Sprayed Concrete Lined Tunnels

An introduction

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Preface

This book seeks to provide an introduction to sprayed concrete lined (SCL) tunnels for people who have little experience of the subject, as well as serving as a reference guide for experienced tunnellers. Tunnels, for any civil engineering purpose, in hard rock, blocky rock and soft ground are covered. In this context, blocky rock is defined as rock where the movement of blocks dominates behaviour. Soft ground is defined as soil or weak rocks where the ground behaves as a continuous mass, rather than discrete blocks.

Opinion is divided on whether engineering is an art or a science. Instinctively, engineers seek to describe the world exactly but nature confounds them. In response they find that an intuitive, semi-empirical approach can function better. Hence this is not a 'cook-book' and *the answer* is not on 'page 42'. However, it is hoped that the book contains sufficient information to guide engineers through their task to an appropriate solution. Given the limitations of space, where necessary this book refers to standard texts or other existing publications.

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

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Contents

1 What is a SCL tunnel?

Considering the title of this book, this seems like the first question that should be answered. A SCL tunnel is a tunnel with a sprayed concrete lining. This generic term makes no claims on how the tunnel was designed, the ground it was built in or what its purpose is. It simply describes the type of lining used.

Modern SCL tunnel construction is described in more detail in Section 3. Figures 1.1 and 1.2 show a typical excavation sequence and cross-section for a large diameter tunnel in soft ground at a shallow depth. The arrangement of the excavation sequence is influenced by the geometry of the tunnel, the stability of the ground and the construction plant. In shallow tunnels

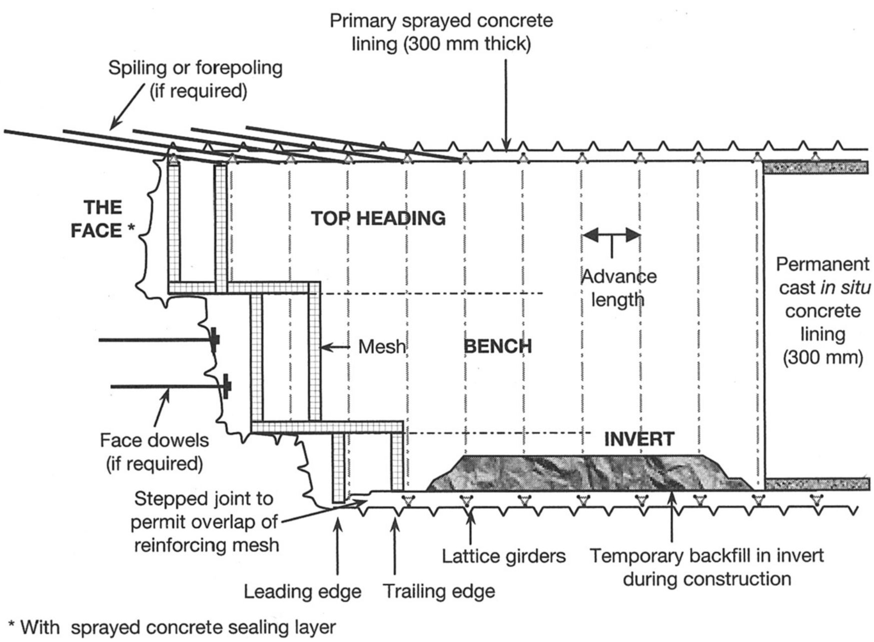


Figure 1.1 Long-section of a SCL tunnel in soft ground

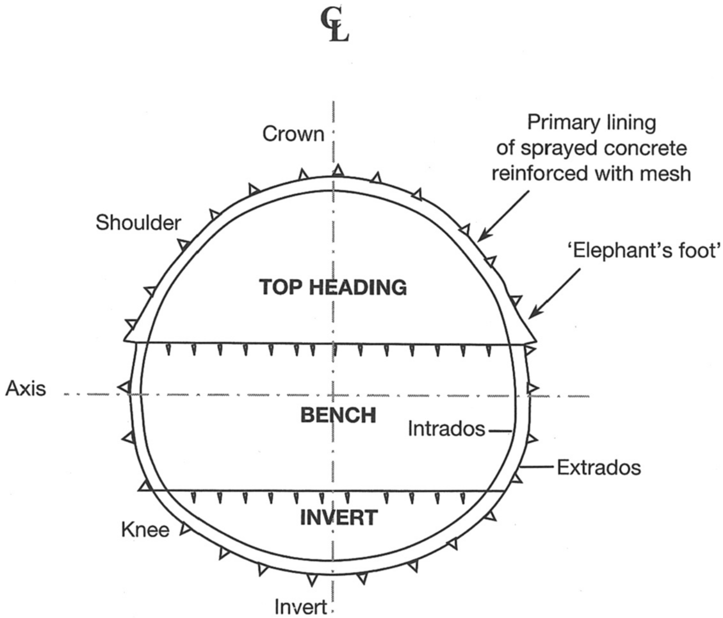


Figure 1.2 Cross-section of a SCL tunnel in soft ground

it is important to close the invert as close to the face as possible, in order to limit ground deformation. However, the designer has a fair degree of freedom in choosing the exact arrangement of the excavation sequence.

After each stage of the excavation sequence has been mucked out, concrete is sprayed on the exposed ground surface. The lining is often built up in several layers with mesh reinforcement inserted between the layers. Alternatively short fibres can be added to the mix to provide some tensile capacity. Once that section of lining is complete, the next stage is excavated and so the process progresses and a closed tunnel lining is formed. Normally, the sprayed concrete lining does not form part of the permanent works and another lining is installed at a later date (see Figure 1.1).

In rock tunnels sprayed concrete plays a lesser role as it is used in combination with rockbolts to support the rock (see Figure 1.3). Nonetheless, sprayed concrete is an important part of the support and often forms part of the permanent support. As with tunnels in soft ground, the degree and timing of support and the excavation sequence are governed by the stability of the ground.

A plethora of other terms exist for tunnels with sprayed concrete linings: most famously in Europe there is NATM – the New Austrian Tunnelling Method; in North America SEM – Sequential Excavation Method – is often used; while elsewhere no particular emphasis is placed on the use of sprayed

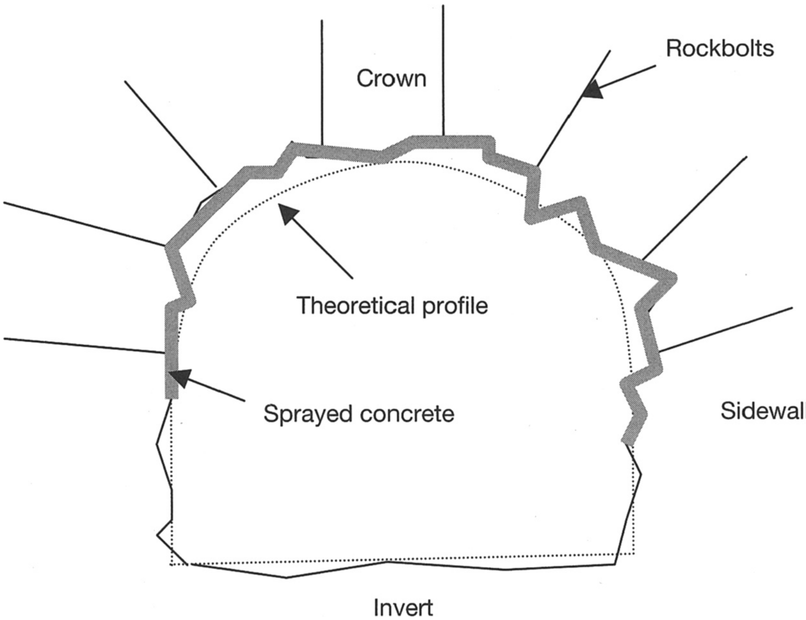


Figure 1.3 Cross-section of a SCL tunnel in rock

concrete as a distinguishing feature, for example in hard rock tunnelling. In this book the term SCL will be used throughout as a descriptive term and as such it opens the door to look at the many and varied uses of sprayed concrete in modern tunnelling.

1.1 Sprayed concrete – the early days

The invention of sprayed concrete is generally attributed to Carl Ethan Akeley in 1907, who used a dry-mix sprayed mortar to apply a durable coating to dinosaur bones. However, in Germany, August Wolfsholz had been developing equipment for spraying cementitious mortar in tunnels for rock support from as early as 1892 (Strubreiter 1998), and Carl Weber patented a method for spraying concrete in 1919 (Atzwanger 1999). While sprayed concrete was used on a few engineering projects to repair concrete structures or for rock support in the first half of that century – it was even trialled by the modernist architect Le Corbusier for one of his projects – this material and method first attracted serious attention after its use on a series of pioneering projects in Venezuela and Austria by Ladislaus von Rabcewicz in the 1950s (Rabcewicz 1969).

The early sprayed concrete was not a high quality product. Large quantities of aggressive accelerating additives were required to get the sprayed concrete to adhere to the ground and so that reasonably thick layers could be sprayed. The environment during spraying was very unhealthy due to the large quantities of dust and the caustic nature of the accelerators. Despite the accelerators, a large quantity of the sprayed concrete failed to adhere and fell as waste material onto the tunnel floor – so-called 'rebound'. The material was very sensitive to the influence of the nozzleman, since he controlled how the material was sprayed (which determines the compaction) and the water content. Because of this, and a deterioration caused by accelerators such as 'waterglass', long-term strengths of sprayed concrete were much lower than conventionally cast concrete and the material was more variable in quality.

Hence, research and development since the 1970s has focused primarily on accelerators and admixtures (to achieve higher early strengths with lower dosages of these expensive and often hazardous additives, without compromising long-term strength, and to reduce dust and rebound) and spraying equipment (to improve the quality, spraying quantity and automation). Research into the durability and mechanical properties of sprayed concrete other than strength and stiffness followed later as the early challenges were overcome and design approaches and usage developed.

1.2 Why use sprayed concrete linings?

To understand the origin and merits of sprayed concrete tunnel linings, one must first appreciate some fundamental tunnelling principles:

1. Tunnelling is a case of three-dimensional soil-structure interaction.
2. The load to be carried by the composite structure of the ground and lining arises from the *in situ* stresses and groundwater pressure.
3. Deformation of the ground is inevitable and it must be controlled to permit a new state of equilibrium to be reached safely.
4. The unsupported ground has a finite 'stand-up time'.
5. Often the strength of the ground depends on how much it is deformed.
6. The load on the lining will depend on how much deformation is permitted and how much stress redistribution ('arching') within the ground is possible.
7. The art of tunnelling is to maintain as far as possible the inherent strength of the ground so that the amount of load carried by the structure is minimised.

These basic principles have been understood implicitly or explicitly by experienced tunnellers since tunnels were first constructed. However, they were brought to the forefront of attention by the pioneering work of engineers, such as Rabcewicz, who developed the tunnelling philosophy that is now marketed as the New Austrian Tunnelling Method (NATM).

In his early work in rock tunnels, Rabcewicz (1969) recognised that sprayed concrete was a material well suited to tunnelling for the reasons below:

- Sprayed concrete is a structural material that can be used as a permanent lining.
- The material behaviour of sprayed concrete (which is initially soft and creeps under load but can withstand large strains at an early age) is compatible with the goal of a lining which permits ground deformation (and therefore stress redistribution in the ground).
- The material behaviour (specifically the increase in stiffness and strength with age) is also compatible with the need to control this deformation so that strain softening in the ground does not lead to failure.
- Sprayed concrete linings can be formed as and when required and in whatever shape is required. Hence the geometry of the tunnel and timing of placement of the lining can be tailored to suit a wide range of ground conditions. Sprayed concrete can also be combined with other forms of support such as rockbolts and steel arches.

One may also note the lower mobilisation times and costs for the major plant items compared to tunnel boring machines (TBMs). The same equipment can be used for shaft construction as well as tunnelling. SCL tunnelling offers a freedom of form that permits tunnels of varying cross-sections and sizes, and junctions to be built more quickly and cost-effectively than if traditional methods are used.

1.3 Development of SCL tunnelling

Sprayed concrete was first used as temporary (and permanent) support in rock tunnels. However, the principles above apply equally to weak rocks and soils. In the 1970s shallow SCL tunnels were successfully constructed in soft ground as part of metro projects in cities such as Frankfurt and Munich.

Taking the UK as one example, Figure 1.4 charts the rise of SCL tunnelling. This technique is relatively new to the UK and has only become widely used within the last 15 years. Initially there was great enthusiasm for SCL tunnelling. However, following the collapse of a series of SCL tunnels in 1994, this construction method came under intense scrutiny. Vociferous sceptics have asserted, and some still do, that SCL tunnelling cannot and should not be used in soft ground at shallow depths (e.g. Kovari 1994).

Reports by the UK Health and Safety Executive (1996) and the Institution of Civil Engineers (ICE 1996) have established that SCL tunnels can be constructed safely in soft ground and the reports provided guidance on how to ensure this during design and construction. The reports also drew attention to the weaknesses of this method:

- The person spraying the concrete (the nozzleman) has a considerable influence over the quality of the lining so the method is vulnerable to poor workmanship. This is particularly true for certain geometries of linings.
- The performance of the linings and ground must be monitored during construction to verify that both are behaving as envisaged in the design. The data from this monitoring must be reviewed regularly in a robust process of construction management to ensure that abnormal behaviour is identified and adequate countermeasures are taken.
- It is difficult to install instrumentation in sprayed concrete linings and to interpret the results (Golser *et al.* 1989, Mair 1998, Clayton *et al.* 2002).
- It is difficult to predict the behaviour of SCL tunnels in advance.

The specific disadvantages of this method, as applied to soft ground, are:

- Minimising deformations is of critical importance otherwise strain-softening and plastic yielding in the ground can lead rapidly to collapse. Complex excavation sequences can lead to a delay in closing the invert of the tunnel (and forming a closed ring). This delay can permit excessive deformations to occur.
- In shallow tunnels the time between the onset of failure and total collapse of a tunnel can be very short, so much tighter control is required during construction.

More general disadvantages include the fact that advance rates are slower than for TBM-driven tunnels, so SCL tunnels are not economic for long tunnels with a constant cross-section (i.e. greater than about 500 m to a few kilometres, depending on ground conditions). A higher level of testing is required for quality control during construction, compared to a segmentally lined tunnel.

Following the HSE and ICE reports, a considerable amount of guidance has been produced on such subjects as certification of nozzlemen (see Austin *et al.* (2000) for the latest guidance), instrumentation and monitoring (HSE 1996) and risk management. The UK tunnelling industry has incorporated much of this into its standard practices. Since the collapse at Heathrow, more than 200,000 m^3 of shallow SCL tunnels have been successfully constructed in a variety of soft ground conditions. Major UK projects such as the Heathrow Baggage Transfer Tunnel (Grose and Eddie 1996) and the CTRL North Downs Tunnel (Watson *et al.* 1999) have demonstrated the great benefits to be gained from this method, not least in terms of time and cost savings compared to traditional construction methods.

So the reputation of sprayed concrete in UK tunnelling has recovered and for certain types of work SCL has supplanted traditional methods. For example, SCL is the method of choice for shafts and short tunnels in the

1930	Mersey Tunnel – the first use of sprayed concrete in the UK?
1977	Dinorwic Power Station – SCL tunnels in hard rock.
1989	Channel Tunnel – SCL used extensively for caverns in chalk.
1990 –92	SCL road tunnels at Brighton and Pen-Y-Clip.
1992	SCL adopted for Heathrow Express stations following the successful HEX Trial tunnel.
1994	The collapse of SCL tunnels at Heathrow halts SCL work on HEX and JLE projects.
1996	ICE report on SCL tunnelling concludes that this method can be used in soft ground.
1996	Heathrow Baggage Transfer Tunnel completed in part with a permanent sprayed concrete lining; permanent sprayed concrete also used on JLE and HEX projects.
1998	Ramsgate Harbour Access tunnel.
1999	CTRL North Downs tunnel.
1999	Guilty verdicts in HSE case against contractor and his designer for HEX collapse.
2000	HSE Final report on HEX collapse is published.
2003	Single-pass fibre reinforced SCL tunnels at Heathrow T5.
2006	Outline design of Crossrail project progresses with SCL tunnels proposed for all mined stations.

Figure 1.4 Development of SCL tunnelling in the UK

London Clay. This pattern of development mirrors the experience of many other countries. Nevertheless, SCL tunnels are still perceived to be difficult to design because of the complex behaviour of sprayed concrete. This uncertainty, coupled with a history of high-profile failures, means that SCL tunnelling is perceived as risky. In truth SCL tunnelling is no more risky than any other type of tunnelling. The risks can be identified clearly and successfully managed.

2 Sprayed concrete

2.1 Constituents and mix design

'Sprayed concrete is concrete which is conveyed under pressure through a pneumatic hose or pipe and projected into place at high velocity, with simultaneous compaction' (DIN 18551 (1992)). It behaves in the same general manner as concrete but the methods of construction of SCL tunnels and of placement of sprayed concrete require a different composition of the concrete and impart different characteristics to the material, compared to conventionally placed concrete. Sprayed concrete consists of water, cement and aggregate, together with various additives. On a point of nomenclature, sprayed concrete is also known as 'shotcrete', while 'gunite' normally refers to sprayed mortar, i.e. a mix with fine aggregates or sand only.

The composition of the concrete is tailored so that:

- it can be conveyed to the nozzle and sprayed with a minimum of effort;
- it will adhere to the excavated surface, support its own weight and the ground loading as it develops;
- it attains the strength and durability requirements for its purpose in the medium- to long-term.

Table 2.1 contains a comparison of the constituents of a high quality sprayed concrete and an equivalent strength cast *in situ* concrete. Considering each component in turn, one may note that:

- the water/cement ratio in sprayed concrete is higher so that the mix can be pumped and sprayed easily;
- ordinary Portland Cement is normally used, in conjunction with cement replacements such as pulverised fly ash (PFA), though special cements are sometimes used;
- the mix is 'over-sanded' to improve pumpability (Norris 1999) (see Figure 2.1 for grading curve);
- the maximum aggregate size is usually limited to 10 or 12 mm;
- additives are used to accelerate the hydration reaction (see Figure 2.2 for the effect of increasing accelerator dosage on strength gain);

Table 2.1 Typical mix design

	High quality wet-mix sprayed concrete (Darby and Leggett 1997)	*Cast* in situ *concrete (from Neville 1995)*
Grade	C40	C40
Water/cement ratio	0.43	0.40
Cement inc. PFA, etc.	430 kg/m^3	375 kg/m^3
Accelerator	4%	–
Plasticiser	1.6% bwc	1.5%
Stabiliser	0.7% bwc	–
Microsilica	60 kg/m^3	–
Max. aggregate size	10 mm	30 mm
Aggregate < 0.6 mm	30–55%	32%

- plasticisers and stabilisers are added to improve workability as in conventional concrete;
- other components may include microsilica, which is added to improve immediate adhesion (which allows the accelerator dosage to be reduced) and to improve long-term density (which improves strength and durability) or steel fibres, which are added for structural reinforcement or crack control.

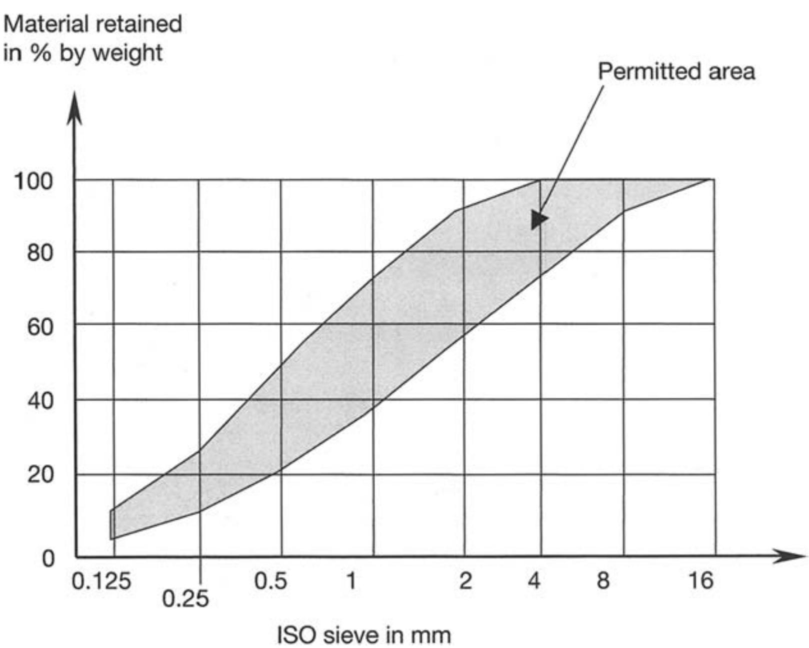


Figure 2.1 Typical grading curve for sprayed concrete (EFNARC 1996)

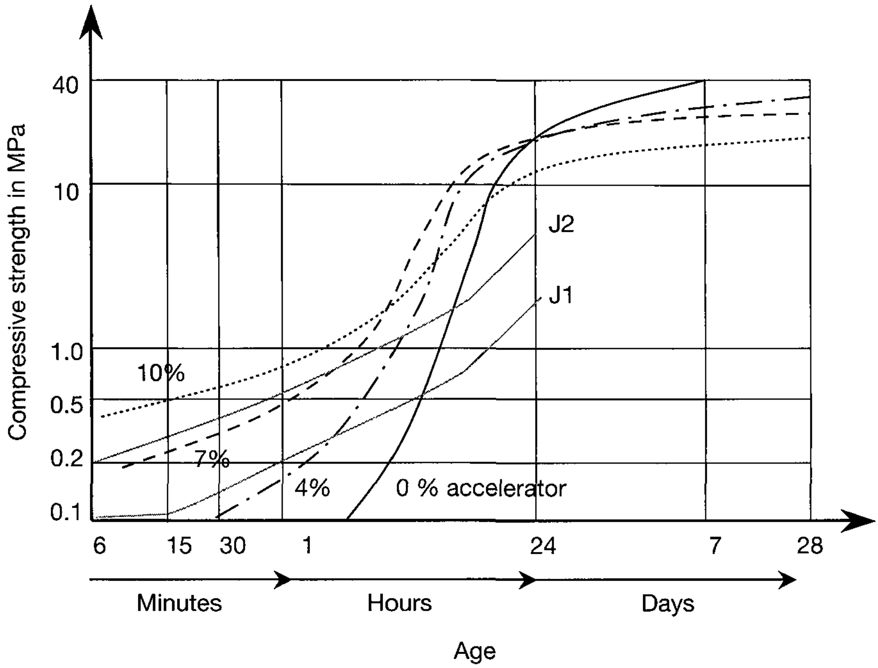


Figure 2.2 Early-age strength gain depending on dosage of accelerator with ÖBV J-curves for minimum strength (after Kusterle 1992)

Each component represents a large subject in itself and more information on most of them can be found in standard textbooks on concrete technology (e.g. Byfors 1980 and Neville 1995). Where relevant, their influence on the mechanical properties of sprayed concrete will be briefly discussed in Section 2.2. More details on the constituents of sprayed concrete can be found in the International Tunnelling Association's state-of-the-art review (Malmberg 1993) and other texts (e.g. ACI 506R (1990), Melbye 2005, Austin and Robins 1995, Brite Euram 1998, Brooks 1999).

2.1.1 Cement

For wet mix sprayed concrete Ordinary Portland Cement (OPC) is normally used. The cement does not have to have any special properties. The accelerator is used to speed up the hydration so a 'rapid hardening cement' is not needed. However, the chemistry of the cement is important. If the percentage of the fast-setting component, tricalcium aluminate, is unusually low, the cement may react too slowly for use in sprayed concrete.

Dry mix sprayed concrete also normally uses OPC. However, to reduce the need for accelerators, new types of cement – so-called 'spray cements' –

have been developed for use with the dry mix process (Testor 1997, Lukas *et al.* 1998). If gypsum (hydrated calcium sulphate) is removed from cement, the speed of the hydration reaction increases dramatically. Normally the gypsum reacts to form a film of calcium sulfoaluminate (ettringite) on the surface of the tricalcium aluminate in cement particles otherwise the tricalcium aluminate is free to react immediately and form hydrated calcium aluminate directly (Neville 1995, Atzwanger 1999). The reaction is so rapid that most of these new cements can only be used with oven-dried aggregate; otherwise hydration may occur in the delivery hoses. No accelerator is required. The latest 'spray cements' can also be used with naturally moist aggregates. While costs are reduced by not having to use accelerators, extra costs are incurred in the preparation and storage of the cement and aggregate.

2.1.2 Water

Ordinary water is used for sprayed concrete. As in conventional concrete, the water/cement ratio has a large influence on the strength of the concrete. In the wet mix process, the water is added during batched concrete as in normal concrete. In the dry mix process, the water is added in the tunnel during spraying. The wet and dry mix processes are described in more detail in Section 3.4.

2.1.3 Sand and aggregate

Sand and aggregate forms the bulk of sprayed concrete. The normal rules for concrete govern the choice of rock for the aggregate. The pumping and spraying process places onerous demands on the mix. A smooth grading curve is essential and rounded aggregates are preferred to angular particles. As noted above there tend to be more fine particles in a sprayed concrete mix. Contamination of the mix with over-sized stones (i.e. larger than about 10 mm in diameter) is a common cause of blockages during spraying. Blockages can cause costly delays and wastage of concrete. Therefore, careful design of the mix and control of the batching is advisable.

Moisture in the aggregate contributes to the water in the mix and this affects properties such as the strength. Sometimes special measures are needed to control the moisture content.

2.1.4 Accelerators

The accelerators were one of the main problems with sprayed concrete in the early days. Although they could achieve high early strengths, the chemicals – such as those based on aluminates – were very caustic and so posed a danger to the workers. Sometimes also the products of hydration were

Table 2.2 Acceptable setting times for accelerated cements (Melbye 2005)

	Caustic accelerators	*Alkali-free accelerators*
Initial set	< 60 seconds	< 300 seconds
Final set	< 240 seconds	< 600 seconds

unstable and the strength of the sprayed concrete actually decreased over time, notably when using waterglass (modified sodium silicate).

Modern accelerators do not have these problems. They are normally based on combinations of aluminium salts (sulphates, hydroxides and hydroxy-sulphates) (DiNoia and Sandberg 2004). The modern accelerators are classed as 'non-caustic' so they are safer to use. They are also 'alkali-free' – equivalent Na_2O content < 1.0% – which reduces the risk of an alkali-silica reaction in the concrete. In wet mix, the accelerator is added in liquid form at the nozzle during spraying. Dry mix uses the same approach, but the accelerator can also be added as a fixed dosage in powder form when using pre-bagged mixes. The only drawback of the modern accelerators is that they do not act as fast as the old caustic ones (see Table 2.2).

Some products on the market today are gelling agents rather than chemicals which accelerate the hydration process. The two products should not be confused. Although a gelling agent will help the concrete to adhere to the substrate, thick layers cannot be sprayed using a gelling agent because it is insufficiently strong to hold the self-weight of the concrete. This also means that until the concrete starts to hydrate it will not be able to carry any load from the ground.

Since this is a specialist field it is best to consult with accelerator manufacturers on how best to use their products. Laboratory and/or field tests are needed to check the performance of the accelerators.

2.1.5 Admixtures

To meet the conflicting demands of the design strengths (both short- and long-term), a long pot life, and ease of pumpability and sprayability, a cocktail of admixtures is added.

Plasticisers (lignosulphates) and superplasticisers (naphthalenes/melamines or modified polycarboxylic esters) increase workability without increasing the water/cement ratio (except for the water contained in the plasticiser itself).

Retarders (also known as 'stabilisers' or 'hydration control agents') delay hydration to extend the pot life of the concrete while activators remove the inhibiting effects of retarders. Some manufacturers claim that their products can extend the pot life of a wet mix from the normal 1.5 hours to as much as 72 hours (Melbye 2005). In practice, if high doses of retarders are used, it can be difficult to reactivate the concrete and achieve acceptable early-age strengths.

The interaction of the admixtures depends on the exact mix recipe and sometimes combinations can produce unexpectedly adverse results. For example, Niederegger and Thomaseth (2006) discuss the effect of certain plasticisers causing excessive stickiness, which in turn can lead to variable early-age strengths. Since this is a specialist field it is best to consult with admixture manufacturers on how best to use their products. Laboratory and/or field tests are needed to check the performance of the admixtures.

2.1.6 Microsilica

Microsilica has many benefits and its usage is discussed in Sections 2.2.1 and 2.2.9.

2.1.7 Mix design

The subject of mix design for concretes is discussed in detail in Neville (1995). The only difference for sprayed concrete is the addition of early-age strength criteria. That is not to say that the process of selecting a mix to meet the design and construction criteria is an easy one. Minor variations in one component can have a large impact on the performance of the mix overall. So, although in practice engineers often rely on an empirical approach, using tried and tested mix designs, it is naïve to believe that a recipe which worked on one project will be as good on another project, using different cement, aggregate or other ingredients. The development of a mix design is guided by the results of laboratory and field tests.

2.2 Material properties and behaviour

Considering sprayed concrete as a construction material, we could start by asking a series of basic questions:

- How strong is sprayed concrete?
- Is it brittle or ductile?
- Do its properties or behaviour change with time?
- Do its properties change with pressure, temperature or other environmental conditions?

The following sections will try to answer these questions and more. As an introduction, Table 2.3 contains the properties of a sprayed concrete and an equivalent strength concrete mix described in Table 2.1. Although the properties of this sprayed concrete are at the higher end of the typical values for sprayed concrete, there is a trend towards using such higher quality sprayed concrete as the norm on major construction projects (Brooks 1999).

Table 2.3 Typical properties of sprayed and cast concrete

Property	*High quality sprayed concrete*	*Cast* in situ *concrete*
Compressive strength @ 1 day in MPa	20	6 (est.)
Compressive strength @ 28 days in MPa	59	44
Elastic modulus @ 28 days in GPa	34	31 (est.)
Poisson's ratio, v, @ 28 days	0.48–0.18[a]	0.15–0.22
Tensile strength @ 28 days in MPa	> 2 (est.)[b]	3.8 (est.)
Initial setting time (start–end) in mins	3–5[c]	45–145 (est.)
Shrinkage after 100 days in %	0.1–0.12	0.03–0.08
Specific creep after 160 days in %/MPa	0.01–0.06	0.008
Density kg/m^3	2140–2235	2200–2600
Total porosity in %	15–20[d]	15–19
Permeability in m/s	2.0×10^{-12} to 10^{-14}	10^{-11} to 10^{-12}
Microcracking @ 28 days in cracks/m	1300	–
Coefficient of thermal expansion in –/K	$8.25–15 \times 10^{-6}$ [e]	10×10^{-6} [f]
Slump in mm	200 (est.)[g]	50

Notes:
a Kuwajima 1999.
b Kuwajima 1999.
c see Table 2.2.
d Blasen 1998 and Lukas *et al.* 1998.
e Kuwajima 1999 and Pöttler 1990.
f Eurocode 2 (2004) Cl. 3.1.3.
g Melbye 2005.

Figure 2.2 shows the strength of sprayed concrete at ages less than one day in comparison to an unaccelerated mix.

2.2.1 *Strength in compression*

Theories and mechanisms

Strength is often the first parameter that an engineer examines when considering a new material. As with all materials, the strength of concrete is governed as much by the flaws and imperfections within the material, as by the intrinsic strengths of the main components and their interaction. In the case of concrete the main components are the hydrated cement paste and the aggregate.

Typical compressive strengths of the hydrated cement paste (in the form of very dense cement paste compacts) can be up to 300 to 500 MPa, while the compressive strength of rocks commonly used for aggregate lies between 130 and 280 MPa (Neville 1995).

The imperfections are voids or pores, microcracks and macrocracks (both due to shrinkage and loading). The total porosity of concrete typically ranges

Table 2.4 Composition of porosity

Pore type	*Pore diameter*	*Mix type*	*% of total volume*
Gel	< 0.1 μm	Dry and wet	3–4 est.[a]
Capillary	0.1–10 μm	– Wet Dry and wet	15–19[a] 13–17[b] 17.5[c]
Entrained air pores and accidental voids	> 10 μm and 0.001–0.1 m	Wet Dry and wet	0.9–4.5[a] 3.7[b]
Total porosity			Wet 17–22[a] Dry 18–20[a] Dry and wet 21.1[b]

Notes:
a Kusterle 1992.
b Cornejo-Malm 1995.
c Blasen 1998 (average values from 337 samples).

between 15 and 20% of the volume (see Table 2.4). The porosity comprises gel pores (between the individual crystals and particles of gel; 10^{-7} to 10^{-9} m in size) and capillary pores (10^{-4} to 10^{-7} m in size), which remain after hydration and are partially occupied by excess water and air pores (10^{-2} to 10^{-4} m in size), which may be either intentional (entrained air pores) or accidental (due to poor compaction). The porosity of sprayed concrete tends to lie at the higher end of the range for concretes (Kusterle 1992, Lukas *et al.* 1998, Blasen 1998, Oberdörfer 1996) – see Table 2.4 – with the highest porosities generally in wet mix sprayed concrete. Considering a wet mix sprayed concrete, a dry mix sprayed concrete (spray cement with moist aggregate) and a normal cast concrete, all with the same water/cement ratio of 0.55, Blasen (1998) found that porosity of the wet mix was 16% greater than the cast concrete, while the porosity of the dry mix was only 8.7% higher. Consequently, wet mix sprayed concretes may tend to achieve lower strengths than comparable dry mixes.

Failure of concrete in compression is governed by cracking under uniaxial or biaxial compression and by crushing under multi-axial stress (Neville 1995, Chen 1982). Existing microcracks due to hydration and drying shrinkage start to grow when the load exceeds about 30% of the maximum compressive strength of mature concrete (Feenstra and de Borst 1993). These microcracks are mainly located at the interface between the aggregate and hardened cement paste. As the size of the microcracks increases, the effective area resisting the applied load decreases and so the stress rises locally faster than the nominal load stress (Neville 1995). This leads to strain hardening and the curved shape of the stress-strain graph for concrete in compression (see Figure 2.3) – i.e. the tangent modulus decreases with increasing strain. Clearly, the higher the initial level of porosity in the concrete, the higher the initial

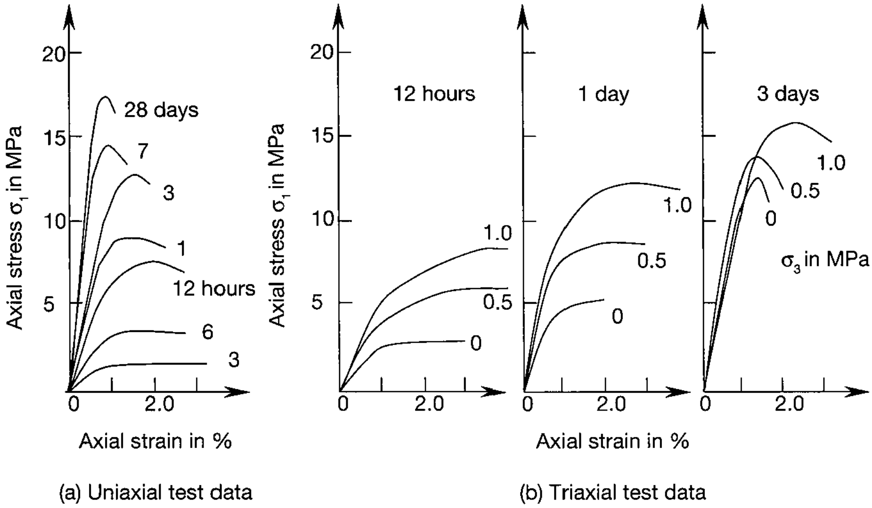


Figure 2.3 Stress-strain curves for sprayed concrete at difference ages (after Aydan *et al.* 1992a)

local stresses will be. Above an applied stress of about 70% of the maximum compressive strength, cracking occurs within the paste and the microcracks start to join up (Rokahr and Lux 1987). After the maximum compressive strength has been reached, macrocracks form as the microcracks localise in narrow bands and the load that the concrete can sustain decreases (Feenstra and de Borst 1993).

Under triaxial compressive stresses, the cracking may be suppressed by the lateral stresses and, if the confinement is high, the mode of failure is crushing. Hence, the maximum compressive stress under triaxial compressive loading is much higher than the uniaxial or biaxial strength (Chen 1982, Neville 1995). However, in the case of a tunnel lining, the stress state is largely biaxial since the radial stresses in the lining are much lower than the tangential and longitudinal stresses (Meschke 1996). In compression, the biaxial strength is only 16% greater than the uniaxial strength, when $\sigma_2/\sigma_1 = 1.0$, and 25% greater, when $\sigma_2/\sigma_1 = 0.5$ (Chen 1982) – see Figure 2.4. In intermediate states of stress between pure compression and pure tension, the presence of a tensile stress reduces the maximum compressive stress attainable (Chen 1982). In the biaxial stress case, it is often assumed that the maximum compressive stress reduces linearly from the uniaxial value (when the tensile stress is zero) to zero (when the tensile stress equals the maximum uniaxial tensile stress) – see Figure 2.4.

To summarise, the strength of concrete depends on one hand on the strength of the main components – the hardened cement paste and aggregate – and on the other hand on the density of the sample. Strength rises with

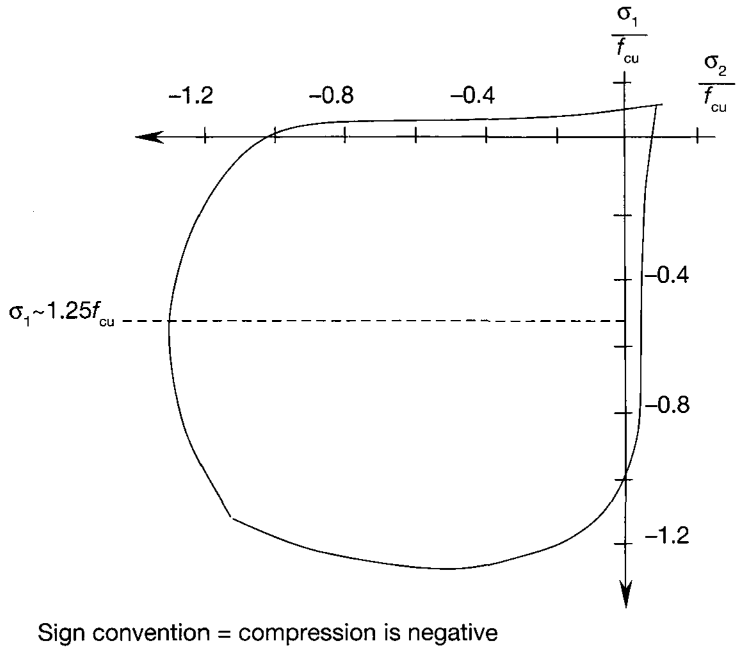


Figure 2.4 Normalised biaxial strength envelope for plain concrete from experimental data (after Chen 1982)

age since the quantity of hardened cement paste increases with age as the hydration process continues and the quantity of voids decreases, rather than any actual change in the mechanical properties of the microscopic constituents (Ulm and Coussy 1995). On a microscopic level, the local stress depends on the effective area of solid material sustaining the stress (if one ignores any contribution of pore water pressure), and the growth of cracks then depends of the strength of the bond between the hardened cement paste and aggregate compared to these local stresses. From this simplified theory of how concrete behaves under compressive loading, one would conclude that, to improve the strength of a concrete, one should improve the density of the material, by maximising hydration and minimising porosity, and one should also improve the hardened cement paste – aggregate interaction.

Influences on behaviour

Modern specifications typically require compressive strengths of 20 MPa (for temporary sprayed concrete) to 40 MPa or higher (for permanent sprayed concrete) at 28 days (Brooks 1999). However, the sprayed concrete must also possess sufficient adhesion to adhere to the ground and to support load, from the ground as well as other sources, such as blasting, soon after it has been sprayed. Hence, in contrast with conventional concrete, the sprayed concrete mix must be designed to attain a relatively high early compressive strength (see Figure 2.2) as well as meeting the long-term criteria. Furthermore, the mix must meet more stringent workability and pumpability criteria than conventional concrete. Of these competing criteria, traditionally the early-age strength (which determines the thickness of layers that can be formed and the safety of the tunnel heading) and the pumpability requirements have dominated, at the expense of longer-term strength (Kusterle 1992, Darby and Leggett 1997).

ACCELERATORS

Accelerating the hydration reaction will increase the strength of the sprayed concrete at early ages (see Figure 2.2). Traditionally, high early strengths have been achieved by adding accelerators to the mix in the spraying nozzle. This has several disadvantages. Firstly, accelerating the hydration reaction causes more, smaller hydrated calcium-silicate crystals to grow. A slower reaction permits larger crystals to grow, resulting in higher strengths in the long term (Fischnaller 1992, Atzwanger 1999).[1] Secondly, many of the early accelerators were very alkaline and hazardous to the health of workers in the tunnel. Some accelerators, such as waterglass (sodium silicate), not only led to low strengths at 28 days but due to their instability, the strength actually decreased with age (Kusterle 1992). The concerns over low long-term strengths and health and safety have forced the introduction of new accelerators – so-called 'alkali-free' or 'low-alkali' accelerators (Brooks 1999). With these new products and other new additives, the compressive strength gain of sprayed concrete can be controlled with a fair degree of accuracy and tailored to suit the particular requirements of the project. Together with new additives such as microsilica, the competing demands of high early strength and long-term strength can be met more satisfactorily (see Table 2.3).

CEMENT

Ordinary Portland Cement is normally used. 'Spray cements' can be used in the dry mix process (see Section 2.1.1).

CEMENT REPLACEMENT

Pulverised fly ash (PFA) and Ground granulated blast furnace slag (GGBFS) are added to the sprayed concrete mix as cement replacements in the normal manner, though GGBFS cannot be used in the same quantities as in conventional concrete. Because of its particles' angular shape GGBFS can only be used to replace up to 35% of the cement (Brite Euram 1998). Above this level there are problems pumping the mix. Since these materials react more slowly than cement, their beneficial contributions to durability characteristics, density and strength are only seen over the longer term (i.e. at ages greater than 28 days).

WATER/CEMENT RATIO

The lower the water/cement ratio, the higher the strength because fewer voids are left after hydration. Complete hydration of cement requires a water/cement ratio of approximately 0.23. However, pumpability requirements dictate that higher water/cement ratios are used for wet mix sprayed concrete than for cast concrete. In the dry mix process, the water/cement ratio is controlled by the nozzleman. Typically average values are 0.3 to 0.55; the ratio for wet mixes lies in the range of 0.4 to 0.65 (ITA 1993).

GRADING CURVE AND AGGREGATE

The maximum aggregate diameter is usually limited to about 10 to 12 mm, compared to around 20 mm for cast concrete. Strength increases with increasing maximum diameter of aggregate but the larger the pieces of aggregate, the more of them are lost in rebound (Kusterle 1992, Brite Euram 1998, Austin *et al.* 1998). As a whole, the grading curve for sprayed concrete is biased towards the finer end (see Figure 2.1) for ease of pumping (Norris 1999). Both crushed and round gravel can be used as aggregate. Some experimental evidence suggests that the type of gravel causes little difference in the quality of the sprayed concrete (Springenschmid *et al.* 1998). However, anecdotal evidence from various sites suggests that the grading curve and sometimes the type of aggregate too may have a great influence on the sprayed concrete. Aggregate that has a smooth grading curve should be used and angular particles should be avoided since they are more difficult to pump. If necessary, either the sand or the aggregate may be angular but one of the two should be rounded.

MICROSILICA

The addition of microsilica has two main advantages. Firstly, it improves the adhesion of the sprayed concrete, permitting accelerator dosages to be reduced or thicker layers of sprayed concrete to be placed. The higher adhesion reduces dust and rebound (Brite Euram 1998). Secondly, acting as a very reactive pozzolanic pore-filler, microsilica improves the long-term density, which is beneficial for strength and durability. In general, microsilica improves the quality of the sprayed concrete, improving durability as well as mechanical properties (Kusterle 1992, Norris 1999). The main disadvantage is its high water demand, which requires more plasticiser or water or both (Norris 1999, Brooks 1999). In fact, it has been suggested that, in the case of dry mix sprayed concrete, this additional water may be partially responsible for the reduction in dust and rebound (Austin *et al.* 1998).

FIBRES

Contradictory evidence exists as to whether the addition of steel fibres alters the compressive strength of sprayed concrete. Vandewalle (1996) suggests that they have little beneficial effect, while Brite Euram (1998) suggests that steel fibres increase the compressive strength by 10 to 35%. Polypropylene fibres were also found to enhance strength but they also increase the water demand so that there is little overall benefit (Brite Euram 1998).

OTHER ADDITIVES AND ADMIXTURES

Individually, plasticisers, stabilisers and other additives may not have a detrimental impact on the mechanical properties of sprayed concrete but one must always be aware that combinations of accelerators and additives may produce unfavourable results, such as significantly reduced strengths (Brite Euram 1998). Compatibility testing before work begins on site is used to identify such unfavourable combinations.

ANISOTROPY

Concrete is not naturally anisotropic and the anisotropy seen in sprayed concrete is a consequence of the way in which it is produced. Compressive strengths have been found to be 10 to 25% higher in the plane perpendicular to the direction of spraying (Cornejo-Malm 1995, Huber 1991, Fischnaller 1992). However, others have reported no variation in strength with direction of testing (Purrer 1990, Brite Euram 1998). At first sight, higher strengths perpendicular to the direction of spraying may seem paradoxical since the spraying jet is the sole means of compaction for sprayed concrete. This 'softer response' may be due to compaction at the less dense interfaces between layers of sprayed concrete (Aldrian 1991). The strength is normally tested in the direction of spraying, since the samples are usually cored from sprayed test panels or the lining itself, whereas the major compressive stresses are in the plane perpendicular to this (Golser and Kienberger 1997, Probst 1999). Hence the use of strength values from cores could be considered as conservative. Steel fibre-reinforced sprayed concrete exhibits pronounced anisotropy in its behaviour under both compression and tension (see Section 2.2.2). Normally anisotropy of the sprayed concrete (and indeed the stiffening effect of the layers of mesh or fibres) is ignored.

TEMPERATURE

Cervera *et al.* (1999a) proposed a reduction factor for the ultimate compressive peak stress, to account for the effects of (constant) elevated ambient temperatures during curing. The reduction factor is $k^{iso} = [(100 - T^{iso})/(100 - 20)]^{nT}$, where $nT = 0.25$ to 0.4 and T^{iso} is the (constant) temperature

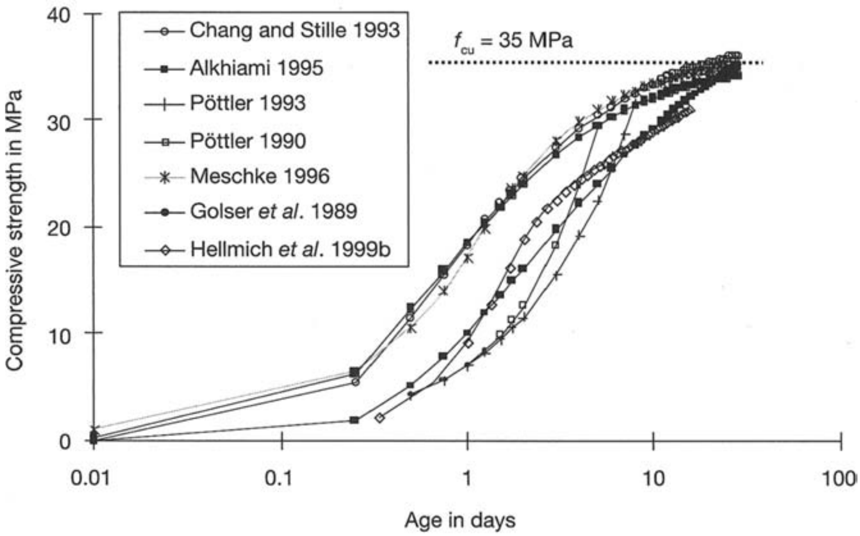


Figure 2.5 Predictions of strength development vs age

during hydration. This gives comparable reductions to those found experimentally by Seith (1995), e.g. 25% reduction in strength for curing at 60°C compared to curing at 16°C.

In conclusion, if properly produced, sprayed concrete can achieve high early strengths and long-term strengths (see Table 2.3). The exact shape of the strength gain curve will depend on the sprayed concrete mix and additives. Because of the interest in early-age strength gain, several authors have proposed equations that can be used to relate the compressive strength to age (e.g. Aldrian 1991, Chang 1994, Alkhiami 1995, Yin 1996 (after Weber 1979), Pöttler 1990, Meschke 1996) – see Figure 2.5 and Appendix A. Other more complex approaches have been developed to include ageing in numerical analyses (see Section 5).

2.2.2 *Strength in tension*

This section covers the tensile strength of both plain and reinforced sprayed concrete.

Theories and mechanisms

Even more so than in the case of compression, when under tension, cracking governs the behaviour. Up to 60% or more of the maximum uniaxial tensile stress few new microcracks are created and so the behaviour is linearly elastic (Chen 1982). The period of stable crack propagation under tension is shorter than compression. At about 75% of the maximum uniaxial tensile stress unstable crack propagation begins and a few cracks grow rapidly until failure occurs. The exact cause of tensile rupture is unknown but it is believed to originate in flaws in the hardened cement paste itself and at the paste/aggregate interface, rather than in the voids and pores, although these features contribute to the formation of stress concentrations (Neville 1995).

Normally the tensile strength of concrete is ignored in design because it is low – typically about one-tenth of the compressive strength – and because of the brittle nature of the failure once the maximum is reached. To counteract this, tensile reinforcement is added to concrete. Reinforcement in sprayed concrete tunnel linings is normally by steel mesh or steel fibres, although experiments have been performed with other materials, such as polypropylene fibres (Brite Euram 1998). When reinforced concrete is loaded with a tensile stress, cracking occurs in the concrete as before. However, the bond between the uncracked concrete and the steel bars permits a gradual transfer of the tensile load from the cracking concrete to the steel as the load increases (see Figure 2.6). The reinforced concrete continues to act as a composite and hence it has a stiffer response to loading than the reinforcement or concrete alone. This phenomenon is known as 'tension stiffening' (Feenstra and de Borst 1993).

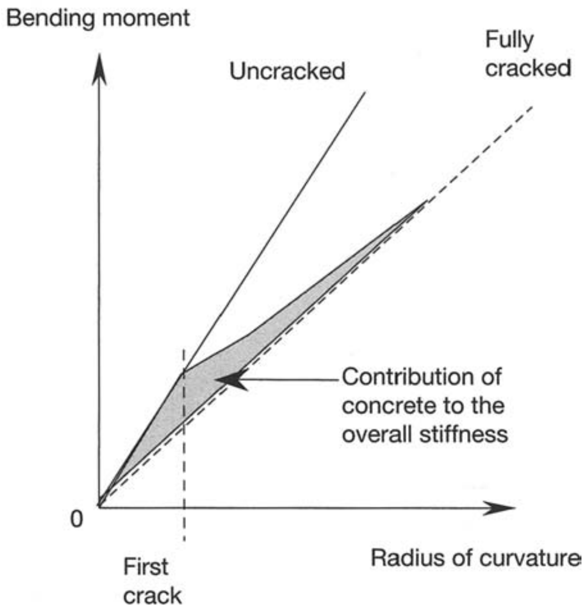


Figure 2.6 Tension stiffening of reinforced concrete (after Feenstra and de Borst 1993)

Table 2.5 Typical properties of structural fibres for sprayed concrete

	Steel	*Structural synthetic*
Density (kg/m^3)	7800	910
Fibre length	30–50	40–60
Aspect ratio	30–65	50–90
No. of fibres per kg	14,500	35,000
Elastic modulus (kN/mm^2)	200	5.0–9.5
Tensile strength (N/mm^2)	1100	500–550

Fibre reinforcement has a similar effect, although the interaction between the fibres and the matrix is more complex. The fibres bridge the opening cracks, thereby continuing to carry tensile forces across the cracks. The fibres are usually deformed in some manner to improve their resistance to being pulled out of the concrete as a crack opens. High grade steel is used for the fibres (typically yield strengths around 1000 MPa) so that failure occurs by means of a 'ductile' process in which individual fibres are pulled out of the concrete. If lower strengths of steel are used, the individual fibres would snap and the overall failure process would be a brittle one. Even with high strength steels there remains the risk that as the strength increases with age, the mode of failure becomes more brittle. The typical dosage for steel fibres ranges from 20 to 60 kg/m^3. In practical terms fibres can only be used with wet mix sprayed concrete. The difficulties in mixing fibres in dry mix lead to excessive rebound.

