings)*			
F	Sandy particles, clay-free disintegrating rock, etc	25–30	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous but < 5 mm thickness)	16–24	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5 mm thickness)	12–16	8.0
J	Swelling-clay fillings, i.e. montmorillonite (continuous but > 5 mm thickness. Value of J_a depends on percent of swelling clay-sized particles, and access to water, etc.	6–12	8–12
(c) *No rock-wall contact when sheared (thick mineral fillings)*			
KLM	Zones or bands of disintegrated or crushed rock and clay	6–24	6, 8, or 8–12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	——	5.0
OPR	Thick, continuous zones or bands of clay (see G, H, J, for description of clay condition)	6–24	10, 13 or 13–20

Table A.6 Joint water reduction factor (after Barton 2002)

		Approx. water pressure (kg/cm²)	J_w
A	Dry excavations or minor inflow	< 1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1–2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5–10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5–10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2–0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1–0.05

Notes: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered. (iii) For general characterisation of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33, etc as depth increases from say 0–5, 5–25, 25–250 to > 250 m is recommended, assuming that RQD/J_n is low enough for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterisation values of the Stress Reduction Factor. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

Table A.7 Stress Reduction Factor (SRF) (after Barton 2002)

		σ_c/σ_1	σ_Θ/σ_c	*SRF*
(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)			5
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)			2.5
D	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5
E	Single shear zones in competent rock (clay-free), (depth of excavation ≤ 50 m)			5
F	Single shear zones in competent rock (clay-free), (depth of excavation > 50 m)			2.5
G	Loose, open joints, heavily jointed or 'sugar-cube', etc. (any depth)			5
(b) *Competent rock, rock stress problems*				
H	Low stress, near surface, open joints	> 200	< 0.01	2.5
J	Medium stress, favourable stress condition	200–10	0.01–0.3	1
K	High stress, very tight structure. Usually favourable to stability, maybe unfavourable for wall stability	10–5	0.3–0.4	0.5–2
L	Moderate slabbing after > 1 h in massive rock	5–3	0.5–0.65	5–50
M	Slabbing and rock burst after a few minutes in massive rock	3–2	0.65–1	50–200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	< 2	> 1	200–400
(c) *Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure*				
O	Mild squeezing rock pressure		1–5	5–10
P	Heavy squeezing rock pressure		> 5	10–20
(d) *Swelling rock: chemical swelling activity depending on the presence of water*				
R	Mild swelling rock pressure			5–10
S	Heavy swelling rock pressure			10–15

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

A.2.1 Use of the Q-method for predicting TBM performance

Barton (1999), with further explanation in Barton (2000), developed a method for predicting the penetration rate and advance rate for TBM tunnelling. This method is based on an expanded Q-method of rock mass classification and average cutter force in relation to the appropriate rock mass strength. The parameter Q_{TBM} can be estimated during feasibility studies, and can also be back calculated from TBM performance during tunnelling. Equation A.1 shows the expression used to calculate Q_{TBM} and is based on equation 2.10 presented for the standard Q-system.

$$Q_{TBM} = \frac{RQD_0}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \times \frac{SIGMA}{F^{10}/20^9} \times \frac{20}{CLI} \times \frac{q}{20} \times \frac{\sigma_\theta}{5} \tag{A.1}$$

where RQD_0 = RQD (%) interpreted in the tunnelling direction. J_n, J_r, J_a, J_w and SRF are unchanged, except that J_r and J_a should refer to the joint set that most assists (or hinders) boring. F is the average cutter load (tnf) through the same zone, normalized by 20 tnf. SIGMA is the rock mass strength estimate (MPa) in the same zone. CLI is the cutter life index (for example 4 for quartzite and 90 for limestone). q is the quartz content in percentage terms and σ_θ is the induced biaxial stress on the tunnel face (approx. MPa) in the same zone, normalized to an approximate depth of 100 m. SIGMA incorporates the Q-value. The choice between $SIGMA_{cm}$ and $SIGMA_{tm}$ (equations A.2 and A.3) will depend on orientation (Barton 2000).

$$SIGMA_{cm} = 5\gamma Q_c^{1/3} \tag{A.2}$$

$$SIGMA_{tm} = 5\gamma Q_t^{1/3} \tag{A.3}$$

where $Q_c = Q\sigma_c/100$, $Q_t = Q.I_{50}/4$, γ = density (g/cm^3), σ_c is the uniaxial strength, I_{50} is the point load strength.