The first non-metallic fibres to be widely used in sprayed concrete were polypropylene fibres. These fibres did not have a meaningful beneficial effect on tensile behaviour except at an early age when they resist shrinkage (see Sections 2.2.6 and 6.8.9). A new generation of structural plastic fibres emerged during the 1990s, with mechanical properties that rival the performance of SFRS (e.g. see Hauck *et al.* 2004, DiNoia and Rieder 2004, Denney and Hagan 2004, ITA 2006). Pioneered in the mining industry these fibres have similar dimensions to steel fibres but they are lighter, softer and less strong. Given the difference in density, a dosage of 5 to 10 kg/m^3 is equivalent to 30 to 40 kg/m^3 of steel fibres (Melbye 2005). Experiments have also been conducted recently with textile glass fibre meshes made from alkali resistant glass. At present these meshes can only be used in gunite with a maximum size of aggregate of less than 2 mm due to the close spacing of the threads in the mesh (Schorn 2004).

The use of non-metallic fibres removes the residual concern over durability but does introduce a new source of worry – namely, creep. Sprayed concrete with synthetic fibres has a higher creep coefficient, possibly as high as twice that of steel fibre reinforced sprayed concrete (Bernard 2004a, MacKay and Trottier 2004).

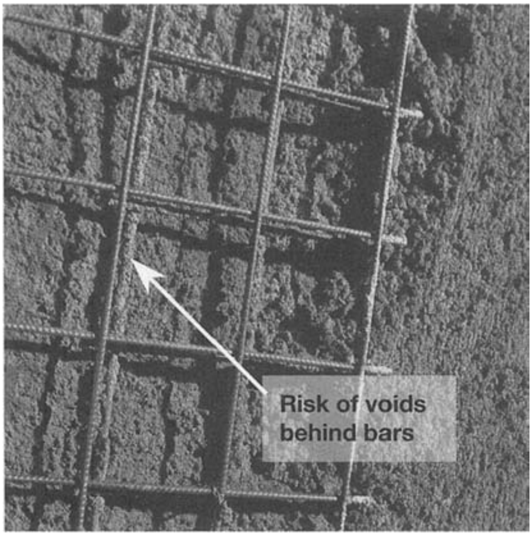


Figure 2.7 Shadowing

Influences on behaviour

The tensile strength of sprayed concrete is subject to the same influences and can be improved in the same ways as the compressive strength (see Section 2.2.1). A spin-off of this is that the tensile strength of concrete can be reliably estimated from the compressive strength. Various empirical formulae have been proposed (see Appendix A). It is generally assumed that tensile strength increases with age at the same rate as compressive strength.

Sprayed concrete tunnel linings are often formed by spraying several layers of concrete. Mesh or bar reinforcement is placed on the surface of the last layer sprayed and encased in concrete by the next layer (see Figure 2.7). Because the sprayed concrete must be sprayed through the mesh, complete encasement is difficult to achieve (Podjadtke 1998). Sprayed concrete rebounds off the bars and 'shadows' are left behind the individual bars (see Figure 2.7). Not only does this reduce the bonded length of the mesh but it also provides an ideal location for corrosion of the steel to occur, if water permeates through the lining.

Steel fibre reinforced shotcrete (SFRS) has many advantages (Brite Euram D1 1997, Vanderwalle *et al.* 1998):

- SFRS can behave in an almost elasticperfectly plastic manner (Norris and Powell 1999), withstanding very large post-yield strains;
- they can be included in the sprayed mix, reducing the cycle time and improving safety, since there is no mesh to be fixed at the face;

- fibres are more effective in controlling shrinkage cracking than typical mesh or bar reinforcement;
- corrosion of the fibres is not generally thought to be a significant problem (Nordstrom 2001) and there are no problems of shadowing.

These qualities make SFRS popular for support in tunnels in blocky ground. They are also desirable in tunnels in high stress environments, where large deformations are expected and SFRS can be used in conjunction with rock-bolts or in tunnels with permanent sprayed concrete linings, which are acting mainly in compression (Annett *et al.* 1997, Rose 1999).

Although the fibres orientate themselves mainly in the plane perpendicular to the direction of spraying (e.g. Cornejo-Malm 1995, Norris and Powell 1999), the moment capacity of SFRS is quite small, at typical fibre dosages. If a large moment capacity is required, bars or mesh reinforcement are needed. This is usually assumed to be the case in soft ground tunnels, especially at junctions. Hence, SFRS alone is rarely used in soft ground tunnelling. If one could be certain that the tensile stresses due to bending were well below the yield strength for SFRS, this material could be used more widely, with potentially large cost savings (Norris 1999). The German Concrete Society's guidelines on SFRS include a design method in which the strength from flexural tests can be converted into a permissible tensile strength (DBV 1992).

Because the fibres orientate themselves in the plane perpendicular to the direction of spraying, when tested in compression in this plane, SFRS exhibits a stiffer response pre-peak, higher peak stresses and a softer post-peak response, compared to tests performed in the same direction as spraying (Brite Euram 1998). The axial and lateral strains at ultimate stress are lower for the same reason.

2.2.3 Strength in other modes of loading

The input parameters for concrete models, such as Drucker–Prager or Mohr–Coulomb plasticity models, are generally derived from the compressive and tensile strengths. Unlike soils, the shear strength of sprayed concrete is not normally tested directly. However, the shear strength may be critical to the performance of the sprayed concrete lining (Barrett and McCreath 1995, Kusterle 1992), particularly if the lining thickness is very small (NCA 1993). Information on the shear strength of sprayed concrete bonded to various rocks is contained in Figure 2.8 (see also NCA (1993) for information on bond strengths to rock).

Similarly, the bond strength of sprayed concrete (both to the substrate and between successive layers of sprayed concrete) is important to the performance of sprayed concrete. In rock tunnels, when considering the lining, acted upon by a single wedge, the failure of sprayed concrete linings has been found to occur in two stages – usually by debonding, followed by

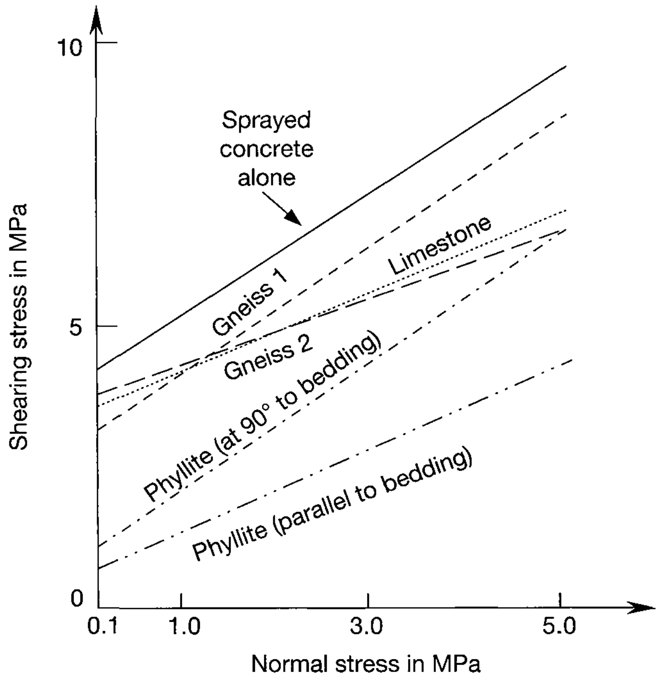


Figure 2.8 Bond strength in shear to various substrates (after Kusterle 1992)

failure in flexure (Barrett and McCreath 1995, NCA 1993). Table 2.6 contains typical values for the peak strengths of sprayed concrete in other loading modes. The bond strength between layers of sprayed concrete has been examined in the context of permanent sprayed concrete linings, in which the final layer may be added months after the first (Kusterle 1992,

Table 2.6 Strength in other modes of loading (after Barrett and McCreath 1995)[a]

Strength (in MPa)	*8 hours*	*1 day*	*7 days*	*28 days*
'Poor' bond strength	–	–	–	0.5
'Good' bond strength	–	–	–	1.0
Direct shear strength	1.0	2.0	6.0	8.0
Flexural strength	–	–	4.0	6.0
Diagonal tensile strength	0.75	1.0	1.75	2.0
Uniaxial compressive strength	5.0	10.0	30.0	40.0

Note:
a All sprayed concrete strengths are for an unreinforced mix with silica fume added.

Brite Euram 1998). Typical values of bond strength between layers of sprayed concrete range between 0.8 and 2.6 MPa (Brite Euram 1998), although Clements *et al.* (2004) reported values for bond of sprayed concrete to rock ranging from 2.83 to 11.3 MPa at 150 days. Jolin *et al.* (2004) report much larger values of more than 20 MPa for bond strengths to reinforcing bars at an age of 28 days. Provided that the substrate has been cleaned well, acceptable bond strengths can be achieved. Having said that, the failure is generally at the contact with the rock or in the rock, rather than in the concrete itself (Clements *et al.* 2004). Ansell (2004) reported results from Malmgren and Svensson that showed how the adhesion increases with age from starting values of between 0.125 to 0.35 MPa (when sprayed) to between 1.0 and 1.4 MPa (at 28 days).

In soft ground the tunnel lining is subjected to a more even loading than in rock tunnels so the bond strength may not be an important parameter for soft ground tunnels.

2.2.4 Stress–strain relationship in compression

Behaviour and influences

The mechanisms behind the stress-strain behaviour of concrete in compression have been described in Section 2.2.1. A stress-strain curve for a uniaxial test typically shows a linear elastic response up to the limit of proportionality, followed by what becomes an increasingly softer response as the maximum compressive strength is approached (see Figure 2.9). After reaching a peak value, the stress that can be sustained falls with increasing strain until the ultimate compressive strain is reached and the sample fails completely. In fact the onset of failure may occur before the peak stress, since the maximum volumetric strain is reached at a stress of between 0.85 and 0.95 of the peak and after this point dilation starts (Brite Euram C2 1997). The observed shape of the post-peak descending branch of the stress-strain curve depends heavily on the confinement and the boundary conditions imposed by the experimental equipment, due to the localisation of cracking (Choi *et al.* 1996, Swoboda *et al.* 1993). For that reason, one could describe concrete as being a 'near-brittle' material and ignore the post-peak region, concentrating rather on the pre-peak region. Generalised mathematical relationships have been developed for this region (e.g. Eurocode 2 (2004), BS8110 Part 2 (1985)) that agree well with a large range of uniaxial, biaxial and triaxial data, including tests on sprayed concrete (Brite Euram C2 1997).

The stress-strain behaviour of concrete under multi-axial stress states is very complex. While the increase in compressive strength has been clearly established, it is more difficult to form a definitive picture of the strain behaviour since it depends heavily on the boundary conditions in the experiments (Chen 1982). That said, increasing the confining pressure appears

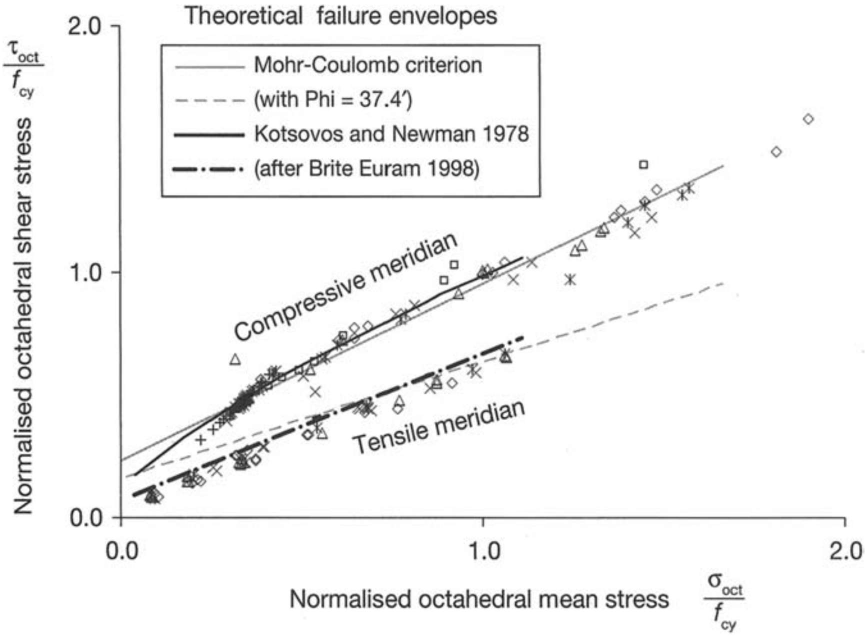


Figure 2.9 Normalised octahedral stress envelope for sprayed concrete (with published data from Aydan *et al.* 1992a, Brite Euram 1998 and Probst 1999)

to lead to more ductile behaviour (Michelis 1987, Aydan *et al.* 1992a) – see Figure 2.9. Triaxial behaviour will not be discussed further since this stress state in a tunnel lining is basically biaxial. The effect of tension in mixed biaxial loading is to reduce the peak (and failure) principal compressive and tensile strains (Chen 1982). The maximum strength envelope under biaxial loading can be considered to be independent of the stress path (Chen 1982).

In most cases the stress level in a tunnel lining is relatively low. Considering a typical tunnel in soft ground, where the principal stresses in the lining might be 5.0, 5.0 and 0.5 MPa and the 28 day strength is 25 MPa, the normalised octahedral mean stress (σ_{oct}/f_{cyl}) is only 0.14. Hence one can ignore those effects, which occur at moderate to high stress levels, such as the curved nature of the yield surface meridians (see Figure 2.9).

Particular points of interest to the designer are the initial elastic modulus, the limit of proportionality (i.e. limit of elastic range), the peak stress and strain. The behaviour post-peak and at high stress/strength ratios (e.g. >0.85) will not be discussed further here on the grounds that structures are not normally designed to operate in this region.

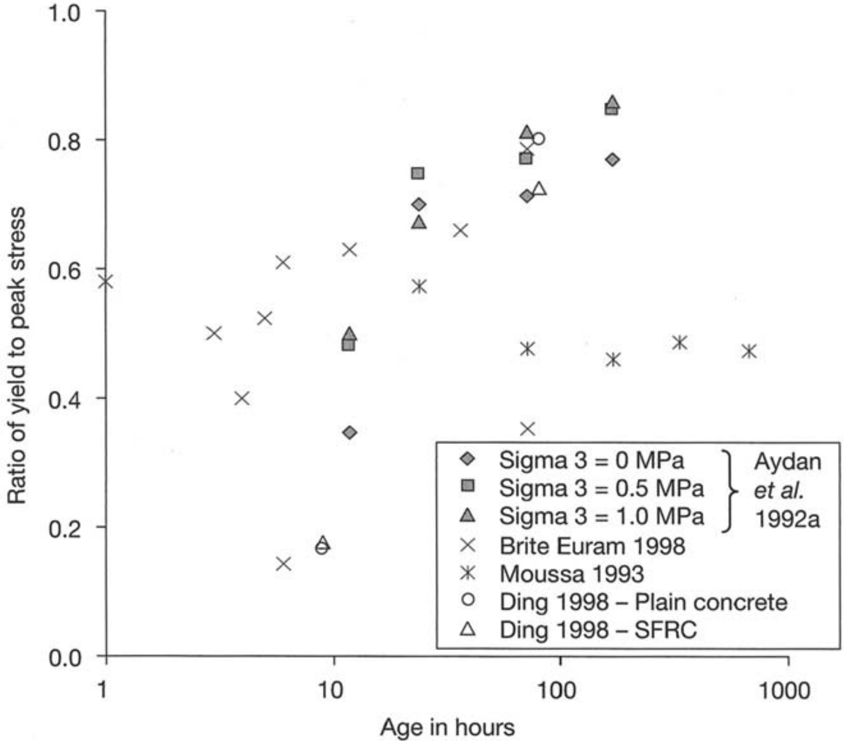


Figure 2.10 Yield stress/peak stress ratio (published data, including data from triaxial tests by Aydan *et al.* 1992a at various confining pressures – sigma 3)

Elastic region

ELASTIC LIMIT

The behaviour of sprayed concrete at early age, in compression tests, has been characterised as viscous (from 0 to 1 or 2 hours old), viscoelastic (1 to 11 hours) and elastoplastic (from 11 hours onwards) (Brite Euram 1998). This behaviour may vary depending on the level of loading. Figure 2.10 shows how the ratio of yield stress to peak stress for sprayed concrete (estimated visually from stress-strain curves or from published data from (Aydan *et al.* 1992a)) varies with age. Some data suggest that the yield point is relatively high – 0.70 to 0.85 of the peak stress (Aydan *et al.* 1992a). Other data suggest much lower ratios, tending towards the generally accepted yield ratio for mature concrete of 0.3 to 0.4 (Chen 1982, Feenstra and de Borst 1993). If one examines tests that included unloading/reloading cycles (e.g. Moussa 1993, Probst 1999), one can see that all the strain is not recovered upon unloading even at low stresses. This supports the view that the elastic limit is low.

ELASTIC MODULUS

Considerable data exists for the elastic modulus (calculated from uniaxial compression tests) and how it varies with age (e.g. Chang 1994, Kuwajima 1999). The modulus grows rapidly with age in a similar way to compressive strength, although it appears to grow at a faster rate (Byfors 1980, Chang 1994). Various formulae have been proposed to relate the elastic modulus to age (see Figure 2.11 and Appendix A). Other more complex approaches have been developed to include ageing in numerical analyses (see Section 5). Sprayed concrete may exhibit anisotropy, with the elastic modulus in the plane perpendicular to the direction of spraying being higher than in the plane parallel to direction of spraying. Celestino *et al.* (1999) reports that it is 40% higher while Cornejo-Malm (1995) recorded an increase of 10% (see also Section 2.2.1). Such anisotropy is generally ignored in design.

Ansell (2004) reported that Nagy proposed that the dynamic elastic modulus is related to the static modulus by the following equation:

$$E_{dyn} = E(1 + \eta^{0.35}) \tag{2.1}$$

where $\eta = 0.14$ for very young concrete and decreases to 0.05 at an age of 2 days.

Since concrete behaves linearly elastic up to a limit of about 0.4 times the uniaxial compressive strength, the elastic modulus should be determined from loading within this range only.

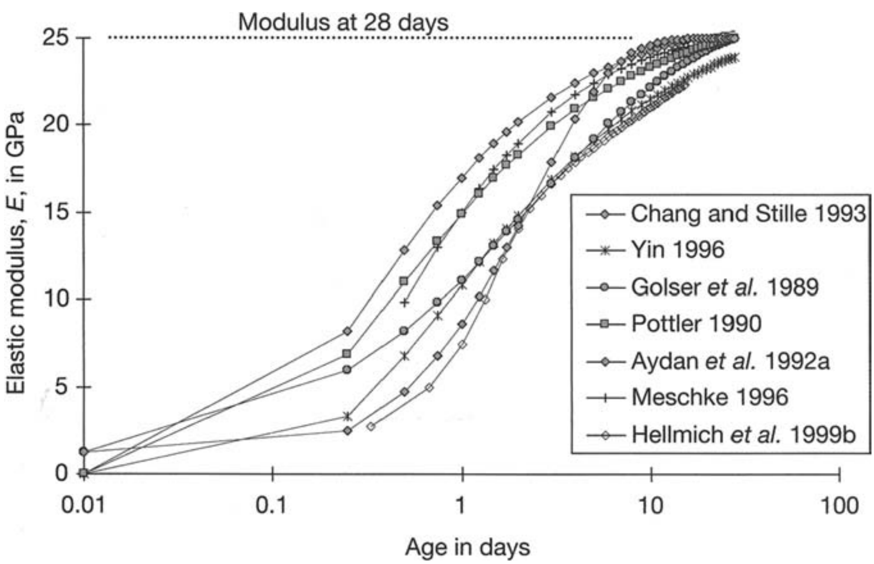


Figure 2.11 Predictions of the development of elastic modulus with age

POISSON'S RATIO

Within the elastic range and up to 80% of the maximum stress, Poisson's ratio remains constant for mature concrete, ranging between 0.15 and 0.22 and with an average of about 0.2 (Chen 1982). The actual value of the Poisson's ratio depends mainly on the type of aggregate, with lower values in concrete with lightweight aggregate (Neville 1995). Mature sprayed concrete exhibits the same behaviour but there is some evidence that the Poisson's ratio varies with age. Kuwajima (1999) measured the dynamic Poisson's ratio using ultrasound and found that it decreased with age from close to 0.5 to about 0.28 (see Figure 2.12). Dynamic Poisson's ratio values are usually higher than static ones (Neville 1995). Aydan *et al.* (1992a) and Aydan *et al.* (1992b) report a similar variation with age. They measured values of Poisson's ratio close to 0.45 initially falling to about 0.2 at 12 hours but they do not state how these values were obtained (see Appendix A for the equation relating Poisson's ratio to age). Plane strain compression tests from the Brite Euram project (Brite Euram 1998) suggest that the Poisson's ratio at early ages (3 to 16 hours) is closer to the mature values.

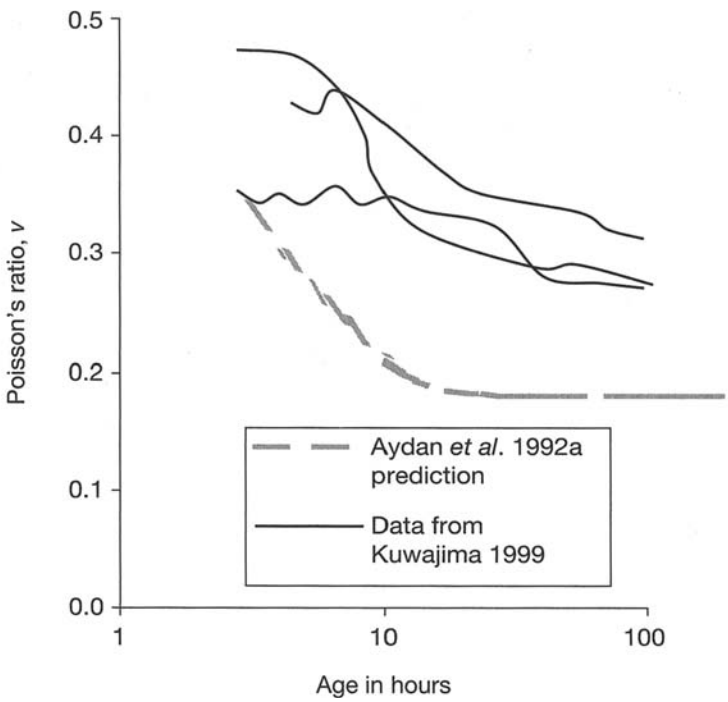


Figure 2.12 Variation of Poisson's ratio with age

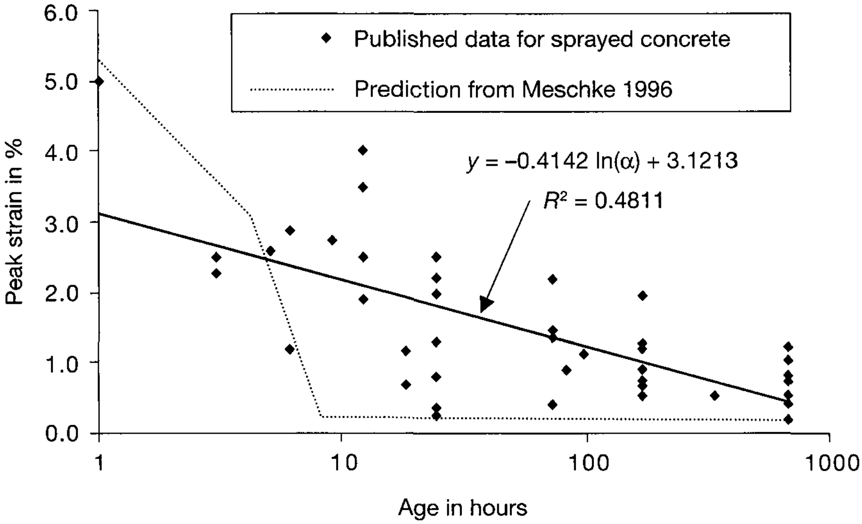


Figure 2.13 Peak compressive strain vs age

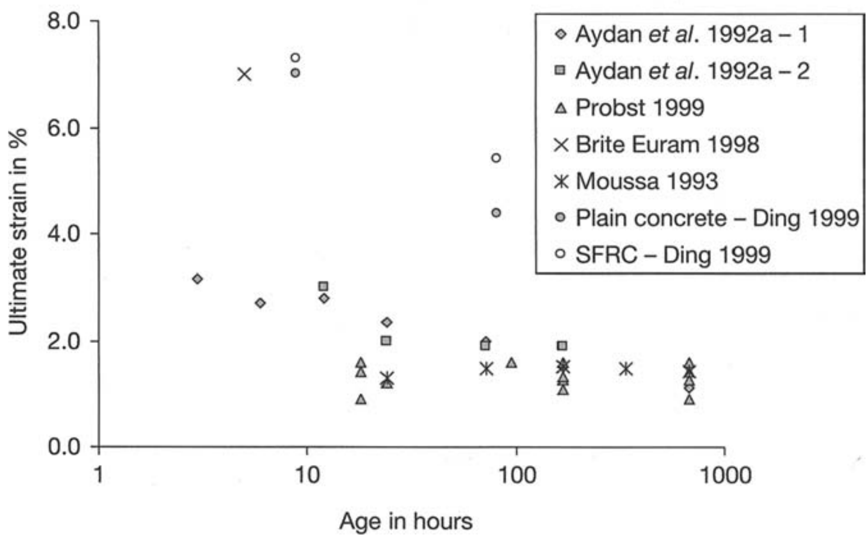


Figure 2.14 Ultimate compressive strain vs age

Plastic region up to peak stress

Sprayed concrete can withstand very large plastic strains at an early age. The strain at peak stress decreases with increasing age (see Figure 2.13), from as high as 5.0% at 1 hour old to a relatively constant value of 1.0%, from 100 hours onwards. The peak strain of mature concrete is normally assumed to be about 0.3% in uniaxial and biaxial loading (Chen 1982, BS8110 Part 1 (1997)). The ultimate strain (at failure) also decreases with age (see Figure 2.14) and the behaviour becomes more brittle (see Figure 2.3) (Swoboda *et al.* 1993). Swoboda and Moussa (1994) observed a similar trend, when plotting a graph of maximum strain against the logarithm of the compressive strength of the sprayed concrete rather than its age. It is believed that the deformation behaviour of mature sprayed concrete is not much affected by changes in mix constituents (Brite Euram 1998). For example, the normalised stress at maximum volumetric strain does not change with variation in accelerator dosage and the strain at peak stress is also independent of accelerator dosage (Swoboda and Moussa 1992).

The addition of steel fibres appears to make the sprayed concrete more ductile in compression, with a strain at peak stress of about 0.42% (28 days), compared to 0.20% for plain sprayed concrete (Brite Euram 1998).

Unloading

In uniaxial compression tests, the unloading (and reloading) modulus is stiffer than the initial loading modulus (Michelis 1987, Probst 1999) – see Figure 2.15. Probst (1999) suggests multiplying the current value of (initial) modulus by a factor of 1.1 to 1.5 to account for this, while the average of results from Aldrian (1991) was 1.27. For loads up to 70% of the peak stress, Moussa (1993) found that, when reloaded after unloading, the stress-strain path would rejoin the original curve at the point where it had departed on unloading and continue as if the unloading had not occurred (as one would expect for concrete (Chen 1982)). Using a new type of testing rig, which does not require demoulding and therefore may reduce disturbance of the samples, Probst (1999) observed the same behaviour as Moussa in uniaxial tests, even up to 80% of peak stress.

Damage due to loading

The question of whether or not early loading damages the concrete is of great importance in the debate over the use of permanent sprayed concrete linings (see Section 4.2.5). Little research has been done on this aspect of sprayed concrete. Moussa (1993) concluded from experimental work that stresses below a utilisation factor, α, of 70% of the current peak stress had no detrimental effect on the later peak stress. He proposed a linear relationship between the reduction factor and the stress above this level:

$$R_d/f_c = 2.532(\sigma/f_{ct1} - 0.69) \quad (2.2)$$

which ranges from 0 at $\alpha = 0.69$ to 0.78 at $\alpha = 1.0$, where $\alpha = \sigma/f_{ct1}$ and σ = the stress applied and f_{ct1} = the uniaxial strength at the age of loading, $t1$. However, the experimental data were scattered (Swoboda *et al.* 1993). By comparing the strength of samples from creep tests with samples from parallel shrinkage tests, Huber (1991) found that the strength of loaded samples was 80% of the unloaded ones. The utilisation factors in the creep tests ranged from 20 to 70%. Chen (1982) suggested that for normal concrete unstable crack propagation occurs when utilisations exceed 75%.

2.2.5 Stress-strain relationship in tension

The mechanisms behind the stress-strain behaviour of unreinforced concrete in tension have been described already in Section 2.2.2. Experimental data on conventional concrete in tension are scarce, especially for concrete at early ages. A stress-strain curve for a uniaxial test on mature concrete typically shows a linear elastic response up to 60% of the maximum stress (Chen 1982). As more and more microcracking occurs, the response becomes softer until the maximum stress is reached. After the peak stress, the stress quickly drops to zero for unreinforced concrete. The precise

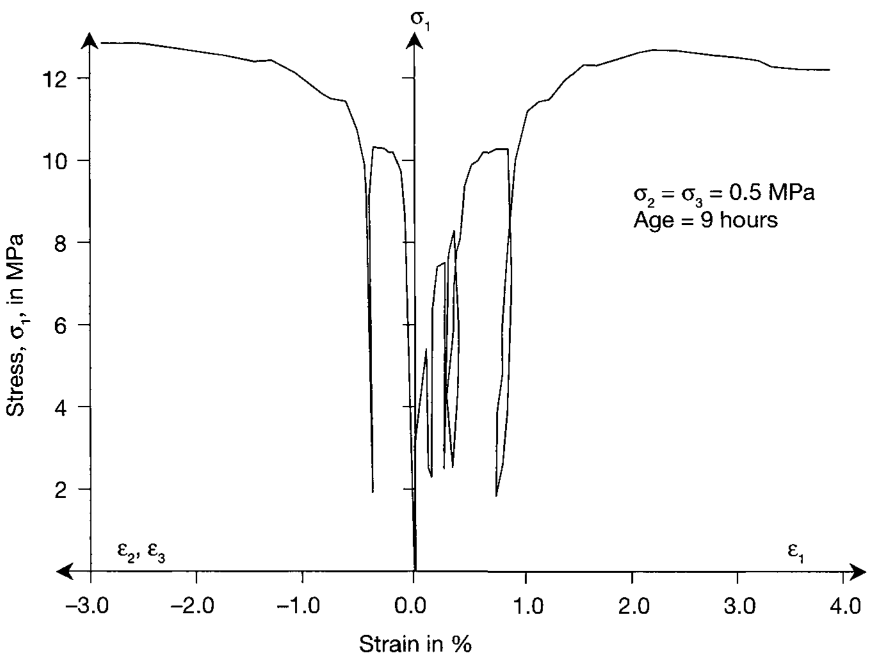


Figure 2.15 Compressive test on sprayed concrete (Brite Euram 1998)

nature of the descending branch of the stress-strain curve depends heavily on the arrangement of the testing rig (Hannant *et al.* 1999, Chen 1982). Reinforcement enables tensile forces to be carried even though the concrete has cracked, as discussed earlier (see Section 2.2.2 and Figure 2.6). The utilisation factor in parts of a tunnel lining under tensile stress is likely to be much higher than in areas of compressive stress because the tensile strength is much lower.

At very early ages (i.e. less than 4 hours old), cast concrete appears to behave plastically and can be strained by up to 0.5% or more (Hannant *et al.* 1999). However, this ultimate strain reduces sharply with increasing age and is about 0.05% at 5 hours.

Sprayed concrete exhibits the same behaviour (see Figure 2.16). In uniaxial tensile tests, plain sprayed concrete and fibre-reinforced sprayed concrete (SFRS) behave similarly (Brite Euram 1998) and, as for plain concrete, the ultimate strain reduces sharply in the first few hours. The effect of the steel fibres can be seen in Figure 2.16 as converting an otherwise brittle failure into a more ductile one, in which the stress-strain curve descends slowly from the peak. A similar effect is observed in flexural tests on SFRS beams. Taking the toughness index, I_{10}, values of 4 and 6 as indicative of 'fair' and 'good' performance, SFRS typically falls into the 'good'

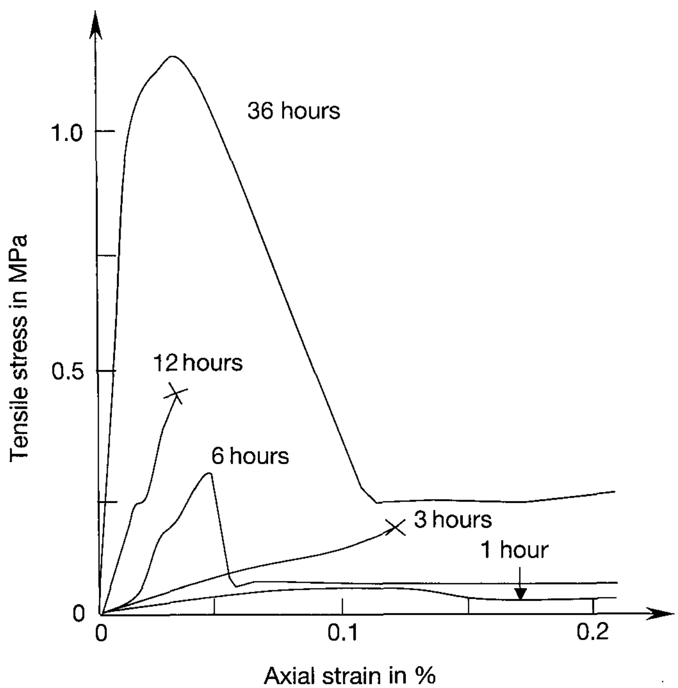


Figure 2.16 Uniaxial tensile tests on samples of mix IK013 at different ages (Brite Euram 1998)

category while polypropylene fibre-reinforced sprayed concrete falls into the 'fair' category (Brite Euram 1998).[2] However, new High Performance Polymers are being developed which can provide comparable performance to SFRS (Tatnall and Brooks 2001, and Melbye 2005) – see Section 2.2.2.

The elastic modulus under tension is also assumed to be equal to that under compression for unreinforced concrete. Poisson's ratio is the same in tension as in compression in the elastic region.

The variation in the material properties of sprayed concrete with age has already been discussed. The variation with material properties with time is covered under durability in Section 2.2.7. In the following sections, the variation of sprayed concrete's behaviour with time will be considered. This can be subdivided into two categories – stress-independent changes (due to shrinkage and temperature effects – Section 2.2.6) and stress-dependent changes (due to creep – Section 2.2.7).

2.2.6 Shrinkage and temperature effects

The following sections will cover the various forms of shrinkage (plastic, autogeneous, drying and carbonation shrinkage) and temperature effects that induce strains in sprayed concrete linings.

Shrinkage

PLASTIC SHRINKAGE

Plastic shrinkage is the contraction caused by the loss of water from the fresh concrete's surface due to evaporation or suction, by adjacent dry soil or existing concrete, while the concrete is still plastic (Neville 1995). If the water lost exceeds the volume brought to the surface by bleeding, surface cracks may appear. Plastic shrinkage increases with increasing evaporation, cement content and water/cement ratio and increases with a decreasing tendency for bleeding. A typical value for (linear) shrinkage after 24 hours is 0.2% (for 400 kg/m^3 cement, air temp = 20°C, relative humidity = 50%, air velocity of 1.0 m/s – Neville 1995).

PLASTIC SETTLEMENT

Plastic settlement also occurs in the first hours after casting and is sometimes confused with plastic shrinkage. Plastic settlement is caused by differential settlement of the concrete over obstructions such as large aggregate or reinforcement (Neville 1995). Plastic shrinkage is the early part of drying shrinkage (see below) that occurs while the concrete is still plastic. Two key factors influencing this are the degree of compaction and the rate of build-up of concrete. In the case of sprayed concrete, in the crown of the tunnel the sprayed concrete is loaded by its own self-weight from the moment it is sprayed. This represents some of the most extreme conditions for sprayed concrete. The degree of compaction is least in the crown due to vertical spraying. The bond to the substrate depends heavily on the preparation of that surface and the early-age strength gain of the sprayed concrete, which also controls the adhesion between subsequent layers of sprayed concrete. If one of these properties is inadequate or too thick a layer of sprayed concrete is sprayed, lumps of sprayed concrete will sag or simply fall out of the lining (sloughing). The presence of reinforcement will help to prevent this but this would imply that the sprayed concrete is hanging off the reinforcement – potentially leading to plastic settlement cracking.

AUTOGENEOUS SHRINKAGE

Autogeneous shrinkage occurs when there is no movement of water to or from the concrete. During hydration water is drawn from the capillary pores. This 'self-desiccation' causes the cement matrix to contract. Typical values of autogeneous shrinkage are 0.004% after 1 month, i.e. an order of magnitude smaller than plastic shrinkage (Neville 1995). The magnitude of autogeneous shrinkage is likely to be greater in sprayed concrete due to the faster rate of hydration and high cement content.

DRYING SHRINKAGE

Drying shrinkage occurs in the hardened cement paste as water is lost to the air.[3] First the water from the larger voids and capillary pores is lost and this causes no shrinkage. However, when the absorbed water in the hardened cement paste is removed, shrinkage occurs. The constituents of sprayed concrete and its curing mean that sprayed concrete is likely to shrink more than a similar strength cast *in situ* concrete.

Considering the constituents, drying shrinkage increases primarily with increasing cement content, decreasing quantity of aggregate and decreasing stiffness of the aggregate (Neville 1995). The reasons lie in the increased quantity of hardened cement paste and the decreased restraining effect of aggregate. Most natural aggregate itself does not shrink but shrinkage does vary considerably depending on which aggregates are used. The actual grading curve has little influence other than indirectly by altering the relative proportions of cement and aggregate (Powers 1959). Water/cement ratio has no direct influence but increasing the ratio reduces the proportion of aggregate. Cement type generally has little influence on shrinkage, though cements which have low gypsum contents tend to shrink more than normal. More accurately, for each cement there is an optimum gypsum content, which minimises shrinkage (Powers 1959). Low gypsum content also means a fast reaction, which produces a different gel structure and porosity. This would suggest that the fast reacting sprayed concrete mixes and especially those made with dry-mix 'spray cements' will produce sprayed concrete that exhibits high shrinkage and shrinkage cracking. The fineness of the cement also influences shrinkage, because increasing fineness reduces the number of larger particles that restrain shrinkage (Powers 1959). Silica fume, fly ash and ground granulated blast furnace slag are known to increase shrinkage and are all often used in sprayed concrete. The use of plasticisers and other water reducing admixtures implies a higher cement content in the mix and hence higher shrinkage, although the admixtures themselves are not believed to cause additional shrinkage.

Considering curing and the tunnel environment, one would expect that any measure that reduces moisture loss from the concrete would reduce drying shrinkage. In the extreme, concrete stored underwater actually swells rather than shrinking. Drying shrinkage increases considerably with decreasing relative humidity. Shrinkage at a relative humidity of 40% can be 3 times greater than at a relative humidity of 80%. However, concrete is subject to a series of competing influences. For example, prolonged moist curing reduces drying but also reduces the quantity of unhydrated cement available to restrain shrinkage (Neville 1995). Well cured concrete shrinks faster and, since it is more mature, the capacity for creep is much reduced. This reduces the ability to reduce the stresses due shrinkage. On the other hand, the more mature concrete is stronger.

The effect of ventilation depends on the rate at which moisture can move within the concrete. During the early stages, increased ventilation may increase shrinkage (Kuwajima 1999). At later ages, the rate of evaporation is much greater than the rate of movement of water in the concrete and so increased ventilation has much less effect. In the case of tunnels the movement of air stems from tunnel ventilation and may be highly localised in nature, since the forced ventilation is provided by means of ventilation ducts. Typically, relative humidity in a tunnel is around 50% (though it may be higher) and the temperature is fairly constant within the range from 12 to 24°C, depending on the time of year. The flow of air from ventilation ducts will dry out the sprayed concrete adjacent to them. Concrete with a temperature of 25°C (in air at 20°C and 50% RH), being dried by a current of air at 10 km/hr (which is 2.8 m/s), would lose around 0.5 kg of water per m^2 per hour (see Figure 2.17). If there was no flow of air, it would lose around 0.15 kg per m^2 per hour. Considering one can see that given the high local air velocities at the end of a vent duct (around 20 km/hr), the high temperature of the sprayed concrete during initial hydration (typically 30 to 45°C) and the large surface area, considerable volumes of water could be lost (adjacent to the vent duct) in the early stages of hydration (Oberdörfer 1996). However, the most vulnerable area, at the outlet of the duct, only represents a small proportion of the total surface of the lining.

Furthermore, experiments on shrinkage of sprayed concrete have yielded some contradictory results. Increased ventilation has been found to increase the rate of shrinkage but to reduce the total magnitude (Cornejo-Malm

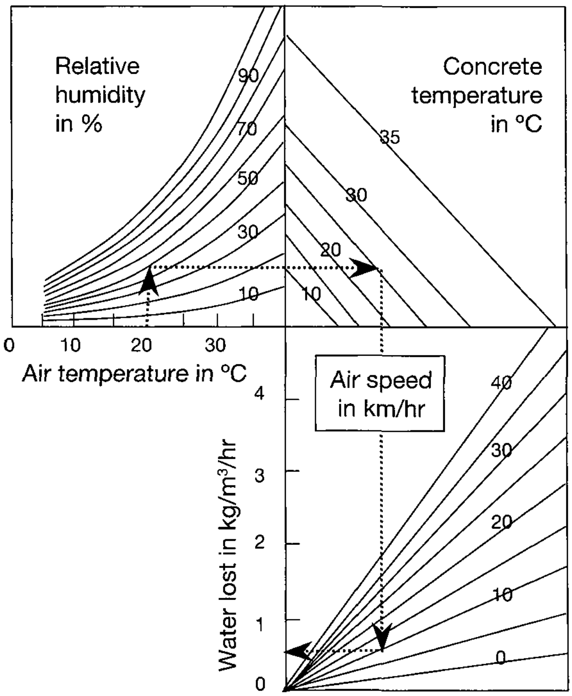


Figure 2.17 Water loss from concrete (after Oberdörfer 1996)

1995). The magnitude of the shrinkage will depend on the origin of the water that is being removed, i.e. 'free' water in capillary pores or 'absorbed' water in gel pores, which in turn depends on the original water/cement ratio, the degree of hydration and the porosity of the aggregate (Powers 1959).

Drying shrinkage continues to take place over years, albeit at a much reduced rate. Typically only 20 to 50% of the total shrinkage will have occurred within the first month and about 80% of the total within the first year (Neville 1995).

CURING

Curing – the prevention of water loss – is recognised to be important for proper hydration of concrete. While moist curing is often specified for a period of between 4 to 7 days after construction, it is difficult to achieve in tunnels. Covering with impermeable sheets or wet matting is usually deemed impractical in a tunnel during construction. Similarly, spraying with water is not preferred by contractors but arguably it is simple to do. Curing compounds can be applied to external faces or internally as special additives. Externally applied compounds have to be removed before additional layers of concrete are cast or sprayed to ensure that the bond is not impaired.

CARBONATION SHRINKAGE

Carbonation shrinkage occurs in the surface layers of concrete. Carbon dioxide from the air forms carbonic acid, which reacts with various hydrates in the hardened cement paste, notably calcium hydroxide. Hence, this shrinkage is irreversible. The rate of carbonation slows as the depth of carbonation increases, because the carbon dioxide has further to permeate and because the products of carbonation reduce the porosity of the concrete (Blasen 1998). Carbonation is greatest at moderate levels of relative humidity – i.e. 50 to 75% – since both a lack of water and saturation slow the process. Although the levels of carbon dioxide may be higher than normal in a tunnel due to construction traffic, the fact that carbonation occurs at the same time as drying is liable to reduce the overall contribution of carbonation to shrinkage, because the carbonation will be occurring while the relative humidity is quite high. Typical values for the depth of carbonation are 2 to 3 mm after 6 months (Oberdörfer 1996). Environmental factors may increase this. For example, in road tunnels there is a higher concentration of carbon dioxide in the air.

Temperature effects

Expansion and contraction due to temperature changes occur in tunnel linings during the first few days due to the heat of hydration and subsequent cooling. The thermal coefficient of expansion for cast *in situ* concrete is largely determined by the coefficients of expansion for the cement and aggregate and their proportions in the mix. Typical values for mature concrete range from 4 to 14 $\times 10^{-6}$ per °C (Neville 1995, ACI 209R 1992) and codes often assume an average value of 10 $\times 10^{-6}$ per °C (DIN 1045 1988, ACI 209R 1992). Similar values have been suggested for sprayed concrete (see Table 2.3) but the coefficient may vary with age. Laplante and Boulay (1994) reported values decreasing from about 21 $\times 10^{-6}$ per °C at 8.4 hours to 12 $\times 10^{-6}$ per °C at 16.4 hours. After that the coefficient remained constant.

Typical profiles of temperature in sprayed concrete linings can be found in Kusterle (1992), Fischnaller (1992) and Hellmich and Mang (1999). Figure 2.18 displays readings from pressure cell temperature transducers and shows how the temperature rises and then decays with time. As one would expect, the maximum rise in temperature depends heavily on the thickness of the sprayed concrete layer (see Table 2.7), the initial temperature of the mix and the rate of hydration. The peak rise in temperature occurs about 7 to 10 hours after spraying for dry mix sprayed concrete

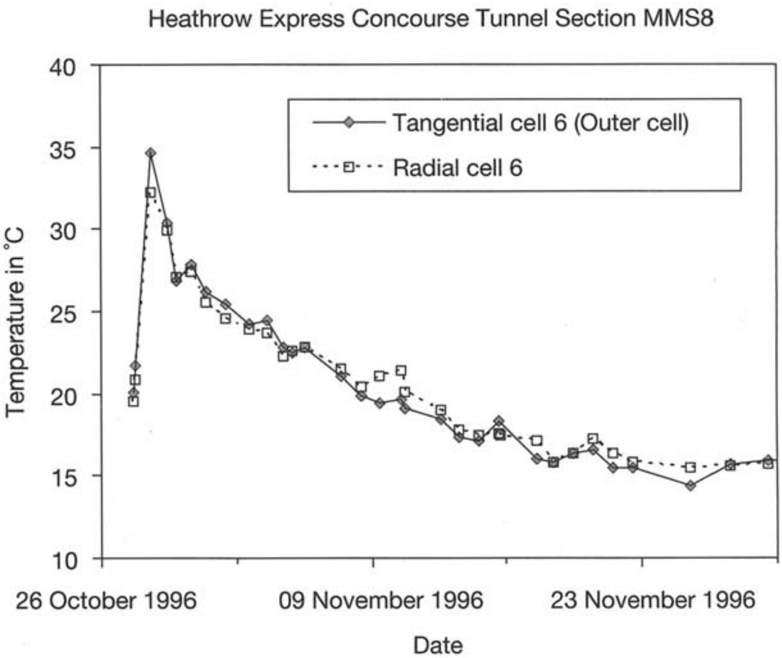


Figure 2.18 Temperature profile in a sprayed concrete lining

and slightly later, at 10 to 15 hours, for wet mix sprayed concrete (Cornejo-Malm 1995). Typically the maximum temperature lies between the centre of the lining and the extrados and ranges from between 28 to 45°C (i.e. 10 to 25°C above the ambient temperature). Fischnaller (1992) suggests that wet mix sprayed concrete produces higher temperature rises while Cornejo-Malm (1995), quoting lower figures, suggested that wet and dry mix produce similar temperature rises. Typically after 48 hours the maximum temperature rise (above ambient temperature) has fallen to less than 20 to 30% of the peak temperature rise (Kusterle 1992). Given the short-lived nature of the high temperatures, they are not believed to have a detrimental impact on the strength of the concrete itself (see also Section 2.2.1).

Table 2.7 Maximum temperature rises in sprayed concrete linings (Kusterle 1992)

Thickness of lining in mm	*Max. temperature rise in °C*
50–100	6–9
100–150	10–15
300	25

Assuming a thermal coefficient of 10×10^{-6} per °C and a rise of 20°C, the heat of hydration would induce a maximum compressive linear strain of 0.02% for a perfectly confined sample of concrete. As the concrete cools, it will contract by 0.02% over the following 48 hours. The initial tendency to expand tends not to induce much compressive stress (because the elastic modulus is still small and creep rates are high) whereas the contraction could induce significant tensile stresses in lightly loaded linings. Although shrinkage of a uniform ring would not induce tensile stresses, sprayed concrete linings are made up of a series of panels of different ages. Therefore there is the potential for differential shrinkage and partial restraint. That said, since the concrete in a lining is not fully restrained so the influence of shrinkage may be small.

Cracking due to shrinkage and temperature effects

The very early ages at which cracking is most likely to occur, the variation in conditions within the tunnel (e.g. ventilation duct in the crown, invert covered with excavated material), the heat of hydration and the possibility of water transfer from the ground all complicate the prediction of cracking due to shrinkage. Indeed it has been reported that water from the ground can actually lead to swelling in the invert and generally a reduction in shrinkage (Kuwajima 1999). Golser *et al.* (1989) reported that the shrinkage of the tunnel lining is greatest in the crown, 50% smaller at axis level and negligible in the invert. Creep of tensile stresses (due to shrinkage or bending moments) may lead to additional cracking (Negro *et al.* 1998), although the case study cited may not be representative of general conditions in tunnels. Although the temperature in the thin shell of a tunnel lining peaks and falls much more quickly than, for example, a base slab, which might take several weeks to return to ambient temperature (Eierle and Schikora 1999), the stiffness of the sprayed concrete rises much faster than normal concrete and so there is just as much risk of the stress induced by the contraction exceeding the tensile strength of the concrete. It has been suggested that the stiffness of concrete rises faster than strength (Eierle and Schikora 1999, Chang 1994). Typical values for shrinkage are 0.10 to 0.12% for wet mixes after 100 days and 0.06 to 0.08% for dry mixes after 180 days (Cornejo-Malm 1995). Given that the concrete is not fully restrained in a tunnel lining, it is the non-uniform nature of the volume change, rather than merely the magnitude of the shrinkage, which causes cracking.

According to some experimental evidence, the general restraint of shrinkage by reinforcement is quite small. The uniaxial shrinkage strain for fibre reinforced sprayed concrete (both polypropylene and steel fibres at low to moderate dosages) was only 8% less than that of ordinary sprayed concrete after 300 hours while 0.39% (by area) steel bar reinforcement reduced the shrinkage strain by 16% (Ding 1998). The addition of fibres may increase porosity (Chang 1994, Ding 1998, Brite Euram 1998), leading to higher shrinkage and creep at higher dosages (60 kg/m^3 or more) (Ding 1998).

2.2.7 *Creep*

Theories and mechanisms

Creep is defined as the increase in strain with time under a sustained stress and relaxation is the decrease in stress with time in a sample under constant strain (Neville *et al.* 1983). Relaxation is also sometimes referred to as creep and here the comments on creep can be taken to apply equally to relaxation unless otherwise stated. In discussions on creep, the term 'specific (or unit) creep' is often used. Specific creep is the creep strain per unit stress (typically in units of 10^{-6} –/MPa).

Creep can be divided into two components, depending on moisture movement. 'Basic creep' is the creep that occurs under conditions of no moisture movement to or from the sample (i.e. conditions of hygral equilibrium). 'Drying creep' is the additional creep, which occurs during drying of the sample. The total creep is the sum of these two components. Furthermore, creep components can be divided into reversible and irreversible parts (see Figure 2.19 and England and Illston 1965). On unloading, along with the instantaneous elastic recovery, there will be a gradual recovery of a portion of the creep. While this is relevant for conditions of varying stress, it can be ignored if unloading does not occur. In any case, experimental evidence for sprayed concrete suggests that reversible (viscoelastic) creep forms a very small percentage of the total strain (typically less than 10%) in samples that are loaded for a prolonged period, i.e. more than 7 days – see Figure 2.20 (Huber 1991, Abler 1992, Fischnaller 1992, Ding 1998, Probst 1999).

The mechanisms behind creep are not fully understood, although it is recognised that its origin lies within the cement paste. Shrinkage and creep are normally assumed to be independent and a simple superposition of strains is used. In reality they probably are not independent since both are related to movement of water within and from the concrete (Neville *et al.* 1983). In the case of drying creep, obviously the movement of water from the concrete plays a role and in practice it may be difficult to distinguish this from strain due to drying shrinkage.[4] In the case of basic creep, movement of water from the absorbed layers on the cement paste to internal voids may be a cause of the creep. The fact that creep increases with increasing porosity tends to support this theory (Neville 1995). However, the largely irreversible nature of creep would suggest that the viscous movement of gel particles and, to a lesser extent (at higher stresses), microcracking may also play a significant role. Like shrinkage, creep occurs over a prolonged period and for conventional concrete 60 to 70% of the final magnitude of creep strain occurs within the first year (Neville 1995).

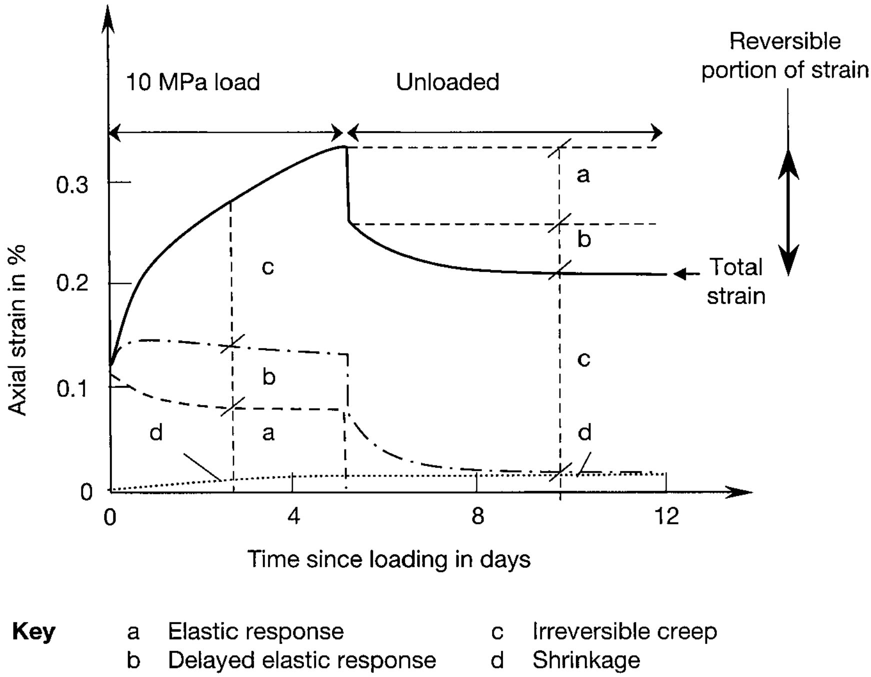


Figure 2.19 Decomposition of strains according to the Rate of Flow Method (after Golser *et al.* 1989)

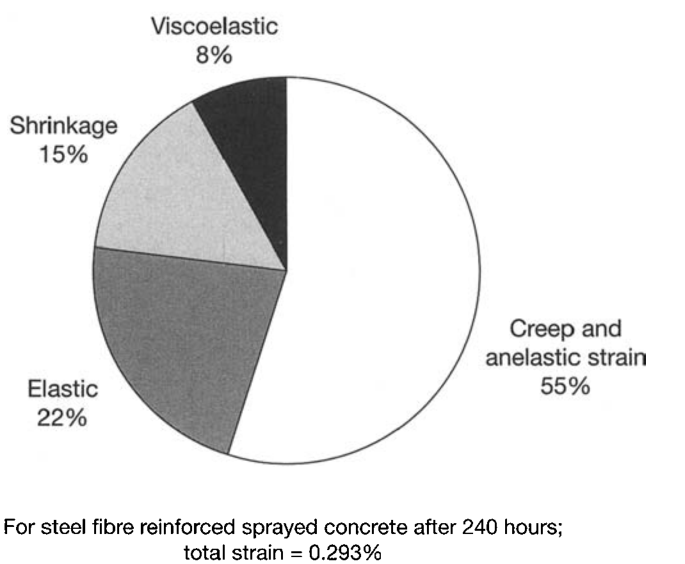


Figure 2.20 Composition of strains in a creep test (after Ding 1998)

Creep strains after one year are typically 2 to 3 times the magnitude of the elastic strain.[5]

It appears that creep of concrete under uniaxial tension may be 20 to 30% higher than in compression, but relatively little work exists on this subject and some of it is contradictory (Neville *et al.* 1983). No specific work on creep in tension of sprayed concrete has been found in the course of researching this book. Creep in tension reduces the risk of cracking due to uneven shrinkage. Creep in compression will reduce the compressive stresses induced by thermal expansion during hydration and therefore increase the risk of tensile stresses forming on cooling.

Lateral creep has the effect of increasing the apparent Poisson's ratio (creep Poisson's ratio) in uniaxial tests, as creep strain occurs in the direction of the lateral expansion, unless the stress is lower than half the strength (Neville 1995). At lower stresses the Poisson's ratio is the same as normal (i.e. about 0.20). However, experimental results from uniaxial compression tests on sprayed concrete vary considerably – $\upsilon = 0.11$ to 0.5 (Golser and Kienberger 1997, Rathmair 1997). Even so, one can at least say that the effect of creep is an overall decrease in volume. Under multi-axial stress, the apparent Poisson's ratio is normally lower – 0.09 to 0.17 and considerable creep will occur even under hydrostatic compression (Neville 1995). The simple superposition of the creep strains due to the stress in a given direction and the Poisson's ratio effect of the creep strains in the two other normal directions is unlikely to be valid (Neville 1995, Mosser 1993) but any errors may be small in the case of tunnel linings due to the predominately biaxial stress conditions.

Influences on behaviour

Creep of concrete and sprayed concrete alike increases with decreasing relative humidity (i.e. increasing drying), increasing cement content, increasing stress and decreasing strength (Fischnaller 1992, Huber 1991, Neville *et al.* 1983). The latter two explain why under a constant stress, applied at an early age, creep is greater for more slowly hydrating concretes (since the stress/strength ratio will be higher). However, if one considers concretes loaded with the same stress/strength ratio, the creep is lower for more slowly hydrating concretes (since the magnitude of the stress applied is lower). In the case of uniaxial compression, creep is proportional to the applied stress at low stresses (up to 40% of the uniaxial strength (Pöttler 1990, Huber 1991, Aldrian 1991)). Above this level it is believed to increase at an increasing rate. At very high stresses (>80% f_{cu} – Abler 1992), creep will lead eventually to failure (so-called 'tertiary' creep – Jaeger and Cook 1979).

Creep and creep rates are significantly higher at an early age of loading since the strength is lower. Byfors (1980) found that for plain concrete the creep strain of a sample loaded at 10 hours could be 50 times the creep strain when loaded at 28 days. This is of importance for SCL tunnels, since the lining is loaded from the moment it is formed. A sample loaded at an age of 8 days may creep by 25% more than a similar sample loaded at 28 days (Huber 1991). On the other hand, sprayed concrete exhibits a rapid development in strength so, after 24 or 48 hours, the creep behaviour is relatively close to that at greater ages (Kuwajima 1999).

Creep is also influenced by the aggregate in the mix, since it restrains the creep. Increasing the proportion of aggregate reduces the magnitude of creep but the effect is small for the ranges of proportion of aggregate in normal mixes. Creep decreases if stiffer aggregates are used.

Other mix parameters (such as water/cement ratio and cement type) appear to influence creep only insofar as they influence strength and its growth with time (Neville *et al.* 1983). Hence the types of cement or cement replacements used do not themselves appear to affect (basic) creep behaviour, but their affect on the rate of strength gain will influence creep. Cement replacements, which reduce the porosity (such as microsilica), may well reduce drying creep, since they will restrict water movement.

Considering other influences, the creep of concrete increases with temperature (ACI 209R 1992) but for the case of most tunnel linings, since the increase in temperature due to hydration is relatively small and short-lived, this effect can probably be ignored. This may not be the case in deep tunnels where the ambient temperature of the rock is relatively high. Creep decreases with increasing size of the specimen since this affects drying (ACI 209R 1992, Huber 1991). Some experimental evidence has shown that reinforcement (both bars and fibres) reduces creep (see Figure 2.21 and Ding 1998). Presumably this is due to its restraining effect. Typically, 20 kg/m^3 of steel fibres (0.21% steel by volume) and 0.39% bar reinforcement reduce the magnitude of creep by the same amount, roughly 25% after 180 hours compared to plain concrete (Ding 1998). Due to their distributed nature, the fibres have more effect than bar reinforcement. Notwithstanding this, steel fibre reinforced sprayed concrete can exhibit considerable creep potential, with creep coefficients of 3 to 6 after 1 year (depending on the degree of loading) – MacKay and Trottier (2004). Structural synthetic fibres have a higher creep capacity and the creep coefficient for sprayed concrete reinforced with them can be twice as high as that for steel fibre reinforced sprayed concrete (Bernard 2004a, MacKay and Trottier 2004).

2.2.8 Variation in properties with environmental conditions

The effects of relative humidity in tunnels on shrinkage and creep, and of temperature on strength gain and creep have already been discussed. Air pressure, production influences (including curing and spraying) and the loading on the lining (including exceptional events such as fires) are the other main environmental influences on material behaviour.

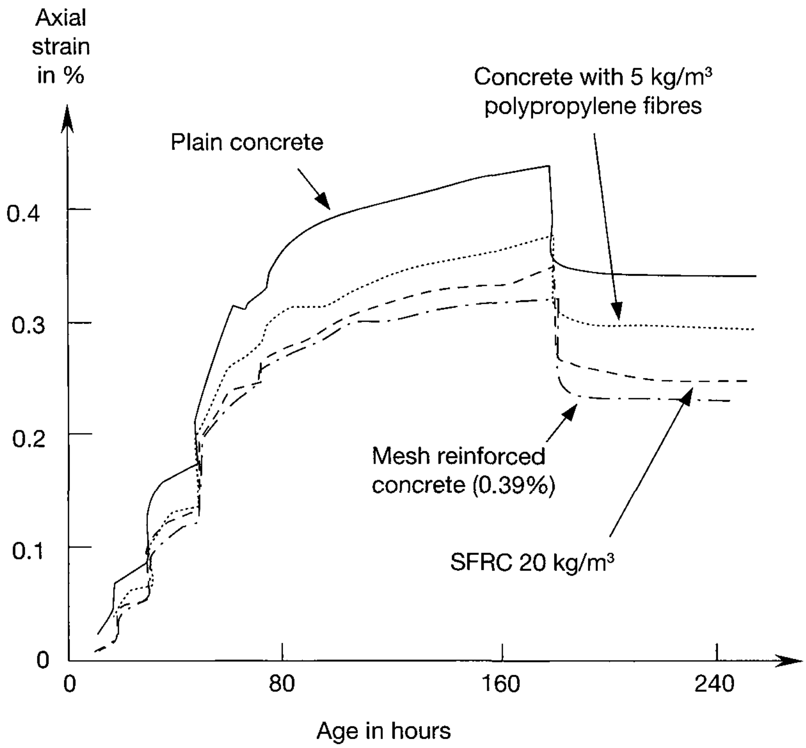


Figure 2.21 Creep test (after Ding 1998)

Due to several recent high profile tunnel fires (Bolton 1999, Bolton and Jones 1999), the fire resistance of tunnel linings has come under scrutiny. Sprayed concrete has in the past been used as fire protection in tunnels (Kompen 1990). However, the fire loading in a tunnel tends to be quite severe (Varley and Both 1999). Explosive spalling due to the build-up of moisture within the concrete or differential expansion in the concrete would be likely in most conventional sprayed concrete or segmental tunnel linings. Although less dense than the concrete used for segmental linings, sprayed concrete may have a higher coefficient of thermal expansion and so is unlikely to perform any better than conventional concrete. In the worst case the entire lining thickness could be destroyed. One countermeasure is the addition of polypropylene fibres to the concrete mix. During a fire, the polypropylene fibres melt and the resulting capillaries provide an escape path for moisture in the concrete, thus avoiding spalling. Winterberg and Dietze (2004) contains a good review of the state-of-the-art in passive fire protection for sprayed concrete.

Dynamic behaviour is rarely a concern. The seismic design of tunnels is discussed in Section 6.8.1 and the effects of blasting in Section 3.3.2. Since sprayed concrete tunnel linings are not generally subject to cyclic loading, fatigue is not of concern, though one may note that steel fibre reinforced concrete performs significantly better under cyclic loads than plain concrete (Vandewalle *et al.* 1998). Cyclic loading could be applied to a sprayed concrete lining, in a railway tunnel, where it forms the permanent lining and is in intimate contact with the trackbed or is subject to changes in air pressure due to the piston effect as trains pass by at high speed.

Sprayed concrete is rarely used in compressed air tunnelling. Some evidence exists to suggest that considerable quantities of air are lost through the sprayed concrete lining (Strobl 1991). Research has focused on measuring the air permeability of sprayed concrete linings, so that air losses and supply requirements, as well as surface settlements, can be estimated more accurately (Kammerer and Semprich 1999) and the numerical modelling of construction under compressed air (Hofstetter *et al.* 1999) – see also Section 6.8.6.

SCL tunnels can also be subjected to variable air pressures due to the piston effect as high speed trains pass through them. This typically imposes a load of ± 1 to 4 kPa. So long as there is sufficient adhesion of the sprayed concrete to the substrate there should not be any damage to the lining otherwise additional measures may be needed to pin the lining back onto the rock (Holmgren 2004).

2.2.9 Durability and construction defects

Durability

Where permanent sprayed concrete linings are used, a design life of 50 to 100 years or more is normally specified. Concerns over durability of sprayed concrete tunnel lining have prevented the more widespread use of the material in the permanent lining of tunnels (see also Section 4.2.4). The situation is not helped by the fact that the concept of durability is often poorly understood. The fundamental questions are:

- Will the sprayed concrete maintain its ability to carry the loads during its design life?
- Will the lining satisfy the desired watertightness during its life?

With this in mind, the specific concerns centre on the stability of the components of hydration, the susceptibility of steel reinforcement to corrosion (see Section 2.2.2) and possible damage to the structure of the concrete due to early loading (see Section 5.1). Depending on the waterproofing system, the intrinsic impermeability of the sprayed concrete may also be relevant. The following section will cover only those aspects of durability that are different in sprayed concrete. The durability of reinforced concrete in general – for example, under attack from sulphates or chlorides – is covered in standard texts.

Considering the first of these concerns, while it is widely believed that the latest additives and accelerators do not have any detrimental effects on the sprayed concrete over the long term, as far as the author is aware little detailed evidence has been produced to support this, either from petrographic examinations or accelerated ageing tests. Melbye (2005) reported that in tests the products of hydration of the accelerated sprayed concrete had been found to be the similar to a conventional (durable) concrete. Furthermore, both concretes contained similar patterns of microcracking. Although the sprayed concrete initially showed more microcracking – possibly due to thermal effects – this was reduced with time by autogeneous healing.

Steel fibres are often cited as the cure for corrosion concerns because, unlike bar reinforcement, there is no risk of shadowing. Nevertheless, the concrete does crack and the fibres bridging the crack are exposed. Corrosion of the fibres will not cause spalling but it does reduce the load capacity. Various researchers have examined cracked samples of steel and structural synthetic fibre reinforced sprayed concrete under different exposure conditions (e.g. Nordstrom 2001 and Bernard 2004b). The conclusion appears to be that so long as the cracks are narrow (i.e. < 0.1 mm) the reduction is small for steel fibres and autogeneous healing of the cracks may occur otherwise the loss in capacity can be significant. This is exacerbated by embrittlement. In this process as the strength of the concrete increases there is a tendency for the fibres to snap (brittle failure) rather than be pulled out (ductile failure) – Bernard (2004b). Synthetic fibres perform better and show little signs of deterioration (Bernard 2004b).

Regarding early-age loading, ÖBV (1998) cautions that if sprayed concrete is loaded to more than 80% of its strength progressive creep will occur and the concrete will be damaged. If the lining is designed as part of the permanent works, the safety factors will ensure that loading is well below this level.

Durability can be assessed by examining the permeability of the sprayed concrete (which, typically, must be less than 1×10^{-12} m/s (Watson *et al.* 1999)), oxygen and chloride diffusion, freeze-thaw resistance, resistance to sulphate attack and the progress of carbonation. Water permeability may also be assessed by means of a penetration test, according to the German standard DIN 1048 (1991). Penetration depths of less than 50 mm indicate good quality, 'impermeable' concrete.

Results from the extensive Brite Euram project suggest that average water permeabilities range from 0.5 to 4.5×10^{-12} m/s, oxygen diffusion coefficients range from 1.79 to 14.2×10^{-9} m/s and chloride diffusion coefficients range from 1.57 to 9.21×10^{-12} m/s. The overall assessment was that sprayed concrete could be produced with as good durability characteristics as a

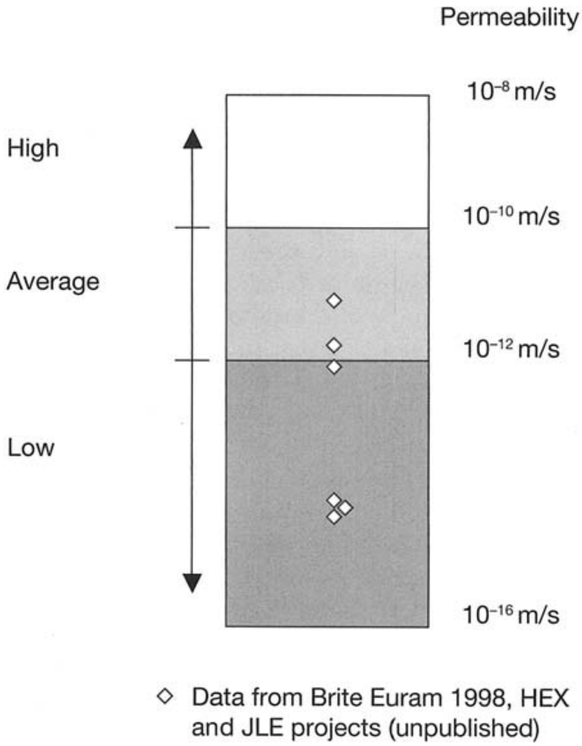


Figure 2.22 Permeabilities of sprayed concrete vs categories according to Concrete Society Technical Report 31 (1988)

similar conventionally cast concrete (Brite Euram 1998, Norris 1999) (see Figure 2.22). Water penetration depths are typically 14 to 25 mm (i.e. well within the 50-mm limit) (Röthlisberger 1996). Some experimental work has demonstrated the detrimental impact of non-alkali accelerators on the sulphate resistance of sprayed concrete (e.g. Spirig 2004) but, nonetheless, the levels of absorbed SO_3 at 100 days, which ranged from 0.69 to 2.10%, were well below the recommended limit of 3.00% (Lukas *et al.* 1998, Atzwanger 1999). Similarly, depths of carbonation have been found to be satisfactory – typically, 2 to 3 mm after 6 months (Oberdörfer 1996).

However, one may note that the test samples are often stored and cured in more favourable conditions than are present in tunnels. Curing measures are rarely implemented on site, because they would slow the advance of the tunnel. Consequently it is believed that the quality of the sprayed concrete suffers. Experimental evidence suggests that the detrimental impacts on the sprayed concrete of less favourable curing conditions and drying are limited (Hefti 1988, Cornejo-Malm 1995, Oberdörfer 1996, Bernard and Clements 2001). Therefore, the benefits of improving curing may not justify the additional cost and disruption. The addition of additives, such as microsilica, may offer a more cost-effective means of improving durability characteristics.

Construction defects

As already mentioned, sprayed concrete, and in particular dry-mix sprayed concrete, is susceptible to poor workmanship. This manifests itself as areas which are too thin or areas of low strength material. Common problems include the failure to clean the substrate, the inclusion of rebound,[6] voids, shadowing behind bars and an intermittent flow of shotcrete (leading to a film of pure accelerator being sprayed on the substrate). The manner in which it is sprayed as well as the quantities of accelerator and water added at the nozzle has a strong influence over the quality of the sprayed concrete. Sprayed concrete is therefore inherently more variable as a material than conventionally cast, ready-mixed concrete. Typically, the standard deviation in 28 day compressive strengths might be 5 MPa for a 35 MPa mix (Brite Euram 1998 – from field trials, Bonapace 1997). This would give a rating of Fair to Poor according to ACI 214–77 (Neville 1995). A similar cast *in situ* concrete typically might have a standard deviation of about 3.5 MPa, which rates as Very Good to Good (Neville 1995). There is evidence that the quality can be worse where the lining is more difficult to form, such as at joints (HSE 2000).

Notes

1 A more detailed discussion of the hydration of cement and its chemistry can be found in Neville (1995).
2 For explanations of the term I_{10} see the ITA report (ITA 1993) or Vandewalle (1996).
3 Plastic shrinkage (see above) is the early part of drying shrinkage while the concrete is still plastic.
4 Creep is time-dependent strain due to applied load; shrinkage is time-dependent strain independent of the applied load.
5 As estimated from Figure 7.1, BS8110 Part 2 (1985), assuming RH = 50%, age at loading = 1 day and an effective section depth of 300 mm. Eurocode 2 (2004) Figure 3.1 suggests large values of about 3 to 5.
6 Rebound is loose material – usually predominately the larger pieces of aggregate and fibres – which fails to adhere during spraying and falls down.

3 Construction methods

This chapter briefly outlines the typical construction methods and plant used to build SCL tunnels. Further details about tunnelling can be found in established texts (e.g. BTS 2004, Hoek and Brown 1980). Typical support measures are also described. The excavation sequence is as important in supporting the ground as the sprayed concrete lining or any other additional measures. Hence as a precursor to designing the sprayed concrete lining we must first understand how the sprayed concrete functions as part of the support for the tunnel.

Broadly speaking, the ground beneath our feet falls into one of three categories: soft ground, blocky rock or hard rock (see Table 3.1). Obviously this is a simplistic categorisation and the borders between each class are not clearly defined. Nonetheless, this classification is useful because each type of ground behaves in a different way and knowledge of the mode of behaviour guides the design and construction of the tunnel, including the sprayed concrete lining.

To explain the difference in behaviour of the ground we could think of the ground as being like cheese. Soft ground is like soft cheese, for example, Brie. It deforms as one body – a continuum. Because of the weak strength of the cheese, it deforms plastically when loaded. Similarly massive hard

Table 3.1 Types of ground

Category	*Soft ground*	*Blocky rock*	*Hard rock*
Description	Soils and weak rocks	Weak to moderately strong rocks	Massive strong rocks
Mode of behaviour	Continuum	Discontinuum	Continuum
Strengths	$\sigma_c < 1$ MPa	$1 \leq \sigma_c \leq 50$ MPa	50 MPa $\leq \sigma_c$
Stress/strength ratio[a]	≤ 1	≤ 1	$\ll 1$
Examples	Sands, clays, chalk	Limestone, sandstone	Basalt, granite

Note:
a This is indicative only since the stress conditions are governed by external factors and any rock type can be subjected to overstressing.

rock, like a hard cheese such as Parmesan or a mature Cheddar behaves like a continuum when cut or loaded. Deformation is often purely elastic due to the high strength. In between these two extremes are weak or moderately strong, blocky rocks – Wensleydale or Stilton being the equivalent cheeses. Deformation is governed by failure on pre-existing lines of weakness (e.g. joints). In other words the material behaves as a collection of discrete bodies – a discontinuum.

3.1 Soft ground

Soft ground is defined as soil or weak rock that generally behaves as a single mass (a continuum). Weak rocks can include chalk, breccia or conglomerate. The strength of soft ground ranges approximately from 0 to 10 MPa. The ground requires full support immediately or within a short space of time (i.e. when unsupported it has a stand-up time of less than a few hours). The key mechanisms of behaviour are listed in Table 3.2.

Control of deformations is critical to the success of tunnelling in soft ground – or more precisely, the control of the redistribution of stress is the key. As the tunnel is excavated, there is a redistribution of stress around the opening and this is accompanied by deformation of the ground. There is often some plastic yielding in the ground. If this process is not controlled, the yielding can lead to excessive deformations or complete collapse. Installation of the tunnel lining – and most importantly the formation of a structural ring – provides support and restricts the deformation of the ground.

3.1.1 Method of excavation

Given the low strength of soft ground it can be mechanically excavated using bucket excavators (see Figure 3.1) or roadheaders. Tunnel boring

Table 3.2 Key mechanisms of behaviour in soft ground

Behaviour	*Type of ground*	*Support measure*
Plastic yielding	Clays, sands, weak rocks	Early ring closure; subdivision of face; sprayed concrete
Ravelling	Sands	Ground treatment; subdivision of face; forepoling; sprayed concrete
Heave (of tunnel invert)	Clays, sands, weak rocks and/ or high water pressure	Early ring closure
Running	Sands – loose or under the water table	Ground treatment; subdivision of face; forepoling; compressed air; sprayed concrete
Block failure	Stiff jointed clays and weak rocks	Spiling; rockbolting; sprayed concrete

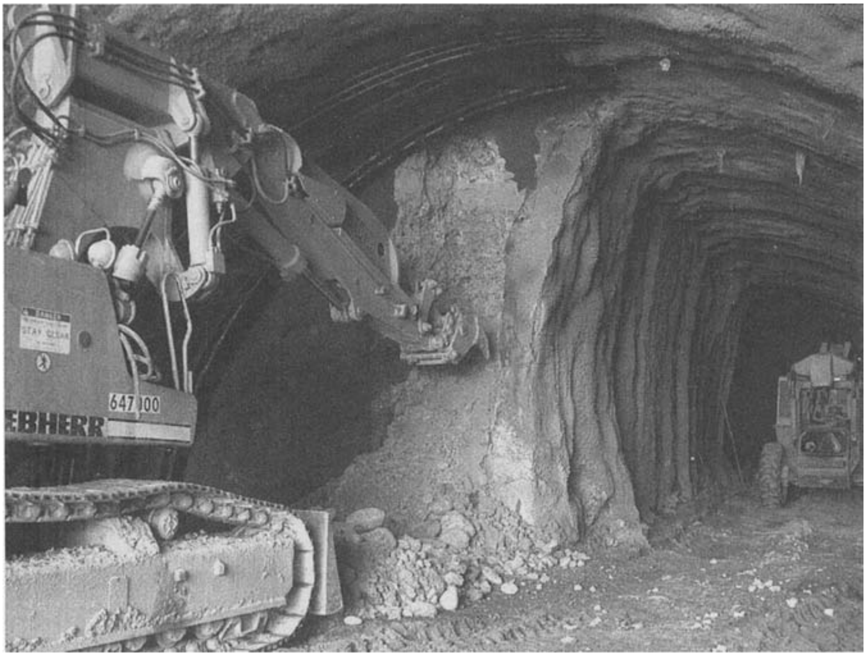


Figure 3.1 Excavation of a SCL tunnel in soft ground

machines (TBMs) are often used but rarely in combination with sprayed concrete. In soft ground the speed of advance of TBMs normally outstrips the ability of sprayed concrete to be applied and to support the ground.

3.1.2 Support and excavation sequences

Support measures have been developed to cope with the mechanisms of behaviour listed in Table 3.2. The role of sprayed concrete is to provide immediate/short-term support to the ground as a whole, as well as providing support in the long term.

Typically the lining will consist of 150 to 350 mm of sprayed concrete reinforced with one or two layers of wire mesh, depending on the size of tunnel and the loads on it (see Figure 3.2 for typical details). For smaller tunnels steel fibre reinforced sprayed concrete is sometimes used instead of mesh reinforcement. Lattice girders are often used to control the shape of the tunnel and to support the mesh during spraying.

If a tunnel cannot be safely constructed using full-face excavation, the face is subdivided into smaller headings. A variety of excavation sequences are used in soft ground (see Figure 3.3). Sometimes they are grouped in the categories of 'horizontal division' and 'vertical division' of the face.

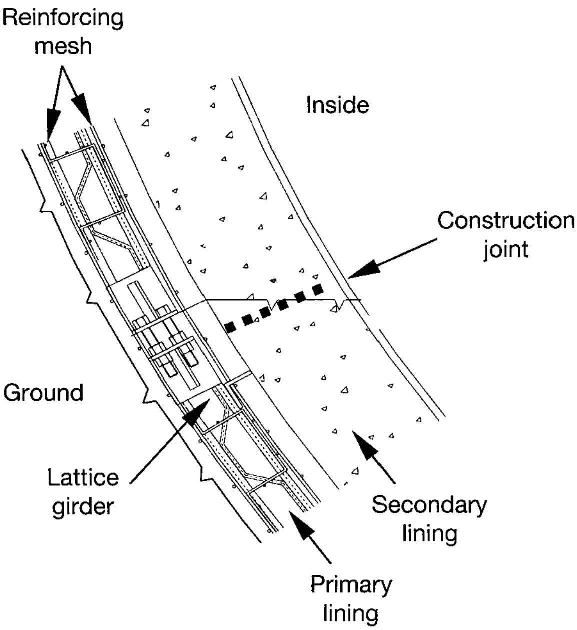
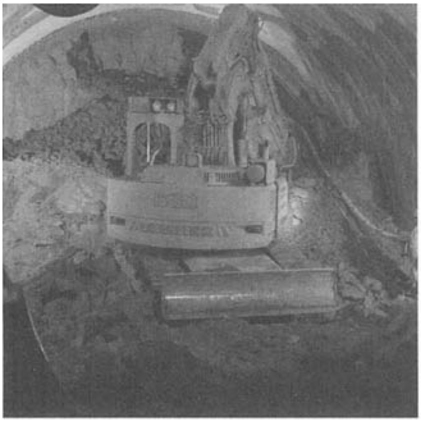


Figure 3.2 Lining detail

Side galleries are an example of the former and a top-heading drive followed later by a bench and/or invert is an example of the latter. The choice of excavation sequence is driven by the stability of the unsupported face and heading but it is also influenced by other factors such as the programme and the equipment available. Determining the loads on the sub-headings of a tunnel and the stresses in the linings is complex and will be discussed in more detail later (see Chapter 4). Collapses of SCL tunnels have taught us that these intermediate stages of construction require a full design as they may be more critical than the load cases acting upon the finished tunnel.

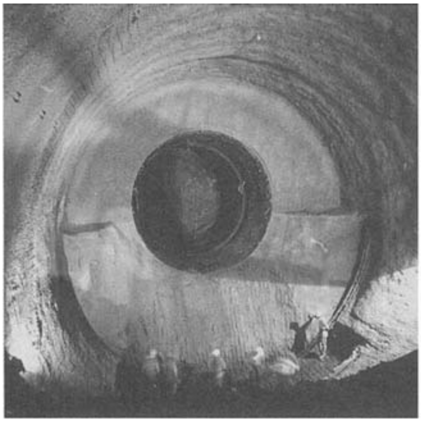
If the lining is loaded it will generally continue to deform until a closed structural ring is formed – so-called 'ring closure'. This applies equally to the temporary headings of the tunnel and the whole tunnel itself. Early ring closure is often specified in designs to control ground movement and settlement of adjacent structures. As an example, Ruzicka *et al.* (2007) found that a top heading drive ('vertical division' of the face) resulted in more than twice as much movement as a side gallery and enlargement sequence ('horizontal division'). If plastic yielding is limited to the area under the footings of the top heading, other measures can be used to increase the bearing capacity of the footings. The most simple is enlarging the footing to form an 'elephant's foot'. The most complex may involve temporary inverts or mini-piles.

Subdivision of the face introduces joints into the lining. For structural integrity there must be continuity of the steel reinforcement across these joints. Traditionally this was achieved using complex arrangements of lapping bars (see Figure 6.12). It is difficult to build these joints without damaging the lap bars in the process and without trapping rebound when spraying. As a result the quality of the joints was sometimes poor – both in terms of structural capacity and watertightness. Prefabricated starter bar units, such as KWIK-A-STRIP have simplified these joints (see Figure 6.13).

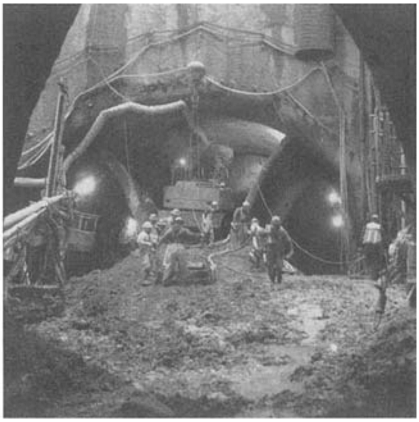


(a) Top heading, bench and invert

(b) Side gallery and enlargement



(c) Pilot and enlargement



(d) Twin side gallery and central core (provided by kind permission of M. Murray)

Figure 3.3 Excavation sequences in soft ground

3.1.3 *Special cases*

Shafts

The same principles for tunnels outlined above apply to shafts in soft ground (see Figure 3.4). Often the shaft cannot be excavated full face and to maintain stability it is divided into sections. Each section is excavated and supported sequentially. Additional measures such as dewatering may be required to ensure the stability of the invert. SCL construction can be used in conjunction with other methods. For example, if the ground consists of water-bearing deposits overlying impermeable ground, the shaft can be started using caisson sinking and continued as SCL after the caisson has been toed into the impermeable layer (e.g. Audsley *et al.* 1999).

The practical limitation for the gradient of an inclined tunnel is the ability of equipment to move up and down the shaft. Tunnels have been driven downhill at gradients of up to 32% (ITC 2006).

Junctions

At the junction between tunnels or tunnels and a shaft, the lining is reinforced to cope with the re-distribution of lining stresses around the

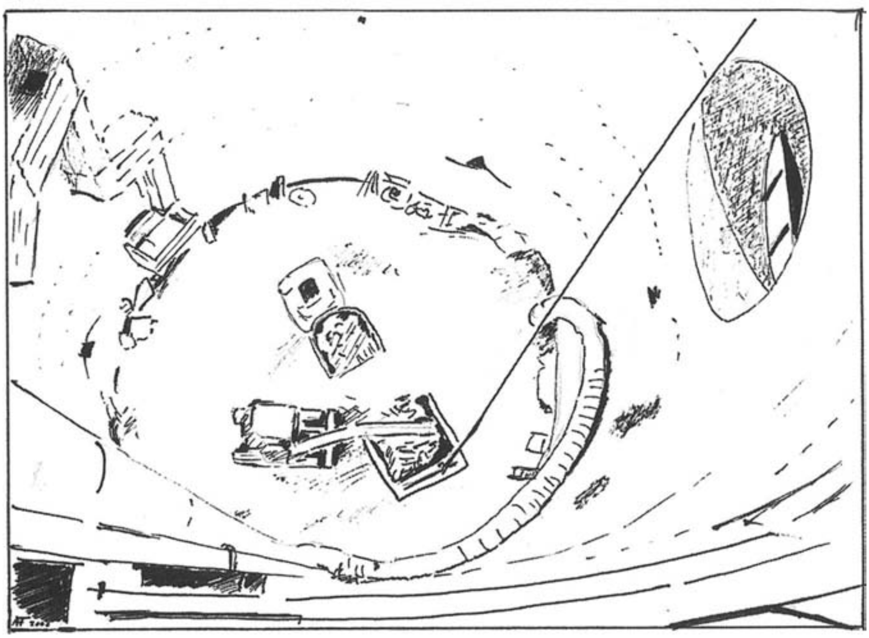


Figure 3.4 Excavation of a SCL shaft

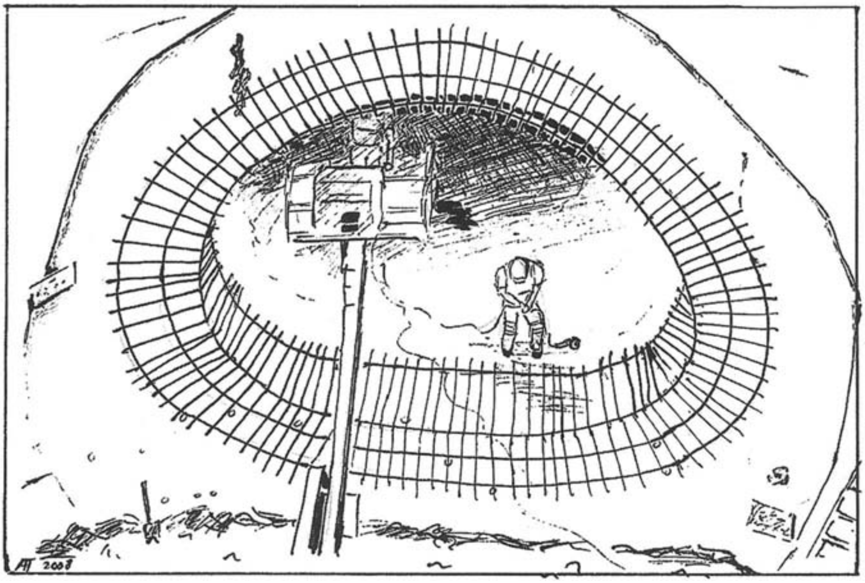


Figure 3.5 Reinforcement around a tunnel junction

openings (see Figure 3.5). As the 'parent' tunnel is constructed it is normal to form the tunnel eye in preparation for the construction of the 'child' tunnel. The tunnel eye is where the second tunnel meets the first and the lining here is thinner and/or lightly reinforced as it will be broken out later. Additional reinforcement is placed around the eye in the shape of a circle, square or bands (see Figure 6.5).

The choice of construction sequence is important at junctions because, when breaking out of (or into) the parent tunnel, the loaded lining is cut and those loads must be redistributed to the adjacent areas. This will cause the tunnels to deform until a complete structural ring is formed again. Hence subdivision of the face, temporary propping and early ring closure are often used to manage the stress redistribution and keep the tunnels both stable and within the required limits of deformation.

3.2 Blocky rock

Blocky rock is defined as rock that generally behaves as a collection of discrete blocks (a discontinuum). Blocky rocks include limestones, sandstones and mudstones. The strength of blocky rock ranges approximately from 10 to 50 MPa but it may be much higher. The defining characteristic is joint spacing, which ranges from extremely close to wide (< 20 mm to 2 m). The ground requires full support within a short space of time (i.e. stand-up time

Table 3.3 Key mechanisms of behaviour in blocky rock

Behaviour	*Type of ground*	*Support measure*
Block failure	Rocks with intersecting joints and a weak bond across the joint	Spiling; rockbolts; sprayed concrete with mesh or fibres; steel arches
Plastic yielding	Rocks where the stress/strength ratio is greater than 1	Early ring closure; subdivision of face; sprayed concrete
Ravelling	Loose blocky rock, e.g. breccia	Ground treatment; subdivision of face; forepoling; sprayed concrete
Heave (of tunnel invert)	Weak rocks and/or high water pressure	Early ring closure
Squeezing	Where the stress/strength ratio is much greater than 1	'Yielding support': rockbolts with sprayed concrete; slots in sprayed concrete; deformable supports; yielding arches
Swelling	E.g. phyllites, anhydrite, some clays	Subdivision of face; heavily reinforced lining; ring closure

for the unsupported excavation ranges from a few hours to a few days). The key mechanisms of behaviour are listed in Table 3.3.

3.2.1 Method of excavation

Depending on the strength of blocky rock and type of jointing, it can either be mechanically excavated using roadheaders (see Figure 3.6) or by drill and blast. Tunnel boring machines (TBMs) are sometimes used, for longer tunnels. Overbreak along the lines of joints often occurs, leading to an irregular shape of the tunnel surface. Sprayed concrete can be used to backfill this overbreak.

3.2.2 Support and excavation sequences

Support measures have been developed to cope with the mechanisms of behaviour listed in Table 3.3. The role of sprayed concrete is to provide support to blocks and to prevent deterioration of the rock mass over time. The sprayed concrete is often used in combination with rockbolts. Typically the layer of sprayed concrete will consist of 50 to 150 mm of sprayed concrete reinforced with steel fibres or wire mesh, depending on the loads on it. The majority of the *in situ* ground stresses are redistributed around the tunnel and carried within the rock itself.

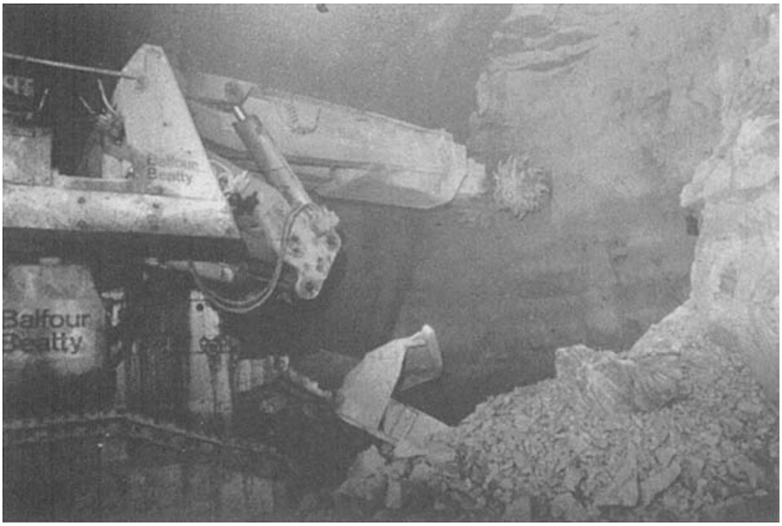


Figure 3.6
Excavation of a SCL tunnel using a roadheader

Because the behaviour of the ground is governed primarily by the movement of individual blocks, a full ring of sprayed concrete is not normally needed (see Figure 3.7). Support classes III and IV in Figure 3.7 refer to ground that is heavily jointed and is behaving more like soft ground. Again, because of the greater stability of block rock compared to soft ground, less restrictive excavation sequences can be used. For example, support does not necessarily have to be installed immediately behind the face and top heading drives can advance far ahead of the rest of the tunnel. However, the boundary between soft ground and blocky rock is not clear cut. Care must be taken to ensure that stability is always maintained. A common cause of tunnel failure is that a top heading drive is advanced too far into unstable ground.

3.2.3 Special cases

Shafts and junctions

Depending on the ground conditions, many of the comments in Section 3.1.3 may apply. Blocky rock tends to be stronger and so the support required is less and also it is tailored to control block stability. Longer advance lengths and less restrictive excavation sequences may be employed but only if the stability of the ground permits. At junctions there is more scope to reinforce the surrounding ground (e.g. with rockbolts) and to let it carry the redistributed loads.

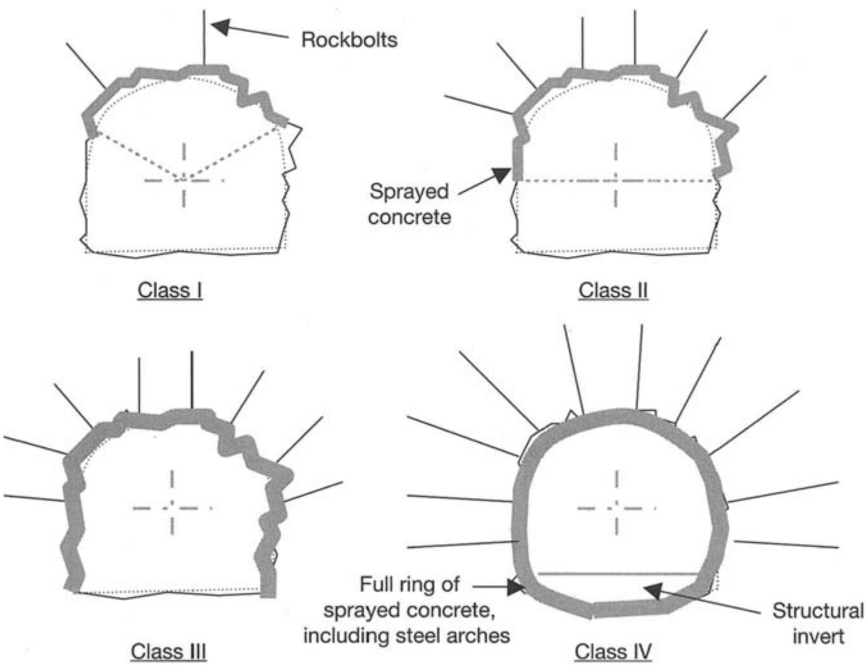


Figure 3.7 Rock support classes

Swelling

Certain minerals such as phyllite and anhydrite swell when the confining stress is relieved and/or in the presence of water. This can impose enormous loads on a tunnel lining (e.g. Wittke-Gattermann 1998). Apart from minimising the exposure to water, there is little that can be done to prevent swelling. Instead a heavily reinforced final lining is installed. The best one can do is to manage the convergence of the tunnel and thereby return the situation to equilibrium. During construction the primary lining may suffer damage and large deformations (tens of centimetres or more) may occur. 'Ductile' support may be required, including yielding arches or supports and slots in the lining – see the section below for more details.

Squeezing

Squeezing ground is essentially another form of plastic yielding. Like rockburst it occurs where the *in situ* rock stresses are very high. When the uniaxial compressive strength of the rock is less than 30% of the *in situ* stress, severe or extreme squeezing can occur (see Hoek and Marinos (2000) for a method of predicting squeezing potential). The NATM design philosophy was originally conceived in part to deal with squeezing ground.

It is not economic to install a lining to resist the ground stresses, rather it is better to manage the redistribution of the stresses in the ground as it yields and to install a lining when most of the stresses (acting radially) have relaxed. The deformations that occur before installing a final lining can be up to one metre or more. Rabcewicz (1969) identified that sprayed concrete in combination with rockbolts was very effective because it provides ductile support to the ground as it deforms. The rockbolts should be long enough to reinforce the whole of the plastic zone that develops around the tunnel. However, the primary lining can be effectively destroyed in this process. Over the years this approach has been developed further. Longitudinal slots can be left open in the tunnel lining to accommodate the deformations without excessive damage to the sprayed concrete (e.g. the Arlberg tunnel in Austria (John 1978)). The slots can either be left open or deformable steel supports can be installed in them (e.g. Aggistalis *et al.* 2004).

Creeping

Rock salt, chalk, coal and certain other rocks may exhibit creep behaviour. They will continue to deform when under load. This undermines the arching of stresses in the ground and exerts additional load on the tunnel lining. A full structural ring has to be installed and reinforced to resist these creep loads.

3.3 Hard rock

Hard rock is defined as massive strong rocks that mainly behave as a continuous mass. The joints are widely to extremely widely spaced. The strength is usually greater than 50 MPa and the stress/strength ratio is less than one. While block failure and plastic deformation may occur, in general the rock responds elastically to the excavation of the tunnel. Stand-up time for the unsupported excavation as a whole ranges from days to years. However, individual blocks may need immediate support. The key mechanisms of behaviour are listed in Table 3.4.

3.3.1 Method of excavation

For long tunnels with a constant shape, tunnel boring machines (TBMs) are increasingly preferred (see Figure 3.8) while, for shorter tunnels or underground works with a complex shape, the drill and blast method remains the more economic option (see Figure 3.9). Support measures can be installed near the face of the TBM or after mucking and scaling in a drill and blast tunnel. However, if the support is installed immediately behind the face it may risk being damaged by the next blast (see Section 3.3.2).

Depending on rock conditions and the blasting method, the surface of the tunnel can be very irregular in a drill and blast tunnel. Additional sprayed concrete may be required as a smoothing layer to fill up the overbreak.