Based on empirical data, Barton (1999) suggested an approximate relationship between penetration rate (PR) and Q_{TBM} as shown in equation A.4.

$$PR \approx 5(Q_{TBM})^{-0.2} \tag{A.4}$$

and advance rate (AR) as shown in equation A.5.

$$AR \approx 5(Q_{TBM})^{-0.2} \times T^m \tag{A.5}$$

where T is total time in hours (24/day, 168/week, etc.) and m is defined from the empirical data as follows:

Best performance	m ~ –0.13 to –0.17 (variable)
Good	m ~ –0.17
Fair	m ~ –0.19
Poor	m ~ –0.21
Exceptionally poor	m ~ –0.25

m can be further refined based on the diameter of the tunnel D, CLI, q and n using equation A.6.

$$m \approx m_1 \times \left(\frac{D}{5}\right)^{0.20} \times \left(\frac{20}{CLI}\right)^{0.15} \times \left(\frac{q}{20}\right)^{0.10} \times \left(\frac{n}{2}\right)^{0.05} \tag{A.6}$$

where n = porosity (%). Some case history data using Q_{TBM} were reported by Sapigni *et al.* (2002) and Figure A.2 is reproduced from Palmström and Broch (2006).

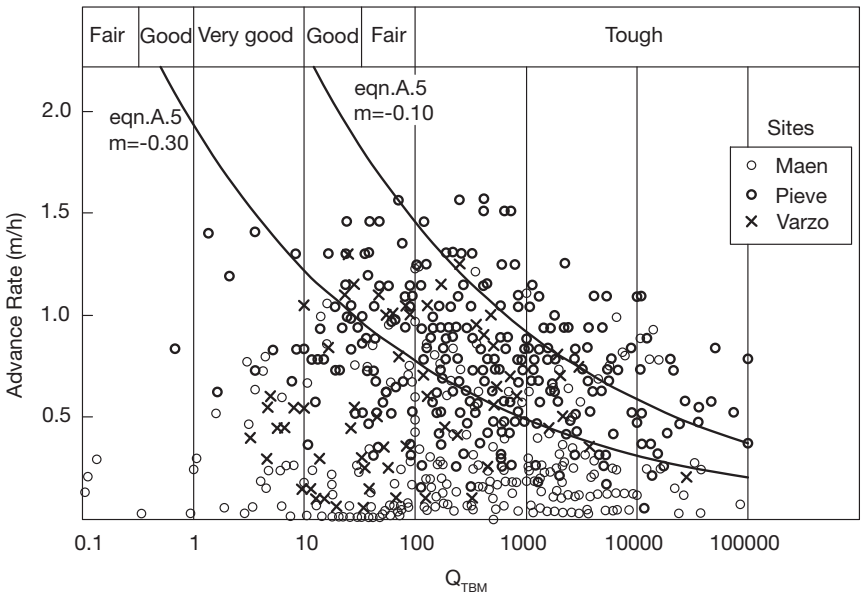


Figure A.2 Advance rate for three TBM tunnels plotted against Q_{TBM} (Sapigni *et al.* 2002, reproduced from Palmström and Broch 2006).

Appendix B

Analytical calculation of a sprayed concrete lining using the continuum method

B.1 Introduction

There are different analytical methods to estimate the internal forces in a tunnel lining and give an indication on the type of support needed (see section 3.5). In this section the focus is on tunnels which have a large overburden (h ≥ D). This allows the ground to be treated as a continuum, i.e. a plate with deformations in one plane (Figure B.1). The plate has a circular hole (the tunnel), which is stiffened by a circular ring (the lining). It can be assumed that the area above the tunnel is not softened and can carry some load. The primary stresses can be calculated without the associated deformations and the lateral coefficient of earth pressure is K_0.