Table 3.4 Key mechanisms of behaviour in hard rock

Behaviour	*Type of ground*	*Support measure*
Plastic yielding	Rocks where the stress/strength ratio is greater than 1	Early ring closure; subdivision of face; sprayed concrete
Block failure	Stiff jointed clays and weak rocks	Spiling; rockbolts; sprayed concrete
Rockburst	Where the stress/strength ratio is much greater than 1	Rockbolts with mesh or SFRS
Squeezing	Where the stress/strength ratio is much greater than 1	'Yielding support' rockbolts with sprayed concrete; slots in sprayed concrete; deformable supports; yielding arches

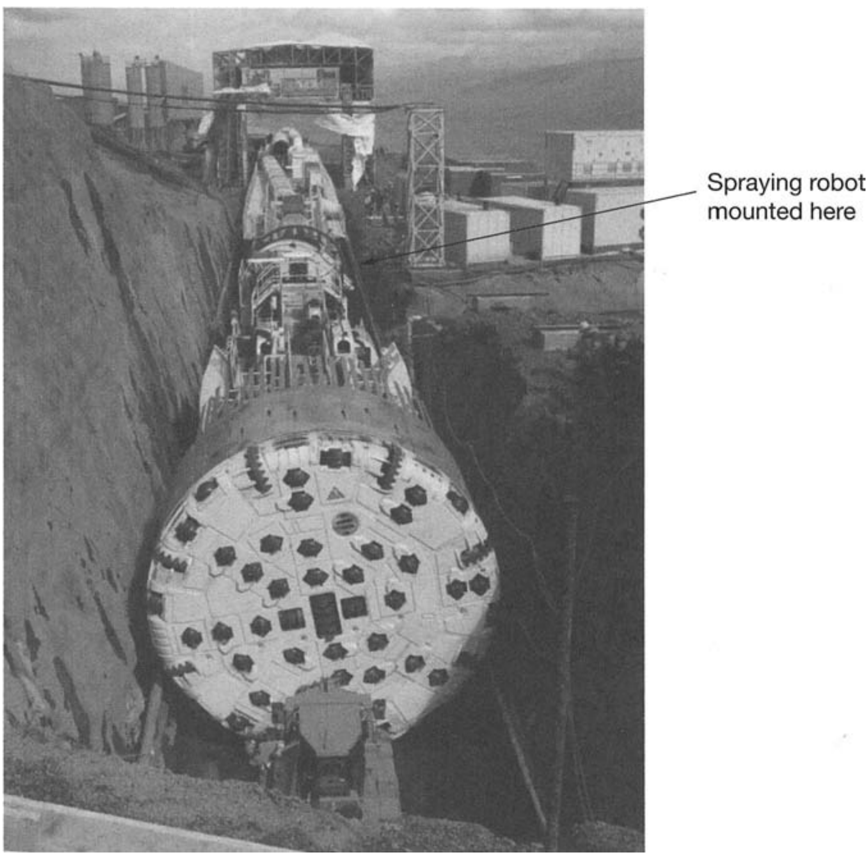


Figure 3.8 Hard rock tunnel boring machine

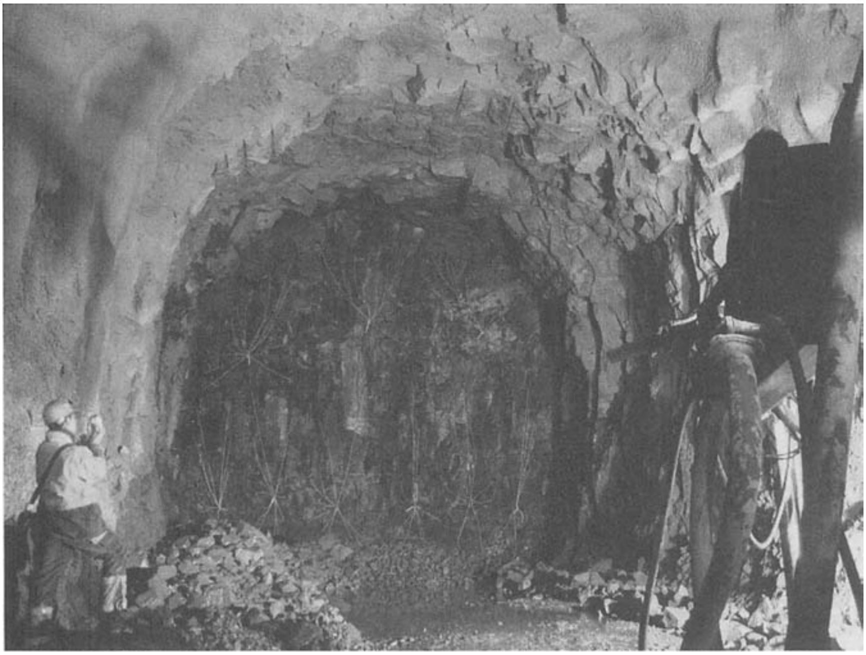


Figure 3.9
Excavation of a SCL tunnel using the drill and blast method

3.3.2 Support and excavation sequences

The role of sprayed concrete is primarily to support isolated blocks, rather than the rock mass as a whole. The rock itself is often stronger than the sprayed concrete. If the rock is very jointed, then it behaves more like a discontinuous mass and the support is as described in Section 3.2. The sprayed concrete is often used in conjunction with rockbolts. Steel fibres are tending to replace mesh as reinforcement due to the ease of application and saving in time. There is a slight risk of the sprayed concrete being damaged by the blasting but this usually only happens at high particle velocities. The threshold for damage within the first 24 hours is around 150 mm/s (Geoguide 4, 1992). Ellison *et al.* (2002) suggested that damage can occur at velocities above 500 mm/s but that after 24 hours the sprayed concrete could withstand even this level of vibration. Ansell (2004) has provided more detailed guidance for plain sprayed concrete, based on *in situ* tests and dynamic numerical modelling of sprayed concrete (see Table 3.5). The time for the adhesion of the concrete to grow to greater than the maximum stress from blasting was found to depend on the thickness of the layer as well as the proximity of the charge and its size.

Table 3.5 Recommended minimum age for sprayed concrete to be subjected to blasting (Ansell 2004)*

Thickness of sprayed concrete (mm)	*Distance to charge (m)*	*Weight of explosives (Ammonium nitrate and fuel oil – ANFO)*		
		0.5 kg	*1.0 kg*	*2.0 kg*
25	4	1 hour	2 hours	4 hours
	2	8 hours	25 hours	65 hours
50	4	3 hours	11 hours	35 hours
	2	45 hours	5 days	9 days
100	4	15 hours	72 hours	7 days
	2	8 days	14 days	Not possible

* See original paper for the additional notes for this table.

Since relatively small quantities of sprayed concrete are often required in hard rock tunnels so it may be more convenient to use pre-bagged dry mix sprayed concrete. This is especially true when the working space is limited such as in small diameter tunnels or shafts.

3.3.3 Special cases

Shafts and junctions

Given the stability of the rock neither shafts nor junctions place particular demands on the sprayed concrete lining. The sprayed concrete is merely to control block stability. At junctions additional reinforcement of the rock mass is required but this is most effectively achieved using rockbolts. The same applies to pillars between tunnels.

Rockburst

Although the introduction to this category suggested that in hard rock tunnels the stress/strength ratio is less than one, this may not be the case. In deep mines or tunnels in regions of high tectonic stresses (e.g. the Alps or Himalayas), the rock stresses can exceed the strength. The result is rockburst – the sudden, brittle fracturing of rock around the edge of the excavation. Given the size of the forces involved, rockburst can rarely be prevented but it can be controlled. Steel fibre reinforced sprayed concrete (SFRS) can absorb a lot of energy as it is deformed. Hence SFRS is often used in combination with rockbolts for protection from rockburst.

Fault zones

Within a massive rock mass there will be major joints or fault zones. Sometimes these are narrow and sealed tight; sometimes they are wide, open and contain water or loose material. These can be very difficult areas because the ground is highly overstressed or the water is at high pressure. There are numerous instances of tunnel collapse at fault zones. In these cases the ground behaves more like a blocky rock or soft ground and the support measures described earlier should be applied.

3.4 Modern sprayed concrete

Sprayed concrete technology is a rapidly developing field. While many of the principles remain the same, the equipment has improved greatly in recent years in terms of its ease of use and its capacity. Health and safety considerations and the need for higher production rates are leading to increasing levels of automation. Mix design has also advanced in leaps and bounds. This has been discussed earlier in Section 2.1. Sprayed concrete is produced in two ways: the dry mix process and the wet mix process.

3.4.1 Dry mix sprayed concrete

In the dry mix process, a mixture of naturally moist or oven-dried aggregate, cement and additives is conveyed by compressed air to the nozzle, where the mixing water (and accelerator, if liquid) is added (see Figures 3.10 and 3.11). The dosage of accelerator and the water/cement ratio are controlled by the nozzleman during spraying. In the past the dry mix has been preferred, because it could produce sprayed concrete with higher early strengths, and some countries, notably Austria, retain a preference for the dry mix process.

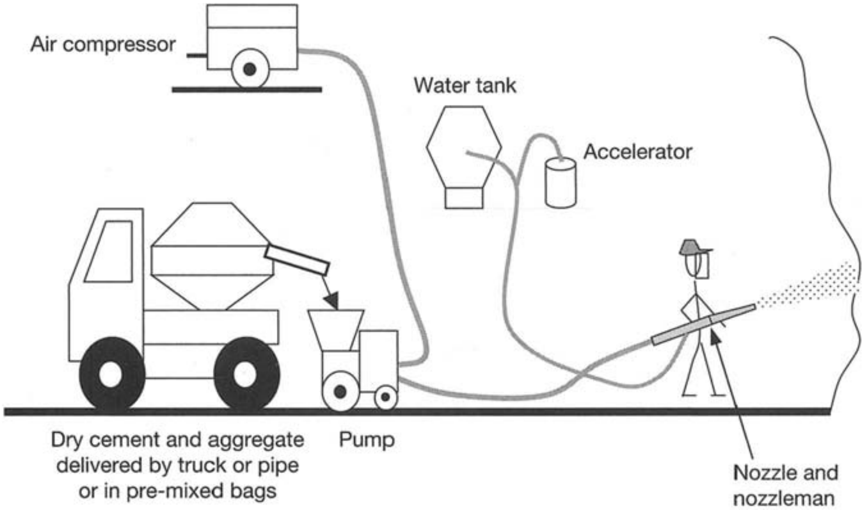


Figure 3.10 Dry mix process

Figure 3.11 Dry mix pump

Some of the reasons for choosing dry mix sprayed concrete are listed below:

- higher early-age strengths (see Table 3.6);
- lower plant costs;
- small space requirements on site, especially if pre-bagged mixes are used (this is particularly advantageous in urban sites);
- more flexibility during operation (sprayed concrete can be effectively available 'on tap', less cleaning required).

Broadly speaking, dry mix sprayed concrete is best suited to projects that require small and intermittent volumes of sprayed concrete and where there are space constraints on site or long journey times from the point of batching to the face. Dry mix sprayed concrete can be batched and stored in bags ready for use. Accelerator in powder form can be added to the pre-bagged mix so that only water and compressed air are needed when spraying. This simplifies the equipment needed too, but there is no opportunity to vary the dosage of accelerator.

The main disadvantages of the dry mix method are the high levels of dust (see Figure 3.14) and the variability of the product due to the influence of the nozzleman. To counteract this, pre-wetting nozzles (to reduce dust)

Table 3.6 Compressive strengths of modern mixes (after Lukas *et al.* 1998)

Age	*Dry mix spray cement (oven-dry agg.)*	*Dry mix spray cement (moist agg.)*	*Wet mix 6% alkali-free acc.*[a]	*Dry mix 6% alkali-free acc.*
6 minutes	0.95	0.5	0.5	–
1 hour	1.3	1.0	1.0	–
1 day	23.0	21.0	15.0	17.0
56 days	41.0	39.0	61.0	39.0

Note:
a Higher equivalent cement content in this mix; refer to original report for the full mix details.

and special spray cements, which require no additional accelerator, have been developed (Testor 1997).

3.4.2 Wet mix sprayed concrete

In the wet mix process, ready-mixed (wet) concrete is conveyed by compressed air or pumped to the nozzle, where the liquid accelerator is added (see Figures 3.12 and 3.13). The water/cement ratio is fixed when the concrete is batched outside the tunnel. The dosage of accelerator is controlled by the nozzleman during spraying. There is a global trend towards using the wet

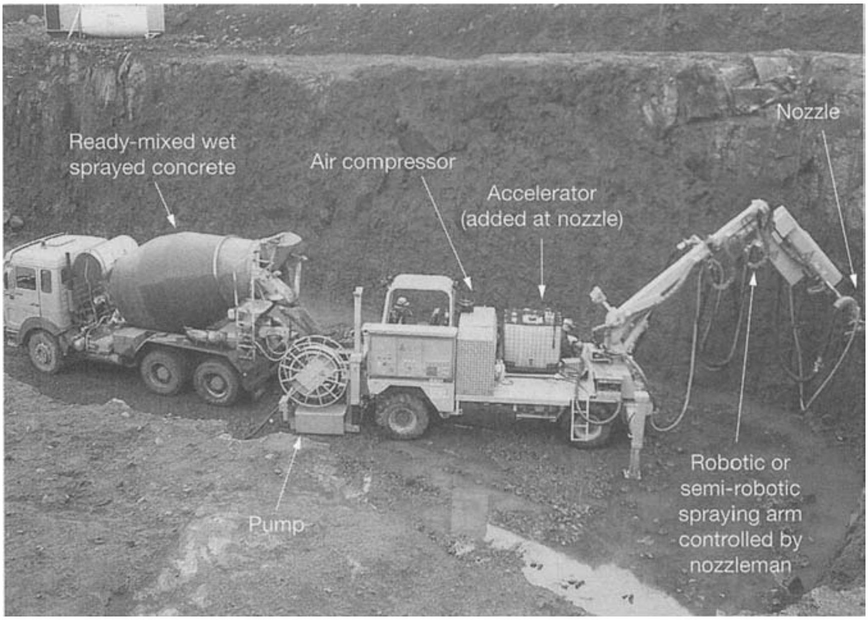


Figure 3.12 Wet mix process

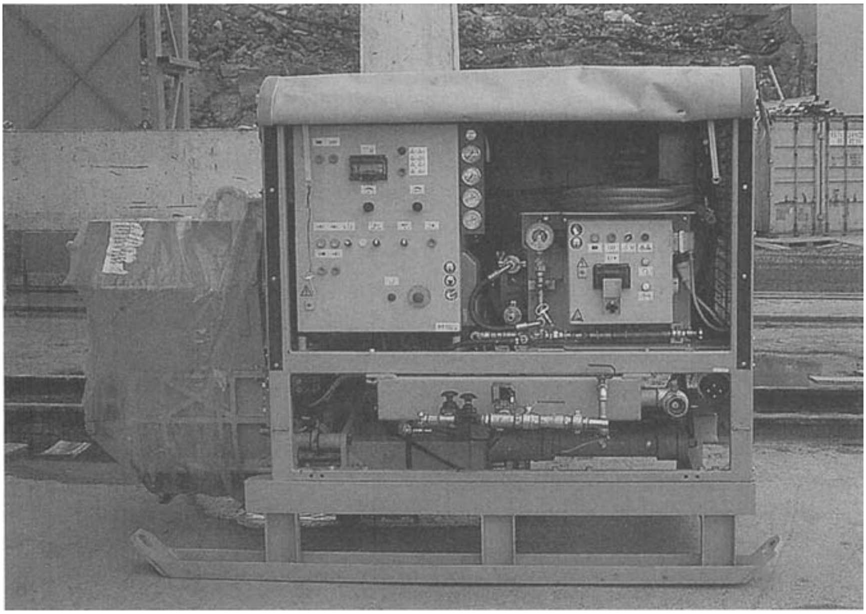


Figure 3.13 Wet mix pump

mix process in preference to dry mix. The wet mix process is perceived to permit greater control over quality, to be more suited to automation and to be safer (since dust levels are lower). It has been estimated that worldwide about 60% of sprayed concrete is produced by the wet mix method (Brooks 1999).

Some of the reasons for choosing wet mix sprayed concrete are listed below:

- Greater quality control (the mix is batched at a plant and the water/cement ratio cannot be altered by the nozzleman).
- Robotic spraying is required because of the weight of the nozzle and hose but this leads to higher outputs than dry mix (up to 18 to 20 m^3 per hour) and reduces variability due to the human factor. The additional plant cost is partially offset by the reduction in labour costs.
- Lower rebound (typically 16%, compared to 21 to 37% for dry mixes (Lukas *et al.* 1998)).
- Less dust (with dust levels within acceptable limits – see Figure 3.14).
- The use of ready-mix batches and robotic spraying permits records of the exact mix and quantities sprayed to be kept more easily (Davik and Markey 1997).

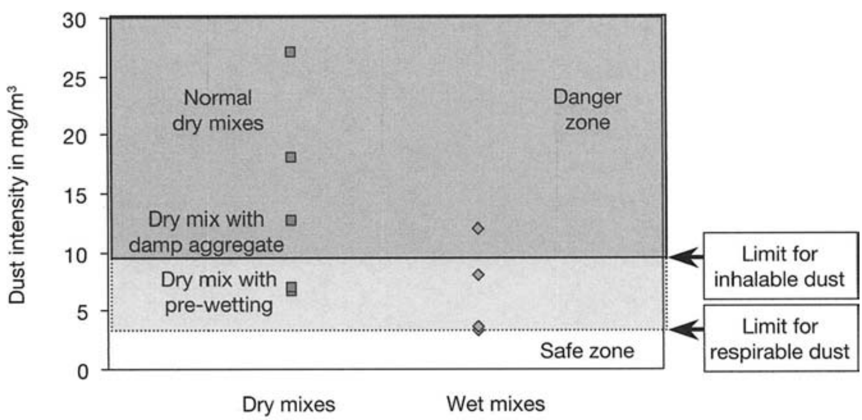
Wet mix sprayed concrete is best suited to projects that require regular and large volumes of sprayed concrete and where a batching plant can be

Table 3.7 Normalised cost comparisons between dry and wet mix sprayed concrete

Source and notes	*Dry mix*	*Wet mix*
Lukas *et al.* (1998) – all costs exc. rebound	0.72	1.00
Röthlisberger (1996) – all costs	1.12	1.00
Strubreiter (1998) – whole costs over 1000 m tunnel	0.91	1.00
Melbye (2005) – all costs over 2000 m tunnel	1.00	0.42

located close to the point of use. The cost difference between modern high quality dry and wet mix sprayed concrete has reduced to the extent that, if one includes all relevant factors (such as rebound, labour costs and cycle time), there is little to choose between the two methods (see Table 3.7). On longer tunnels the higher investment in equipment can be easily justified.

The main disadvantages of wet mix sprayed concrete are the higher plant costs, lower strengths and the limited pot life of the sprayed concrete once mixed. However, the extra plant cost is partially offset by the benefits of automation. As health and safety regulations become increasingly strict, the question of dust levels will strengthen the case for using wet mix sprayed concrete.



(Sample data from Testor and Pfeuffer 1999 and Melbye 2005 with the shaded area denoting the danger zones according to UK health and safety limits)

Figure 3.14 Dust levels for different types of sprayed concrete (after Testor and Pfeuffer 1999)

3.4.3 Pumping

Large-scale pumps (see Figure 3.13) have two cylinders to provide as smooth a flow as possible. Typical pumping rates are around 8 m^3/hr, although the capacity of the pumps can be as high as 20 m^3/hr. Before starting to pump concrete, the pump and line should always be washed through with a weak cement grout, otherwise the water-borne cement in the concrete mix is absorbed as it passes along the line and the first concrete is ruined. If the pump is not operating correctly, the flow will tend to pulse – i.e. the flow is not continuous. This can lead to poor compaction and lamination.

The role of the pump operator is often undervalued. A skilled operative monitors the quality of the concrete entering the pump and the performance of the machine. By doing so he can avert blockages and damage to the pump.

Typical pump lengths are between 20 to 40 m but with the addition of suitable admixtures this can be extended up to 150 m (Melbye 2005). Spirig (2004) describes one extreme example from the Gotthard Tunnel in Switzerland where careful mix design enabled sprayed concrete to be delivered down a vertical pipe of up to 800 m deep and sprayed without the use of a remixer.

The length of steel fibres should be less than 50% of the diameter of the pumping hose to avoid blockages.

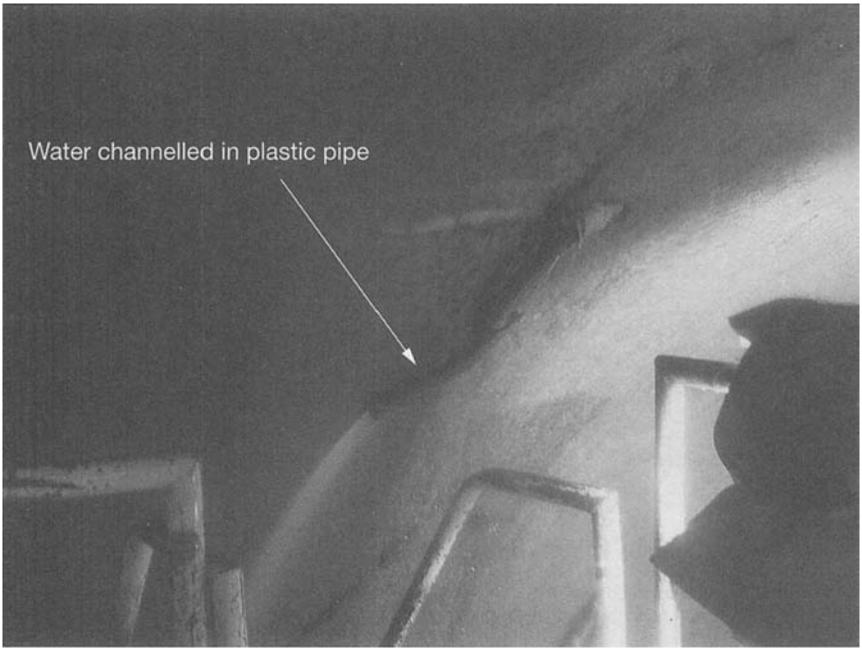


Figure 3.15 Control of water ingress

3.4.4 Spraying

Substrate surface

The ideal substrate for sprayed concrete is a slightly rough, moist surface as this permits a good bond.

Concrete cannot be sprayed onto surfaces that it cannot bond to – for example, wet surfaces with running water or smooth plastic surfaces. If there is water present, steps must be taken to manage it so that the sprayed concrete can be applied (e.g. see Figure 3.15). Dry mix sprayed concrete can be sprayed onto wet surfaces if the force of the jet of concrete can displace the water and permit the concrete to bond to the ground. Pinning a geotextile over a wet area has the disadvantage of preventing any bond between the sprayed concrete and the ground. As an alternative, drainage holes can be drilled into the ground to concentrate the ingress at discrete points and dry up the surface.

When spraying onto a plastic sheet membrane a layer of thin wire mesh is often placed in front of the mesh, firstly to hold the membrane in place and prevent it from flapping around (so-called 'drumming') and secondly to provide something to hold up the sprayed concrete while it is fresh.

Spraying technique

The skill of the nozzleman has a great influence on the quality of the sprayed concrete. Several guides exist on best practice for sprayed concrete (e.g. ACI 506R-90 (1990), EFNARC 1996).

Poor spraying technique can lead to the following defects (see also Figure 3.16):

- **Voids** – when spraying on an irregular surface, in awkward geometries (such as sharp corners) or around obstructions (such as reinforcing bars), there is a danger of forming voids if the angle of the jet of concrete is wrong.
- **Shadowing** – voids are formed behind reinforcement bars, exposing the steel to a greater risk of corrosion and reducing the effectiveness of the reinforcement.
- **Sloughing** – sections of sprayed concrete fall off under their own weight, either because the bond is too weak or because the layer that has been applied is too thick.
- **Laminations** – rather than being a homogeneous mass, the sprayed concrete may consist of layers with a poor bond between the layers. This may be due to inadequate surface preparation between applications of sprayed concrete or variations in compaction during spraying. White staining may indicate that a film of pure accelerator was sprayed on the surface due to an interruption in the flow of concrete.

- **Rebound** – if rebound is not cleared away during spraying it may become incorporated into the lining, forming a zone of weakness. Also excessive rebound is a costly waste of sprayed concrete – see Figure 3.17 for the influences on rebound.
- **Low strength** – if there is overdosing of the accelerator, there is a risk that the sprayed concrete will have a low strength, either because it

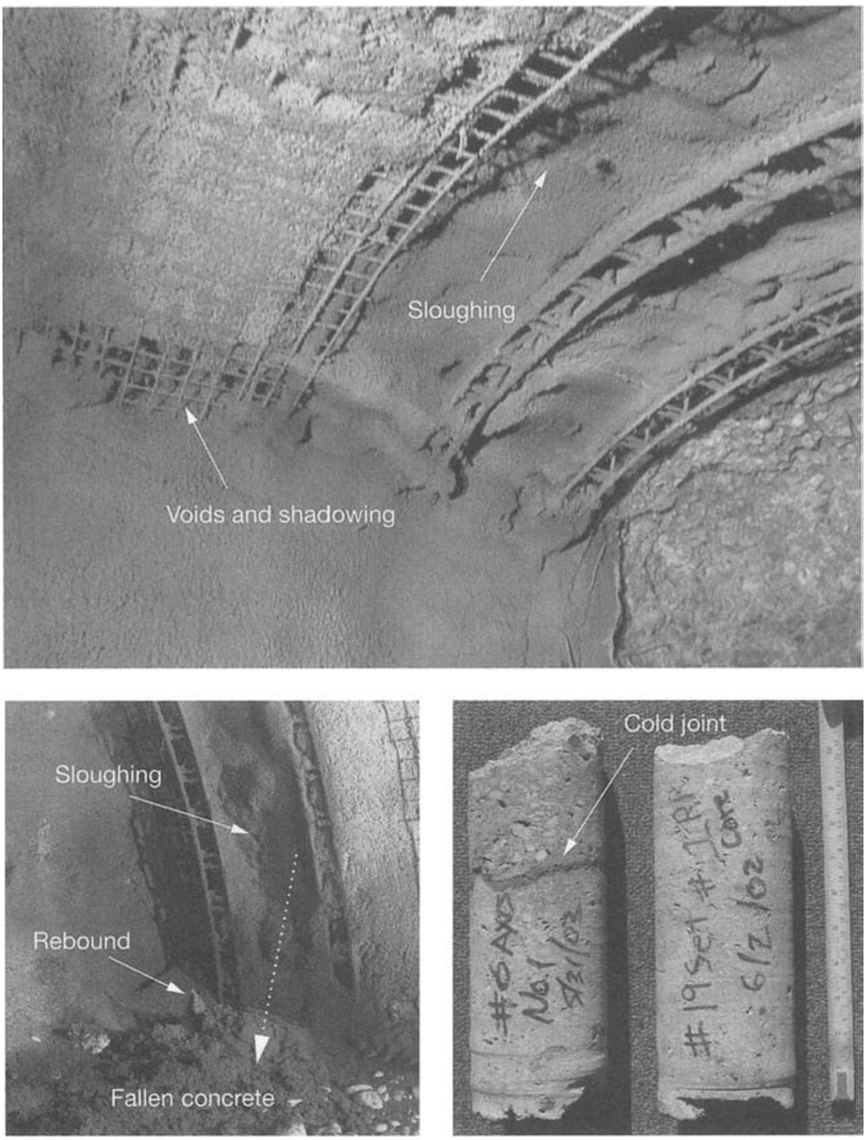


Figure 3.16 Spraying defects

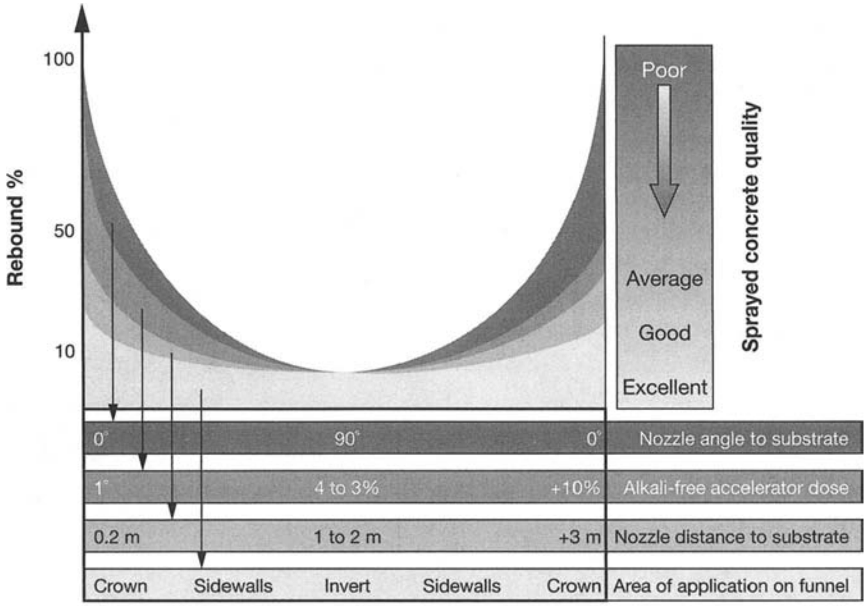


Figure 3.17 Effect on rebound and quality of the principal spraying parameters (Melbye 2005)

has a more porous structure (as the compaction is less effective) or because of a long-term reduction in strength (although this phenomenon does not seem to occur with modern accelerators).

A good design will make the job of the nozzleman easier. Geometries that are awkward to spray should be avoided and the spacing of reinforcing bars should not be less than 150 mm. Prefabricated starter bar units simplify joints.

The latest technology focuses on reducing the scope for human error. Various distance measuring devices (e.g. laser distometers such as TunnelBeamer[1] or photogrametric devices such as DIBIT) are in use to check the profiles of the sprayed concrete. However, the only current system that can be used interactively during spraying to check the profile and thickness is the TunnelBeamer (Hilar *et al.* 2005). Therein lies the strength of this device, outweighing the fact that it can only take spot measurements. The advantage of a device such as DIBIT is that it provides a check on the whole surface but work at the face has to stop for the survey to be done. In a tunnel, the typical accuracy of these systems is ±20 mm, which is adequate given that tolerances for spraying are typically ±15 to 25 mm.

Fully automatic (robotic) spraying has been trialled (e.g. for fire protection coatings) but it has yet to be used in a production situation. Undoubtedly this development will come soon.

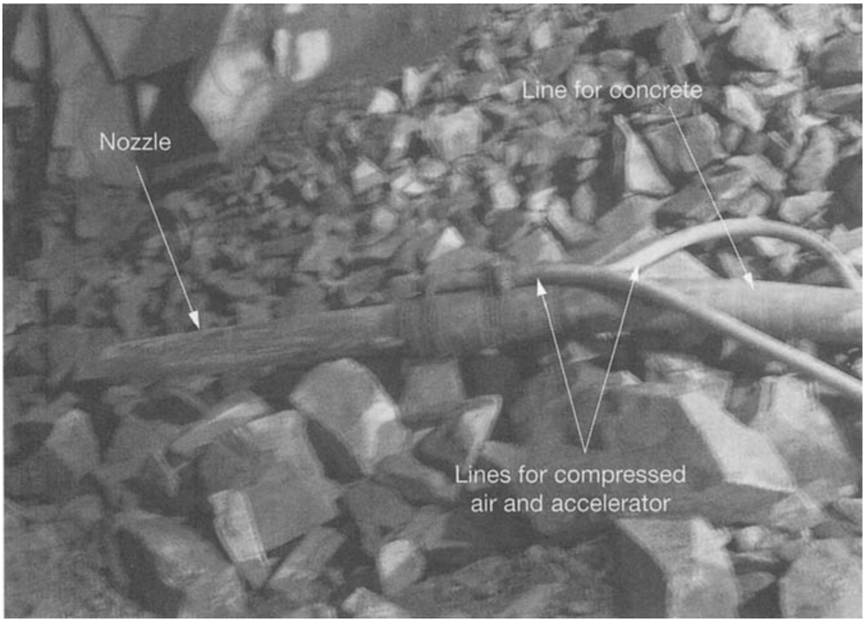


Figure 3.18 Nozzle

Nozzles

The nozzle is the device at the end of the hose that converts the stream of concrete (dry or wet) into a jet of sprayed concrete by the addition of compressed air (see Figure 3.18). Nozzles are designed to ensure good mixing of the accelerator and compressed air and, in the case of dry mix, the water too. Special nozzles have been developed for dry mixes to permit prewetting of the aggregate just before the main body of water and accelerator is added. This helps to reduce dust.

The nozzle should be cleaned after every use so that it does not become clogged with hardened concrete. The end of the nozzle is designed to blow off if the nozzle becomes blocked during spraying. Nozzles can suffer greatly from wear and tear due to the abrupt reduction in diameter of the pipe. This can be mitigated by curving the interior of the nozzle or using so-called 'stream converter' nozzles (Spirig 2004).

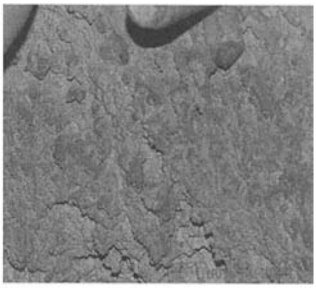
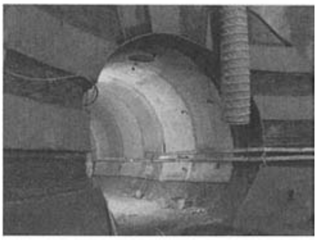
The length of steel fibres should be less than 75% of the diameter of the nozzle to avoid blockages.

Finishing

Depending on the end-user's requirements for the SCL, various finishes can be created (see Table 3.8). The quality of the as-sprayed surface can be improved by reducing the dosage of the accelerator in the final pass when spraying.

The sharp steel fibres protrude from the as-sprayed surface, so where people might come into contact with the lining a 'smoothing' layer of gunite is often applied as a finishing coat.

Table 3.8 Finished surfaces of sprayed concrete

Finish	*Class acc. to SHW Clause 1708 (HA 2006)*	*Example*
As sprayed – poor quality finish		
Unfinished	U1 = The concrete shall be levelled and screeded to produce a uniform plain or ridged surface as specified, surplus concrete being struck off by a straight edge immediately after compaction.	
Wood float finish	U2 = after the concrete has hardened sufficiently, it shall be floated by hand or machine sufficient only to produce a uniform surface free from screed marks	
Steel float finish	U3 = When the moisture film has disappeared and the concrete has hardened sufficiently to prevent laitance from being worked to the surface, it shall be steel-trowelled under firm pressure to produce a dense smooth uniform surface free from trowel marks.	

Curing

Unlike conventional cast concrete, sprayed concrete is rarely cured. Examples of curing are normally associated with the use of permanent sprayed concrete. A common view is that the environment in the tunnel is sufficiently humid (often more than 50% relative humidity) for effective curing to take place. This is a convenient assumption as curing interferes with the construction process in a tunnel. However, it is doubtful that the humidity is ever this high (Holmgren 2004). On the other hand, if we are aspiring to create concrete that is as good as cast concrete, it seems odd that no curing is applied. Curing can be helped by spraying a mist of water onto the sprayed concrete (e.g. Kusterle 1992) or by applying a curing compound to the surface.

Note

1 TunnelBeamer™ is patented by and a registered trademark of Morgan Est and Beton-und-Monierbau.

4 Design approaches

As stated in the Preface, this book is not a cookbook and, in my opinion, nor should it be. Given the variability of nature and the limitless plausible scenarios for a tunnel, it is impossible to write a simple guide that dictates how to design a tunnel under every imaginable circumstance. The art of tunnelling lies in applying knowledge to the unique situation facing an engineer. To aid this process, this chapter discusses the logic that underpins good design practice.

4.1 Observation vs prediction

In terms of approaches to design, in theory, there is a spectrum from a reliance on pure empiricism to total faith in prediction. In practice neither extremes are used (see Figure 4.1). Given the heterogeneity of the ground, it is impossible to predict in advance precisely how a tunnel will behave, even if the exact construction method is known. Engineers must observe the performance of the tunnel and, if necessary, adjust the excavation sequence and support to suit the prevailing conditions. Similarly, an observational approach does not equate to launching into a tunnel without any concept of what the support will be. Peck's definition of the Observational Method was succinctly summarised by Everton (1998) as:

> a continuous, managed and integrated process of design, construction control, monitoring and review, which enables previously designed modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety.

The terms 'robustly engineered' or 'fully engineered' design have been introduced to describe a suitable approach to modern SCL tunnelling (e.g. Powell *et al.* 1997). In a robust design, all the load cases that could be reasonably foreseen should be considered and the support designed to ensure that there is an adequate factor of safety, not just for the completed structure but also for all intermediate stages. What distinguishes SCL tunnels from others is the fact that these intermediate stages may be more critical than the final condition.

Designs for SCL tunnels often feature a range of support classes. The choice of support class is based (implicitly or explicitly) on an assessment of which one will manage the prevailing risks best. Even where support classes are not specified in the design, there is usually scope to vary aspects of the design such as advance length and timing of part or all of the support. This should be done within the framework of a set of pre-designed measures. In other words, the key distinguishing feature of modern design

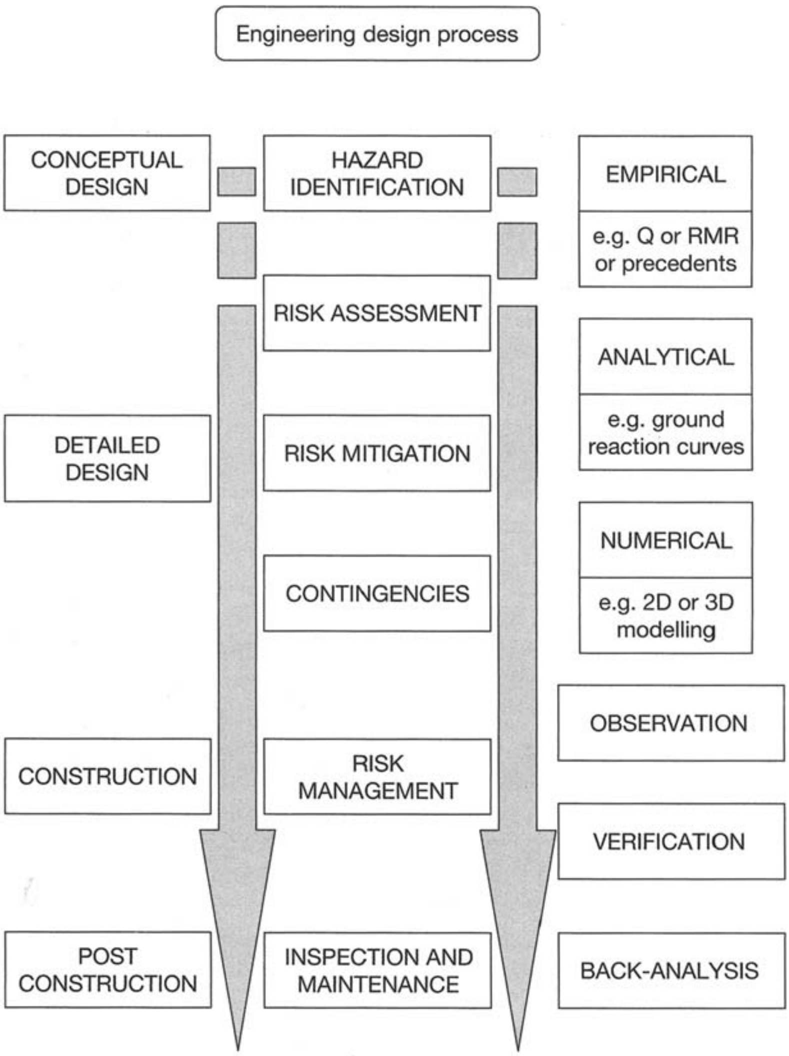


Figure 4.1 Empiricism vs prediction in design

practice is that a robust design is produced before the tunnel begins and all changes are made from this firm foundation. A robust design is most easily produced as part of a risk-based approach (see Section 4.1.1).

Like SCL tunnels in hard rock, instrumentation is installed in and around a soft ground SCL tunnel to monitor its behaviour. However, its purpose is different. For soft ground tunnels with a robust, fully engineered design, the monitoring is to verify that the tunnels are behaving as predicted rather than to determine the support required. Design changes may still be made but in general these are within the boundaries of the original design. For example, advance lengths could be varied or optional additional support measures such as spiling used or omitted but the thickness of the lining and its reinforcement are not normally altered. In rock tunnels under high stresses, the monitoring often feeds into the design in a more active sense. For example, decisions on timing of the placement of the inner lining are sometimes made on the basis of rates of convergence of the lining. If more extreme conditions are encountered than those envisaged, parts of the tunnel may have to be redesigned.

The competence of the site team is critical to the success of SCL construction and it is useful to have a designer's representative on site (see Section 7.3).

4.1.1 Risk-based designs

The UK safety legislation places a duty on designers to identify hazards and avoid or mitigate the associated risks (i.e. reduce them until they are As Low As Reasonably Practical – ALARP). The systematic consideration of risks helps to lead to the production of a 'robust' or 'fully engineered design' in which all aspects (including temporary cases) are considered in detail before construction begins. These demands complement the needs of shallow, soft ground SCL tunnels since temporary cases are often more critical than the permanent case and the time between the onset of failure and total collapse of a tunnel can be very short. It is this lack of time that means the more observational approach traditionally adopted with the NATM is not appropriate for soft ground.

Risk-based design methods assist all parties to understand how construction, design and safety interact. Inevitably some residual risks will remain and they are communicated to the contractor via the residual risk register.

4.1.2 General loading

The lining will have to be designed to carry all loads that it is expected to carry (see BTS 2004 for detailed discussion). Each tunnel is subjected to its own unique set of loads. In addition to the ground and water loads, these might include compensation grouting, loads from internal fixtures (e.g. road signs, cable trays, jet fans) and accidental loads (e.g. fire or impact from train derailment).

4.2 Basic principles

The fundamental principles that underpin all tunnelling have been outlined in Section 1.2, so only the concluding comment will be repeated here:

> The art of tunnelling is to maintain as far as possible the inherent strength of the ground so that the amount of load carried by the structure is minimised.

Good design should incorporate these basic principles, such as *soil-structure interaction* and *'arching'* in the ground. The act of excavating the tunnel modifies the stress distribution in the ground. Figure 4.2 shows one illustration of this, using an analytical solution for a hole in an elastoplastic plate under stress. Introducing a hole into the plate converts a distribution of principal stress in the vertical and horizontal directions into one with high tangential stresses arching around the hole and a radial stress of zero at the edge of the hole. At points far from the hole the stress pattern is unaffected by it. By means of arching, a certain amount of the initial stresses are

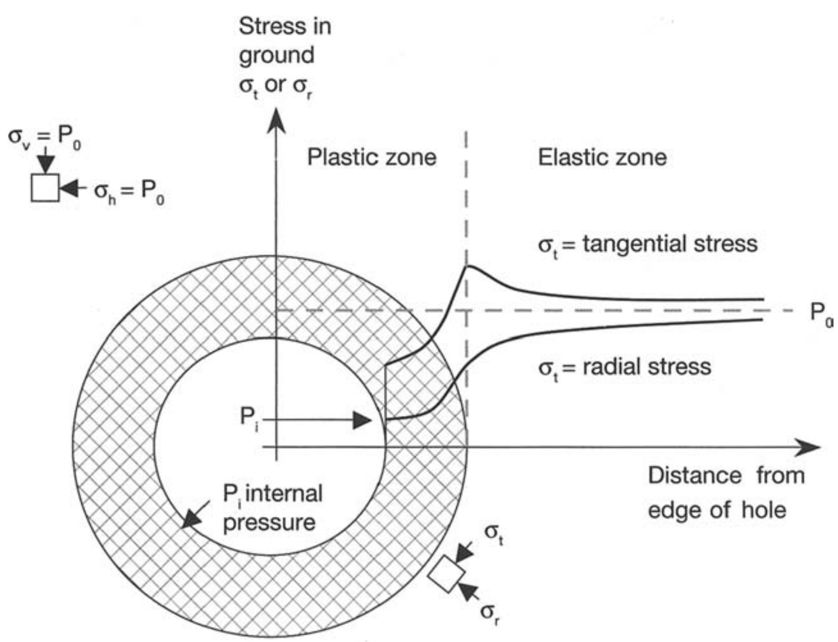


Figure 4.2 'Arching' of stresses around a hole in a stressed plate

redistributed around the tunnel, leaving the remainder to be borne by the lining (internal pressure, P_i). Hence deformation of the ground is inevitable and it must be controlled to permit a new state of equilibrium to be reached safely. The arching occurs in three dimensions and so adjacent excavations may interact. It is important for designers to be able to visualise – even if only in their own mind's eye – their tunnels in three-dimensional form.

The ground can be divided into three categories according to its behaviour:

- soft ground (soils and weak rocks);
- blocky rock;
- hard rock.

Tables 3.2, 3.3 and 3.4 list the typical modes of behaviour of each type of ground. Soft ground and hard rock behave like a continuum – i.e. as if the ground is a single mass. In soft ground immediate support is required, while in massive hard rock the ground is essentially self-supporting, except for a few isolated blocks. In contrast, blocky rock behaves like a discontinuum – i.e. as if the ground is a collection of discrete blocks. Sprayed concrete is used to support blocks, often in conjunction with rockbolts.

The fourth dimension – time – plays a major role in tunnelling. The concept of 'stand-up' time was recognised long before it was formally articulated by Lauffer and others (see Bieniawski 1984). *Stand-up time* is a measure of how long the ground can stand unsupported. If this is less than the time required to install the sprayed concrete lining, additional measures will be required (e.g. ground improvement). Certain types of ground also exhibit *time-dependent behaviour* such as creep. Time appears as a factor in the support too. Support is usually installed at a finite time after the ground has been excavated, ranging from minutes to days. The support may also be built in several stages. Certain support measures, such as sprayed concrete, possess time-dependent behaviour. Time-related aspects of the ground and the support interact due to the progressive nature of tunnelling. For example, each excavation stage throws load onto the part of the tunnel lining nearest the face but, as the face advances, the zone of influence shifts with the tunnel.

This last point highlights another phenomenon, namely that stiffer structures can actually attract load. Installing a heavy lining may not end up being a safer option. Firstly, it is slower to construct and, secondly, it deforms less and the ground tends to arch preferentially onto the stiff lining. Both can lead to higher loads on the lining.

Finally, all good design should embody the principle of *constructability*. In other words, the structure should be easy and safe to construct. Good constructability will reduce mistakes during construction and improve cost-effectiveness. For this reason designers should always keep abreast of current construction techniques and innovations.

4.2.1 Ground loads

Due to the phenomenon of arching in the ground, tunnel linings are rarely subjected to the original *in situ* ground stresses. In most cases it is overly conservative therefore to design the linings to carry the overburden pressure (i.e. the dead weight of the ground above) or the original *in situ* horizontal stresses, with the normal partial safety factor of 1.4 applied. Two more realistic approaches can be used to estimate the ground loads.

Firstly, the loads can be estimated from precedents (e.g. see Table 4.1). The standard partial load factor is then applied to these loads. Unfortunately there is not an extensive database of information on lining loads in different types of ground.

Secondly, where there is inadequate experience of tunnelling particular ground conditions, the ground loads can be estimated from using analytical or numerical methods (see also Section 4.3). A relaxation factor can be used (in the Convergence Confinement Method and numerical modelling)

Table 4.1 Sample long-term ground loads and relaxation factors for shallow tunnels

Ground	*Long-term load (as % of full overburden pressure)*	*Relaxation factor, λ (%)*	*Source*
London clay	50–60	50	Jones 2005
Completely decomposed granite (~ silty sand)	est. 40–60[a]	75	Equivalent to volume loss of 0.2% – Private communication
Conglomerate	–	50	Private communication
Sandstone	–	50–60	Private communication
Chalk Marl	–	50	Hawley and Pöttler 1991
Chalk	50+	25	Watson *et al.* 1999
Limestone	–	50–70	Private communication
Rock	See Table 4.2	Up to 100	

Note:
a Although a literature search yielded no data on SCL tunnels, data from pressure cells in segmentally lined tunnels in granular material suggest values of 40 to 60% overburden and as starting point one could assume that the loads on SCL tunnels are similar.

Table 4.2 Analytical solutions for estimating ground loads

Ground type	*Solution*	*Source*
Soft ground (soils)	Curtis–Muir Wood	Muir Wood 1975, Curtis 1976
Soft ground (soils)	Einstein–Schwartz	Einstein and Schwartz 1979
Soft ground (soils and rock)	CCM	Panet and Guenot 1982
Blocky rock	Protodykianov	Szechy 1973
Hard or blocky rock	P arch	Grimstad and Barton 1993

to simulate the beneficial effects of arching. The lining is only introduced in the model after the initial stresses have been relaxed according to the factor. Clearly the choice of this factor has a large influence on the final loads on the lining. Table 4.1 includes some typical values. These values are an initial guide only as the amount of relaxation depends on the constitutive model and the construction sequence. More recent work – modelling a multi-stage excavation sequence with a sophisticated strain-softening model for an overconsolidated clay – has shown that several stages of relaxation should be used to replicate better the ground movements and development of load on the tunnel that are observed on site.

In a similar manner Muir Wood (1975) recommended that only a fraction of the loads predicted by his analytical solution are applied to the lining. For the specific case of London Clay he proposed a value of 50%.

The experience in high stress rock tunnels and simple analytical models based on stress relaxation have led to the view that, the more the ground is permitted to relax, the lower the ultimate load on the lining will be. This has become enshrined in the NATM philosophy. However, this does not appear to hold true for SCL tunnels in soft ground. In fact the more the ground relaxes the higher the load will be due to strain-softening in the ground and the generation of negative pore pressures which later dissipate (Jones 2005).

For blocky rock, programs such as UNWEDGE can be used to calculate sizes of typical blocks, based on the pattern of joints. Once again the standard partial load factor is then applied to these loads. Alternatively, in numerical modelling, partial safety factors can be applied to soil parameters in line with the procedure described in Eurocode 7 (2004). The numerical model then provides 'factored' loads directly. However, it should be noted that care must be taken when factoring ground parameters to ensure that this does not produce a misleading simulation of the ground behaviour. For example, a reduction in shear strength could lead to much more plastic yielding and deformation than might reasonably be expected. For this reason designers often avoid using Eurocode 7 for tunnels.

4.2.2 *Excavation and support sequence*

The way that a SCL tunnel is constructed has a large bearing on final stresses and the amount of surface settlement. The age of the material at loading is the main reason for the difference in behaviour of sprayed concrete and conventional concrete in tunnel linings. An increasing but variable load is applied to the sprayed concrete from the moment that it is sprayed. The loading of sprayed concrete at an early age means that creep may be important and the material may be 'overstressed' – i.e. loaded to a high percentage of its strength, leading to pronounced nonlinear behaviour and possibly long-term damage to the microstructure. Since the properties of sprayed concrete change considerably with age during the early life of a SCL tunnel, the response of the tunnel lining to loading varies, depending on when the load is applied.

The key elements of the construction sequence are outlined below:

- *Excavation sequence:* A tunnel is often subdivided into headings. The headings advance is a sequence, with each heading increasing the size of the tunnel until the full section is formed. Typical sequences include: top heading, bench and invert; pilot and enlargement; side gallery and enlargement; and twin side galleries and central pillar. The choice of sequence is governed by the overall stability of the temporary headings.
- *Subdivision:* Subdivision of the face of a heading is used to control stability of the face along with measures such as a sealing coat of sprayed concrete.
- *Ring closure:* The key to controlling ground movements in soft ground tunnelling is the formation of a closed structural ring. While the ring is open, ground movements into the tunnel continue but ring closure brings them to a halt.
- *Advance length:* Also called 'round length' in drill and blast tunnels, the advance length is the distance of a heading that is excavated in a single stage.
- *Adjacent excavation:* Because the stresses in the ground arch around a tunnel heading, the adjacent ground experiences an additional load. Any structure in that zone of influence must be designed to cope with that loading.

The influences on the choice of excavation and support sequence have been outlined in Section 3. The main ones are listed below:

- stand-up time (and stability of the ground overall);
- face stability.

In addition, the construction team will have its own preferences on sequence, depending on the construction programme and the equipment available.

The stability of the ground at the face governs the size of the faces in each heading. Various analytical tools can be used to help estimate this, e.g. N – stability number for cohesive materials (Mair 1993); see Leca and Dormieux 1990 for cohesionless ground.

In certain cases it may be advisable to delay installing support (e.g. swelling or squeezing rock) but in general, and especially in soft ground, it is better to install the tunnel lining sooner rather than later. This will minimise the risk of the ground around the tunnel deforming excessively or weakening.

4.2.3 Water

Groundwater can have two influences on the design of a SCL tunnel. Firstly, its effects on the behaviour of the ground and, secondly, waterproofing measures may be required, depending on the purpose of the tunnel.

Table 4.3 Design approaches for waterproofing

Drained	*Partially tanked*	*Fully tanked*
Water inflow is acceptable	Minor water inflow is acceptable	Water inflow is unacceptable
Class 1 or 2 * inside final lining	Class 1 or 2 * inside final lining	Class 1 or 2
Class 3 or 4 * if no internal lining $Q < 0.02$ or $0.02 < Q < 0.1$ l/m^2 Over 10 m	$0.02 < Q < 0.1$ or $0.1 < Q < 0.2$ l/m^2 Over 10 m	$Q < 0.02$ or $0.02 < Q < 0.1$ l/m^2 Over 10 m
Any tunnel under high water pressure or water tunnels	Public areas, metro tunnels, shallow road or rail tunnels	Public areas, metro tunnels, shallow road or rail tunnels
Low or high water pressure < 1 bar or > 5–8 bar	Low to medium water pressure < 5–8 bar	Low to medium water pressure < 5–8 bar
Permeable or impermeable ground	Impermeable ground	Permeable ground
Design for seepage pressures only e.g. San Diego MVE – Thomas *et al.* 2003	The lining may be designed for full water pressure. e.g. Channel Tunnel UK Crossover – Hawley and Pöttler 1991	The lining must be designed for full water pressure. e.g. Linkou Tunnel, Taiwan high speed railway

Apart from the immediate effect of water on the strength of the ground and its stability, in the longer term water may also impose loads on the lining. If the tunnel acts as a drain it may be loaded by seepage forces. In stiff overconsolidated clays, the equalisation of negative pore pressures, that have been generated around the tunnel, will add load to the lining, although in most cases this increase will be small and actually tend to even out the loading on the lining (Jones 2005).