For the approach of a rigid interconnection between the ground and the tunnel lining it is important to note whether the tangential component of the stresses from the earth pressure can be transferred into the tunnel lining for example through friction. In many cases it is better to assume tangential slippage between the ground and the tunnel lining in order to be on the

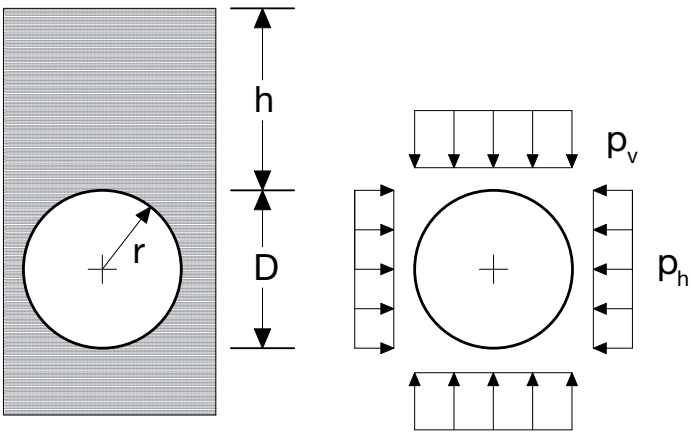


Figure B.1 a) Analytical model for deep tunnels and b) primary loads (Ahrens *et al.* 1982)

safe side. This can sometimes also be supported from a construction point of view by the type of tunnel construction for example for a shield driven tunnel or a tunnel with a membrane layer between the lining and the ground. The earth pressure approach displayed in Figure B.1 has first been suggested by DGGT (1980) and is valid independently of the depth of the tunnel and the chosen analytical model. Earlier analytical models assumed different approaches for the earth pressure using tables and diagrams for the simple determination of internal loads.

B.2 Analytical model using Ahrens *et al.* (1982)

MAIN ASSUMPTIONS AND REQUIREMENTS

- Straight tunnel.
- Load, ground parameters and cross sectional area remain constant along the tunnel.
- The tunnel construction is completed.
- Primary stress condition $\sigma_v^p = -\gamma \times h$; $\sigma_h^p = -K_0 \times \gamma \times h$
- Circular tunnel cross section.
- Homogeneous, isotropic and ideal-elastic material behaviour for the ground and the lining.
- Thin tunnel lining.
- Constant area and constant second moment of area in the φ-direction.

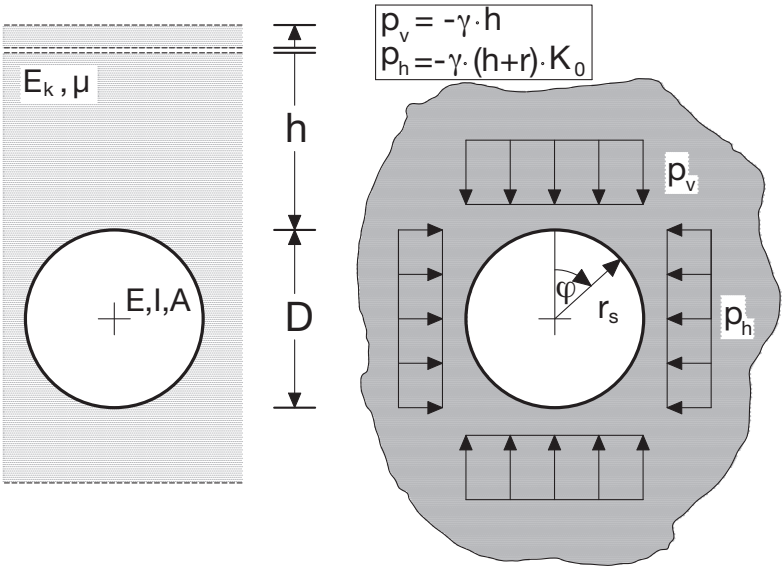


Figure B.2 a) Analytical model for deep tunnels and b) primary loads (Ahrens *et al.* 1982)

If segmental linings are used, the following additional assumptions are made in the analysis:

- Pre-deformations of the segmental linings resulting from the erection of the segments are proportional to the elastic deformations.
- The annular gap between the ground and the lining is completely grouted.
- Linearized theory of second order (small strain, large deformations) can be applied.

B.3 Required equations and calculation process

The calculation of the internal forces of the analytical model is carried out using the displacement method. This requires that the primary stress situation or the stresses determined from the earth pressure approach is used as a load displacement condition, where the, as yet unknown, deformations (in this case the deformation of the tunnel contour) are assumed to be zero.

However, as additional limitations of the deformations do not exist, the forces due to the earth pressure are not in equilibrium around the tunnel contour. Furthermore, the deformations along the tunnel contour are not equal to zero. Instead, these can be calculated taking into account the appropriate forces – the transition condition between the perforated disc

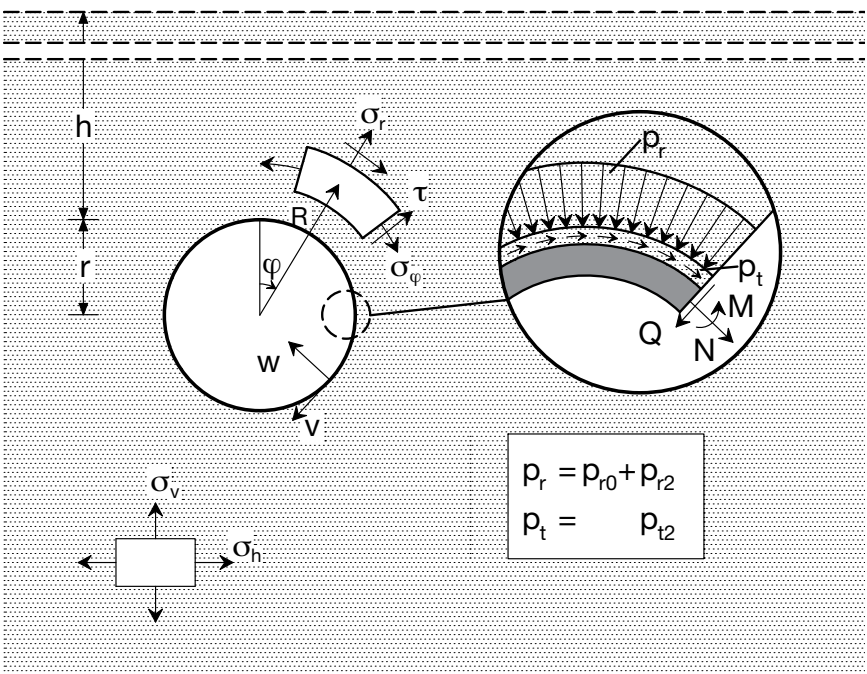


Figure B.3 Definitions of the parameters

and the circular ring – with the help of the unity deformation condition from the equilibrium conditions along the tunnel contour. The consideration of the equilibrium conditions for the individual components of the Fourier series leads to an equilibrium system, which allows for a more or less easy calculation of the unknown deformations, resulting in the internal forces. For the case of a tunnel support with infinite axial stiffness, the equations of the systems can be decoupled so that explicit equations can be given for the calculation of the deformations.

The following are the equations, using first order theory, for the case of a rigid bond between the tunnel support and the ground assuming infinite axial stiffness. If using segmental lining, the pre-deformations of the segments as a result of the installation can be considered using second order theory (Ahrens *et al.* 1982). It is assumed that the tunnel lining has an infinite axial stiffness.

The following equations are from Ahrens *et al.* (1982) and further explanations can be found in this reference.

Earth pressure:

$$p_v = -\gamma \times h \tag{B.1a}$$

$$p_h = -K_0 \times \gamma \times (h + r) \tag{B.1b}$$

Transformation into polar coordinates:

$$p_r = \bar{p}_{r0} + \bar{p}_{r2} \times \cos 2\varphi \tag{B.2a}$$

$$p_t = \bar{p}_{r2} \times \sin 2\varphi \tag{B.2b}$$

With

$$\bar{p}_{r0} = 0.5 \times \gamma \times [h + (h + r) \times K_0] \tag{B.3a}$$

$$\bar{p}_{r2} = \bar{p}_{tr2} = 0.5 \times \gamma \times [h - (h + r) \times K_0] \tag{B.3b}$$

The load and deformation variables of the plate are denoted with a superscript 'D', while the load and deformation variables associated with the circular ring frame are denoted with a superscript 'R'.

Deformations:

$$w(\varphi) = \bar{w}_0 + \bar{w}_2 \cdot \cos 2\varphi \tag{B.4}$$

$$v(\varphi) = + \bar{v}_2 \cdot \sin 2\varphi \tag{B.5}$$

With (EA $\rightarrow \infty$)

$$\bar{w}_0 = 0 \tag{B.6}$$

$$\bar{w}_2 = \frac{\bar{p}_{r2} + 0.5 \times \bar{p}_{t2}}{\dfrac{1}{(3-\mu-4\mu^2)} \times (2.25 - 1.5\mu) \times \dfrac{E_k}{r} + \dfrac{9EI}{r^4}} \tag{B.7}$$