Table 4.3 shows the ranges of application of the different design approaches for waterproofing.

The effects of groundwater (either as a direct load or its influence on the stability) can be accounted for in design analyses when determining ground loads (see Section 4.2.1). If the tunnel has an impermeable lining it will be subject to uplift forces.

As for waterproofing, a wide variety of measures can be taken to keep water out of a tunnel and the choice depends on the hydrogeology, the purpose of the tunnel and the permissible water ingress. The ITA has produced guidelines on the impermeability of tunnel linings, depending on the use of the tunnel. Figure 4.3 shows solutions to achieve the categories for dry tunnels such as public areas in stations or heavily used road tunnels. However, as the external water pressure increases (e.g. at around 50 to 80 m of head), it may be more economic to permit a small controlled water ingress, rather than to exclude it completely. This reduces the load on the lining.

Normally the purpose of the waterproofing is to keep groundwater out but sometimes it is to keep the fluid in the tunnel from leaking out (e.g. water transfer tunnels). However, it should be noted that the approach to waterproofing also varies widely between different countries and different sectors, depending on the client's budget and local design practice. For example, the limit on water ingress in a subsea road in Norway is 14 l/m^2 per day (over a 10 m length), which is far above ITA Class 5, whereas a similar road tunnel in the UK would have to meet Class 2. The difference arises in part from the lower number of tunnel users but partly also from national preferences. Waterproofing is a notoriously difficult part of construction and 100% success cannot be guaranteed even with expensive 'fully tanked' membrane systems. One small flaw can compromise the whole system. The impermeability of the sprayed concrete is only one element of the whole waterproofing system for a tunnel. Joints in the lining or waterproofing system itself and between the tunnel and connected structures, such as shafts, represent weak points and extra care is required to seal them against water ingress.

Waterproofing represents a huge topic in its own right. Further information can be found in BTS Lining Design Guide (BTS 2004). Some notes have only been included below to explain the impact of waterproofing measures on the sprayed concrete lining.

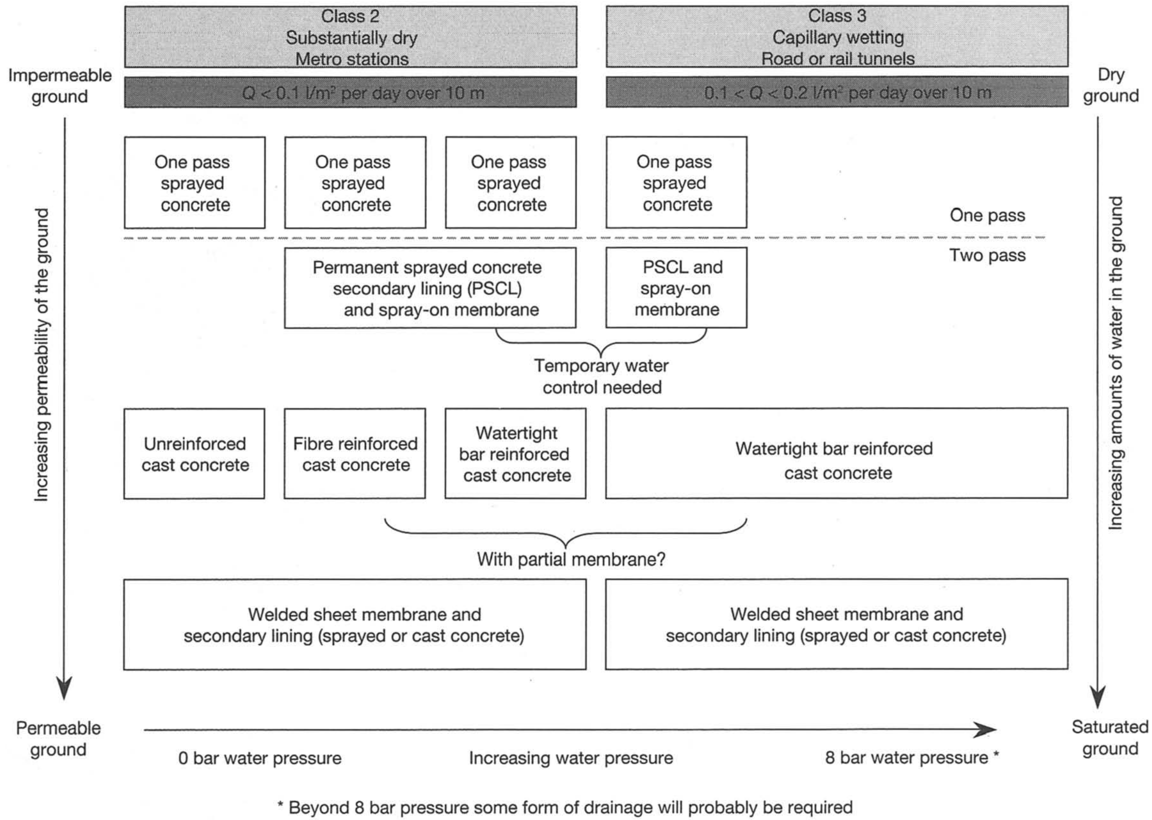


Figure 4.3 Options for 'undrained' solution to achieve a dry tunnel

Controlled inflow (via drainage holes)

A 'drained' solution is often adopted where complete watertightness is not required (e.g. in non-sensitive tunnels which are not open to the public), where the water ingress is transient and low in volume or where the water pressures are so high that it would not be economic to exclude the water.

Figure 4.4 shows a typical arrangement of a drained solution. Small diameter plastic pipes (e.g. 50 mm) with slots cut in them are often inserted into the holes to keep the inflow path open. The pipes can be wrapped in geotextile to prevent fines from the ground being washed in. The inflowing water is then directed into drains running in the invert of the tunnel. Unfortunately salts such as calcium hydroxide can be leached out of the ground and the sprayed concrete lining and then deposited in the drains (Eichler 1994). In the worst case the drains can become completely blocked. Hence in some cases where a drained solution is adopted, the tunnel lining is checked against the load case of a full hydrostatic head (i.e. assuming the drains become blocked) – for example, the Channel Tunnel UK Crossover cavern.

Drainage holes can also be used during construction to control inflow. Instead of the water seeping in through the whole lining, it becomes concentrated at the holes and so it is easier to handle. It may also be possible to draw down the water table locally which reduces the inflow pressure.

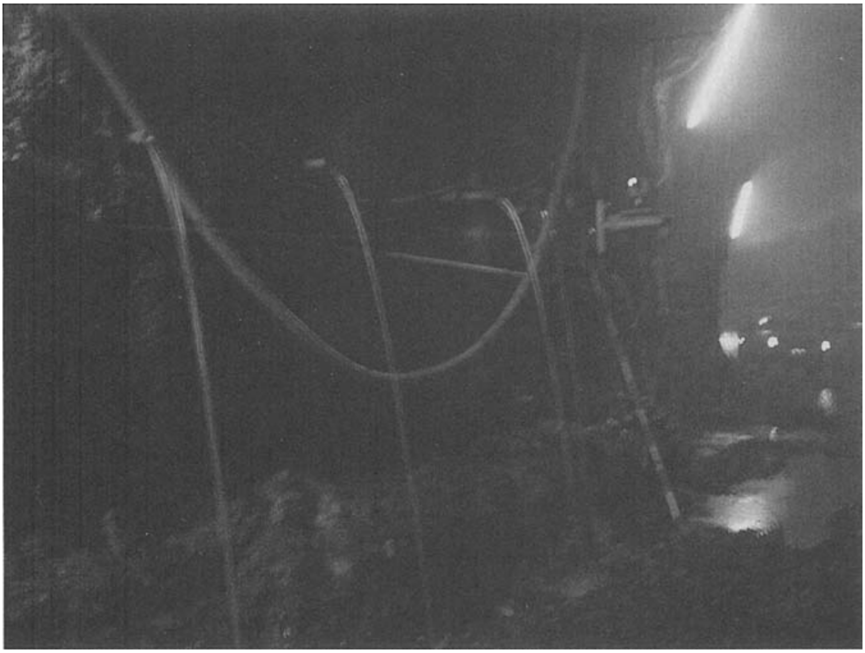


Figure 4.4 Drainage pipes

Sheet membranes

In the past sprayed concrete on its own has not been considered watertight and additional measures have been required. Also, concerns over the durability of sprayed concrete encouraged designers to ignore the primary lining once the secondary lining had been installed. A common solution is to install a sheet membrane inside the primary sprayed concrete lining. In this so-called 'fully tanked' or 'partially tanked' solution, a secondary lining is installed inside the membrane to carry the water load and normally the entire ground load in the long term. Even if the primary lining is included in the long-term design, because the membrane introduces a frictionless interface between the two linings, it must be assumed that no composite structural action occurs. Adopting this approach has a major impact on the design of the lining (see Section 4.2.4) but the sheet membrane itself too has an influence on the sprayed concrete lining.

Typically the surface of a SCL tunnel will have to be prepared before a sheet membrane is installed to prevent it from being punctured. Sometimes the drainage fleece can serve the additional purpose of a protective layer. If there are sudden changes in profile or deep depressions in the surface of the lining there is a risk of over-stretching the membrane during concreting. To avoid this, smoothness criteria are set (e.g. see Table 4.4).

Table 4.4 An example of criteria for smoothness of SCL tunnels

Parameter		*Limit*
D/L – as shown in Figure 4.5 (over a 3 m reference length)	temporary linings permanent linings	less than 1:5 less than 1:20
Transitions and intersections of underground structures shall be rounded off with a minimum internal radius of the minimum thickness of the final finishing layer		500 mm
The radius of curvature of the finishing surface over protruding steel parts such as rockbolts		25 mm
All protruding steel shall be cut flush with the surface unless treated with additional shotcrete		greater than 200 mm

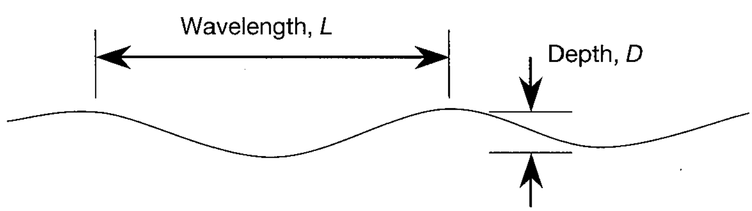


Figure 4.5 Depth to wavelength criterion for smoothness

Various types of waterproofing sheet membranes are available (see BTS 2004) and their installation is a highly specialised job. Temporary drainage measures may be required during the installation before the secondary lining is in place to resist the water pressure.

In a 'fully tanked' solution the membrane is designed to form a completely impermeable barrier to water inflow (or outflow). In a 'partially tanked' solution the membrane does not extend around the full circumference of the tunnel. It terminates at either side of the invert in a drain and it is combined with a geotextile fleece to transmit the water to the drains. The advantage of this is that the inner lining does not need to be designed to withstand the full hydrostatic pressure. On the downside measures must be taken to prevent sintering (see the section on controlled inflow on page 90).

Even in a 'fully tanked' solution it is prudent to install some drainage in the tunnel as it is almost inevitable that there will be a leak somewhere in the membrane.

Spray-on membranes

These products are a relatively new innovation and are usually aimed at providing a more cost-effective solution than sheet membranes in situations with low hydrostatic pressures or complex geometries. A key advantage is

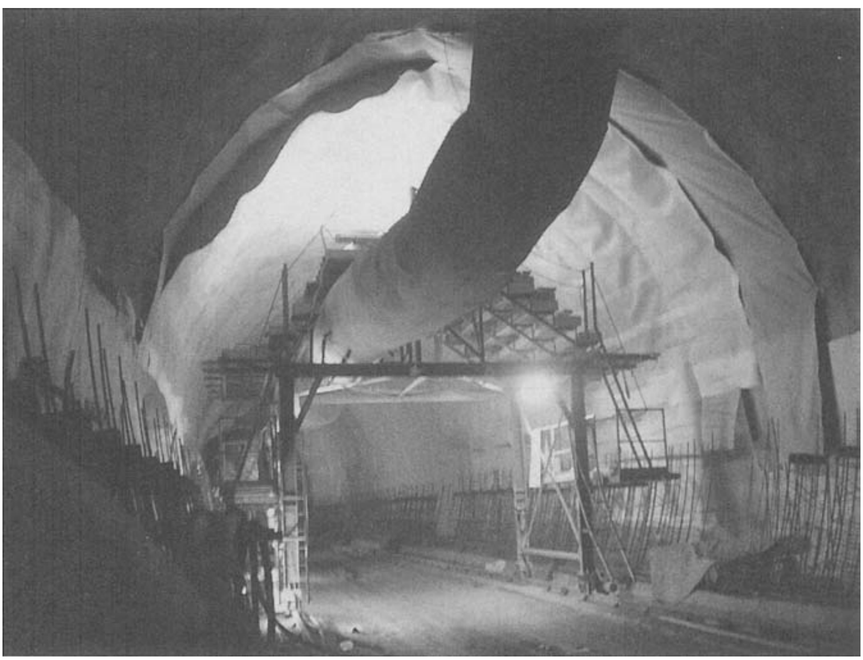


Figure 4.6 Sheet membrane installation

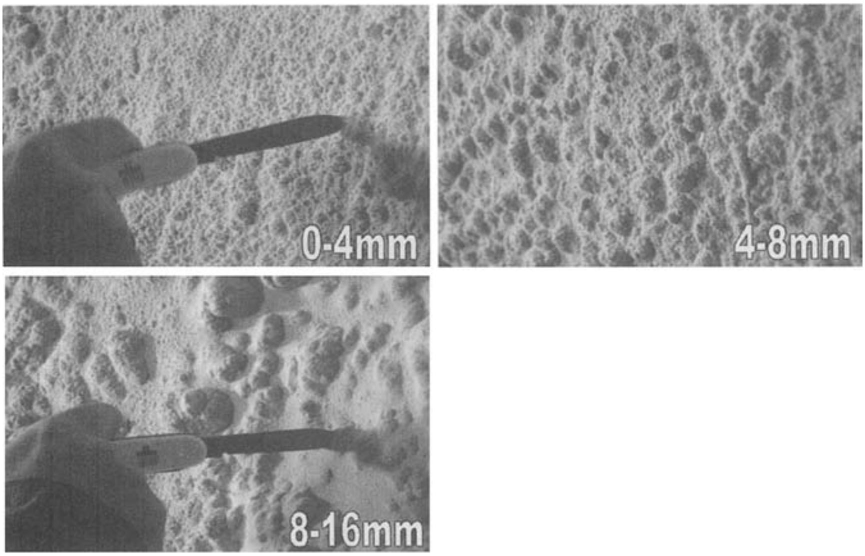


Figure 4.7 Sprayed concrete surfaces covered by MS 345 with different maximum sizes of aggregate (courtesy of BASF)

Figure 4.8 Spray-on membrane installation (courtesy of BASF)

the ease of application, using simple equipment that is easy and quick to operate. Depending on the bond across the membrane it may be possible to use spray-on membranes between layers of a 'single shell' lining (see Section 4.2.5).

Good surface preparation is critical to the successful application of a spray-on membrane. The manufacturers provide detailed guidance on this. As with sheet membranes, particular care is required when forming joints although, in the case of a spray-on membrane, the joints are simply formed by overlapping the adjacent section onto the previous one. The surface must not be too rough or else there will be a high consumption of the expensive membrane material and flaws (such as pinholes where the membrane fails to bridge across a deep depression in the concrete surface). To avoid this, the sprayed concrete should be a U1 finish or better (see Table 3.8), or a smoothing layer of gunite should be applied. Figure 4.7 shows how the as-sprayed surface roughness can vary depending on the maximum size of aggregate in the sprayed concrete.

A disadvantage of spray-on membranes is that – like sprayed concrete itself – the product is created *in situ* in the tunnel and hence is vulnerable to environmental influences (such as low temperatures which can harm the curing process) and poor workmanship.

4.2.4 *Primary plus secondary lining – two pass lining*

Secondary linings are normally formed of cast *in situ* concrete, although sprayed concrete is increasingly used, especially where the cost of formwork is high (e.g. at junctions or tunnels of varying shape). Since the secondary lining is placed inside a waterproof membrane, it is typically designed to carry the water loading plus most or all of the ground loads (see Sheet membranes above).

Cast concrete linings are placed within formwork (see Figure 4.9) and normal concrete technology is applied. The type of formwork depends on the specific requirements of each tunnel. Mobile steel formworks are used for longer tunnels with a constant cross-section. Timber formwork is cheaper in terms of materials but more labour intensive. Hence it is used in countries where wage costs are low or for special cases, such as junctions, where it is not cost-effective to buy steel formwork.

Ideally inner linings are designed to be unreinforced concrete. The shape of the tunnel can be chosen to minimise bending moments and, depending on the compressive hoop load in the tunnel, they may be low enough to be safely within the capacity of plain concrete. The risks of cracking due to thermal or shrinkage effects can be reduced by good mix design (e.g. using cement replacements like pulverised fly ash (PFA) to slow the hydration process) and casting the lining in short lengths (e.g. less than 10 m long bays). If there is no waterproofing membrane, it is sometimes advisable to install a plastic separation sheet to reduce friction on the contact with the primary lining. Again this reduces the risk of cracking.

Where lining loads are high, reinforcement is added to the secondary lining.

Permanent sprayed concrete

Alternatively the inner lining can be formed of sprayed concrete. Sprayed concrete can be produced with acceptable durability characteristics (i.e. equal to that of *in situ* concrete, as indicated by its permeability and porosity values (Neville 1995, Palermo and Helene 1998, Norris 1999)), although this increases the unit cost of the material. To be permanent, this sprayed concrete must be durable enough to last for the design life of the tunnel. Sprayed concrete is still not widely used as part of the permanent works (at least in public tunnels) (Golser and Kienberger 1997, Watson *et al.* 1999) and no clear performance specification has been established (see also Section 2.2.9). In general terms, the strength should not degrade over time and the

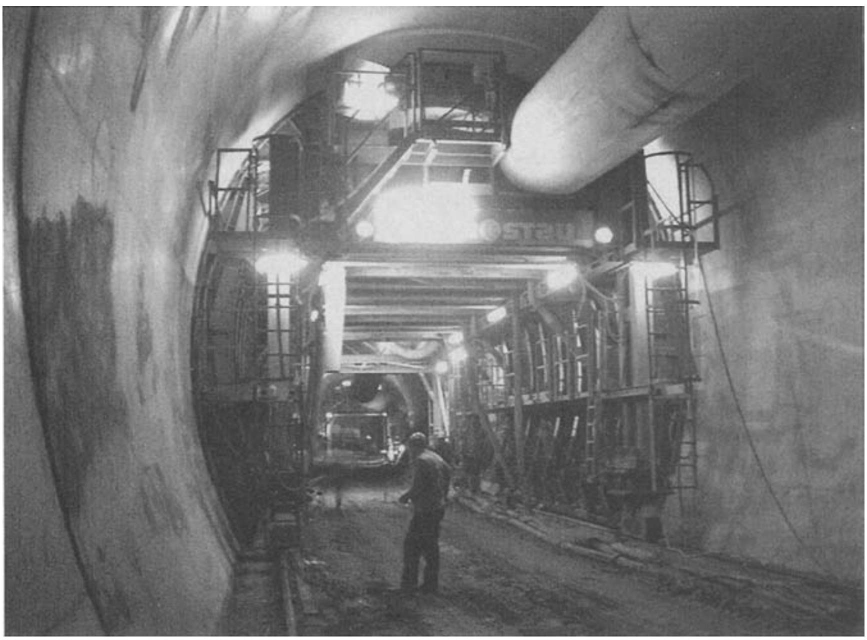


Figure 4.9
Formwork for cast concrete secondary lining

concrete should be dense and have a low permeability for water. The latter criteria are aimed to reduce the potential for water ingress through the body of the lining and corrosion of reinforcement within the lining. Table 4.5 contains typical requirements specified for a 'permanent' sprayed concrete to achieve those basic criteria. There has been some concern that the loading experienced by the sprayed concrete at an early age may damage its long-term strength. This is more relevant to 'single shell' linings (see Section 4.2.5) but in any case the normal compressive strength testing of the *in situ* sprayed concrete should detect any significant damage. Hence a higher specification is required than for sprayed concrete used in temporary works. Table 2.1 contains an example of mix design. Higher standards of workmanship hand in hand with greater quality control are necessary. Occasionally curing is applied but as noted in Section 3 this introduces an additional activity in the tunnel which the construction team prefers to avoid.

Accounts of pioneering projects in the field of permanent sprayed concrete linings can be found in Gebauer (1990), Kusterle (1992), Arnold and Neumann (1995), Darby and Leggett (1997), Zangerle (1998), Palermo and Helene (1998).

Clearly installing a secondary lining and ignoring the primary lining costs more, both in time and money, than a lining which uses all the concrete sprayed as part of the permanent lining. Hence attempts have been made to improve upon this simple approach.

Table 4.5 Typical design requirements for permanent sprayed concrete linings

Parameter	*Value*	*Source/comments*
Max. water/binder ratio	0.45	–
Min. cement content	400 kg/m^3	Good to add 5–10% of microsilica (bwc)
Min. compressive strength	Depends on lining loads – typically 30–40 MPa	No reduction with time after 28 days
Max. accelerator dosage	Keep as low as possible	–
Water permeability	$\leqslant 10^{-12}$ m/s	Darby and Leggett 1997
Water penetration	$\leqslant$50 mm	Lukas *et al.* 1998
Max. crack width	0.4 mm	ÖBV 1998
Curing period	7 days	By spraying with water
Bond between layers of concrete	1 MPa	EFNARC 1996

Table 4.6 Design approaches for 'grey rock' – degraded sprayed concrete

Approach	*Values*
50% degradation[a]	E = 10 GPa, c = 2 MPa and Φ = 45°
Full degradation[a]	E = 0.1 GPa and zero strength
Channel Tunnel	c = 0 MPa and Φ = 30°; stiffness not stated
Terminal 5 project, UK	Ignore outer 75 mm of lining thickness but the remainder retains full strength; E = 15 GPa for long term in line with Eurocode 2

Note:
a Presented by M. John at Nove Trendy v Navhovani Tunelu II seminar, Prague 2006.

'Grey rock'

In the past it has been assumed that the initial concrete sprayed is of low quality and that in the long term it will degrade to a cohesionless gravel. There is no evidence that modern good quality sprayed concrete exhibits such a drastic deterioration but nevertheless a project may insist on this assumption. An alternative to ignoring the primary lining completely in the long-term design is to recognise that it remains in place and to ascribe the mechanical properties of a gravel to it in the design calculations. This is the so-called 'grey rock' design philosophy. In practice this approach is only used in numerical analyses as it is too complex to incorporate into analytical solutions. The benefit in terms of the reduction in axial loads on the secondary lining is often very small but it may help because high 'predicted' bending moments in the primary lining may disappear when it is turned into gravel in the analysis. Given the limited benefit, engineers may prefer to spend their energies demonstrating that the primary lining sprayed concrete is durable in the long term rather than juggling with 'grey rock' parameters in numerical analyses. Table 4.6 contains some examples of these parameters.

4.2.5 Single shell lining – one pass lining

Considerable cost savings are possible if the concrete, sprayed as the initial ground support, can be included in the permanent lining (the so-called 'single shell' or 'monocoque' approach) (Golser and Kienberger 1997). Permanent sprayed concrete linings may be formed in several ways and in actual fact the 'single shell' may consist of several layers of sprayed concrete, placed at different times. However, the underlying principle is that all the sprayed concrete carries load over the life of the tunnel and the different layers normally act together as a composite structure. This approach is common in certain sectors – notably on hydroelectric power projects – and in certain

Table 4.7 Examples of single shell SCL tunnels

Project	*Type of tunnel*	*Reference*
Munich sewer	Sewer	Gebauer 1990
Munich metro	Metro	Kusterle 1992
Heathrow Baggage Transfer tunnel	Non-public	Grose and Eddie 1996
Heathrow Terminal 5 'Lasershell' method	Water, road and rail tunnels	Hilar *et al.* 2005

ground conditions – such as dry hard rock. More recently it has been extended to water-bearing soft ground situations and transport tunnels. Table 4.7 contains a short list of some examples. A much more comprehensive listing of more than 150 tunnels of all types and from all parts of the world can be found in Franzen *et al.* (2001).

There are normally two questions that must be answered before a project accepts the use of a single shell lining:

1 Is the sprayed concrete durable?
2 Is the lining sufficiently watertight?

Modern good quality sprayed concrete is a durable material (see the previous section for further discussion). Poor workmanship, water inflow during construction or excessive loading at an early age are the only real risks to durability of the concrete. Corrosion of reinforcement steel embedded in the sprayed concrete presents the main residual risk. One way to remove this is to use steel fibre reinforcement. However, there may still be cases such as junctions where heavier, steel bar reinforcement is required. In those cases good workmanship should ensure that the steel is safely encased in dense concrete with a low permeability – just as in cast concrete.

Considering watertightness, Figure 4.10 shows that, if one considers a simple calculation of water inflow into a tunnel under various conditions (see Equation 4.1, after Celestino 2005, Franzen 2005), single shell linings can only be used in relatively impermeable ground if the tunnel is to exceed ITA Class 3 (the minimum criterion for a railway running tunnel). The mass permeability of the ground would have to be less than 1×10^{-10} m/s. For example, single shell linings have been used successfully in London Clay which has a permeability in the range of 5×10^{-11} to 5×10^{-12} m/s.

Figure 4.10 assumes that single shell linings (as a whole) can only achieve an impermeability that is about one order of magnitude less than the permeability of individual samples of sprayed concrete, i.e. 1×10^{-10} m/s to 1×10^{-11} m/s is achievable (see Figure 2.22 and Celestino 2005). That said, there is always the option of improving the permeability of the ground (e.g. by systematic grouting (Franzen 2005)):

$$Q = \frac{2\pi k_1 (h - r)}{\ln\left(\left(\frac{r+t}{r}\right) + \left(\xi \frac{k_1}{k_g}\right)\right)} \tag{4.1}$$

where Q is inflow in m^3/s per m of tunnel, k_l and k_g are the permeabilities of the lining and ground, h is the depth below groundwater level, t is the thickness of the lining and ξ is the 'skin factor'.

It is also worth noting at this point that there is some debate about the acceptable levels of water ingress. The limits proposed by ITA (1993) and shown on Figure 4.10 may be too tight. Franzen (2005) noted that a single dripping spot per m^2 in a tunnel could result in about 1.5 l/m^2 per day in 10 m of tunnel while Celestino (2005) quoted 1 l/m^2 per day as an acceptable value for metro tunnels and 14 l/m^2 per day as the accepted limit in Norwegian subsea road tunnels (inferred as 0.1 and 1.4 l/m^2 per day measured over 10 m of tunnel respectively).

The impermeability of the lining is important with respect to achieving the specified dryness of the tunnel but also in preventing corrosion of reinforcement. The biggest challenges for a single shell lining lie in the joints. Although the permeability of high quality sprayed concrete can be as low as 10^{-12} m/s or even lower (see Figure 2.22), the permeability of the lining as a whole is probably closer to 10^{-10} m/s due to inflow at joints.

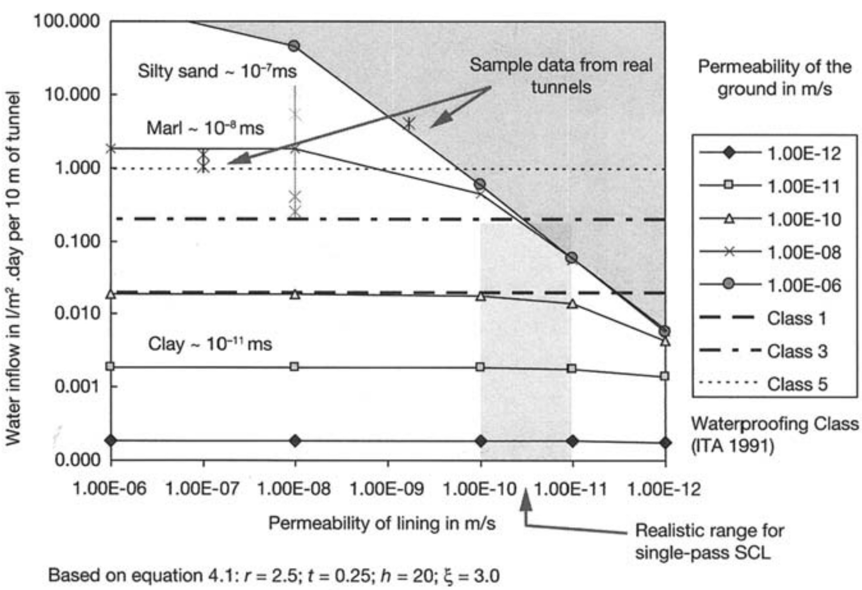


Figure 4.10 Water inflows into tunnels of various lining and ground permeabilities

Minimising the number of joints reduces the potential for inflow (e.g. by using a full face excavation sequence rather than top heading and invert – if possible) but the scope for this is limited. Because of the large number of joints in the initial support (advance lengths during excavation are between 1 and 4 m), it is more sensible to ensure a good bond between adjacent panels of sprayed concrete than to install seals. The integrity of individual joints depends on achieving a good bond. Good spraying techniques are sufficient to ensure this. Sloping joints are sometimes preferred for ease of cleaning and compaction during spraying. They also have the merit of creating long and tortuous water paths. Staggering joints between the initial support and subsequent layers of concrete helps by extending the water path for incoming groundwater.

Suitable mix designs are discussed above and in Section 2.1. Microsilica is often added because it fills pores and improves the density of the concrete.

At major joints – for example, tunnel junctions – it may be necessary to install additional protection. Given their complicated shapes it is almost impossible to spray around conventional water-stops and encase them in concrete. Therefore injectable grout tubes (see Figure 4.11) are preferred for sealing major construction joints where differential movement may occur.

In terms of the design calculations for a single shell the main difference arises from the composite action of the various layers of sprayed concrete. Few case studies exist to illustrate this aspect of the design. However, it appears that a modest bond strength (~ 0.5 MPa) is required in the radial direction between layers to permit composite action (Kupfer and Kupfer 1990). This is well within the achievable bond strengths for sprayed concrete (see Section 2.2.3). Having checked the integrity of the bond between layers,

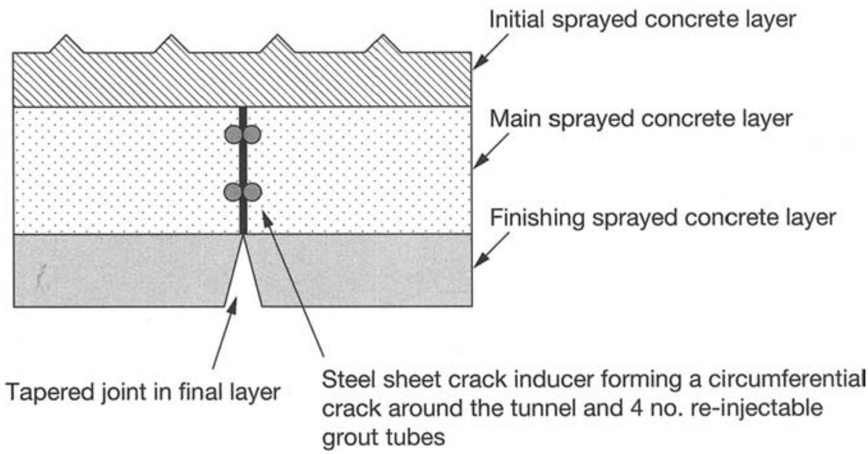


Figure 4.11 Joint detail in sprayed concrete lining

the full thickness of the sprayed concrete lining can then be used to carry the loads on it.

4.3 Design tools

A range of design tools exists for designers as outlined below. A more comprehensive discussion of the strengths, weaknesses and ranges of application of empirical, analytical, numerical and physical modelling tools for tunnel design can be found in the BTS Lining Design Guide (BTS 2004).

It is important to understand that all design tools are approximations of reality. The design tool simplifies the real case, analyses the simplified case and produces an answer – which is an estimation of how the real tunnel will behave. In very simple or well-understood cases, the estimation produced by the design tool may be identical to the real behaviour. However, generally this is not the case and hence factors of safety are applied to the results of design calculations to ensure that the risk of tunnel failing to meet the specified design criteria (for both Ultimate and Serviceability Limit States) is reduced to an acceptable level (e.g. 1 in 1,000,000).

In principle, there are six main sources of errors in modelling (after Woods and Clayton 1993):

- modelling the geometry of the problem;
- modelling of construction method and its effects;
- constitutive modelling and parameter selection;
- theoretical basis of the solution method;
- interpretation of results;
- human error

and these should be borne in mind when choosing the design tools for a particular tunnel.

4.3.1 Empirical tools

Commonly used empirical design methods like the RMR system (Bieniawski 1984) and the Q-system (Barton *et al.* 1975) have been developed for blocky or hard rock tunnels. They are quick and easy to use, at least for those with experience in estimating the input parameters. These methods employ a combination of parameters such as the strength of the rock, its quality (using Rock Quality Designation – RQD values), joints (number of sets, frequency, spacing and condition) and groundwater conditions to produce a rock mass classification. From design charts or tables, the support measures required are quantified based on the product of these parameters.

However, there are limitations to these tools. For example, they are based on drill and blast tunnels. In TBM-driven tunnels the rock is less disturbed by excavation and so requires less support than might be predicted. On the other hand there may be particular requirements for a project that mean more support is required than is suggested. These methods, and others, are reviewed in Hoek and Brown (1980) and Hoek *et al.* (1998).

Empirical methods for soft ground tunnels are rarely used now.

4.3.2 Analytical tools

These include continuum 'closed-form solution' models (e.g. Curtis and Muir Wood) and Panet's Convergence Confinement Method (CCM) – see also Table 4.2. The continuum analytical methods are relatively simple and provide information on stresses in the lining and its deformation. These are often used in the early stages of design. Some of them may be extended to include features such as plasticity in the ground or the timing of placement of the lining.

However, they share several fundamental limitations: they assume plane strain or axisymmetry and the solutions are almost invariably developed only for circular tunnels, constructed in full-face excavation in homogeneous ground. This is a major weakness from the point of view of designing SCL tunnels. Furthermore, the modelling of soil-structure interaction is limited yet this is fundamental to all tunnels. Many closed form solutions in their basic forms make no allowance for stress redistribution ahead of the face, but sometimes this can be incorporated using a relaxation factor (see Table 4.1).

For the case of blocky rock, limit-equilibrium calculations can be used to calculate the bolting required to support individual wedges. Typical factors of safety are 1.5 for grouted and 2.0 for ungrouted rockbolts (Hoek and Brown 1980). However, in practice these are performed by computer rather than by hand (e.g. using programs such as UNWEDGE). Similar calculations for a variety of failure mechanisms can be performed to determine the required thickness of sprayed concrete between rockbolts in rock support (Barrett and McCreath 1995). The governing failure mechanism has been found to be generally a loss of adhesion to the rock, followed by flexural failure of the sprayed concrete (Barrett and McCreath 1995).

4.3.3 Numerical modelling

To overcome the limitations of empirical and analytical tools, one must turn to numerical methods, such as the finite element (FE) and finite difference (FD) methods, which can model the full complexity of a SCL tunnel explicitly. Ideally the design calculation will consider the following:

- the construction sequence and the three-dimensional stress redistribution around the tunnel face;
- the age- and time-dependent behaviour of the sprayed concrete;
- nonlinear material behaviour of the sprayed concrete and the ground.[1]

Despite their advantages, numerical models are still mere approximations of reality and it is important to understand the limitations of the design models. Sound engineering judgement remains the key to good design. The more simple design tools still have a vital role to play as a check on the results from numerical models. Where possible the models should also be calibrated with data from site. Further advice on numerical modelling can be found in the BTS guide (2004). In the sections below comments have been made on some of the main sources of errors in numerical modelling and how to avoid them.

Modelling the geometry of the problem

The most obvious question in the numerical modelling of tunnels is whether to use 2D or 3D analyses. The value of using 3D models and avoiding such correction factors has been increasingly recognised (Haugeneder *et al.* 1990, Hafez 1995, Yin 1996, Burd *et al.* 2000, Thomas 2003, Jones 2007), not least because the situation becomes increasingly difficult as more complex models are analysed (such as cross-sections with multiple headings or coupled consolidations analyses (e.g. Abu-Krisha 1998)). While 3D models are still too time-consuming for general use, they are being used on projects to determine input parameters (i.e. the relaxation factor) for 2D analyses or in complex cases such as junctions.

As an aside, a longitudinal plane strain model of a tunnel is not valid because this represents an infinitely wide slot, cut through the ground. Similarly, for shallow tunnels (i.e. C/D ~ 2), axisymmetry is not a valid assumption (Rowe and Lee (1992)).

Where there are adjacent structures (both above and below ground) or features such as slopes, it may be important to model these too.

Modelling of construction method and its effects

SCL tunnels are built in a sequential process that often involves a sub-division of the tunnel face. This should be modelled so that the stability of these intermediate stages can be checked. Failure usually occurs at one of these stages rather than after the full tunnel lining has been installed. A fuller discussion can be found in Section 5.8. All elements of the support system should be modelled – either explicitly or implicitly. For example, spiling or forepoling cannot be modelled in a 2D model explicitly but the effect can be incorporated by enhancing the properties of the relevant area around the tunnel. Other construction activities may also be relevant and therefore should be modelled. One example of this is compensation grouting which can impose additional loads on the lining.

Constitutive modelling and parameter selection

Typically, relatively simple models are used for the sprayed concrete lining. The norm is to assume a homogeneous, isotropic, linear elastic constitutive model, albeit including some variation in elastic modulus with age. It is usually assumed that the lining has been constructed to the exact (nominal) geometry specified.

The choice of constitutive model for the sprayed concrete has been found to affect the predicted stress distribution in the lining (Thomas 2003). This was more pronounced if the utilisation factor in the lining exceeded 50%. Assuming that the tunnel face is stable, the loads in the lining and its movements are governed by the relative stiffness of the ground and lining, since this is a soil-structure interaction problem. Bending moments were found to be more strongly influenced than hoop forces by the constitutive model (both for the sprayed concrete and ground). A discussion of possible constitutive models for sprayed concrete is contained in Chapter 5.

Looking more broadly, the constitutive model for the sprayed concrete may not have a large influence on the far-field behaviour of the ground. As might be expected, the assumed *in situ* stress state and the constitutive model for the ground can both have a considerable influence on the predicted loads on the lining (Thomas 2003). There is a growing appreciation of the need to model the ground with sophisticated constitutive models.

Theoretical basis of the solution method

In Chapter 3 the concepts of continua – soft ground or hard rock – and discontinua – blocky rock – were introduced. The numerical modelling program should be appropriate for the type of ground under consideration. FE and FD programs model continua while Discrete Element Method programs (e.g. UDEC and 3DEC) or UNWEDGE model discontinua. Bedded Beam Models are used less and less these days, because of their limited ability to model the soil-structure interaction. They have been superseded by 2D numerical models.

There are ways to model discontinua using conventional FE or FD programs. For example, one can use the Hoek–Brown failure criteria to approximate the behaviour of a jointed rock mass in a continuum model or one can introduce interface elements to model major discontinuities.

4.3.4 Physical tools

Trial tunnels are used occasionally for research or when SCL tunnelling is proposed for a new or particularly difficult area. Trial tunnels can provide the most readily accessible and realistic data on the performance of tunnel linings, although at considerable expense. Examples from the UK include: the Kielder experimental tunnel (Ward *et al.* 1983), Castle Hill trial heading (Penny *et al.* 1991), the trial tunnels for Heathrow Express (Deane and Bassett 1995) and Jubilee Line Extension (Kimmance and Allen 1996). On a broader note, the results from monitoring during and after construction bolster the general understanding of tunnel behaviour (e.g. the loads on linings (Barratt *et al.* 1994)) and can be used to enhance empirical design methods.

Small-scale and full-scale models are rarely used directly in the design of sprayed concrete linings but have been used in research. Large-scale models of tunnel linings have been constructed and tested to examine behaviour under working and collapse loads (e.g. as part of a recent Brite Euram project[2] – Norris and Powell (1999)). Other examples include: Stelzer and Golser (2002); Stärk (2002) and Trottier *et al.* (2002).

4.4 Code compliance

All tunnels must be designed in accordance with the relevant national design standards. However, most countries do not have specific design codes for underground excavations. Hence code compliance becomes a 'grey area'. Most European design codes are based on the principle of Ultimate Limit State (and Serviceability Limit State). The principle states that the probability of the loads exceeding the strength of the structure should be so small as to be negligible. To achieve the 'worst credible' estimate of the loads, 'moderately conservative' geotechnical parameters are first used to estimate the loads which are then multiplied by a load factor (typically 1.40). The specified strength of the structure is divided by a material factor (typically 1.50 for concrete). The factored strengths should always be higher than the factored loads. The combined load and material factors provide an overall factor of safety that is typically greater than 2.10. Furthermore, the structure should have sufficient redundancy that it does not fail in a sudden, brittle manner.

Compliance with the basic principle seems straightforward. Design codes exist for reinforced concrete structures (e.g. Eurocode 2 (2004)) and provide calculations to check compliance under different forms of loading (e.g. axial force, bending, shear, etc). A tunnel lining in soft ground could be seen as a structural member under combined axial force and bending moments. Shear loading is more important in blocky rock tunnels. Existing design codes for concrete can be used directly for any aspects that are independent of the fact that the concrete has been sprayed or is a tunnel lining – for example, determining cover to reinforcement.

Firstly, difficulty arises in the estimation of the credible ground loading. Section 4.2.1 describes two approaches for determining ground loads in design.

Secondly, design codes for reinforced concrete are written for conventionally placed concrete at ages greater than 7 days. To comply with the normal factors of safety and the stress-strain behaviour set out in the codes, typically the utilisation factors[3] in the tunnel lining should be less than 38% ($0.8f_{cu} \div \{1.4(\gamma_f)*1.5(\gamma_m)\}$). While some codes (e.g. BS8110 Part 2 (1985)) permit some latitude on the basis of experimental data and engineering judgement, it would still be difficult to justify utilisation factors that exceed 55% ($1.0f_{cu} \div \{1.2(\gamma_f)^*1.5(\gamma_m)\}$) for short-term loads near the face.[4] As Figure 4.12 shows, the utilisation factor in the lining may be greater than 55% within the first few metres of the lining. The concrete here is less than 7 days old.

This raises two questions. Should normal design codes be applied at these ages? How can one prove the safety of the tunnel in this critical area?

One could assume that the codes do not apply and resort to other means to prove the stability of the tunnel, e.g. empirical methods based on the stability number, N, or limit equilibrium solutions. The limitations of this approach and the difficulty in application to SCL tunnels have been discussed in Section 4.3.1. One could strengthen this approach by combining it with a process of risk management which culminates in the use of monitoring data during construction to verify that the tunnel is behaving as intended in the design (Powell *et al.* 1997), although one does not have the opportunity to take many readings in the critical area which is near the tunnel face. The differences between this approach and the classical NATM are discussed in the BTS Lining Design Guide (BTS 2004).

Alternatively one could use some of the more elaborate constitutive models such as those presented in Section 5. By taking advantage of factors such as creep, the numerical models may predict utilisation factors that are

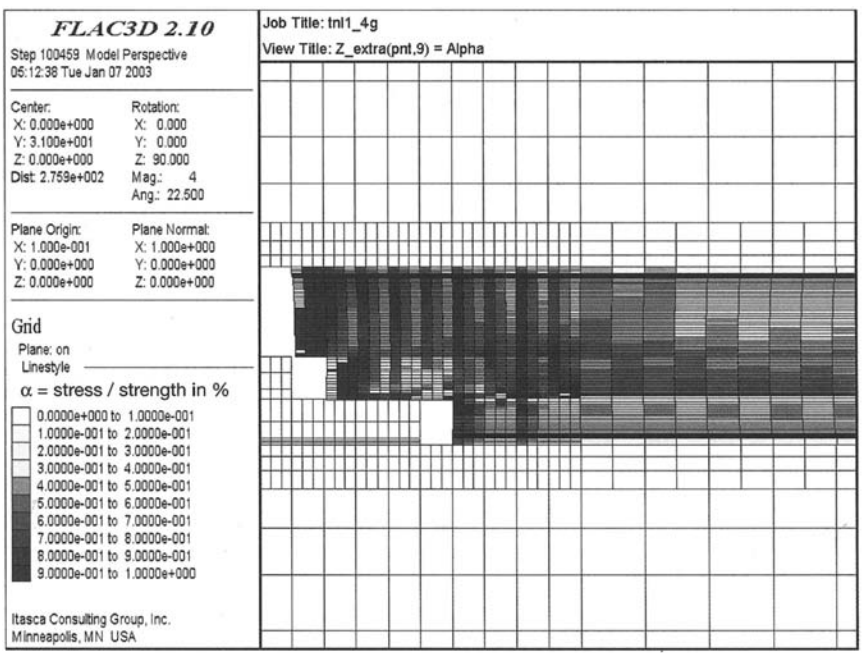


Figure 4.12 Utilisation factors in a shallow SCL tunnel in soft ground (Thomas 2003)

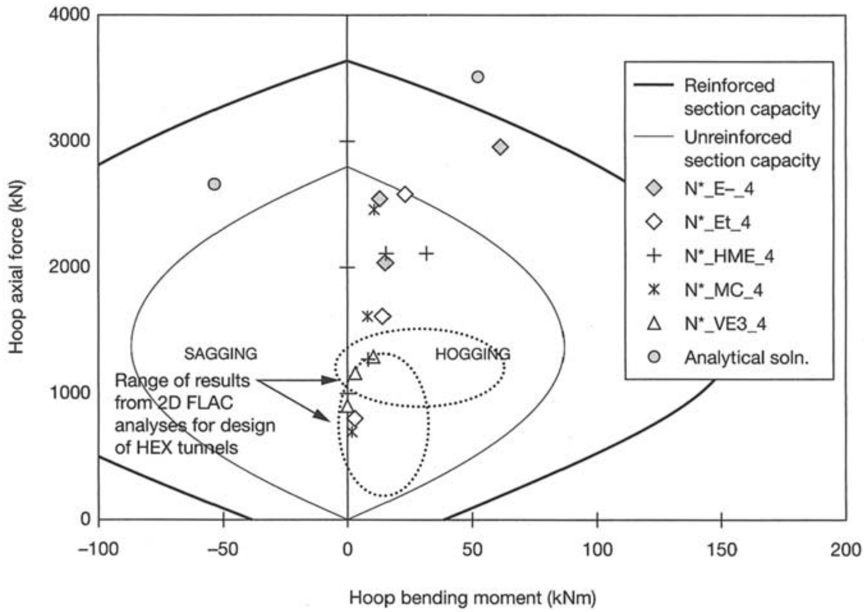


Figure 4.13 Results from numerical model of a shallow SCL tunnel in soft ground (Thomas 2003)

Note: See Appendix E for explanation of key.

low enough to comply with the codes (see Figure 4.13). However, this approach is vulnerable to scrutiny. Few projects can afford an extensive pre-construction test programme. At the same time, there is currently an insufficient pool of data from which to determine many of the model parameters with the certainty normally required. The results from this study have shown that small variations in key parameters (e.g. advance rate or creep parameters) may have significant effects on the results. Therefore the results of the design analyses might remain open to question.

This problem of code compliance has been concealed to a degree in the past since designs were usually based on 2D analytical methods with empirical correction factors (such as in the Hypothetical Modulus of Elasticity approach). The complete stress history of the lining and variations in stress within each ring were rarely investigated. Furthermore, the primary lining was often regarded as part of the temporary works.

A pragmatic approach may be:

- to acknowledge that, while the principles of conventional design codes apply to the heading of a SCL tunnel, the detail may not apply at very early ages;

- to embrace the new information provided by 3D numerical analyses as a complement to existing design approaches; and
- to accept that numerical models are still only an approximation of reality – so sound engineering judgement remains as important as ever.

Finally, little guidance exists for the design of steel fibre reinforced concrete although a design method has been proposed by the German Concrete Society (DBV 1992).

4.5 Continuity between design and construction

Surprising as it may seem, a bundle of drawings and a hefty sheaf of contract documents are not enough to convey everything that the designer has in mind for a tunnel. A certain amount of the design philosophy – at least in current practice – is not captured and passed on to the construction team. This creates a risk that changes made during construction may compromise the design criteria or that abnormal behaviour of the structure will not be identified before lasting damage is done.

Several simple steps can be taken to mitigate this risk. Firstly, a design report can be included with the construction drawings. This brief report describes the design criteria, the philosophy adopted to meet them and may include details of the design methods.

Second, a good dialogue between construction teams and designers improves the mutual understanding and leads to more economic designs and greater safety during construction. The benefits of the presence of a representative of the designer on site are discussed further in Section 7.3.

Notes

1 Soft ground often exhibits complex material behaviour, with features such as nonlinear stress-strain behaviour, plasticity, variable K_0 values, anisotropy and consolidation (e.g. London Clay – see Thomas 2003)
2 BRITE-EURAM BRE-CT92-0231 project on New Materials, Design and Construction Techniques for Underground Structures in Soft Rock and Clay Media, part funded by the Commission of the European Communities, 1994–98.
3 The calculated utilisation factors from the numerical model are based on deviatoric stresses whereas BS8110 Part 1 (1997) considers stress components in each direction separately – e.g. the hoop axial load vs f_{cu} Eurocode 2 (2004) (Cl. 3.1.9) permits the increase in strength due to confinement to be considered.
4 A partial factor of safety of 1.20 is commonly used for temporary loads.

5 Modelling sprayed concrete

As Fourier said, 'nature is oblivious to the difficulties that it poses for mathematicians'. Similarly, sprayed concrete is blissfully unaware of the headaches it causes tunnel engineers. As explained in Section 2.2, sprayed concrete exhibits complex behaviour:

- the mechanical properties of sprayed concrete change considerably as the hydration reaction proceeds and a hardened concrete is formed (i.e. it ages);
- the rate of change of properties is rapid initially, due to the accelerated hydration, and slows with age;
- in its hardened (and hardening) form, sprayed concrete is a nonlinear elastoplastic material when under compression;
- in tension it is initially a linear elastic material which then fails in a brittle manner;
- to overcome the brittle failure, tensile reinforcement is often added;
- sprayed concrete exhibits shrinkage;
- the creep behaviour of sprayed concrete can be pronounced and changes with age.

These features must be borne in mind during the design process but not all of them will necessarily be relevant in every case and so it may be possible to simplify the design. Such simplifications should be made on a reasoned basis and not just to make the designer's life easier, otherwise the design may become unsafe or over-conservative. In contrast to the geotechnical model, relatively crude models are normally used for sprayed concrete (e.g. Bolton *et al.* 1996, Watson *et al.* 1999 and Sharma *et al.* 2000). Therefore, it is perhaps unsurprising that there is often a significant discrepancy between the behaviour predicted by numerical analyses and that observed on site (e.g. Addenbrooke 1996, van der Berg 1999). The results of design analyses have been shown to depend strongly on how the sprayed concrete is modelled, although the difference is most pronounced when the lining is heavily loaded – i.e. the stress is higher than 50% the strength (Thomas 2003).[1] Improving the modelling of the sprayed concrete could make the prediction of the behaviour of these tunnel linings more reliable.