$$\bar{v}_2 = 0.5 \times \bar{w}_2 \tag{B.8}$$

The proportion of the earth pressure acting on the circular frame and the continuum is equivalent to their relative stiffnesses. The circular frame load is:

$$\bar{p}_{r0}^{R} = \bar{p}_{r0} - \bar{p}_{r0}^{D}$$

EA $\rightarrow \infty : \bar{p}_{r0}^{D} = 0$ (as a result of the infinite axial stiffness, the complete constant load is supported by the circular ring frame)

$$\bar{p}_{r0}^{R} = \bar{p}_{r0}$$

$$\bar{p}_{r2}^{R} = \bar{p}_{r2} - \bar{p}_{r2}^{D}$$

$$\bar{p}_{r2}^{D} = \frac{E_c}{r} \times \frac{1}{(3-\mu-4\mu^2)} \times \left[(5-6\mu) \times \bar{w}_2^{D} + (-4+6\mu)\bar{v}_2^{D}\right] \tag{B.9a}$$

$$\bar{p}_{t2}^{R} = \bar{p}_{t2} - \bar{p}_{t2}^{D}$$

$$\bar{p}_{t2}^{D} = \frac{E_c}{r} \times \frac{1}{(3-\mu-4\mu^2)} \times \left[(-4+6\mu) \times \bar{w}_2^{D} + (5-6\mu)\bar{v}_2^{D}\right] \tag{B.9b}$$

Proportional internal force parameter:

Load part $\bar{p}_{r0}^{R}$:

$$N_0 = -r \times \bar{p}_{r0}^{R} \tag{B.10a}$$

$$Q_0 = 0 \tag{B.10b}$$

$$M_0 = 0 \tag{B.10c}$$

Load part $\bar{p}_{r2}^{R}$, $\bar{p}_{t2}^{R}$:

$$N_2 = \frac{r}{3} \times \left(2 \times \bar{p}_{t2}^{R} + \bar{p}_{r2}^{R}\right) \times \cos 2\varphi \tag{B.11a}$$

$$Q_2 = -\frac{r}{3} \times \left(\bar{p}_{t2}^{R} + 2 \times \bar{p}_{r2}^{R}\right) \times \sin 2\varphi \tag{B.11b}$$

$$M_2 = \frac{r^2}{6} \times \left(\bar{p}_{t2}^{R} + 2 \times \bar{p}_{r2}^{R}\right) \times \cos 2\varphi \tag{B.11c}$$

Or simpler:

$$Q_2 = -\frac{6EI}{r^3} \times \bar{w}_2^{R} \times \sin 2\varphi \tag{B.11d}$$

$$M_2 = \frac{3EI}{r^2} \times \bar{w}_2^{R} \times \cos 2\varphi \tag{B.11e}$$

Final internal parameters and deformations:

$$N = N_0 + N_2 \tag{B.12}$$

$$Q = Q_2 \tag{B.13}$$

$$M = M_2 \tag{B.14}$$

$$w = w_2 \tag{B.15}$$

B.4 Example for a tunnel at King's Cross Station, London

Figure B.4 shows the schematic of the geology associated with a tunnel at King's Cross Station in London, UK. The overburden is assumed to be h = 11.0 m.

A) GROUND PARAMETERS

Density	$\rho = 2000\ kg/m^3$
Stiffness	$E_S = 87\ MN/m^2$ $(E_c = f(E_S))$
Poisson's ratio	$\mu = 0.15$
Coefficient of Lateral Earth Pressure	$K_0 = 1.2$

The ground is idealized as a homogenous continuum assuming an average density for the ground of 2000 kg/m^3.

B) STRUCTURAL SYSTEM, LOADS AND PARAMETERS

In order to simplify the calculation, it is assumed that the construction has been completed. The effects of groundwater are neglected as the tunnel was constructed in London Clay. No dead load is considered at the ground

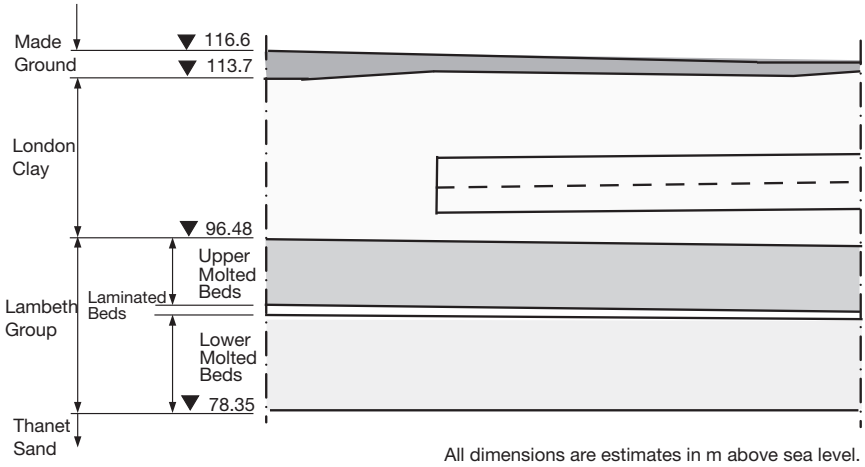


Figure B.4 Schematic showing the geology around the tunnel

surface now or in the future. A rigid bond exists between the lining and the ground.

Weight of the ground:	γ	= 20 kN/m^3	The tunnel weight is neglected.
Radius of the system axis:	r_S	= 3.05 m	
Overburden:	h_O	= h = 11.0 m	

$p_v, p_h \rightarrow p_r, p_t$

$$\bar{p}_{r0} = 0.5 \times \gamma \times [h + (h + r) \times K_0]$$
$$= 0.5 \times 20 \times [11.0 + (11.0 + 3.05) \times 1.2] = 278.6 \text{ kN/m}^2$$

$$\bar{p}_{r2} = 0.5 \times \gamma \times [h - (h + r) \times K_0]$$
$$= 0.5 \times 20 \times [11.0 - (11.0 + 3.05) \times 1.2] = -58.6 \text{ kN/m}^2$$

C) TUNNEL SUPPORT PARAMETERS

The material is sprayed concrete (C20/25) with a Young's modulus of $E = 2.88 \times 10^4$ MPa. The key parameters are:

Profile parameters:	A	= 0.175 m^2/m
	I	= 4.466×10^{-4} m^4/m
	W_o	= 5.104×10^{-3} m^3/m
	W_i	= 5.104×10^{-3} m^3/m

Allowable compressive stress:

$\sigma_{c,c}$ = 11.3 MPa (includes factor of safety and long-term influences)

D) STIFFNESS PARAMETERS

Ground stiffness: E_S = 87 MPa (stiffness parameter)

In general, the stiffness parameter E_S is used as the deformation parameter for soft ground and is determined in laboratory experiments using a compression test with restricted strain. This parameter cannot simply be used as an Elasticity modulus E_C when applying the continuity calculation. In this case, the Elasticity modulus for the three-dimensional continuum can be determined with a Poisson's ratio of $\mu = 0.15$ using Equation B.16:

$$E_C = \frac{(1+\mu)(1-2\mu)}{(1-\mu)} E_S \tag{B.16}$$

A further conversion of the Elasticity modulus of a disc like structure in a plane strain state is not required, as these specific modifications for the model are already included in the following equations. With the given parameters the Elasticity modulus can be calculated

$$E_C = \frac{(1+0.15)(1-2\times 0.15)}{(1-0.15)} 87 = 82.4 \text{ MPa} \tag{B.17}$$

Tunnel support stiffness: EI = 2.88 $10^4 \times$ 4.466 10^{-4} = 12.86 MNm^2/m

E) DETERMINATION OF THE INTERNAL FORCES

Assumption: Tunnel lining with an infinite axial stiffness; 1st Order Theory
Assumption: EA $\rightarrow \infty$
Radial displacement $\bar{w}_0 = 0$
Radial displacement $\bar{w}_2$

$$\bar{w}_2 = \frac{\bar{p}_{r2} + 0.5 \times \bar{p}_{t2}}{\frac{1}{(3-\mu-4\mu^2)} \times (2.25 - 1.5\mu) \times \frac{E_C}{r} + \frac{9EI}{r^4}}$$

$$\bar{w}_2 = \frac{-58.6 + 0.5 \times (-58.6)}{\frac{1}{(3-0.15-4\times 0.15^2)} \times (2.25 - 1.5 \times 0.15) \times \frac{82000}{3.05} + \frac{9 \times 12860}{3.05^4}}$$