Table 5.1 below lists some of the design parameters for sprayed concrete that may be required, as well as the type of ground where they may be relevant. Section 2.2 provides more detail on each parameter.

There is no single perfect constitutive model for sprayed concrete. This chapter will present some of the models that exist and highlight their strengths and limitations. The opportunities to explicitly incorporate the more complex aspects of this behaviour in design calculations are very limited in analytical design tools and non-existent in empirical tools. Hence the comments below are mainly angled towards the numerical modelling of sprayed concrete structures.

5.1 Linear elastic models

The most commonly used model is a linear elastic one with a constant stiffness, because of its simplicity and computational efficiency. Typically, elastic models predict axial forces and bending moments in linings that are unrealistically high compared with field data from strain gauges and pressure cells (Golser *et al.* 1989, Pöttler 1990, Yin 1996, Rokahr and Zachow 1997). This is no surprise since sprayed concrete only behaves in a linear elastic manner up to about 30% of its uniaxial compressive strength (Feenstra and de Borst 1993, Hafez 1995) and the stiffness varies considerably during the early age of the sprayed concrete.

Table 5.1 Common design parameters for sprayed concrete

Parameter	*Description*	*Typical range*	*Type of ground*
E	Elastic modulus	30–35 GPa at 28 days NB: E varies widely, from zero at age = 0	Soft/blocky
f_{cu} (or f_{cyl})	Compressive strength (from cubes or cylinders)	25–40 MPa at 28 days NB: f_{cu} varies widely, from zero at age = 0	All
$f_{cu}(t)$	Compressive strength (from Hilti gun tests)	J2 – ÖBV 1998	All
v	Poisson's ratio	0.20[a]	Soft/blocky
–	Bond strength	0.125–0.35 MPa after spraying, rising to 0.5 to 1.4 MPa at 28 days	Blocky/hard
–	Flexural strength – for steel fibre sprayed concrete	3.1 MPa at 28 days	Blocky/hard

Note:
a 0.20 is a reasonable value except when the concrete is close to failure (Chen 1982).

A logical improvement on a simple elastic analysis is to incorporate the increase in magnitude of the stiffness with age (see Figure 2.11 and Appendix A). In most cases, at the design stage, there is no experimental data for the stiffness (e.g. elastic modulus, E, and Poisson's ratio, ν) at different ages of the sprayed concrete mix in question. Instead the elastic modulus may be estimated from the strength of the sprayed concrete (Chang and Stille 1993; see also Appendix A), using the equation:

$$E = 3.86\sigma^{0.60} \tag{5.1}$$

where σ is the uniaxial compressive strength. If the elastic modulus is known at 28 days (E_{28}), the value at other ages may be estimated from any one of a number of equations (see Appendix A), e.g.:

$$E = E_{28}(1 - e^{-0.42t}) \tag{5.2}$$

where t is the age in days and E_{28} is the stiffness at 28 days (Aydan *et al.* 1992b). The Poisson's ratio can be assumed to be constant with age and equal to 0.20. In numerical models, this is often implemented as a 'stepped' approximation of the curve, since the excavation sequence is modelled as a series of steps. The increase in stiffness with age will lead to irrecoverable strains on unloading at later times (Meschke 1996).

Influence on the predictions of numerical models

There is plenty of evidence to support the use of an age-dependent linear elastic model for sprayed concrete. In a 3D numerical model, Berwanger (1986) also found that the ultimate stiffness of the sprayed concrete had a limited influence on surface settlement but it did have a large influence on the stresses in certain parts of the lining, notably the footing of the top heading. Similarly, in 2D numerical models, Pöttler (1990), Huang (1991), Hirschbock (1997) and Cosciotti *et al.* (2001) found that increasing the stiffness of the lining increased the stresses in it. Considering a tunnel constructed in one stage, in a 2D analysis, Hellmich *et al.* (1999c) stated that the stiffness of the lining affected only the hoop bending moments and not the hoop forces. However, later in the same paper they show that the axial forces are lower when rate of hydration is slower. By extrapolating their finding that the stresses in the lining are influenced by how long it takes for the lining to start to carry meaningful loads, it would seem that in a multi-stage construction sequence the rate of growth of stiffness will be an important factor.[2]

Several authors have suggested that the relative stiffness of the lining compared with that of the ground would affect the amount of influence of a time-varying modulus (Hellmich *et al.* 1999c, Cosciotti *et al.* 2001). In their set of 2D analyses Hellmich *et al.* (1999c) found that the rate of stiffening of the lining is important in ground that creeps at a similar rate to the sprayed concrete but when the creep in the ground is much slower, the rate of hydration of the concrete has virtually no effect on the axial forces and moments. In a detailed study using 3D numerical models, Soliman *et al.* (1994) reported that a variable elastic modulus led to significantly larger lining deformations (20–30% more) and lower bending moments (reduced by up to 50%) compared to a constant elastic modulus. The thrust loads were slightly lower – reduced by about 20% – and hence the stresses in the ground were not increased by much. This may well explain why surface settlements seem to be independent of the constitutive model of the lining (see also Moussa 1993).

In conclusion, in a multi-stage construction sequence, using an age-dependent elastic modulus for the lining will result in lower stresses in the lining (e.g. see Figure 5.1). The bending moments are reduced more than the hoop forces (e.g. see Figure 5.2). The reduction appears to be due more to the lower stiffnesses during early loading (i.e. ages less than 48 hours), compared to a (high) constant stiffness model, than how the stiffness develops beyond that period.[3] If the lining is not heavily loaded (i.e. loaded to a utilisation of 40% or less), it is probably not necessary to use a more complicated model than the age-dependent linear elastic

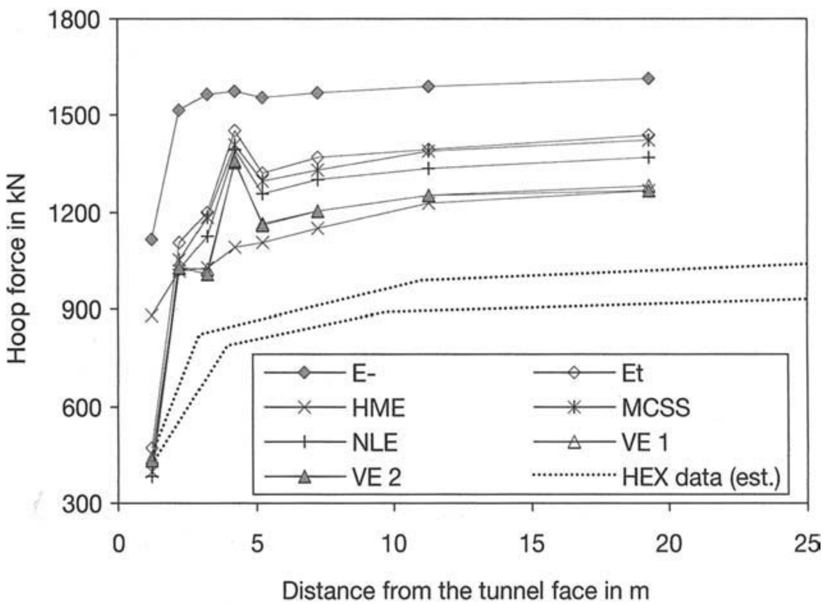


Figure 5.1 Hoop axial force in crown vs distance from face for different sprayed concrete models (Thomas 2003)

Note: See Appendix E for explanation of key.

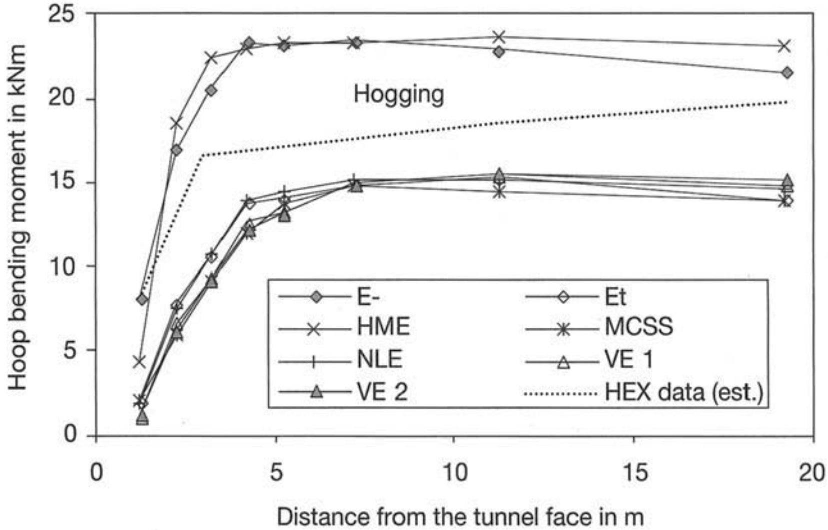


Figure 5.2 Hoop bending moment in crown vs distance from face for different sprayed concrete models (Thomas 2003)

Note: See Appendix E for explanation of key.

model. Sprayed concrete behaves linearly up to stresses of 30 to 40% of the uniaxial compressive strength (see Section 2.2.4).

State-of-the-art SCL designs often employ a relatively crude usage of the Hypothetical Modulus of Elasticity (HME). It is suggested that there is sufficient data to use more realistic constitutive models – e.g. an ageing linear elastic perfectly plastic model based on site data for strength development with age (and stiffness estimated from the strength) or an ageing linear elastic model with the stiffness reduced to account for creep – a refined HME method.

5.2 Hypothetical Modulus of Elasticity (HME)

A widely-used and very successful attempt to improve numerical modelling came in the form of the Hypothetical Modulus of Elasticity (HME) method (Pöttler 1985). In this, several reduced values of elastic modulus are used in an analysis. Typical values are shown in Table 5.2. The 'softer' lining leads to more realistic results – larger lining deformations and lower stresses – without excessive computation time. Though the concept of an Effective Modulus is not new to creep analysis (e.g. BS8110 Part 2 (1985)),[4] the HME is intended to account for ageing of the elastic stiffness, shrinkage and 3D effects as well as creep. According to the original formulation:

$$E_{HME} = E_T f_v f_{s,k} f_{vv} \tag{5.3}$$

where E_T = the stiffness at the time in question, f_v = correction factor for the age-dependent stiffness during the loading up to the time in question, $f_{s,k}$ = correction factor for creep and shrinkage, f_{vv} = the crown deformation occurring before lining placement as a fraction of total deformation of ground at the crown – i.e. the effects of 3D and timing of placement. Sometimes the HME includes an allowance for the nonlinear stress-strain behaviour of the concrete (e.g. John and Mattle 2003).

The correction factors in Equation 5.3 require knowledge of how the lining and ground will deform as well as how creep will alter the stresses in the lining. While the first can be estimated using various analytical tools, the second cannot. So the choice of the values of HME is usually empirical. Sometimes the HME is combined with relaxation of the ground stresses ahead of the face in a 2D numerical model (i.e. f_{vv} = 1 but the relaxation is catered for explicitly in the model – e.g. Pöttler 1990, John and Mattle 2003). While John and Mattle (2003) provide the most detailed description of how to choose the values of the HME, their approach should be treated

Table 5.2 Values of Hypothetical Modulus of Elasticity

Project	*HME*	*Application*
Channel Tunnel (Pöttler and Rock 1991)	1.0 GPa	Age < 14 days; back-analysed from measurements of deformation and pressure cells
(Pöttler 1990)	7.0 GPa	Plus relaxation of the ground of 30%; based on parametric study with a 2D numerical model
CTRL North Downs	7.5 GPa	Age < 10 days; Strength limited to 5 MPa
(Watson *et al.* 1999)	15.0 GPa	Age > 10 days; Strength limited to 16.75 MPa (= 0.67 f_{cu})
Heathrow Express	0.75 GPa	Initial value
(Powell *et al.* 1997)	2.0 GPa	Value after adjacent section is constructed and until lining is complete
	25.0 GPa	Mature sprayed concrete
(John and Mattle 2003)	1.0–3.0 GPa	For sprayed concrete with a 1 day strength < 10 MPa and light reinforcement (i.e. high creep potential)
	3.0–7.0 GPa	For sprayed concrete with a 1 day strength > 10 MPa and moderate to heavy reinforcement (i.e. low creep potential)
	15.0 GPa	Mature sprayed concrete

with caution as it seems to rely on knowing the answer before the calculation is done – i.e. you must know how heavily the lining will be reinforced, how fast the ground will apply the load to the lining and which is the most critical stage of the tunnel construction in order to select the correct HME value.

Influence on the predictions of numerical models

Clearly, given the low stiffnesses at early ages, one would expect that using the HME approach would result in significantly lower predictions of stresses than using a (high) constant stiffness model.

5.3 Nonlinear stress-strain behaviour

As noted earlier, the stress-strain curve for concrete is nonlinear at stresses above about 30% of its uniaxial compressive strength (see Figure 2.3). This nonlinearity can be implemented in the theoretical frameworks of either strain-hardening plasticity or nonlinear elasticity.

5.3.1 Nonlinear elastic models

Models, such as the Cauchy, Hyperelastic and Hypoelastic models, attempt to replicate the nonlinear stress-strain behaviour of concrete. This nonlinear behaviour begins at relatively low stresses[5] and is due to microcracking at the interface between the aggregate and cement paste, which themselves are still responding elastically (Neville 1995). Since this plastic deformation lies behind the nonlinearity, plasticity models are required if unloading occurs. However, if unloading can be neglected, nonlinear elastic models represent an economic means of modelling the nonlinear response of concrete to loading and a significant improvement on linear elastic models (Chen 1982). Consequently they have been widely used in the analysis of concrete structures, although they have been rarely used in the analysis of sprayed concrete tunnels.

Specific nonlinear elastic models used for the analysis of SCL tunnels include: Saenz's Equation (see Chen 1982) which Kuwajima (1999) found fitted experimental data for stress-strain curves well; the Rate of Flow method (see Section 5.6.5) and the parabolic equation below (Moussa 1993):[6]

$$\sigma_c = f_c\left(\frac{\varepsilon_c}{\varepsilon_1}\right)\left(2-\frac{\varepsilon_c}{\varepsilon_1}\right) \tag{5.4}$$

where f_c is the peak stress, ε_1 is the strain at peak stress and σ_c and ε_c are the equivalent uniaxial stress and strain respectively.

In these models, the behaviour of concrete is treated as an equivalent uniaxial stress-strain relationship. Biaxial effects may be accounted for in the tangent moduli.

The Kostovos-Newman model

This model (Kotsovos and Newman 1978, Brite Euram C2 1997) is formulated in terms of octahedral stresses and so, unlike the others, it has the advantage of being designed for generalised states of stress. Although quite lengthy in its formulation, another advantage of this model is that all the parameters can be determined from the uniaxial compressive strength of a cylindrical sample and its initial stiffness – see Appendix B.

Considering it in more detail, its other key advantages are that:

- the model includes the effects of deviatoric stress on hydrostatic strains:
- the model has been shown to agree well with existing test data for concrete and sprayed concrete (Brite Euram C2 1997, Eberhardsteiner *et al.* 1987, Thomas 2003) and hence it has been recommended for use in modelling mature sprayed concrete (Brite Euram 1998):
- the increase in strength with increasing hydrostatic stress is accounted for and the predicted failure surface agrees better than the Mohr–Coulomb model – see Figure 2.9.

However, it should be noted that, as with nonlinear elastic models, it is only valid up to the point of onset of ultimate failure, which is at about 85% of the ultimate strength.

The formulae for the tangent shear and bulk moduli in this model are shown below, with full details of all parameters explained in Appendix B:

$$G_{\tan} = \frac{G_0}{1 + Cd\left(\dfrac{\tau_0}{f_{\mathrm{cyl}}}\right)^{d-1}} \tag{5.5}$$

$$K_{\tan} = \frac{K_0}{\left(1 + \left(Ab\left(\dfrac{\sigma_0}{f_{\mathrm{cyl}}}\right)^{b-1}\right) - \left(klme\left(\dfrac{\tau_o}{f_{\mathrm{cyl}}}\right)^{n}\right)\right)} \tag{5.6}$$

Since this is a tangent modulus model, its accuracy depends on the size of the load increments in comparison to the peak strength. Thomas (2003) suggested that a minimum of seven equal increments is required for acceptable performance. The size of load intervals in numerical models is a trade-off between accuracy and speed. Minor modifications have been made to the formulae to extend them to strengths less than 15 MPa. The shear modulus in the original formulation by Kotsovos and Newman does not decrease to zero at the peak strength. To overcome this, Thomas (2003) proposed that the modulus is reduced gradually to 5% of G_0 (the initial shear modulus) as the actual shear stress approaches the peak shear stress (at 85% of the peak stress). Above the same point the bulk modulus is reset to 0.33 K_0 (the initial bulk modulus) in line with recommendations from Gerstle (1981).

Figures 5.3 and 5.4 suggest that this constitutive model functions well both under uniaxial and triaxial loading. Despite using the same input parameters as a strain-hardening plasticity model, Thomas (2003) found that the nonlinear elastic model agrees better with the test data in the triaxial case (see Figure 5.3). The nonlinear elastic model had been optimised to fit the test data by altering the point at which the moduli reduce to the low values, as described above. This leads to predicted stresses that exceed the model's own estimate of the strength of the concrete (by about 12%) given the confining stresses and the uniaxial strength. This is possible because it is an elastic model, whereas the strain-hardening plasticity model is capped at its predicted peak strength.

Ageing is a major complication in the implementation of the constitutive models for sprayed concrete but it can be successfully handled by the Kotsovos and Newman model.

The effects of loading, unloading or reloading can be included by using the Masing rules for loading cycles (Dasari 1996). Unloading is probably of

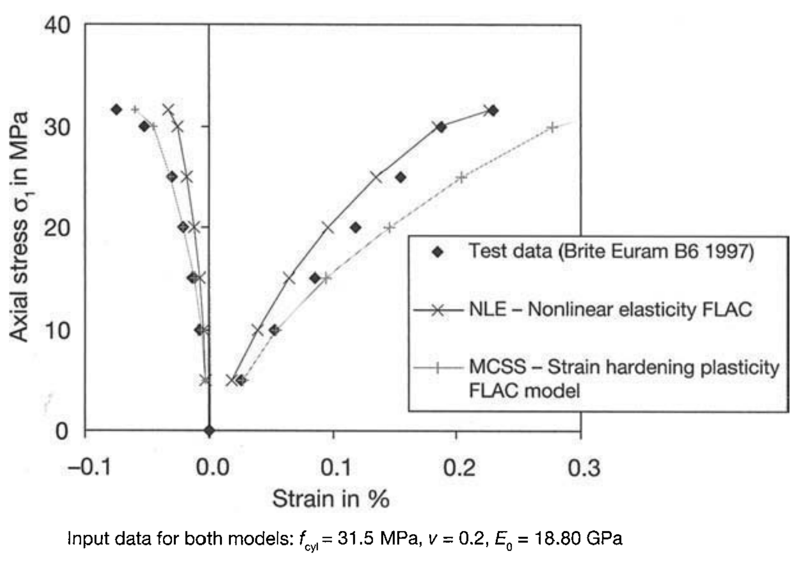


Figure 5.3 Back-analysis of a uniaxial compression test on sprayed concrete (Thomas 2003)

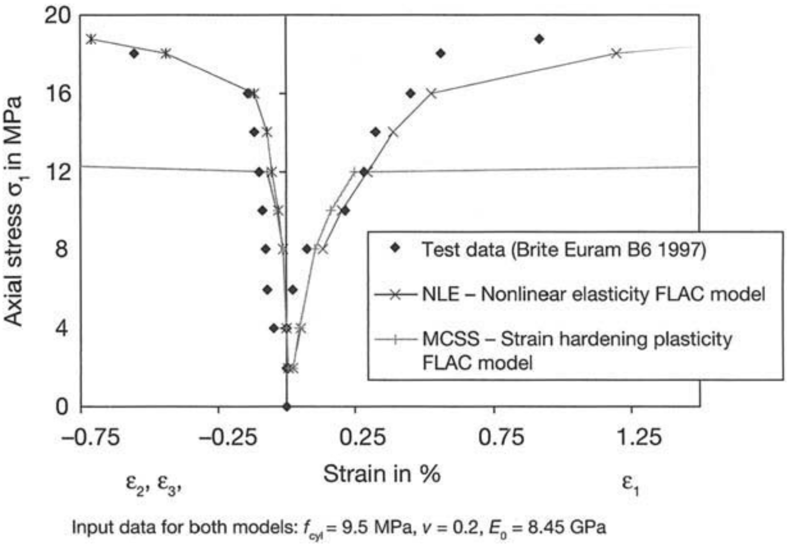


Figure 5.4 Back-analysis of a triaxial compressive test on sprayed concrete (Thomas 2003)

limited relevance to the lining of a single tunnel constructed on its own with a top heading, bench and invert excavation sequence, since little unloading would be expected to occur. However, it is likely to be of relevance where other tunnels are constructed near to an existing tunnel, at junctions and in more complex excavation sequences. Since loading is determined on the basis of changes in octahedral deviatoric stress alone, changes in hydrostatic stress are not recognised in terms of loading/unloading. However, a tunnel lining is predominately in a biaxial stress state and so the most likely changes in load are primarily deviatoric ones, rather than purely hydrostatic.

Influence on the predictions of numerical models

In a 2D analysis of a shallow tunnel in soft ground, Moussa (1993) found that incorporating nonlinearity into the elastic model resulted in a reduction in hoop forces of about 20% and a reduction of up to 50% for bending moments. The surface settlements were virtually unchanged and there was only slightly more plastic deformation in the ground adjacent to the tunnel. It is not clear what the utilisation factors were but they probably ranged from 40 to 80%. Thomas (2003) found that in a relatively lightly loaded lining (with a utilisation factor of around 40%) the nonlinear elastic model had less impact because the lining was still mainly in the linear elastic region. Nevertheless, hoop forces were reduced by 5% and deformations increased slightly.

5.3.2 *Plastic models*

A general elastic perfectly plastic constitutive model requires an explicit stress-strain relationship within the elastic region, a yield surface (failure criterion) defining when plastic strains begin and a flow-rule (governing the plastic strains). In the following sections, the three components of a plasticity model will be discussed, firstly for the compressive region, secondly for the tensile region and finally for intermediate regions, before outlining the impact of elastoplasticity on the results of tunnel analyses. Appendix C contains details of plasticity models already used for analysing sprayed concrete tunnel linings.

Elastic behaviour

Isotropic linear elasticity is generally assumed up to the yield point (about 30 to 40% of the compressive strength).

Yield criteria

Since the first parameter yield criteria were proposed by Rankine, Tresca and von Mises, many others have been formulated (Owen and Hinton 1980). More and more complex criteria have been proposed to match experimental data more accurately over a wider range of stresses. In the case of concrete, the two parameter Mohr–Coulomb[7] and Drucker–Prager yield criteria have been used often in the past (see Figure 5.5). New yield criteria have been developed which can replicate the curved nature of the yield surface meridians (see Figure 2.9) and also the shape of the surface in the deviatoric stress plane, which is initially almost triangular but tends to an almost circular shape at high hydrostatic stresses (Hafez 1995, Chen 1982). Curved yield surfaces are also advantageous since the corners and edges are difficult to handle in numerical analysis (Hafez 1995). Hence, for the purposes of analysis of sprayed concrete tunnel linings, the Drucker–Prager criterion has been the most widely used (see Appendix C).

Considering a moderately heavily loaded tunnel, where the principal stresses in the lining might be 8, 3 and 0.5 MPa and the 28 day strength equals 25 MPa, the normalised octahedral stress, σ_{oct}/f_{cu}, is only 0.15 which is quite low. So the assumption of straight meridians in the Drucker–Prager criterion is reasonable. The Drucker–Prager criterion can also be amended to reflect the increase in yield stress in biaxial states of stress (Hafez 1995, Meschke 1996). However, the shape of the Mohr–Coulomb criterion in the deviatoric plane agrees better with the almost triangular yield surface suggested by test data in the deviatoric plane at low hydrostatic stress. Figure 2.9 suggests that this Mohr–Coulomb failure surface agrees reasonably well with experimental data along the compressive meridian but less well along the tensile meridian.

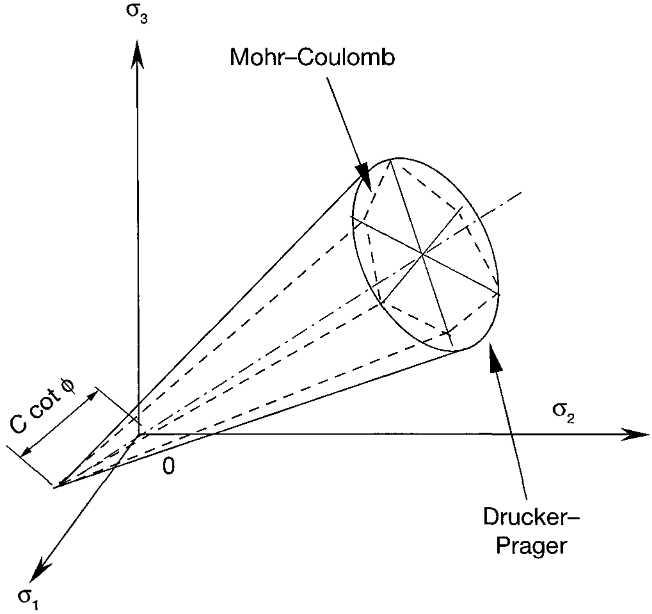


Figure 5.5 Yield surfaces in 3D stress space

Post-yield stress-strain relationships

Various theories have been proposed for post yield behaviour: perfect plasticity, isotropic work-hardening (or softening), kinematic work-hardening or a combination of isotropic and kinematic hardening. Kinematic hardening is only really needed for cyclic loading in concrete (Chen 1982). Most of the models (see Appendix C) assume isotropic hardening up to a peak stress and perfect plasticity thereafter. However, in experiments it has been observed that stress decreases with increasing strain after a peak value (see Figure 2.3). The shape of this descending branch of the stress-strain curve depends heavily on the confinement and the boundary conditions imposed by the experimental equipment (see Section 2.2.4). It is usually assumed that the hardening behaviour does not vary with age.

In classical plasticity models the plastic strains vectors are obtained from the plastic potential and a flow rule, which can be either associated or non-associated. Associated flow rules assume that the plastic potential coincides with the yield function. Hence the vectors of plastic strain are normal to the yield surface. In the absence of experimental evidence to support a particular non-associated flow rule, the assumption of an associated flow rule is a common simplification (Chen 1982; Hellmich *et al.*, 1999b).

Tension

In the plasticity models used in tunnel analyses, the Rankine criterion is generally used for yield in tension (see Appendix C). According to this, brittle fracture occurs when the maximum principal stress reaches a value equal to the tensile strength (Chen 1982). The tensile strength is usually estimated from the compressive strength, using relationships for normal cast *in situ* concrete. Post-failure behaviour in tension will be discussed later in Section 5.4.

Compression and tension

For states in which one of more of the principal stresses is compressive and the others are tensile, an assumption must be made about the nature of the yield surface. It is usually assumed that the presence of tension reduces the compressive strength linearly (see Figure 2.4, Chen 1982). However, there is some doubt about the exact effect (Feenstra and de Borst 1993). In multisurface plasticity models (e.g. Meschke 1996, Lackner 1995), a check is performed in each principal stress direction to see which of the yield surfaces is active and the relevant yield criteria is then applied.

In a plasticity model, the material is assumed to behave in a linear elastic manner until the yield point is reached. Beyond that point the stress increases (or decreases) in accordance with a hardening (or softening) rule relating the cohesion to the plastic strain up to the peak plastic strain. The plastic strains occur according to a flow rule in addition to the elastic strains. In the generalised case, the yield point becomes a surface in 3D stress space (e.g. see Figure 5.5) and is usually defined in terms of stress invariants (Chen 1982). The ratio of yield strength, f_{cy}, to ultimate strength, f_{cu}, (f_{cy}/f_{cu}) is about 0.40.

Figure 5.6 shows the variation of peak strains (i.e. strain at peak stress) with age, t. There is considerable scatter. However, a possible relationship between age in hours and peak strain in % is:

$$\varepsilon_{peak} = -0.4142\ln(t)+3.1213 \tag{5.7}$$

Figure 5.7 shows the ultimate peak strain plotted against age. As with Figure 5.6 the data has been extracted from published laboratory test results. All the values are considerably larger than the 0.35% limit stated in design codes (e.g. BS8110 1997, Eurocode 2 2004). This supports the view of early pioneers that sprayed concrete can withstand large strains at young ages. Meschke (1996) proposed a quadratic hardening law is used to calculate the change in cohesion (see Figure 5.8). This agrees well with the idealised stress-strain curve proposed in BS8110 Part 2 (1985):[8]

$$f = f_{cy} + 2(f_{cu} - f_{cy})\left(\frac{\varepsilon_{pl}}{\varepsilon_{pl\cdot peak}}\right) - (f_{cu} - f_{cy})\left(\frac{\varepsilon_{pl}}{\varepsilon_{pl\cdot peak}}\right)^2 \tag{5.8}$$

where f is the cohesion (or strength), f_{cy} is the yield strength, f_{cu} is the ultimate strength, ε_{pl} is the current plastic strain and $\varepsilon_{pl \cdot peak}$ is the plastic strain at ultimate strength. The cohesion (on the compression meridian – Lode angle = 60°) can be related to the uniaxial strength, f_c, by Equation 5.9 taking ϕ = 37.43°:[9]

$$\text{cohesion} = f_c\left(\frac{(1-\sin\phi)}{2\ \cos\phi}\right) \tag{5.9}$$

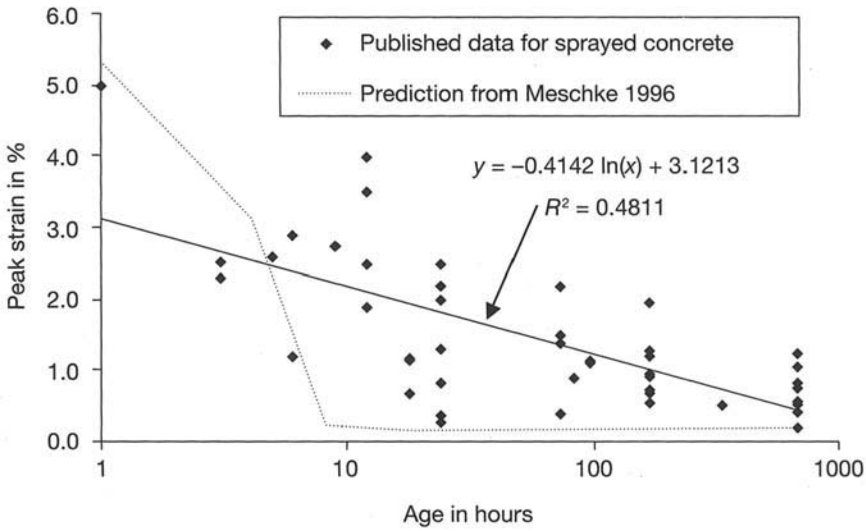


Figure 5.6 Peak compressive strain vs age

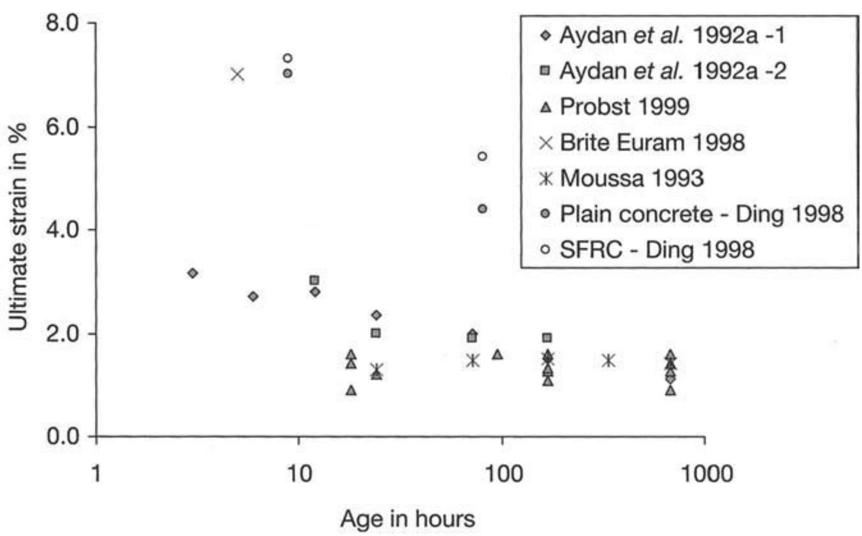


Figure 5.7 Ultimate compressive strain vs age

Influence on the predictions of numerical models

Typically strain-hardening plasticity models predict an increase of 15 to 30% in the magnitude of deformations and a reduction of 10 to 25% in the magnitude of bending moments in the concrete shell, compared to an age-dependent elastic model (Moussa 1993, Hafez 1995, Hellmich *et al.* 1999c, Thomas 2003) – for example, see Figure 5.1 and Figure 5.2. However, the influence of plasticity obviously depends on how heavily the lining is loaded and the ability of the ground to sustain the stress, which is redistributed back into it. If the ground is close to failure, the stress redistribution due to plastic deformation in the lining may exacerbate the situation (Hafez 1995).

5.4 Tensile strength

Since the tensile strength of concrete is much lower than the compressive strength, in many normal load cases, failure in tension (i.e. cracking) may well occur while the compressive stresses are well below failure levels (Chen 1982). While some *in situ* investigations have revealed tensile stresses in sprayed concrete tunnel linings (Hughes 1996, Negro *et al.* 1998) and cracking is of major concern, when considering permanent sprayed concrete linings, only the simplest tensile models have usually been used in design analyses. Namely, the concrete behaves in a linear elastic manner up to a

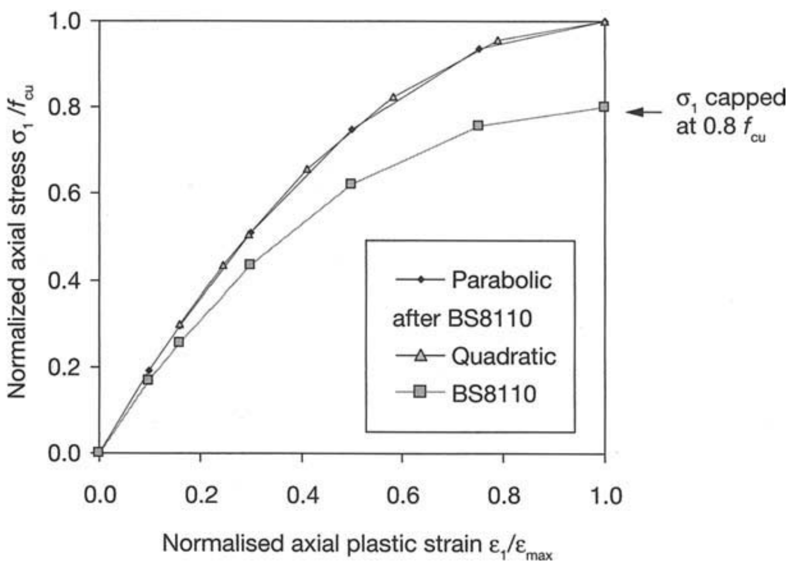


Figure 5.8 Theoretical strain hardening curves

tensile cut-off at the uniaxial tensile strength. This approach may simply have been adopted due to limits on computing power. The sections below include information on more sophisticated constitutive models even though they are not used in current design practice.

5.4.1 Unreinforced sprayed concrete

The tensile strength of unreinforced sprayed concrete, f_{tu}, is rarely tested. In the absence of other data, the tensile strength could be estimated from the compressive strength, using relationships for normal cast *in situ* concrete (e.g. Neville 1995):

$$f_{tu} = 0.30 f_{cu}^{0.67} \tag{5.10}$$

The maximum principal stress failure criterion is the most commonly used. According to it, once the tensile stress acting on a plane exceeds the strength, a crack is formed and the stress carried across the crack falls to zero. In reality, if the width of the crack is not too great, 40 to 60% of the shear forces can still be carried due to aggregate interlocking (Chen 1982). Therefore the behaviour of concrete after cracking is highly nonlinear and orthotropic (Kullaa 1997). Alternatively one can assume a maximum principal strain criterion, where cracks form once a limiting strain value is exceeded (Chen 1982).

A more sophisticated approach is to assume that the tensile stress of plain concrete decreases linearly, bilinearly or exponentially with increasing strain (Lackner 1995). The difficulty in determining the softening curve has led to the use of fracture energy to calculate the parameters for use in these models (Feenstra and de Borst 1993). The fracture energy is the area under a stress-deformation curve and a correction is required to account for the size of the mesh elements. This approach can also be used for the post-peak softening behaviour in compression too (Feenstra and de Borst 1993). Meschke (1996) and Kropik (1994) used a (Rankine) maximum stress failure criterion followed by linear tension softening. The gradient of the descending stress-strain line is assumed to be *E*/100 – one hundredth of the initial elastic modulus. While this would seem to model correctly the pre-crack and post-crack behaviour of plain concrete, the composite material of reinforced concrete actually exhibits tension hardening.

The cracks can be modelled either discretely or smeared over the elements in question. The concept of the smeared crack can be further sub-divided into nonlinear elastic, plastic or damage theory models (Lackner 1995). Smeared cracks can be either fixed (once they have formed) or rotate their orientation as the direction of the tensile stresses changes. To avoid re-meshing, the smeared crack approach is usually adopted (Kullaa 1997).

5.4.2 *Reinforced sprayed concrete*

Like cracks, bar reinforcement can be modelled discretely (Kullaa 1997, Eierle and Schikora 1999). However, for both features, this can be a laborious process even in a simple 2D mesh and is too complex an approach at this time for the 3D analysis of tunnels. Reinforcement and the tensile behaviour of reinforced concrete are rarely simulated in numerical models, either explicitly (e.g. Haugeneder *et al.* 1990) or implicitly.

In the case of a smeared crack model, the reinforcing effects of steel and fibres may be incorporated by modifying the post-crack (tension softening) properties of the concrete elements. For example, it can be assumed that a fraction of the tensile stress across the crack can still be sustained. In the case of steel fibre reinforced shotcrete, a value of 0.3 has been proposed for this (Brite Euram C2 1997). Obviously, a single fixed value does not take account of the variation in behaviour with crack width or the anisotropic distribution of the fibres. Moussa (1993) chose to multiply the value of the ultimate tensile strain by a factor of 10 to account for the presence of reinforcement.

Influence on the predictions of numerical models

Given the dearth of information on this subject, it is not possible to comment in detail on the influence of the model of tensile behaviour. In general one may note that the assumption of an infinite tensile capacity (e.g. in an elastic model) obviously overestimates the capacity of the lining, while a brittle tensile cut-off would underestimate the capacity and result in an overestimate of stress redistribution. A pragmatic approach, which could be adopted, is to use a simple tension model and to compare the predictions of tensile stresses with the tensile capacity. More complex analyses can then be undertaken if required.

As part of a broader study, Thomas (2003) modelled a lattice girder in a ring of sprayed concrete using cable elements for the three main steel bars. The presence of the lattice girder did not have any significant influence on the results of the numerical model due to its small area relative to a 1 m width of the concrete lining.

5.5 Shrinkage

Figure 5.9 shows the variation of shrinkage strain with age. Given the scatter in experimental data, it appears that the simple ACI Equation, with the constant B = 20 days and an ultimate shrinkage strain, ε_{shr00} of 0.1%, may be used to predict the development of shrinkage with age (ACI 209R 1992) as a first approximation. Obviously the value of B may vary depending on the characteristics of each mix (e.g. Jones (2007) quotes a value of B = 55):

$$\varepsilon_{shr} = \frac{\varepsilon_{shr00}t}{(B+t)} \quad \text{where } t \text{ is in days} \tag{5.11}$$

Shrinkage is rarely modelled in design analyses on the assumption that the effects are much smaller than the ground loads. However, this may not be the case in lightly loaded linings (e.g. Jones 2007). Hellmich *et al.* (2000) found that in one set of 2D analyses bending moments were reduced by shrinkage but axial forces where relatively unaffected, while in another example (back-analysis of the Sieberg Tunnel), their model predicted a large reduction in hoop forces due to shrinkage.

5.6 Creep models

Traditionally the high creep capacity of sprayed concrete has been hailed as a great benefit since this can dissipate stress concentrations and avoid overloading. While it may be very important in high stress environments, in shallow soft ground it may be less important than the phenomenon of arching in the ground.

Rheology is the study of flow. Hence the term 'rheological' is often used to cover empirically based creep models, such as those that have been

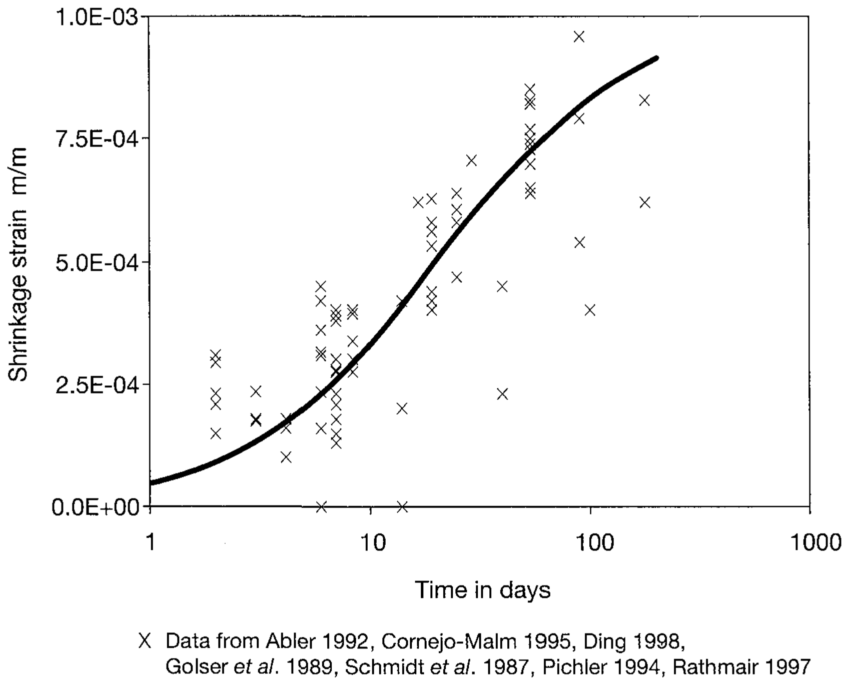


Figure 5.9 Shrinkage of sprayed concrete

idealised as an arrangement of simple units, each with certain defined behaviour (Neville *et al.* 1983). In the following sections other creep models – such as power law models – will also be discussed, with some comments on the effect of incorporating creep into the numerical model.

5.6.1 Rheological models

Typically these models consist of Hookeian springs, Newtonian dashpots and St Venant plastic elements, arranged in series or parallel (Jaeger and Cook 1979, Neville *et al.* 1983), although more exotic units have been devised (e.g. springs in dashpots or Power's sorption elements). Figure 5.10 (a, b and c) shows the three most commonly used rheological models, in the analysis of sprayed concrete – namely, the generalised Kelvin (Voigt) model, the Maxwell model and the Burgers model, which consists of a Maxwell model in series with a Kelvin model.

The three models listed above are viscoelastic models and so the principle of superposition can be applied. The spring stiffnesses and dashpot viscosities can be either linear or nonlinear. From Figure 5.10(a), it can be seen

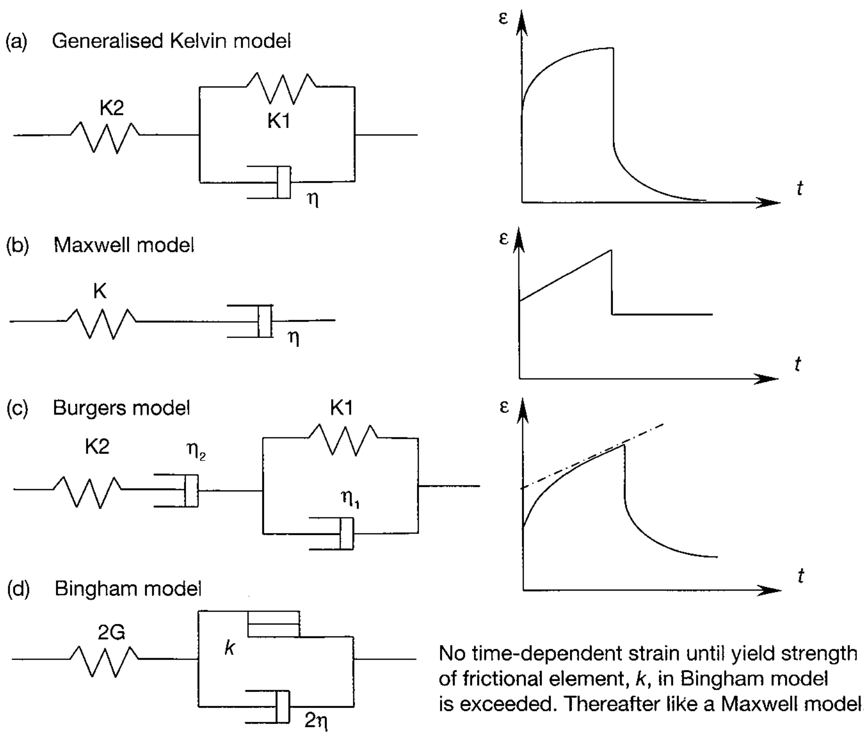


Figure 5.10 Rheological models

that the Kelvin model produces a complete recovery on unloading and so it is often used for fully recoverable transient creep. The Maxwell model produces no recovery and is used for steady-state creep, as well as stress relaxation, unlike the Kelvin model.[10] When combined in a Burgers model, one could say that they cover the whole of concrete's creep behaviour, with the Kelvin model replicating young concrete's behaviour and the Maxwell model the mature concrete behaviour. However, it would be unreasonable to expect that such a simple model could cover such complex behaviour and more elaborate rheological models (e.g. Freudenthal–Roll model with one Maxwell and three Kelvin elements in series) have been proposed (see Neville *et al.* 1983, Chapter 14 for an overview).

Appendix D contains a list of rheological models used in analyses of sprayed concrete linings, together with their parameters. In addition to the generalised Kelvin model and Burgers model, a modified Burgers model (with two Kelvin elements) and a Bingham model have been proposed. All of the rheological models appear to have been formulated for deviatoric stresses only, although there is some experimental evidence to suggest that considerable creep may occur under hydrostatic loading too (Neville 1995). The models are almost exclusively based on the results of uniaxial creep tests. In addition to creep in the direction of loading, lateral creep occurs (see Section 2.2.7).

5.6.2 Generalised Kelvin model

In its mathematical formulation, the Kelvin model requires two parameters, G_k and η, in addition to the normal elastic moduli, as shown below:

For a uniaxial case:

$$\varepsilon_{xx} = \frac{\sigma_{xx}}{9K} + \frac{\sigma_{xx}}{3G} + \frac{\sigma_{xx}}{3G_k}\left(1 - e^{-G_k t/\eta}\right) \tag{5.12}$$

and in the 3D case:

$$\dot{e}_{ij} = \frac{\dot{S}_{ij}}{2G} + \frac{S_{ij}}{2} \cdot \frac{1}{\eta_k}\left(1 - e^{-G_k t/\eta}\right) \tag{5.13}$$

or:

$$\dot{e}_{ij} = \frac{\dot{S}_{ij}}{2G} + \frac{S_{ij}}{2\eta_k}\left(1 - \frac{2G_k e_{ij}}{S_{ij}}\right) \tag{5.14}$$

where ε_{xx} and σ_{xx} are the strain and stress in the x direction, K is the bulk elastic modulus, G is the shear elastic modulus, t is time, S_{ij} is the deviatoric stress, $\dot{S}_{ij}$ is the deviatoric stress rate and $\dot{e}_{ij}$ is the deviatoric strain rate. As noted above, creep is generally assumed to occur under deviatoric loading only (Jaeger and Cook 1979, Neville *et al.* 1983).

Research in the mid-1980s by Rokahr and Lux made a significant contribution to the understanding of creep effects in early-age sprayed concrete. Based on the results of creep experiments on samples, they proposed a generalised Kelvin model, valid for ages from 8 hours to 10 days. They were able to model numerically what Rabcewicz (1969) had intuitively deduced – namely, that creep in the sprayed concrete reduces the stresses in the lining (Rokahr and Lux 1987) (see Figure 5.11). While the utilisation factor (stress/strength ratio) may be high at early ages, as the stress reduces due to creep and the strength increases, the utilisation factor falls and the factor of safety increases. Some subsequent fieldwork has supported this (e.g. Schubert 1988). Although a simple model, both the spring and dashpot parameters are stress-dependent and the viscosity increases with age (see Appendix D).