$$= -0.0042 \text{ m}$$

The tangential displacement

$$\bar{v}_2 = 0.5 \times \bar{w}_2 = 0.5 \times (-0.0042) = -0.0021 \text{ m}$$

Partial load $\bar{p}_{r0}^{R}$, (the total constant partial load $\bar{p}_{r0}$ is carried by the circular ring system (EA $\rightarrow \infty$)):

$$\bar{p}_{r0}^{R} = 278.6 \text{ kN/m}^2$$

Load parts $\bar{p}_{r2}^{R}$, $\bar{p}_{t2}^{R}$

The load part acting on the circular ring support derived from the load parts $\bar{p}_{r2}$ and $\bar{p}_{t2}$ can only be calculated indirectly from the difference between the total load and the portion of the load acting on the plate due to EA $\rightarrow \infty$:

$$\bar{p}_{r2}^{D} = \frac{E_c}{r} \times \frac{1}{(3-\mu-4\mu^2)} \times \left[(5-6\mu)\times\bar{w}_2^{D} + (-4+6\mu)\bar{v}_2^{D})\right]$$

$$\begin{aligned}\bar{p}_{r2}^{D} &= \frac{82000}{3.05} \times \frac{1}{(3-0.15-4\times0.15^2)} \times \\ &\left[(5-6\times0.15)\times(-0.0042)+(-4+6\times0.15)\times(-0.0021)\right] \\ &= 9.787\times\left[-17.03+6.44\right] = -103.7 \text{ kN/m}^2\end{aligned}$$

$$\bar{p}_{t2}^{D} = \frac{E_c}{r} \times \frac{1}{(3-\mu-4\mu^2)} \times \left[(-4+6\mu)\times\bar{w}_2^{D} + (5-6\mu)\bar{v}_2^{D}\right]$$

$$\begin{aligned}\bar{p}_{t2}^{D} &= \frac{82000}{3.05} \times \frac{1}{(3-0.15-4\times0.15^2)} \times \\ &\left[(-4+6\times0.15)\times\left(-0.0042\right)+(5-6\times0.15)\times\left(-0.0021\right)\right] \\ &= 9.788\times\left[12.87-8.51\right] = 42.7 \text{ kN/m}^2\end{aligned}$$

$\rightarrow \bar{p}_{r2}^{R} = \bar{p}_{r2} - \bar{p}_{r2}^{D} = -58.6 - (-103.7) = 45.1 \text{ kN/m}^2$

$\rightarrow \bar{p}_{t2}^{R} = \bar{p}_{t2} - \bar{p}_{t2}^{D} = -58.6 - 42.7 = -101.3 \text{ kN/m}^2$

Internal loads:

Load portion $\bar{p}_{r0}^{R}$

$N_0 = -r \times \bar{p}_{r0}^{R} = -3.05 \times 278.6 = -849.7$ kN/m

$Q_0 = 0$ kN/m

$M_0 = 0$ kNm/m

Load part $\bar{p}_{r2}^{R}$, $\bar{p}_{t2}^{R}$:

$$N_2 = \frac{r}{3} \times \left(2 \times \bar{p}_{t2}^{R} + \bar{p}_{t2}^{R}\right) \times \cos 2\varphi = \frac{3.05}{3} \times \left(2 \times (-101.3) + 45.1\right) \times \cos 2\varphi$$
$$= -160.3 \times \cos 2\varphi \text{ kN/m}$$

$$Q_2 = -\frac{r}{3} \times \left(\bar{p}_{t2}^{R} + 2 \times \bar{p}_{r2}^{R}\right) \times \sin 2\varphi = -\frac{3.05}{3} \times \left(-101.3 + 2 \times 45.1\right) \times \sin 2\varphi$$
$$= 11.3 \times \sin 2\varphi \text{ kN/m}$$

$$M_2 = \frac{r^2}{6} \times \left(\bar{p}_{t2}^{R} + 2 \times \bar{p}_{r2}^{R}\right) \times \cos 2\varphi = \frac{3.05^2}{6} \times \left(-101.3 + 2 \times 45.1\right) \times \cos 2\varphi$$
$$= -17.2 \times \cos 2\varphi \text{ kNm/m}$$