Others too (Swoboda and Wagner 1993, Kuwajima 1999, Sercombe *et al.* 2000) have adopted a generalised Kelvin model, most notably Kuwajima, who investigated how the parameters vary with age over the first 100 hours. However, for the purposes of back-analysing the creep tests, he chose average values, which leads to overestimates of strains. It was suggested that calculations could be simplified further, by assuming that the entire

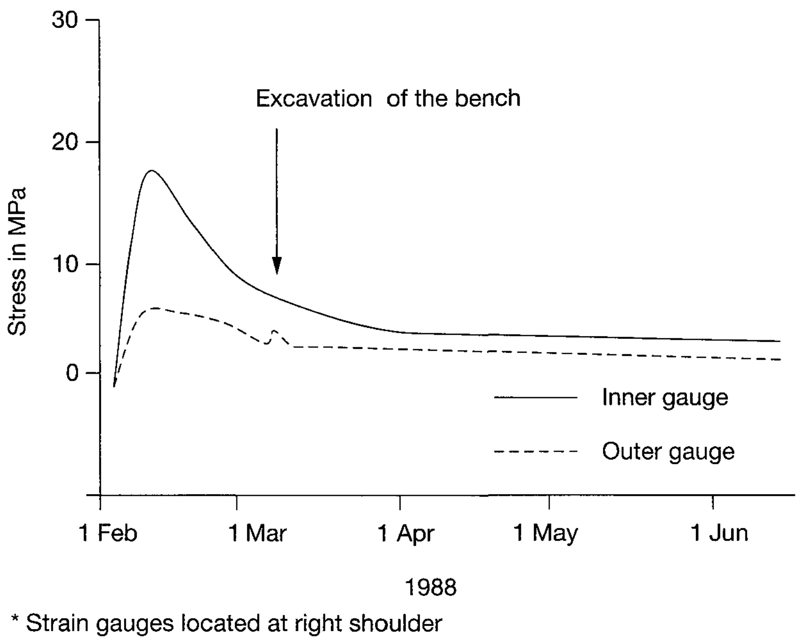


Figure 5.11 Stress reduction due to creep, computed from strain gauge data (Golser *et al.* 1989)

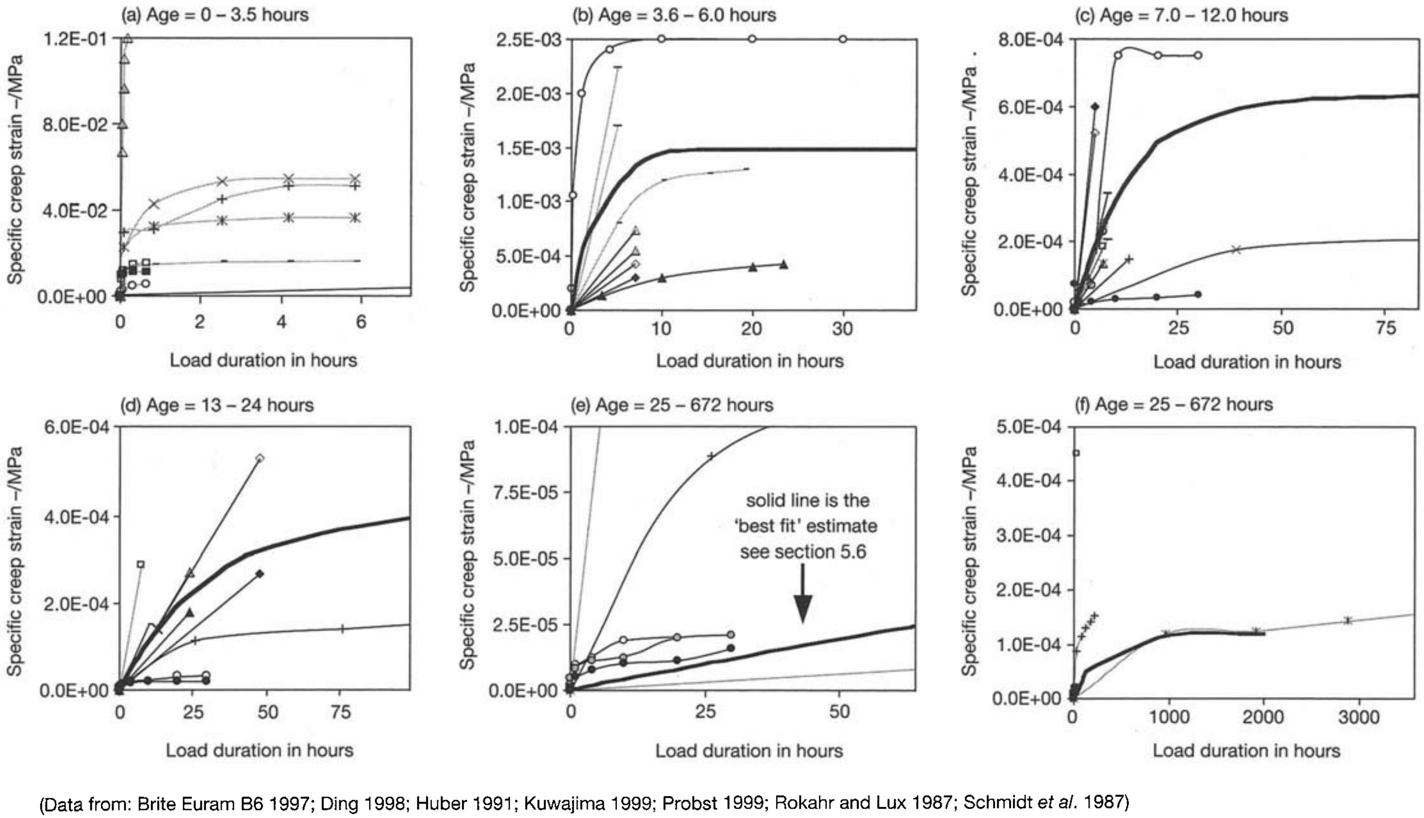


Figure 5.12 Specific creep strain of sprayed concrete, loaded at different ages

viscoelastic strain occurs instantaneously, without affecting the predictions adversely.

One of the weaknesses of research to date is its fragmented nature. Individual researchers have proposed theoretical models to suit the (limited) experimental data that they have been able to collect themselves. It would be preferable to calibrate the models against as much of the existing data as possible.

Figures 5.12(a) to (f) shows over 200 estimates of specific creep strains vs age from seven data sets, presented in groups according to the age at loading (Thomas 2003). Interpretation of creep tests is complicated by the fact that often the loads were applied incrementally at different ages. The total creep strain due to one load increment may not have developed before the next was applied. Furthermore, the utilisation factor may vary considerably during the test due to the ageing of the material (Huber 1991). From Figure 5.12, age is clearly a very important influence on specific creep strain (which is the creep strain divided by the magnitude of the load increment).

The two creep parameters for the Kelvin model can also be described as the specific creep strain increment, $\Delta e_{ij\infty} = 1/(2G_k)$, and the relaxation time, B, where $B = \eta_k/G_k$. In the formulation of Equation 5.13 the physical significance is clear. Namely, B is the time taken for 63.2% of the increment in creep strain to occur. When $t' = B$, $\Delta e_{ij\infty}(2G_k) = 0.632 = 1 - e^{-t'/B})$ or when $t = 3B$, $\Delta e_{ij\infty}(2G_k) = 0.95$. Tables 5.3 and 5.4 summarise the data from Figures 5.12 (a) to (f) in terms of these parameters.[11]

Table 5.3 Specific creep strain increment, $\Delta\varepsilon_{xx\infty}$, in –/MPa

Age at loading in hours	*Lower bound*	*Average*	*Upper bound*
0–3.5	1.5e-3	4.0e-2	8.0e-2
3.6–6.0	5.0e-4	1.5e-3	2.5e-3
7–12	2.0e-4	6.5e-4	7.5e-3
13–24	2.0e-5	4.0e-4	8.0e-4
25–672	5.0e-6	1.0e-4	2.0e-4

Table 5.4 Relaxation time, B, in hours

Age at loading in hours	*Lower bound*	*Average*	*Upper bound*
0–3.5	0.50	0.75	1.00
3.6–6.0	1.0	3.0	10
7–12	5.0	14.0	25
13–24	10	30	50
25–672	100	500	1000

Ageing

Yin (1996) proposed formulae of the form:

$$X = \frac{X_{28} a\, e^{c/t^{0.6}}}{2(1+v)}$$

with established equations for predicting the development of stiffness and strength with age (e.g. Chang and Stille 1993) to account for the ageing of creep behaviour. The parameters for a Kelvin model could be assumed to vary with age in this fashion, according to the equations below:

$$\eta_k = \frac{1.5e^{11}\,1.0e^{-1.5/(T/24)^{0.6}}}{2(1+v)} \text{ in kPa·s} \qquad (5.15)$$

$$G_k = \frac{8.0e^{6}\,1.0\,e^{-1.0/(T/24)^{0.6}}}{2(1+v)} \text{ in kPa} \qquad (5.16)$$

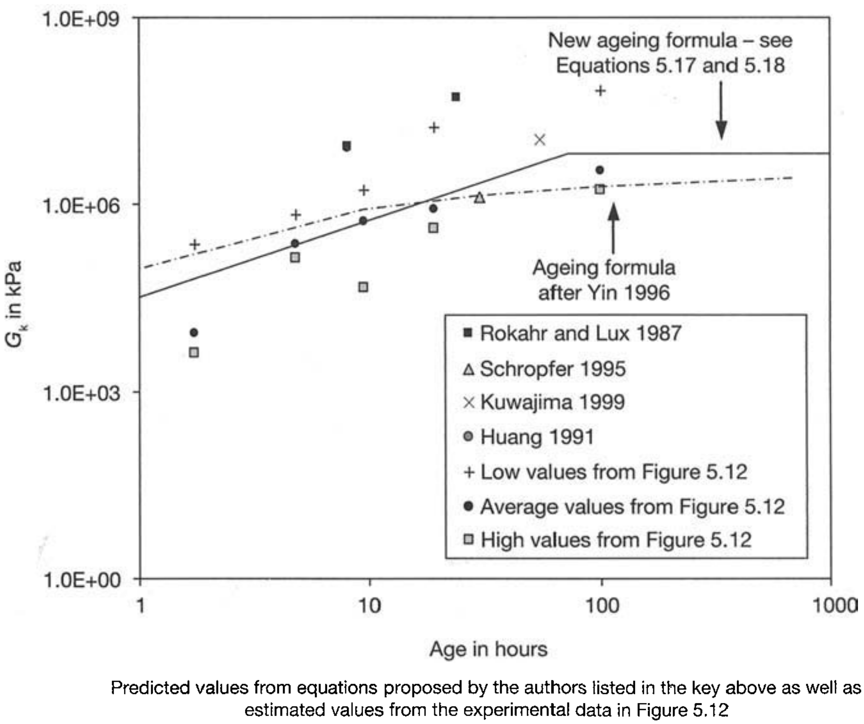


Figure 5.13 Shear stiffness (of spring in Kelvin rheological model), G_k, vs age

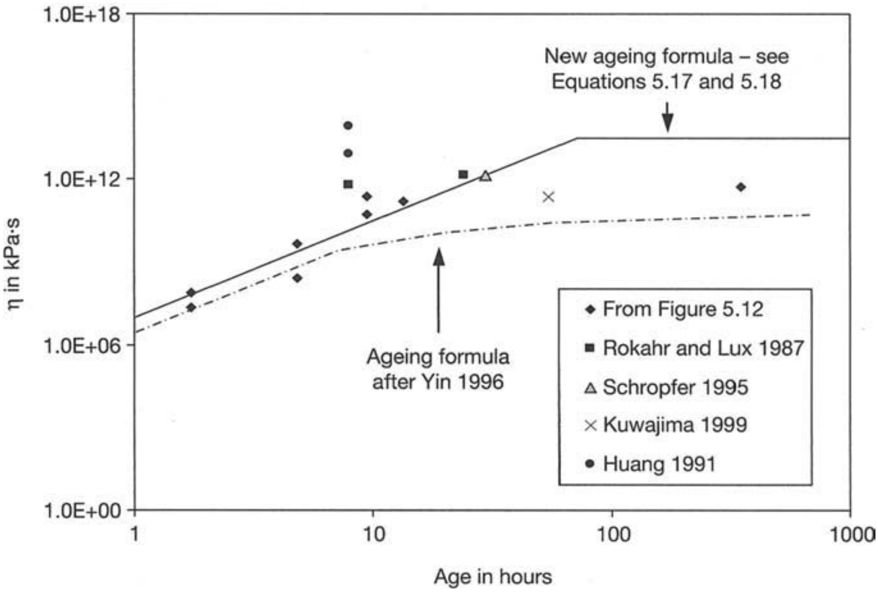


Figure 5.14 Viscosity of damper (in Kelvin rheological model), η_k, vs age

where T is the age of the sprayed concrete in hours, v is Poisson's ratio and the other parameters have been chosen to obtain a reasonable fit to the data (see Figures 5.13 and 5.14). The solid lines on Figures 5.12 (a) to (f) shows the predicted specific creep strains from Equations 5.15 and 5.16. Figure 5.15 shows the relaxation time calculated from data in Figures 5.12 (a) to (f), along with the prediction from Equations 5.15 and 5.16.

Also plotted on Figures 5.13 and 5.14 are approximate lines of 'best-fit' through the data:

$$\log_{10} G_k = a_g \log_{10} (T+b_g) \tag{5.17}$$

$$\log_{10} \eta_k = a_\eta \log_{10} (T+b_\eta) \tag{5.18}$$

where $a_g = 1.25$, $b_g = 4.50$, $a_\eta = 3.50$ and $b_\eta = 7.00$.

According to these equations, if $T = 100$ hours, $G_k = 1.0e^7$ kPa, which equates to a uniaxial specific creep strain, $\Delta\varepsilon_{xx\infty}$, of $3.33e^{-5}$ –/MPa (from Equation 5.12), and $B = 2780$. The modified 'Yin' formula predicts quite different values of $G_k = 1.89e^6$ kPa (equivalent to $1.76e^{-4}$ –/MPa) and $B = 5$. Experimental data suggests that the specific creep strain is about $1.0e^{-4}$ –/MPa and $B = 500$ (see Tables 5.3 and 5.4). One should bear in mind that the majority of the data points for the loading age range of 25 to 672 hours refer to tests that were started at ages less than 80 hours. Hence the results may be biased towards a more pronounced creep behaviour than if more tests had started later.

Given the scant data for loading at ages greater than 100 hours, it would seem reasonable to assume that the sprayed concrete obeys the existing predictions for the creep of mature concrete. According to the ACI method (ACI 209R (1992)), one would expect specific creep strains of about $1.08e^{-4}$ –/MPa and $0.68e^{-4}$ –/MPa after 700 hours for loading at an age 168 and 672 hours, respectively. Eurocode 2 (2004) suggests a specific creep strain of about $1.11e^{-4}$ –/MPa after 700 hours, for normal C25 concrete, loaded at 168 hours.

From a visual inspection of Figures 5.13 and 5.14, it would seem that the ageing formulae after Yin overestimate how fast the creep occurs (i.e. underestimate B – see Figure 5.15) at all ages, and the magnitude of the creep increment for ages greater than 100 hours. The logarithmic ageing formulae agree better, except in the age range greater than 100 hours where the creep increment is probably underestimated and the relaxation time overestimated. Therefore, these formulae are proposed as a good approximation, with the proviso that they are capped at 72 hours[12] (as shown in the figures).

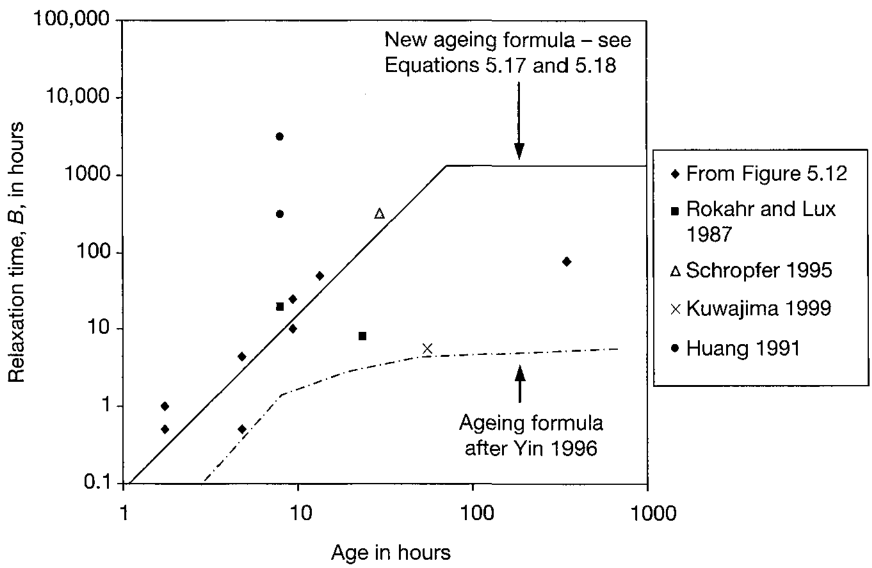


Figure 5.15 Relaxation time, B, vs age ($B = \eta_k/G_k$)

Loading/Unloading

In the case of varying loads, superposition is normally assumed (Neville *et al.* 1983). Unloading can be modelled as the addition of a negative load increment.

In terms of simulating creep in numerical models, the treatment of loading and unloading can be problematic. If the duration of each load increment is longer than 3 B, each load increment could be treated separately, since the creep due to that increment would be essentially complete before the next one is added. If the load duration is shorter, one is faced with the question of whether to use the total stress in a zone/element or the stress increment during the creep calculation for each advance. Applying the normal principle of superposition would be very complicated. Even if one assumed that the creep strain increment during the advance of the tunnel face could be calculated individually for each load increment due to all the previous advance lengths, this would overlook the fact that, in this soil-structure interaction problem, the applied stress is not necessarily constant during the duration of an advance.

As a compromise one approach would be to assume that when the time duration of each advance is greater than $1.5B$,[13] only the increment in stress for that advance is used in the creep calculation (Thomas 2003). If the duration is less, it could be assumed that the time is insufficient for most of the creep strain increment to occur and so the increment in stress is allowed to accumulate.

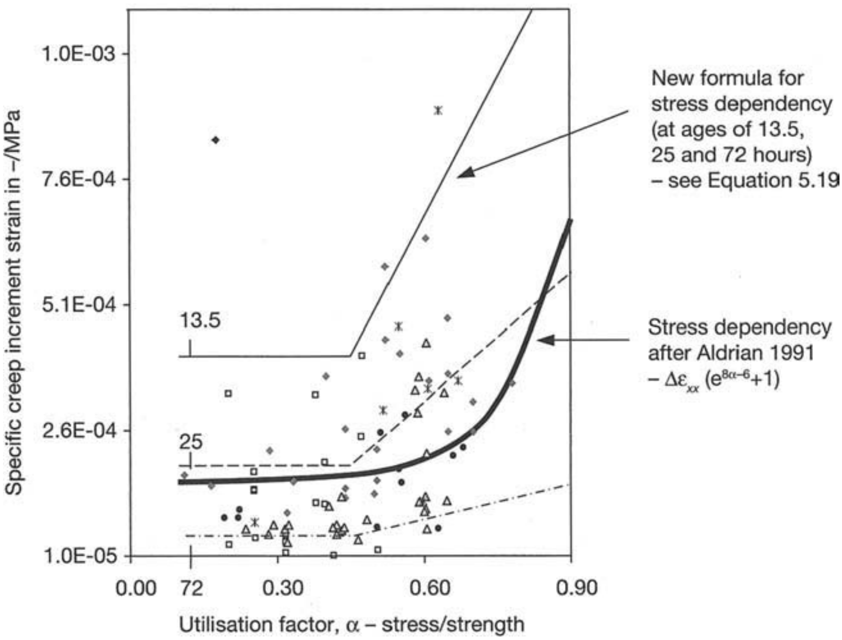
Stress dependency

It has been widely reported that the creep strain rate is more than directly proportional to the applied stress for stresses greater than $0.5f_{cu}$ (Rokahr and Lux 1987, Pöttler 1990, Aldrian 1991). A stress dependence is discernible in Figure 5.16 but, due to the large scatter, its exact nature is unclear. The exponential stress dependence proposed by Aldrian would appear to overestimate the dependence for values of utilisation factors, α, of 0.45 to 0.75 and underestimate it for $\alpha > 0.75$. Based on Figure 5.16, as an initial estimate of the stress dependence, the following relationship for predicting the specific creep increment has been proposed for $\alpha > 0.45$ (Thomas 2003):

$$\Delta e_{ij\infty} = (1 + 2.5{*}(\alpha - 0.45)/0.55)/(2G_k) \qquad (5.19)$$

Validation

In the preceding sections new relationships have been described for the parameters of a generalised Kelvin model. Before any new constitutive model is tried out in a full-scale model of a tunnel, it is important to validate it against the original experimental data it is based on. In this case, the



Data from Ding 1998, Huber 1991, Schmidt *et al.* 1987, Probst 1999

Figure 5.16 Specific creep strain increment vs utilisation factor

creep model above was used to simulate the uniaxial creep test by Huber (1991) – see Thomas (2003) for full details. The model (with parameters chosen to match the test data – 'VE matched') agrees to within –5% of both the analytical solution and the experimental data (see Figure 5.17). Encouragingly, Figure 5.17 also shows that the model (with the average parameters based on results of different researchers – 'VE Kelvin') agrees reasonably well with the experimental data.

5.6.3 Burgers model

Several researchers have proposed using a Burger's model (see Figure 5.10 (c)). Based on a Burgers model but formulated as a time-hardening model, Petersen's model was valid for low levels of stress only (Yin 1996). Pöttler (1990) corrected this and expressed it in polynomial form. However, doubts remain over the validity of the parameters. The original data came from tests performed at ages greater than 30 hours. The creep rate calculated with Pöttler's formulae initially decreases with time but then rises after 1.5 days, in contrast to the observed behaviour. Yin attempted to correct this by re-formulating the model (as a power law creep model) and estimating the parameters, based on the assumption that they increase with age in the

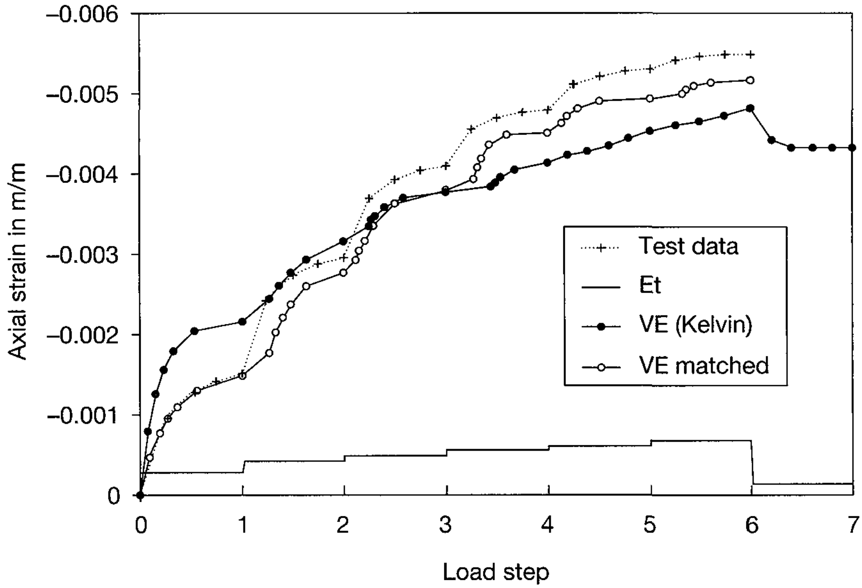


Figure 5.17 Comparison between FLAC models and test data from Huber 1991 (see Appendix F for explanation of key)

same manner as strength and elastic modulus, as proposed by Weber (see Appendix A and Figures 2.5 and 2.11).

Zheng (referred to in Yin 1996) and Huang (1991) utilised an expanded Burgers model, though, in contrast to Neville *et al.* (1983), they viewed the Maxwell element as representative of the behaviour of young concrete and the Kelvin elements of mature concrete. While the irreversible viscous flow of a dashpot may make sense for very young sprayed concrete (or concrete in the case of Zheng, who was examining an extruded concrete lining), creep at mature ages is not fully reversible, as the Kelvin element implies.

5.6.4 Viscoplastic model

In the course of the Brite Euram project on sprayed concrete, a Bingham model for shear stresses was proposed for very young sprayed concrete – i.e. between two and seven hours old (see Figure 5.10 (d)). Four of the five parameters in this viscoplastic model are believed to vary with age, although this was not incorporated into the formulae. The fifth, Poisson's ratio, may also vary with age during the first 12 hours (see Figure 2.12). The resistance of the frictional element (St Venant element) increases with increasing hydrostatic stress. This model may have academic merit but for real tunnels it is not useful because it refers only to the earliest ages of sprayed concrete.

5.6.5 Rate of flow model

A model has been proposed for sprayed concrete, based on the Rate of Flow Method (England and Illston 1965, Schubert 1988, Golser *et al.* 1989) with a view to back-calculating stresses from strain histories (Schropfer 1995). Strain histories can be broken down into four main components, as below (see also Figure 5.18):

(a) instantaneous recoverable strain – elastic strain
(b) recoverable creep – delayed elastic strain
(c) irreversible ('yielding') strain – irreversible creep strain
(d) shrinkage (and thermal strain)

Thermal strains were regarded as negligible (Golser *et al.* 1989). Based on extensive test results, equations have been written to describe each component (Golser *et al.* 1989) and subsequently refined to obtain better agreement at early ages and make the elastic modulus dependent on stress intensity (Aldrian 1991) – see Appendix D.

Aldrian (1991) proposed a relative deformation modulus, V*, which is a reduction factor for the elastic modulus to account for the effect of the

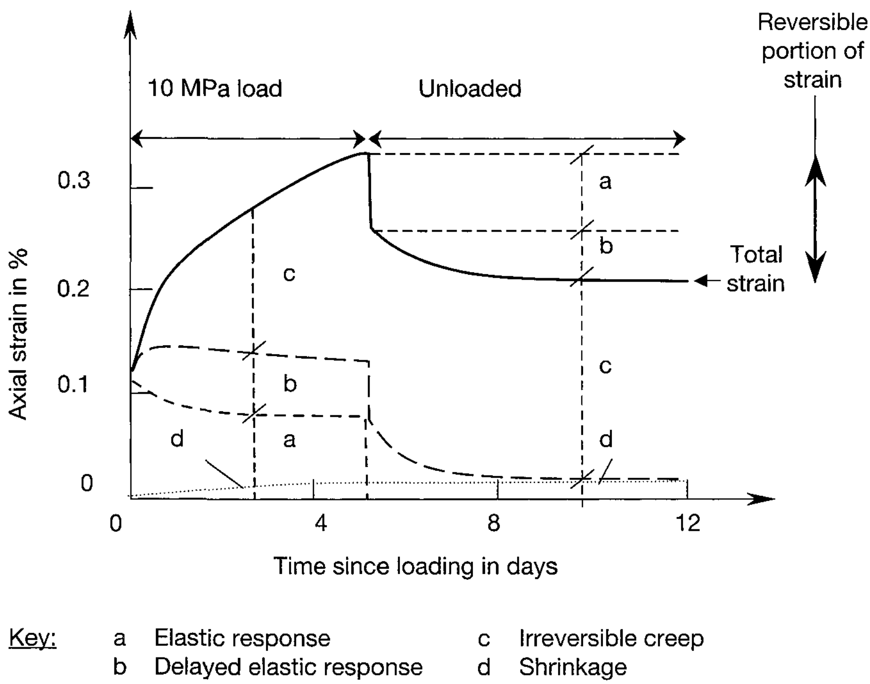


Figure 5.18 Decomposition of strains according to the Rate of Flow Method (after Golser *et al.* 1989)

utilisation factor, α, – the stress/strength ratio – as well as the age of the sprayed concrete (Appendix B). It is unclear what this actually means, since pre-loading, even to high utilisation factors, does not appear to reduce the initial slope of the stress-strain curve on reloading (Moussa 1993, Probst 1999). In contrast, the reloading modulus is usually higher. The factor, V^*, ranges from 1.0 at $\alpha = 0.0$ to 0.13 at $\alpha = 1.0$, at 28 days. It seems that this factor was intended for use in the incremental form of the equations to convert the initial elastic modulus into a tangent modulus and so account for the nonlinear nature of the stress-strain curve.

Many numerical analyses have been performed at the Montanuniversitat Leoben, in Austria, using the modified Rate of Flow Method, most recently to investigate the transfer of loads between primary and secondary linings (and the ground) due to creep (Aldrian 1991, Pichler 1994, Rathmair 1997, Schiesser 1997). Golser and Kienberger (1997) contains an overview of this work. However, it has not been possible to implement this method in 3D finite element analyses (Rathmair 1997) and in 2D analyses it has been noted that agreement becomes poorer with increasing numbers of load steps. The Rate of Flow Method has also been criticised because it relies on the principle of superposition and therefore there is no allowance for plastic strains.

5.6.6 Other creep models

Apart from rheological models, the other existing creep models include power laws (e.g. Andrade's 1/3 power law (Jaeger and Cook 1979)) and creep coefficients, as well as some methods which have been mentioned already – namely, the Effective Modulus (or HME – see Section 5.2) and Rate of Flow methods.

Power laws (Jaeger and Cook 1979) are empirical in nature and were first used to fit curves to data on creep in metals. Of the three stages of creep – transient, secondary and tertiary – only the first is of interest in the case of sprayed concrete linings soon after construction. The general form for a transient creep power law is $\varepsilon = At^n$, where ε is the strain, t is time and A and n are constants. Although a few researchers have used forms of power laws in the analysis of SCL tunnels (Schubert 1988, Alkhiami 1995, Yin 1996, Rathmair 1997), they are not widely used because of their inferior ability to model the complex creep behaviour (e.g. the existence of recoverable and irrecoverable portions of creep). Because of their widespread use in general engineering, power law creep models often come as standard in numerical analysis programs (e.g. ABAQUS, FLAC), but it is not always possible to combine the creep model with more sophisticated elastic or elastoplastic models in those programs (Rathmair 1997).

Standard methods for calculating the effects of creep in concrete have been published by various bodies, such as CEB-FIP and the American Concrete Institute (Neville *et al.* 1983, Chapters 12 and 13). Both use a creep coefficient, which is a multiple of parameters that account for factors such as water/cement ratio, cement content and size of the structural member. While the strength of these methods lies in their ability to obtain a coefficient for a wide range of concretes and situations, their weakness lies in the fact that the parameters are valid for hardened (normal) concrete, aged seven days old or more. They cannot replicate the early-age behaviour of sprayed concrete (Han 1995, Kuwajima 1999) and so they are generally not suitable for sprayed concrete. Furthermore, they are not suitable for the variable loads experienced by a tunnel lining.

Influence on the predictions of numerical models

Creep of sprayed concrete has long been postulated as being responsible for easing stress concentrations in linings (Rabcewicz 1969, Rokahr and Lux 1987, Soliman *et al.* 1994). Many numerical studies (albeit in 2D) have been performed which have demonstrated significant reductions in axial forces and bending moments when a creep model is used for the lining (e.g. Huang 1991, Schropfer 1995, Yin 1996) but hard evidence from the field in support of this is scarce.

Since the load-bearing system is a composite consisting of the ground and the lining, movement of the lining should be expected to cause movement of the ground and a change in the load on the lining. Therefore this case is not as simple as the standard laboratory creep test in which a constant load is applied to a sample and the increasing strain over time is recorded. Whether the creep of the lining results in a reduction in the lining stresses

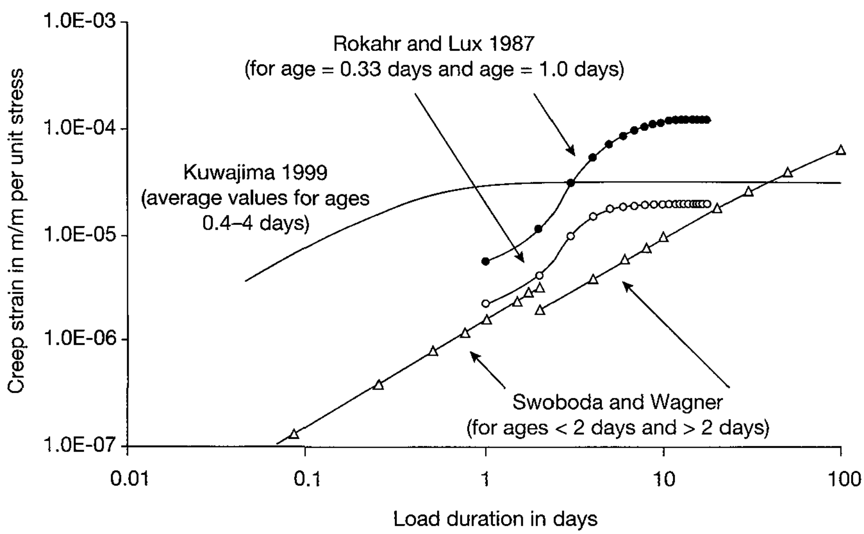


Figure 5.19 Predicted specific creep values

depends on the strength and material behaviour of the surrounding ground. If the ground is elastoplastic or susceptible to creep itself, the load in the lining might be expected to actually increase following creep (Pöttler 1990, Schiesser 1997, Yin 1996, Hellmich *et al.* 2000). The effects also depend on the rate of creep in the ground (Hellmich *et al.* 1999c). It is known that the creep capacity of concrete decreases rapidly with age. Several researchers have proposed relationships to predict this (e.g. Golser *et al.* 1989, Rokahr and Lux 1987, Pöttler 1990, Aldrian 1991) but there is little agreement between them on the rate of change of this property (see Figure 5.19 – normalised age-dependency of creep rate or specific creep). Similarly some have included stress-dependency in their creep models (e.g. Golser *et al.* 1989, Aldrian 1991, Probst 1999).

A few numerical models, that have included creep, have shown unrealistically large reductions in stresses. For example, in analyses performed by Rathmair (1997) the axial force was reduced to 5% of its initial value, although the same author reports a reduction of stress of only 50% in laboratory relaxation tests on sprayed concrete. However, Thomas (2003) did find that even in lightly loaded shallow tunnels, creep may lead to reductions

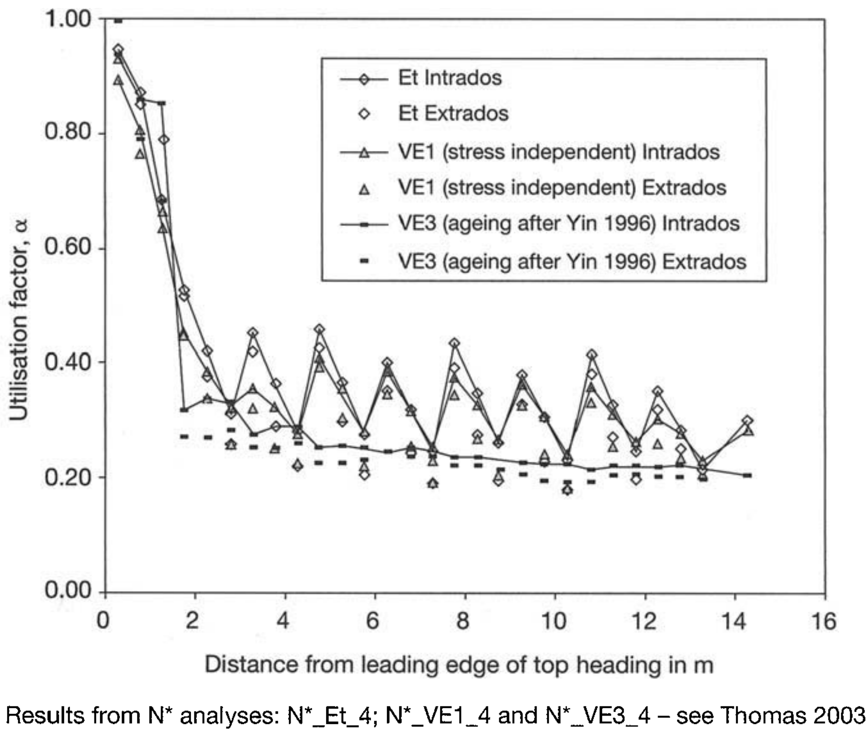


Figure 5.20 The effect of creep on utilisation factors

Note: See Appendix E for explanation of key.

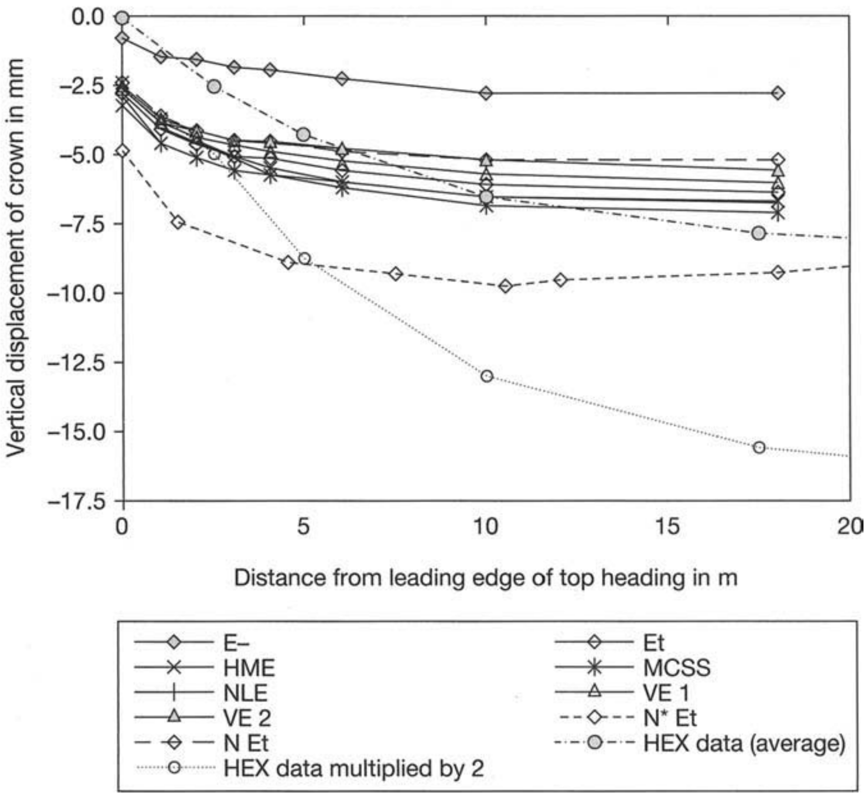


Figure 5.21 Crown displacement vs distance from leading edge of top heading
Note: See Appendix E for explanation of key.

in axial lining loads and bending moments – for example, see Figures 5.1 and 5.2. The extent of the reduction depends heavily on the parameters chosen for the creep model, ranging from 10 to 50%. Of the creep models investigated, the model described in Equations 5.17 to 5.19 was found to give the most realistic predictions. Therefore it is recommended that the creep model is calibrated against laboratory data for the exact sprayed concrete mix planned for each individual project. This particular study also showed how creep can 'smooth' out peaks in the stresses in linings – see Figure 5.20. As one would expect, creep also leads to higher predictions of lining deformations – see Figure 5.21.

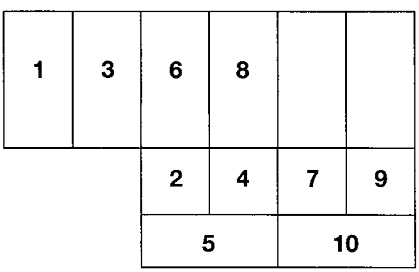
5.7 Ageing

Ageing makes the task of modelling sprayed concrete considerably more complicated than is the case for other lining materials. While in a design calculation one may assume that in a tunnel the load increment due to an advance is applied as soon as the ground is excavated and choose values of parameters (e.g. stiffness) that are consistent with the ages of the different parts of the lining at this moment (see Figure 5.22), one must also check that the new parameters are consistent with the constitutive model. All of the properties of sprayed concrete vary with age and Appendices A and B contain numerous empirical relationships for predicting the most commonly used properties (e.g. strength) at all ages. The age of the individual parts of the lining can be estimated, based on their distance from the tunnel face for a given advance rate and excavation sequence.

5.7.1 Thermo-chemo-mechanically coupled model

The fundamental reason for ageing is the ongoing hydration of the cement. Hence various researchers have sought to quantify how the degree of hydration (ξ) varies with time and then relate all the material properties to this. An overview of the theory can be found in Ulm and Coussy (1995, 1996). The thermo-chemo-mechanically coupled model aims to account for:

- the chemo-mechanical coupling between hydration and the evolution of properties such as strength, stiffness and autogeneous shrinkage;



Schematic of the tunnel, showing the stages of the top heading, bench and invert excavation sequence

Stage	Age at start	E/GPa
1	0	0
2	24	17.0
3	48	20.0
4	72	21.6
5	96	22.4
6	120	23.0
7	144	23.4
8	168	23.7
9	192	23.9
10	216	24.1

Notes: 1 All values estimated using Chang 1994 (see Appendix A)
2 Platform tunnels – 9.2 m OD, progress rate = 0.8 m/day

Figure 5.22 Typical approximation of age-dependent stiffness in a numerical model

- the thermo-mechanical couplings such as dilation due to the exothermic hydration reaction (Hellmich and Mang 1999) or damage criteria (Cervera *et al.* 1999b);
- the thermo-chemical couplings such as the reduction in final strengths and stiffnesses due to increased curing temperatures (Cervera *et al.* 1999b);
- the thermodynamically activated nature of hydration itself.

An underlying intrinsic material function – the chemical affinity or driving force of hydration ($A_{(t)}$ or $A_{(\xi)}$) – can be determined experimentally (see Figure 5.23). Being intrinsic it is meant to be independent of field variables and boundary effects (Hellmich *et al.* 1999a and Hellmich *et al.* 2000). Strength can then be related to the progress of this function with time, *t*, as below:

$$\frac{d\xi}{dt} = \left(1 - \xi_0\right)\left[\frac{df_c \,/\, dt}{f_{c,\infty}}\right] = A_{(\xi)} e^{-E_A/RT_t} \tag{5.20}$$

$$A_{(\xi)} = a_A \left(\frac{1 - e^{-b_A \xi}}{1 + c_A \xi^{d_A}}\right) \tag{5.21}$$

The exponential term in Equation 5.20 accounts for the thermodynamically activated nature of hydration. *T* is temperature in K. ξ_0 is the 'percolation

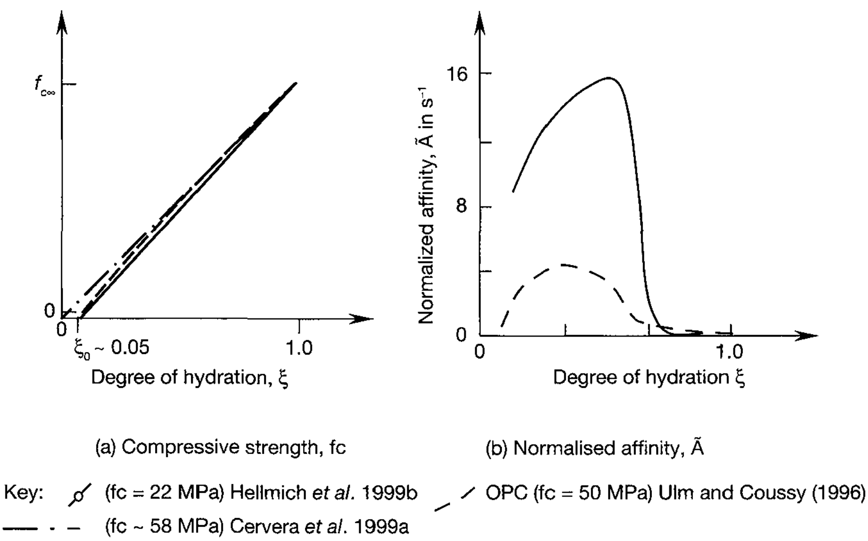


Figure 5.23 Hydration kinetics for shotcrete (after Hellmich *et al.* 1999b)

threshold', below which the concrete cannot sustain any deviatoric stress and f_c is the compressive strength. a_A, b_A, c_A and d_A are all constants (see Appendix F). Similarly, other relationships have been proposed between properties such as tensile strength, autogeneous shrinkage and stiffness and the degree of hydration (ξ) (see Figures 5.24 a and b) – see also Appendix A and Eierle and Schikora (1999). Sercombe *et al.* (2000) contains tentative relationships[14] for the development of short- and long-term creep with the degree of hydration. In common with the empirical formulae, it is assumed that the hydration kinetics are independent of the loading history.

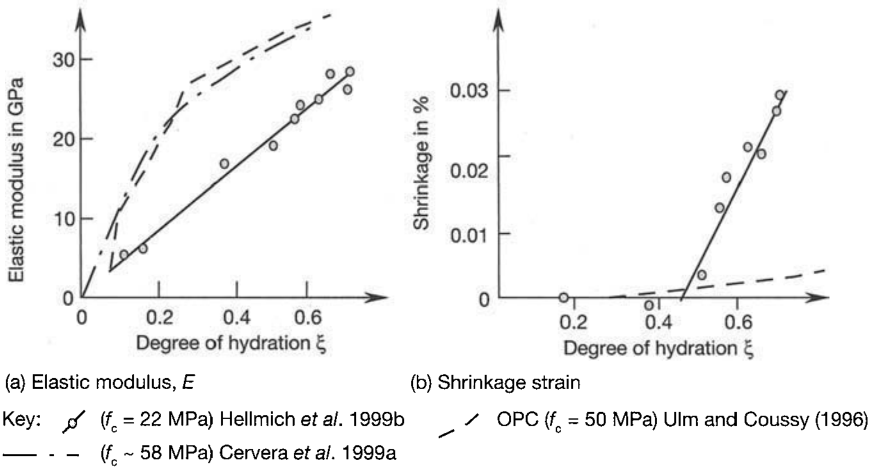


Figure 5.24 Variation of stiffness and shrinkage with the degree of hydration (after Hellmich *et al.* 1999b)

Alternatively, the effect of temperature on hydration can be accounted for by the simpler method of equivalent age (Cervera *et al.* 1999a, D'Aloia and Clement 1999). An elevated temperature speeds up hydration. The value at a given age can be read from a graph of the parameter's growth at a reference temperature, using an age, which has been corrected for the more advanced degree of hydration. Cervera *et al.* (1999a) note that normally all of the cement does not hydrate and the final degree of hydration can be found from:

$$\xi_\infty = \frac{1.031\ \mathrm{w/c}}{0.194 + \mathrm{w/c}} \sim 0.71 \text{ for w/c} = 0.43 \tag{5.22}$$

The initial 'percolation threshold', ξ_0, (sometimes known as the critical degree of hydration) has also been found to vary with water/cement ratio (Byfors 1980):

$$\xi_0 = k\cdot\mathrm{w/c} \tag{5.23}$$

where k varies between 0.40 and 0.46 and w/c is the initial water/cement ratio.

$A_{(\xi)}$ profiles have only been developed for a few concretes and different researchers have proposed widely differing values for some of the key parameters, for example the profile itself or the percolation threshold. A standard means of determining $A_{(\xi)}$ is to work back from data on the development of strength with time. However, if one already knows this, other parameters (e.g. stiffness and tensile strength) can be estimated directly. The benefit of the thermo-chemo-mechanical approach is that it links the properties to the fundamental process behind them – namely, hydration. This provides some further insight into the behaviour of concrete at early ages and makes the inclusion of temperature effects straightforward where this is relevant. While temperature effects are important for massive structures (e.g. Aggoun and Torrenti (1996), Hrstka *et al.* 1999), most tunnel linings are quite slim (i.e. less than 400 mm thick) and the elevated temperatures due to the heat of hydration are short-lived. Figure 2.18 shows that the effect of elevated temperature on hydration is limited for the temperatures experienced in most SCL tunnels.

However, temperature effects may be discernable in lightly loaded tunnels. Jones (2007) coined the term 'ground reaction temperature sensitivity' to describe the stresses induced in radial pressure cells in one shallow tunnel due to the expansion of the ring during hydration of the concrete. Even where temperature does not induce significant stresses compared to the ground load, it should be noted that temperature changes can induce fluctuations in measured stresses in SCL tunnels (see Section 7.2.1).

5.8 Construction sequence

Subdivision of the heading

For segmentally lined tunnels it can be reasonably assumed that the lining is constructed in one action, albeit at a certain distance from the actual face. In SCL tunnels the excavation and construction sequence is much more complex yet this is rarely replicated fully in numerical modelling. The way in which the sequence is simplified can have a great impact on the results. For example, modelling the excavation as full face rather than according to the exact sequence can lead to a more even pattern of lining loads (Guilloux *et al.* 1998) and less unloading in the ground (Minh 1999).

Advance length and rate

Experience and common sense suggest that increasing the advance length or rate may result in higher loading of the lining (Pöttler 1990). Convergence can increase by as much as 50% if the advance rate is increased from 2 to

8 m per day, while the hoop load in the lining could fall by 15%, for a tunnel in soft ground (Cosciotti *et al.* 2001). The difference reduces as the stiffness of the ground increases. The load decreases because the sprayed concrete is more heavily loaded at a young age and deforms more. This permits more stress redistribution in the ground and therefore a lower load in the final case. However, there is the risk that the lining is damaged through overstressing. Kropik (1994) reports a 20% increase in crown deformations when the advance length is increased from 1 to 2 m in a soft ground tunnel. Hellmich *et al.* (1999c) noted the importance of when the lining acts as effective support in determining the final loads, despite the simplifications of the construction sequence in their numerical model. It was also noted that the sooner the lining can carry load (i.e. the sooner the percolation threshold is passed in the thermo-chemo-mechanical constitutive model – see Section 5.7.1), the higher the loads in the lining. This applies for both axial forces and bending moments.[15] The concept of delaying the installation of the completed lining in highly stressed rock tunnels, in order to reduce the load in the final lining, is a basic tenet of the NATM.

The case of soft ground is somewhat different. There is some evidence to suggest that in soft ground the load in the lining is actually lower if the lining is installed sooner rather than later (Jones 2005, Thomas 2003). This is not necessarily at odds with the NATM philosophy since it recognises that, beyond a certain point, delaying the installation of the lining results in higher loads (due to the 'loosening' of the ground around the tunnel). In soft ground plastic deformation or strain-softening could cause higher loads. For tunnels in stiff overconsolidated clays, Jones (2005) ascribed the increase in long-term load to the equalisation of negative pore pressures that had been generated by the unloading of the ground during construction of the tunnel. This occurred mainly in the invert. The greater the unloading (e.g. due to a delay in closing the invert), the larger the negative pore pressures and the higher the subsequent increase in load.

Similarly, in line with experience on site (e.g. Thomas *et al.* 1998), Kropik (1994) noted that closing the invert early (i.e. close to the face) led to a reduction in deformation of the lining.

In the past, 'unlined' analyses have often been used in 2D and 3D simulations (e.g. Gunn 1993, Krenn 1999, Minh 1999, Burd *et al.* 2000). Researchers have often reported that they obtained a good correlation between the results of such analyses and field data of settlement. Obviously such simulations are completely unrealistic and any apparent good match probably stems from peculiarities of the analyses (e.g. the use of a prescribed volume loss or a simple ground model), which prevent the failure that would occur in the real case. Dasari (1996) notes that in his work there was little difference between 2D analyses of lined and unlined tunnels but that the introduction of the lining greatly reduced the settlements in the 3D analyses of the same tunnel.

Load development

The load from the soft ground is generally assumed to increase with time monotonically (e.g. Grose and Eddie 1996). While this may be true for segmentally lined tunnels (e.g. Barratt *et al.* 1994), given the complex excavation sequences and geometries in SCL tunnels, the mode of action of the lining may well change and the loading may vary. For example, one could consider the lining in the top heading as cantilevering off the completed rings behind it initially, resulting in bending in the longitudinal direction (see Figure 1.1). When the ring is completed, the lining acts mainly in compression and the main bending moments act in the hoop direction.

It is worth noting in passing that the longitudinal stresses in tunnel linings have rarely been examined in detail. In his numerical modelling study Thomas (2003) found that in the top part of the lining – i.e. above axis – the extrados of the lining was in tension due to the relative movement of the ground towards the face. The intrados was in compression. This process continued far back from the face and therefore it was the stiffness here that helped to determine the longitudinal forces and bending moments in the final situation – see Figures 5.25 and 5.26. Hence, the constitutive model of the sprayed concrete has a large influence on the predicted longitudinal forces. In general the longitudinal forces are small if the age-dependent

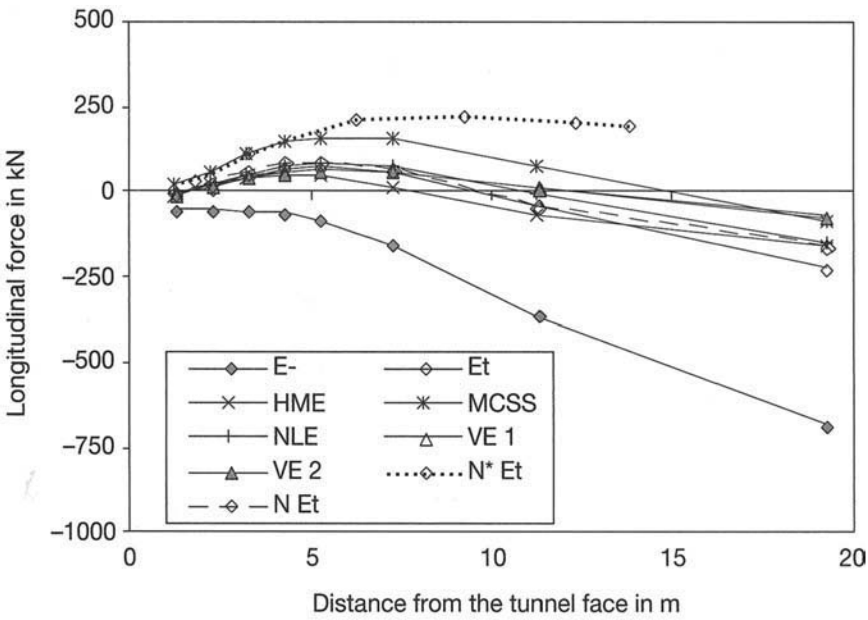


Figure 5.25 Longitudinal axial forces in the crown vs distance to face

Note: Comprehensive forces are positive. See Appendix E for explanation of key.