Calculation to check Q_2 and M_2 from the radial displacement $\bar{w}_2^{R}$:

$$Q_2 = -\frac{6EI}{r^3} \times \bar{w}_2^{R} \times \sin 2\varphi = -\frac{6 \times 12860}{3.05^3} \times \left(-0.0042\right) \times \sin 2\varphi$$
$$= 11.3 \times \sin 2\varphi \text{ kN/m}$$

$$M_2 = \frac{3EI}{r^2} \times \bar{w}_2^{R} \times \cos 2\varphi = \frac{3 \times 12860}{3.05^2} \times \left(-0.0042\right) \times \cos 2\varphi$$
$$= -17.2 \times \cos 2\varphi \text{ kNm/m}$$

Final internal parameters and deformations

$N = N_0 + N_2 = -849.7 - 160.1 \times \cos 2\varphi$ kN/m

$Q = Q_2 = 11.3 \times \sin 2\varphi$ kN/m

$M = M_2 = -17.2 \times \cos 2\varphi$ kNm/m

Radial displacements:

$$w = \quad w_2 = -0.0042 \times \cos 2\varphi \text{ m}$$

F) STRESS ANALYSIS

Using Ahrens *et al.* (1982) section 2.3.4.2 and the following equation,

$$\sigma = \frac{N}{A} \pm \frac{M}{W}$$

The crown stresses are:

Extrados:

$$\sigma_e = -\frac{1010}{0.175} + \frac{17.23}{0.0051} = |-2.4| \text{ MPa} < 11.3 \text{ MPa}$$

Intrados:

$$\sigma_i = -\frac{1010}{0.175} - \frac{17.23}{0.0051} = |-9.15| \text{ MPa} < 11.3 \text{ MPa}$$

The bench stresses are:

Extrados:

$$\sigma_e = -\frac{690}{0.175} - \frac{17.23}{0.0051} = |-7.32| \text{ MPa} < 11.3 \text{ MPa}$$

Intrados:

$$\sigma_i = -\frac{690}{0.175} + \frac{17.23}{0.0051} = |-0.56| \text{ MPa} < 11.3 \text{ MPa}$$

The invert stresses are:

Extrados:

$$\sigma_e = -\frac{1010}{0.175} + \frac{17.23}{0.0051} = |-2.4| \text{ MPa} < 11.3 \text{ MPa}$$

Intrados:

$$\sigma_i = -\frac{1010}{0.175} - \frac{17.23}{0.0051} = |-9.15| \text{ MPa} < 11.3 \text{ MPa}$$

This shows that all the stresses are within the allowable stresses for the sprayed concrete lining.

G) PRESENTATION OF THE INTERNAL FORCES AND THE DEFORMATION

The internal forces are shown in Figure B.5.

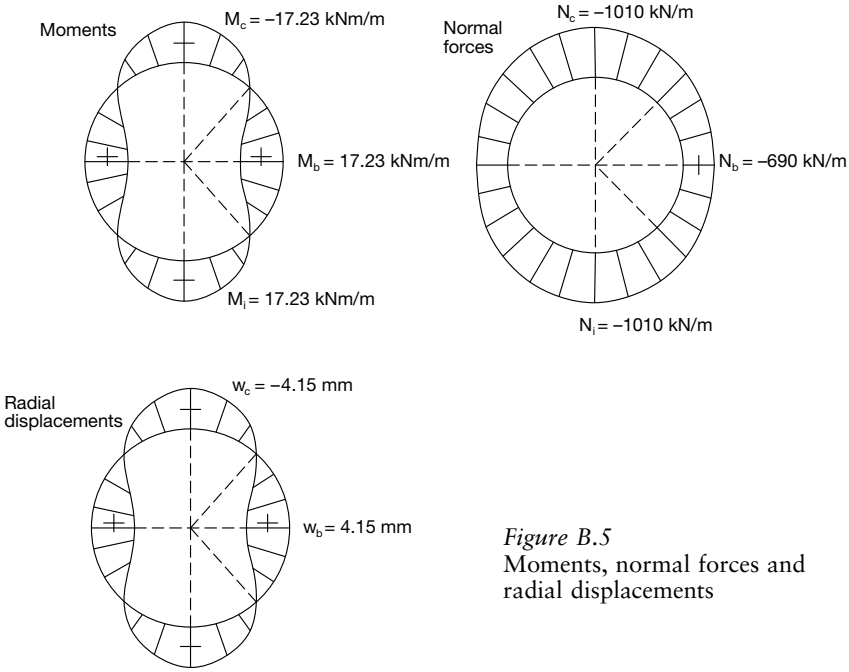


Figure B.5
Moments, normal forces and radial displacements

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