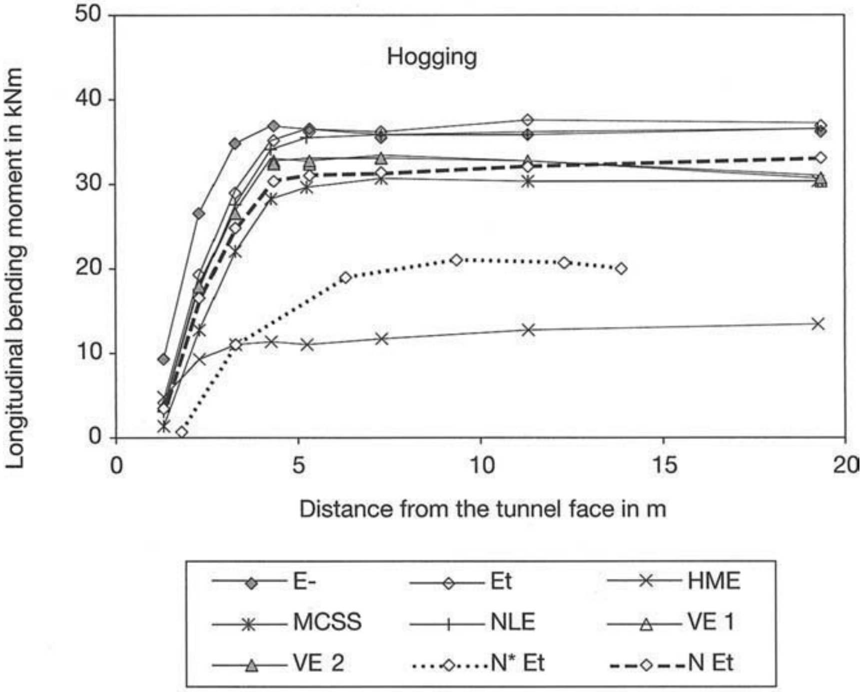


Figure 5.26 Longitudinal bending moments in the crown vs distance to face
Note: See Appendix E for explanation of key.

stiffness is incorporated into the model. However, the bending moments were high for all models, except the HME model. The tension cut-off in the plasticity model and creep in the viscoelastic models helped to reduce the tension and therefore the bending moments. The lowest loads were predicted by the HME model because of its relatively low stiffness throughout the lining.

In the absence of field data on longitudinal loads it is not known how realistic these predictions are. Since the tension is on the extrados, cracking would be hidden from view. Potentially this could cause a durability problem as it would permit water access into the body of the lining.[16]

Stress distribution

Thomas (2003) found in his numerical study that in general the predicted stress distribution in the lining was not uniform. The utilisation factor in the lining was highest near the face and reduced with distance away from the face (see Figures 4.12 and 5.27). Furthermore, as Kropik (1994) found,

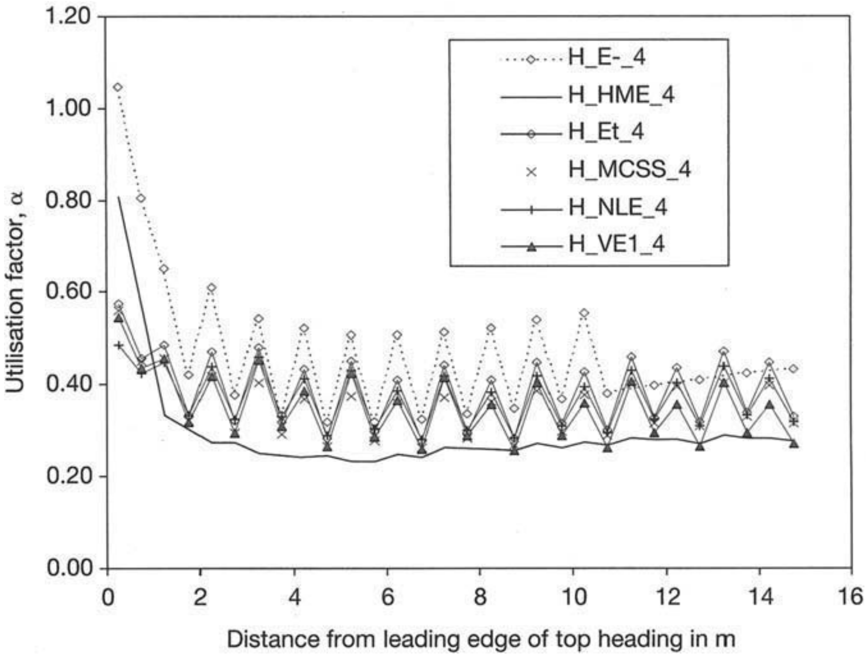


Figure 5.27 Utilisation factors in the crown at extrados vs distance from leading edge (Thomas 2003)

Note: See Appendix E for explanation of key.

the stress at the leading edge of each advance length was found to be greater than at the trailing edge. This may have been due to the difference in the concrete stiffness in adjacent parts of the lining and the higher radial loads at the leading edge. Stresses were higher in the first part of the lining that was constructed – i.e. the stresses were higher in the top heading than the invert.

The Sequence Factor

Given the influence of the construction sequence in SCL tunnels it would be useful to have a means of estimating the impact of the key parameters. Three of the main parameters – advance length (AL), advance rate (AR) and distance to ring closure (RCD) – are interrelated since altering one will affect the others. One way to assess the impact of changes in construction sequence may lie in considering the combination of those key parameters into a new factor – the 'Sequence Factor':

$$\left(\frac{RCD}{AR}\cdot\frac{Ex}{E_{28}}\cdot\frac{AL}{R}\right) \tag{5.24}$$

which consists of:

RCD/AR – ring closure distance/advance rate ~ the time taken to close the ring;

E_x/E_{28} – the ratio of Young's modulus of the sprayed concrete at ring closure to the 28-day value ~ a measure of how stiff the ring is in compression at closure;

AL/R – advance length/tunnel radius (*R*) ~ a measure of the relative size of the unsupported length during excavation.

Results from one numerical study are presented in Figure 5.28 and they suggest that both hoop forces and hoop bending moments – at least in the top part of the tunnel – increase with an increase in the Sequence Factor (Thomas 2003). In other words the loads increase if the time to close the invert is longer, the concrete is older when the invert is closed or the support is installed more slowly (i.e. longer unsupported length).

The pattern was less clear in other parts of the tunnel lining (such as at the axis level or in the invert).

5.9 Construction defects

Despite the fact that SCL tunnelling is known to be vulnerable to poor workmanship, common construction defects such as variation in strength

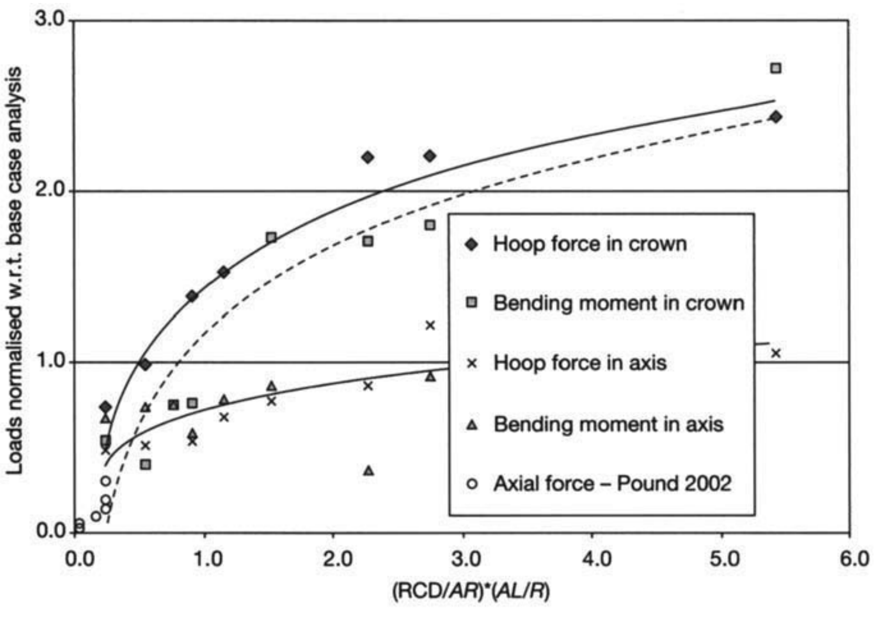


Figure 5.28 Normalised hoop loads vs (*RCD/AR*)*(*AL/R*) corrected for tunnel radius and stiffness at ring closure at 9 m from the face

and quality, poorly constructed joints and variations in shape and thickness are not normally considered in design calculations. In contrast, in the design of segmental linings, it is routine to consider the effects of ovalisation (due to ground deformation or poor build quality) and the misalignment of segments (so-called 'stepping').

Stelzer and Golser (2002) examined the effects of the sort of variations in profile of a sprayed concrete lining that are found in drill and blast tunnels. In their detailed study of both small-scale models and the back-analysis of them with a numerical model, they found that imperfections could reduce the structural capacity of a lining by more than 50%. The imperfect linings tended to deform more too.

SCL tunnels contain many joints and these can be areas of weakness (e.g. HSE (2000) and Figure 3.16). In one study of a SCL tunnel in soft ground, the strength of the lining was reduced by 50% at the joints in the numerical model (see Figure 5.29). Figure 5.30 shows that the presence of weak zones at circumferential and radial joints can alter the stress distribution within the tunnel lining (Thomas 2003). Weak radial joints tended to increase the loads near the face while weak circumferential joints reduced the loads and the lining functioned more like a tube consisting of discrete rings. Weakening both radial and circumferential joints worsened the situation in the critical area near the face.

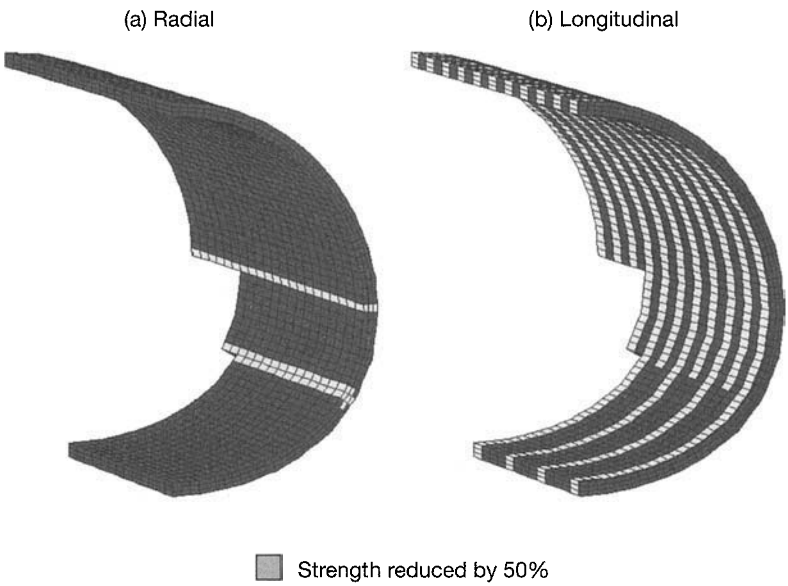


Figure 5.29 Locations of joints in mesh of the SCL tunnel modelled by Thomas (2003)

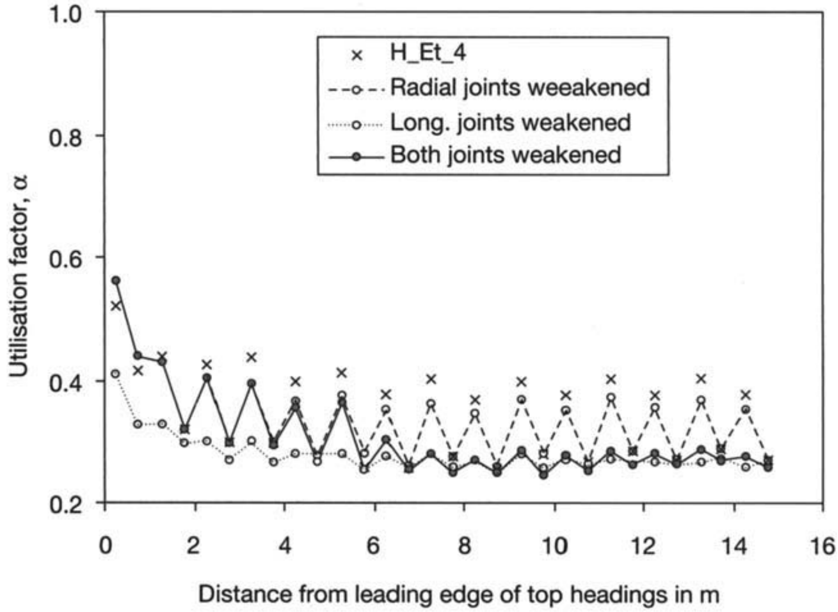


Figure 5.30 Utilisation factors in the crown vs distance from leading edge (for models with weak joints) (Thomas 2003)

5.10 Summary

Numerical modelling plays an increasingly important role in the design of SCL tunnels. The choice of constitutive model for the sprayed concrete can have a significant influence on the results of this numerical modelling. This applies both to the tunnel lining itself and the ground around it. Figure 5.21 illustrates how predictions of lining deformations can vary depending on the model, while Figure 5.31 shows that even the surface settlement can be influenced. Despite the complex behaviour of sprayed concrete, simplistic models tend to be used.

Figure 5.32 demonstrates the potential benefit of using more sophisticated numerical models (Thomas 2003). These models predicted bending moments that would have been small enough to permit the use of steel fibres instead of wire mesh reinforcement. Having removed this major durability concern related to bar reinforcement, a 'one pass' permanent sprayed concrete lining becomes a viable option. Taking one example, this could result in an estimated cost saving of 30%.[17] However, one should note that the lining loads are only one of many considerations in lining design.

There is no single right answer to the question of which constitutive model should be used for sprayed concrete. Above all else, the chosen model must be appropriate for the design calculation. At this point, it is worth

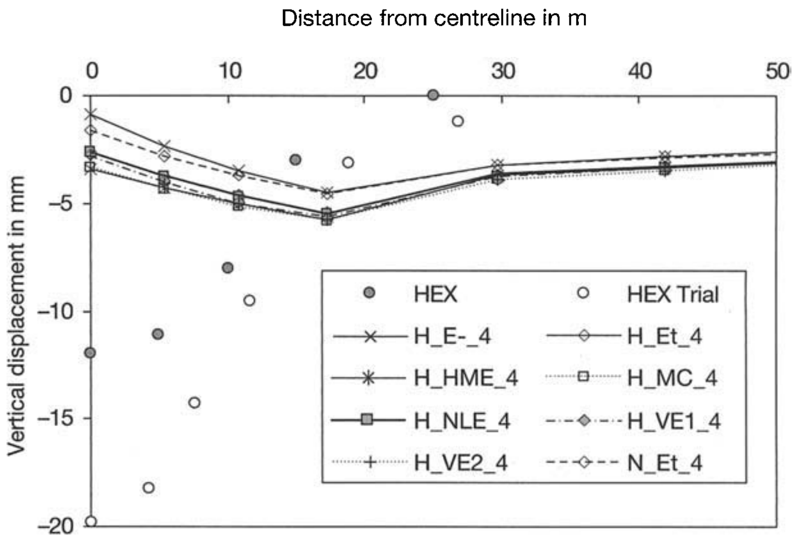


Figure 5.31 Transverse surface settlement profile at 18 m from the face (Thomas 2003)

Note: See Appendix E for explanation of key.

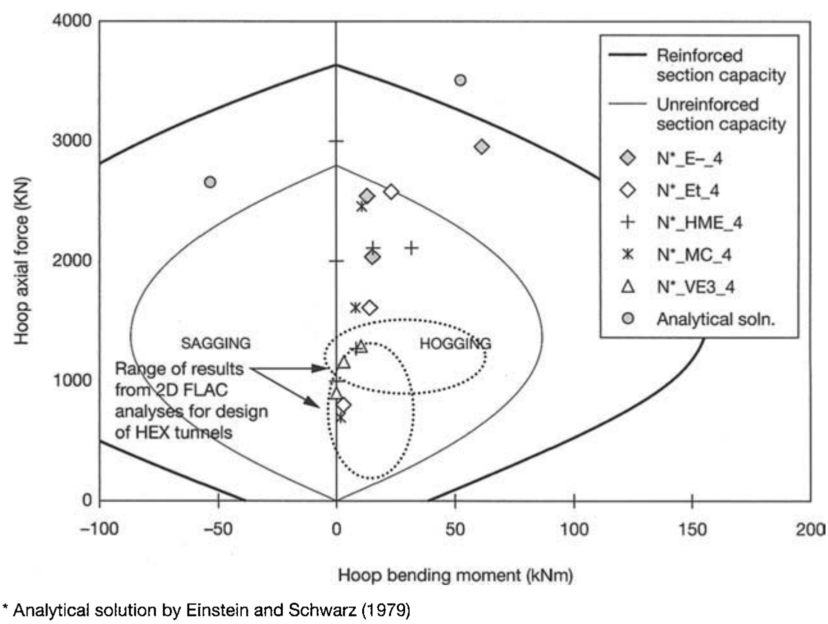


Figure 5.32 Results from numerical model of a shallow SCL tunnel in soft ground (Thomas 2003)

Note: See Appendix E for explanation of key.

reiterating that the constitutive model for the lining is only one part of the numerical model. Different aspects of the model for sprayed concrete may assume more or less importance depending on what is the main focus of interest. For example, the model of the tunnel lining may have little influence on the prediction of surface settlements. One should also bear in mind that, in principle at least, the same outcome could be obtained from a complex model by using (widely) different parameters and/or 'sub-models' for the various elements.

Notes

1 This section draws heavily on that thesis. Appendix E contains the key for the numerical models from that thesis which are referred to in figures in this chapter.
2 For example, if an age-dependent elastic model is used, the invert of the tunnel will be relatively soft when the ring is first closed and will permit much more deformation than if a high stiffness is assigned to that section as soon as it is built.
3 If the load is applied during a period that is much longer than that for hydration, numerical analyses give virtually the same results as one obtains by using a constant stiffness model with the 28-day stiffness, since most of the load is applied when the stiffness of the lining is close to this value (Hellmich *et al.* 1999c).
4 Also the Trost–Bazant creep model uses an age-dependent effective modulus (Neville *et al.* 1983).
5 At early ages, due to the more ductile response, the ratio of yield strength to ultimate strength is higher – between 0.5 and 1.0 (Aydan *et al.* 1992a, Moussa 1993). Rokahr and Lux 1987 report a linear response up to 0.8 at 24 hours (see also Section 2.2.1 and Figure 2.3).
6 Although Moussa (1993) did also propose a seventh-order polynomial function to describe the uniaxial compressive stress-strain curve more precisely.
7 There is a preference in geotechnical engineering for the name Coulomb and in applied mechanics for the name Mohr, hence the name Mohr–Coulomb is used here (Chen 1982).
8 The maximum strength permitted by BS8110 Part 2 (1985) is $0.8f_{cu}$ ($= f_{cyl}$) and this is only for the analysis of non-critical sections.
9 From Chen (1982); Yin (1996) proposed 40°.
10 Strictly speaking creep refers only to increasing strain with time, under a constant load. Relaxation refers to the reduction in stress over time observed in samples held under a constant strain.
11 The data comes from uniaxial creep tests and therefore the parameters have been determined using $\Delta\varepsilon_{xx\infty} = 1/3\ G_k$.
12 Obviously creep remains age-dependent beyond the age of 72 hours and the logarithmic ageing formula could be amended to reflect ageing in line with published formulae or data (e.g. Eurocode 2 or ACI 209R).
13 1.5 B = the time for 0.78 of the creep strain increment to occur
14 The relationships are based on one creep test on a sprayed concrete sample at an age of 28 days.
15 Hellmich *et al.* (1999c) found that varying the stiffness of the lining, which was modelled with a linear elastic model of constant stiffness, made little difference to the hoop forces but influenced the hoop bending moments greatly. From this

they concluded that the hoop force does not depend on the material properties of the lining. However, this is only true for the simplest cases in their study. When there is true interaction between the lining and the ground, the early-age stiffness, and particularly the point at which it becomes effective, had a great influence on the stresses in the lining.

16 There is some anecdotal evidence from segmentally lined pilot tunnels that, during enlargement to the full cross-section, longitudinal movements in the ground can drag pilot tunnel linings into tension, causing the joints between rings to open.

17 Based on a rough estimate of the saving in time and materials cost for the HEX Platform tunnel, assuming that no secondary lining would have been required for the steel fibre reinforced sprayed concrete option (Thomas 2003).

6 Detailed design

Notwithstanding the veracity of the old adage that 'if it can't be designed on the back of a cigarette packet, it can't be built', detailed calculations are essential as justification and evidence of robust engineering in modern design practice. Sadly they are rarely pocket-sized. This chapter provides guidance on detailed design of the sprayed concrete lining only, in a range of ground conditions and special cases. Topics such as face stability or rock-bolt design are not covered. More details on tunnel design in general can be found in the following texts: Szechy 1973, Hoek and Brown 1980 and BTS Lining Design Guide 2004.

When using analytical or numerical design tools, the procedure for detailed design calculations is generally the same. The loads in the lining are estimated. These loads are compared with the capacity of the lining, with normal partial safety factors applied, normally using a moment–force interaction diagram (e.g. see Eurocode 2 (2004)). The deformations (of the lining and ground) are also estimated and these are used to determine trigger values, if required (see Section 7.2).

At the risk of appearing tedious, once again it should be stressed that the sprayed concrete is only one part of the support system for a tunnel. The soil-structure interaction must be modelled realistically. For a given tunnel, the modelling of the ground or the construction sequence may prove more critical to the design than the subtleties of the behaviour of the sprayed concrete.

6.1 Design for tunnels in soft ground

This section covers the design of sprayed concrete linings in soft ground (i.e. where the soil or weak rock behaves as a continuum). The key mechanisms of behaviour are plastic yielding or failure in the ground around the tunnel (see Table 3.2) and the sprayed concrete must provide immediate support. Face stability, along with the available equipment, drives the choice of excavation sequence.

6.1.1 Key behaviour of sprayed concrete

Because of the role of immediate support, the age and time-dependent behaviour of the sprayed concrete may well be relevant. Excavation sequences tend to have multiple stages and the intermediate load cases when the lining is incomplete should also be checked as they may be more critical than the long-term loading. Nonlinear behaviour may occur during these stages.

6.1.2 Determining the loading on the sprayed concrete

The behaviour of soft ground itself may be quite complex: *in situ* horizontal stresses may exceed vertical stresses; K_0 may vary with depth; nonlinearity elastic/plastic behaviour and anisotropy may influence strongly the loads on the tunnel. Depending on the permeability of the ground undrained behaviour may also feature. As an example, Addenbrooke 1996 and van der Berg 1999 describe the features relevant to one type of soft ground, London Clay.

Consequently simple analytical methods alone may not be adequate, although they are often used in early stages of design (see Table 4.2 for examples). Numerical modelling may provide more realistic estimates of the ground loads. Due to the weakness of the ground, support is installed quickly and hence the relaxation factor is low (see Table 4.1), often around 50%. Alternatively a target volume loss can be used in the numerical model (i.e. the ground is permitted to relax until a certain amount of deformation has occurred). Good quality construction results in volume losses around 1.0%.

6.1.3 Lining design

Despite the comments above, a single SCL tunnel in soft ground should not present too many difficulties to a designer.

Some excavation sequences have sharp angles (see Figure 3.3) and design calculations tend to predict high bending moments there. In practice often these do not appear to occur, most probably due to arching in the ground or creep within the sprayed concrete. One way to check how important these stress concentrations may be is to insert a 'pin joint' (i.e. to release the fixity on rotation) and see how this changes the outcome of the analysis. Overall the shape of the tunnel should be fairly circular to minimise stress concentrations in the ground or the lining (see Figure 6.1). Sharp corners may increase the dead loads on the lining as the arching in the ground follows more gentle curves or induce high bending moments in the lining as noted above. Horseshoe-shaped cross-sections tend to experience instability under the footings. Increasing the bearing area of the footing with 'elephant's feet' can combat this but, in the end, a more rounded cross-section and early ring closure may be the only way to prevent instability in the invert.

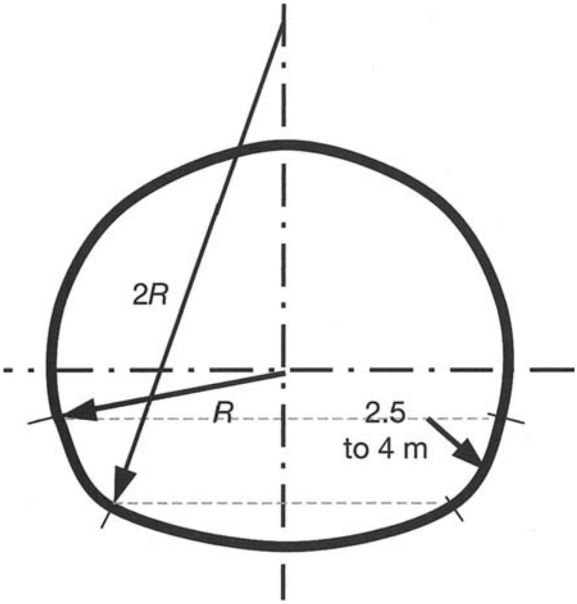


Figure 6.1 Example proportions of a SCL tunnel in soft ground

As noted in Section 3.1.2, the lining tends to continue to deform until a complete structural ring is formed. Hence, early ring closure – i.e. within 0.5 to 1.0 tunnel diameters from the tunnel face – helps to reduce deformations and surface settlement. Typically, advance lengths are limited to around 1.0 m to facilitate early ring closure. The strength of numerical modelling is evident here as it permits different excavation sequences and geometries to be tested in the design phase.

As Figure 6.2 shows, the ratio of tunnel diameter to lining thickness generally lies between 10 to 15 for recent shallow SCL tunnels in the UK. There is considerable scatter in the data but this is probably due more to differences in design assumptions rather than ground conditions.

The arching in the ground occurs in three dimensions around the active face. Hence load is thrown onto the ground ahead of the tunnel and backwards onto the lining. Simple calculations (e.g. Figure 4.2) indicate that arching occurs mainly within one diameter of the tunnel. Experience in the field (e.g. Thomas 2003) supports this. To avoid interaction (e.g. in a pilot tunnel and enlargement sequence), active faces should be kept two tunnel diameters apart. The same applies for adjacent tunnels. If this is unavoidable (e.g. at junctions) or undesirable, the adjacent structures must be designed to cope with the extra loading. The design of junctions will be addressed in more detail in Section 6.5.

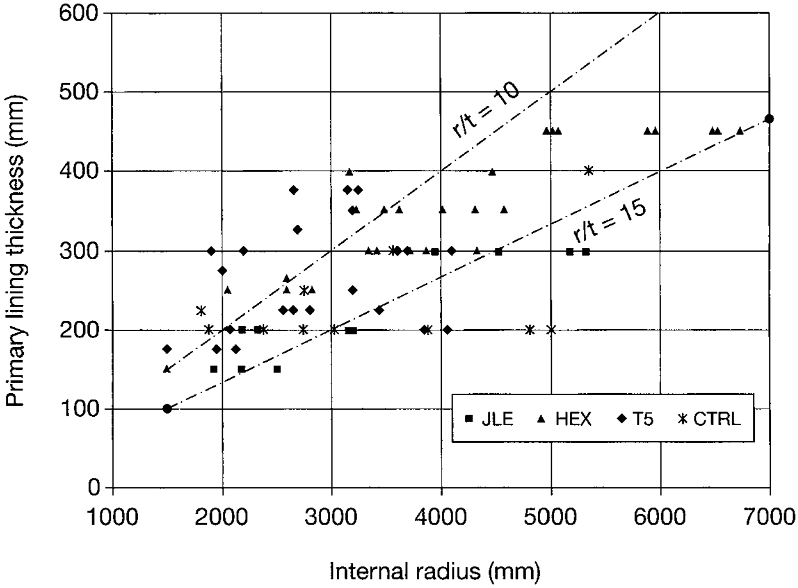


Figure 6.2 Lining thicknesses for SCL tunnels in soft ground

6.2 Design for tunnels in blocky rock

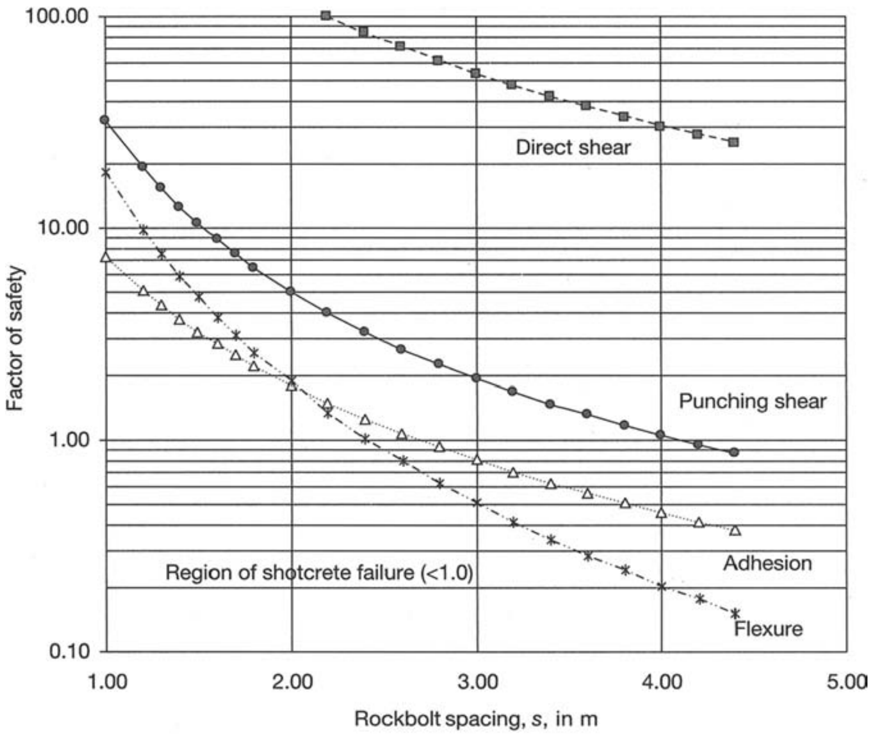
This section covers the design of sprayed concrete linings in jointed rock masses (i.e. where the rock behaves as a discontinuum). The key mechanisms of behaviour are block failure, plastic yielding or general failure in the ground around the tunnel (see Table 3.3). The sprayed concrete may be needed for immediate support (e.g. for blocks or even as a full structural ring) but sometimes installation of the full sprayed concrete lining can be delayed for several advance lengths. Similarly, depending on the type of ground, the timing of ring closure varies and in the best ground a structural invert is not required. Sprayed concrete is usually only one part of the support system and it often functions in combination with rockbolts. As in soft ground, face stability, along with the available equipment, drives the choice of excavation sequence.

6.2.1 Key behaviour of sprayed concrete

The early-age strength of sprayed concrete, especially its bond strength, is the key to supporting blocks. When pushed to its limit, under loading by a single block, a sprayed concrete lining usually fails first in debonding and then in flexure (Barrett and McCreath 1995) (see Figure 6.3).

Steel fibre reinforced sprayed concrete (SFRS) is very effective in supporting blocky rock masses. The fibres reinforce the full thickness of the concrete and absorb a lot of energy while deforming. SFRS is particularly suitable for high stress environments (e.g. Tyler and Clements 2004).

When acting as an arch or a full structural ring, the other aspects of behaviour, such as nonlinearity or creep, may be relevant (see Section 6.1.1).



Input parameters

Unit weight	γ	27	kN/m^3
Width of face plate	c	0.2	m
Adhesion strength	σ_a	750	kN/m^2
Adhesion bond length	a	0.04	m
Direct shear strength	τ_{ds}	8000	kN/m^2
SCL thickness	t	0.25	m
Flexural strength	σ_{flex}	5000	kN/m^2
Diam. tensile strength	σ_{ds}	2000	kN/m^2

Load factor	γ_f	1
Material factor	γ_m	1.5

Figure 6.3 Stability chart for plain sprayed concrete (after Barrett and McCreath 1995).

The importance of time-related characteristics depends on the rate of the development of the ground loads.

Although it may be possible to delay installing the full support, it should be noted that sprayed concrete serves a useful purpose in sealing the rock surface. This prevents deterioration of the rock mass due to drying out, the action of water or air and it seals joints tight.

6.2.2 Determining the loading on the sprayed concrete

Table 4.2 contains some examples of analytical solutions for blocky ground. The disadvantage of semi-empirical methods such as Protodykianov's load distribution (Szechy 1973) is that, while they indicate a ground load or lining stress, they do not provide detailed information on the stresses in the lining. Given the shape of the load distributions it is not a simple task to translate them into the distribution of axial forces and bending moments to be carried by the sprayed concrete lining.

Of the more sophisticated analytical solutions, the Convergence Confinement Method, is the most commonly used in blocky rock. While it does assume the rock mass behaves as a continuum, plasticity can be incorporated using the Hoek–Brown failure criteria which simulates rock better than other failure criteria such as Mohr–Coulomb. The CCM can be used as a quick check on the overall stability of the excavation and check for plastic yielding, bearing in mind its other limitations (such as the assumption of axisymmetry). It is often used to estimate convergence and experiment with the timing of installation of support.

Because of the better stability of the blocky rock, compared to soft ground, non-circular cross-sections can be used more often. This goes beyond the capabilities of most analytical solutions, so designers then turn to numerical modelling. The most well-known numerical modelling program for block rock is the discrete element program, UDEC and its 3D companion, 3DEC. Some of the commercially available finite element or finite difference packages offer constitutive models that try to mimic the behaviour of jointed rock masses. Blocky rock presents an added complication for numerical simulation as it is difficult to determine many of the parameters that govern the behaviour of joints.

6.2.3 Lining design

In blocky rock, the support is designed either using empirical methods or by estimating loads by one of the means mentioned in the previous section, converting them into lining loads and sizing the sprayed concrete to carry them.

The most commonly used empirical methods are the Q-system or RMR (see also Sections 4.3.1 and 6.3.1). These work best in more competent rock, approaching hard rock conditions. At the lower end of the range there is a risk that such simple methods will not provide a robustly engineered solution. Applying support designed for rock to failing soft ground is a recipe for disaster Borderline cases between blocky rock and soft ground deserve more detailed consideration. Also it is worth noting that, like all empirical methods, they provide no information on the factor of safety or the timing of the support. The latter should be specified based on stand-up time.

If the sprayed concrete is acting as an arch or ring, it can be designed as a compression member under bending, in the same way as for soft ground tunnel linings. Where thin layers of sprayed concrete are envisaged or the lining does not form a full ring it may be more appropriate to design the lining as a composite beam acting with the rock. The Voussoir arch theory makes best use of this composite action. Bolts pin the reinforced sprayed concrete (which acts as a tensile membrane) to the rock beam. The axial load is transferred into 'abutments' at the springing point of the arch. Asche and Bernard (2004) noted that this method works well in horizontally bedded competent rock but Banton *et al.* (2004) caution that this assumes no sliding of blocks at the 'abutments'. Banton *et al.* (2004) describe the application of the Voussoir solution as well as other design tools for blocky rock.

If the sprayed concrete is only controlling the stability of individual blocks, effectively spraying between rockbolts, the analytical solution by Barrett and McCreath (1995), can be used to size the lining thickness. The blocks are held in place by adhesion over a strip around the perimeter of the block. This strip is assumed to be 30 mm wide, which enables blocks of 1.5 to 3.0 tonnes per linear metre to be carried (Banton *et al.* 2004). The computer program UNWEDGE offers a more comprehensive treatment of this aspect since, based on inputted joint sets, it calculates all kinematically admissible blocks.

A tunnel usually passes through a variety of ground conditions. It is more economic to specify a range of support classes (typically 3 to 6). The construction team can then choose the most appropriate one, depending on the actual conditions encountered.

The design of junctions will be addressed in more detail in Section 6.5.

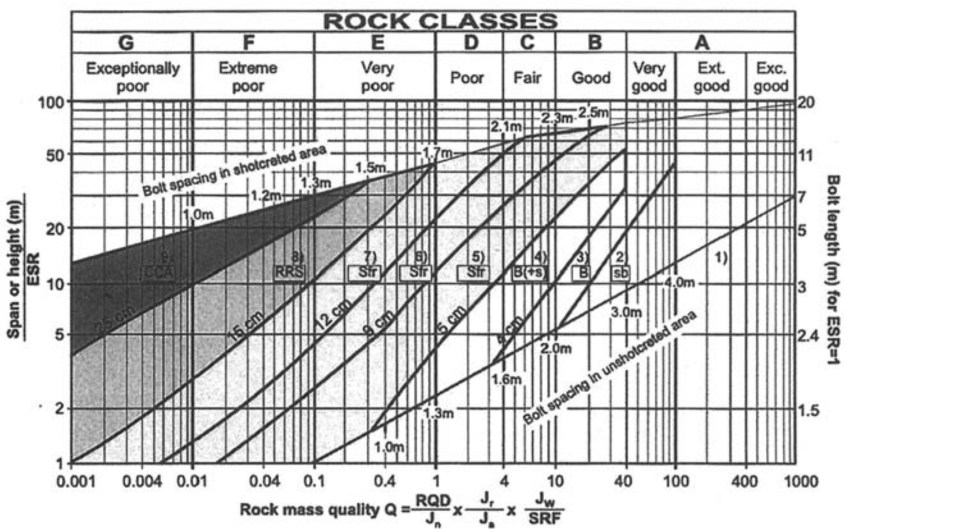
6.3 Design for tunnels in hard rock

This section covers the design of sprayed concrete linings in massive rock masses (i.e. where the rock behaves as a continuum). The key mechanisms of behaviour are usually a stable elastic response of the rock mass around the tunnel and isolated block failure (see Table 3.4). The sprayed concrete must provide immediate support to the blocks but otherwise the tunnel is stable. As before, face stability, along with the available equipment, drives the choice of excavation sequence. Hard rock (in moderate to low stress environments) has long stand-up times so support may not be installed until several rounds behind the face. Round length is typically 3 to 4 m. The special cases of rockburst swelling or squeezing rock are discussed separately below.

6.3.1 Lining design

Unless there are complications, such as high *in situ* stresses, empirical methods are often used to determine support. These may be formal methods such as the Q-system (Figure 6.4) or RMR or simply the application of engineering judgement and informal 'rules of thumb'. It is preferable to assess systematically the risks posed by the ground conditions and use this in combination with the established support charts such as Q or RMR to determine the support. The timing of the support should be specified based on stand-up time. The excavation method too influences the support required. TBM tunnels tend to disturb the rock less than the drill and blast method and so require less support (e.g. Barton 2000).

The sprayed concrete, whether plain or reinforced with fibres or mesh, is usually only one component of the support system. Often it has a secondary



Reinforcement categories

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40–100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50–90 mm, and bolting
6) Fibre reinforced shotcrete, 90–120 mm, and bolting
7) Fibre reinforced shotcrete, 120–150 mm, and bolting
8) Fibre reinforced shotcrete, >150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Figure 6.4 Q-system support chart (Grimstad and Barton 1993) role compared to the rockbolts. The capacity of sprayed concrete to carry blocks, either alone or in combination with rockbolts, can be checked by simple calculations (e.g. Barrett and McCreath 1995 – see Section 6.2.3) or programs like UNWEDGE. The CCM can be used as a quick check on the overall stability of the excavation, a check for plastic yielding and to estimate convergence.

For logistical reasons, small diameter rock tunnels often feature temporary enlargements for passing bays or mucking niches for temporary stock-piling of spoil. These simply extend the span of the excavation. They should be designed in the same way as the main tunnel but using the maximum span. The design of junctions will be addressed in more detail in Section 6.5.

As in all tunnelling, the *in situ* stresses are redistributed around the tunnels. This may lead to overstressing of rock pillars between tunnels. Rockbolts are generally much more effective in reinforcing pillars than sprayed concrete. As expounded by Rabcewicz, the father of the NATM, and others, it is important to think of the support and the rock mass as one system which shares the loads. Rockbolts are more effective because they enhance the ability of the rock mass to carry the loads around the opening whereas the sprayed concrete functions more in supporting the surface of the tunnel.

For complex underground works with adjacent tunnels (e.g. powerhouses), it may be necessary to use numerical modelling in 2D or 3D.

6.4 Shafts

Linings for shafts can be designed in a similar manner to linings for tunnels. Intrinsically a shaft is more stable than a tunnel of the same diameter because gravity exacerbates stability problems in the crown of a tunnel. The same mechanisms of behaviour as outlined before for the three types of ground apply, except that more attention should be paid to invert stability. Sprayed concrete does not normally feature in the temporary measures to ensure the invert stability of the shaft. Subdivision of the excavation round is sometimes employed as is dewatering or construction of a pilot shaft (mined or by drilling and raise-boring). In loose water-bearing ground, vertical forepoling with steel sheets can also be used.

Sprayed concrete linings for shafts do not bear on the base of the shaft during construction but hang off the ground above using skin friction. Several analytical solutions exist for determining the loads on shaft linings in soft ground. The basic CCM can be used although the solutions by Wong and Kaiser (1988 and 1989) are more sophisticated. Notably they incorporate the effects of vertical arching at shallow depths around the shaft and horizontal arching at deeper levels. One phenomenon that is not included in their analytical solution is the reduction in loading at the base of shafts due to vertical arching that they observed in test results and measurements from real shafts. The standard methods described above can be used for support of individual blocks. In numerical modelling an axisymmetric model can be used for single shafts. If there are junctions in the shaft or adjacent structures, it may be necessary to use a 3D numerical model. As you will have probably guessed, the design of junctions will be addressed in more detail in Section 6.5.

Where sprayed concrete is used for the base of a completed shaft, the base is usually domed if a substantial water or ground pressure is expected, since this will help minimise bending in the sprayed concrete and transfer the load in compression into the lining of the shaft. On the down side one must check the shaft for flotation.

Inclined tunnels are essentially a cross between tunnels and shafts in terms of design.

6.5 Junctions

Junctions – between tunnels and shafts or other tunnels – are an almost ubiquitous feature of tunnelling projects. One great advantage of SCL tunnelling is its efficiency in the formation of junctions. The SCL structure acts as a shell, transferring the stresses around the opening. This is believed to be aided by favourable aspects of the behaviour of sprayed concrete such as creep. By a combination of this stress redistribution and the promotion of arching in the ground, a well-designed SCL junction avoids excessive stress concentrations at the edge of opening. However, there will be some increase in stresses so the area around an opening and at the start of the new tunnel may need to be strengthened to cope with this.

This section should be read in conjunction with the relevant section for the type of ground that the junction is located in. The same basic principles apply, with some additional ones as outlined below.

6.5.1 Key behaviour of sprayed concrete

As noted, depending on the ground, different aspects of the behaviour of sprayed concrete will assume differing degrees of importance. In addition to this, assuming that the sprayed concrete lining is acting as a structural ring, at a junction the age-dependent behaviour of the sprayed concrete in the new ('child') tunnel and creep in both linings should be considered. These phenomena produce a 'softer' response, resulting in more deformation and stress redistribution than a stiffer material would. Provided that the overall stability of the junction is maintained, this produces effective stress transfer around the opening. Depending on how old the existing ('parent') tunnel is, age-dependent behaviour in its sprayed concrete may or may not be relevant.

6.5.2 *Determining the loading on the sprayed concrete*

The key for the designer is predicting how the stresses in the lining and the ground will be redistributed as the new tunnel is built. This process is not well understood and so junctions can appear difficult to design. Junctions increase the effective span of the excavation. Consequently, the loads on the lining around a junction are higher than normal.

The stronger the ground is, the less work the sprayed concrete lining will have to do. In very competent, hard rock, rockbolts alone may be used to reinforce the ground and secure the junction. If used at all, sprayed concrete will merely serve to secure individual blocks on the surface of the excavation. In weaker ground, the sprayed concrete lining carries more of the ground load and must function as a full structural ring.

Simplistic design tools – such as beam-spring models – are not suitable for simulating the construction of junctions as they are incapable of replicating realistically the stress redistribution in the ground.

6.5.3 *General arrangement and construction sequence*

Understanding the three-dimensional arrangement of the junction is critical. Due to arching in the ground and the lining, the junction will interact with any nearby objects, i.e. within about one tunnel diameter. Unfortunately there is little published guidance on the design of junctions (e.g. on the relative size of the two tunnels or proximity between adjacent junctions). On the other hand there are many examples of the successful use of SCL tunnel junctions.

Often there are several phases in the construction of a junction. In preparation, it is good practice to form a tunnel 'eye'. The eye is the area which will be broken out. To make this easier it is usually thinner and more lightly reinforced than the rest of the parent tunnel. If possible, starter bars can be added to provide continuity of reinforcement when the new tunnel is started. The area around the eye is then strengthened. This can be done in several ways (see Figure 6.5):

- Reinforcing a patch around the eye. A square patch is easier to construct than a circular one, not least because the reinforcement can be added as pieces of mesh as the tunnel advances. For circular patches, bars are added later individually in a circumferential/radial configuration or as rectangular pieces of mesh.
- Adding reinforcing rings above and below the opening.

The latter is both less elegant and less effective in transferring stresses smoothly around the opening. The rings tend to be large sections so they may intrude on the internal space of the parent tunnel.

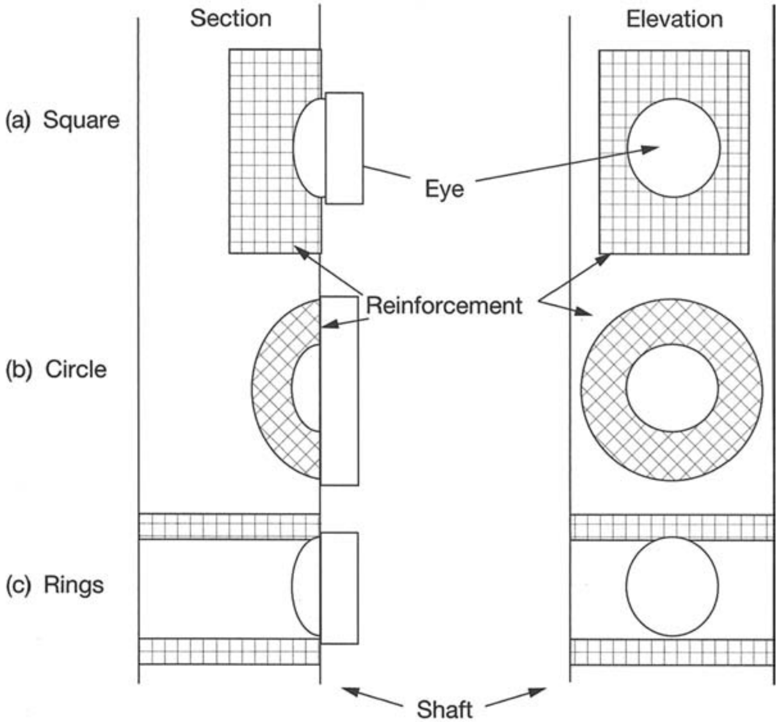


Figure 6.5 Reinforcement around junctions

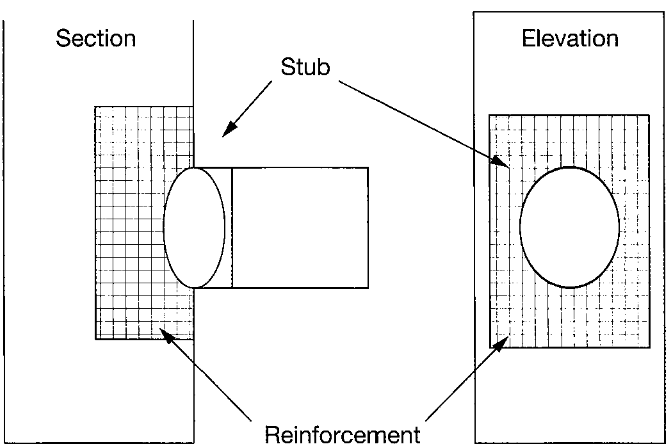


Figure 6.6 Construction sequences for junctions

Several construction sequences are commonly used:

- stub;
- pilot tunnel and enlargement;
- reinforcement of the 'child' tunnel;
- the normal excavation sequence for the tunnel.

The first two are used in weaker ground where the priority is to ease the birth of the child tunnel. Subdivision of the tunnel face improves stability and the early installation of support reduces ground movements. The formation of a full structural ring (e.g. in the form of a stub) should arrest the deformation of the tunnel linings. Having stabilised the junction, the new tunnel can then progress as normal. Alternatively the first part of the new tunnel (a distance equivalent to half a diameter) can be thickened and/or reinforced to cope with the extra loads. The final sequence above should only be used where the support installed already has stabilised the junction – for example, in hard rock where rockbolts have been installed from the parent tunnel.

6.5.4 Lining design

In the past the sprayed concrete lining at a junction has been designed on the basis of precedent or simple models such as the 'hole in an elastic plate' analytical solution (see Figure 6.7).

Knowing the loads in the lining of the parent tunnel, the latter can be used to model the redistribution in the sprayed concrete shell. The effect of adjacent junctions can be estimated by simply assuming superposition. An important question when using this method is: what is the stress in the longitudinal direction? Any stress in this direction will reduce the peak tangential stresses at the edge of the opening although the shear stresses increase (see the effect of varying K in Figure 6.7).

The obvious limitations are that the plate is flat, unlike the curved lining, and it is assumed that the total of the ground load remains the same. These new loads are used to determine the additional reinforcement around the opening. Rather than take the highest forces, which occur at the edge of the opening, sometimes the forces are averaged over the area to be reinforced. Typically this is 1 to 2 m from the edge of the opening. It can be argued that creep and nonlinearity in the sprayed concrete will tend to smooth out stress concentrations. Bending moments are derived by assuming the axial forces act at a nominal eccentricity (typically 20 mm – Eurocode 2 (2004)).[1] Overall this method is viewed as being conservative as it tends to predict thicker linings and heavier reinforcement than precedent practice suggests is necessary. Often the lining of the 'child' tunnel is not increased beyond its normal thickness, on the basis that all additional loading at the junction is carried by the (larger diameter) 'parent' tunnel.

A minor refinement of the method above is to use a 2D numerical model of the opening in the flat plate. As well as being able to include non-circular geometries, this opens the door for nonlinearity and creep to be included in the model.

The final option is the use of a 3D numerical model. Before embarking on this, it is worth remembering that this will prolong the design process. The geometry of a junction is more complex so it takes longer to build the model and requires more elements. To get the best out of the model, the constitutive modelling should be as realistic as possible. As a result, the models tend to be large and therefore slow to run.

As with all other structures, care should be taken with the detailing to keep the construction as simple as possible. For example, concentrations of reinforcing bars should be avoided.

The potential for differential movement at junctions between SCL tunnels and other structures should be considered. Special measures may be required to avoid structural damage or water ingress.

6.6 Tunnels in close proximity

Building a tunnel modifies the stress in the ground around it. Therefore, if another tunnel is built close to the first one, it may encounter higher ground

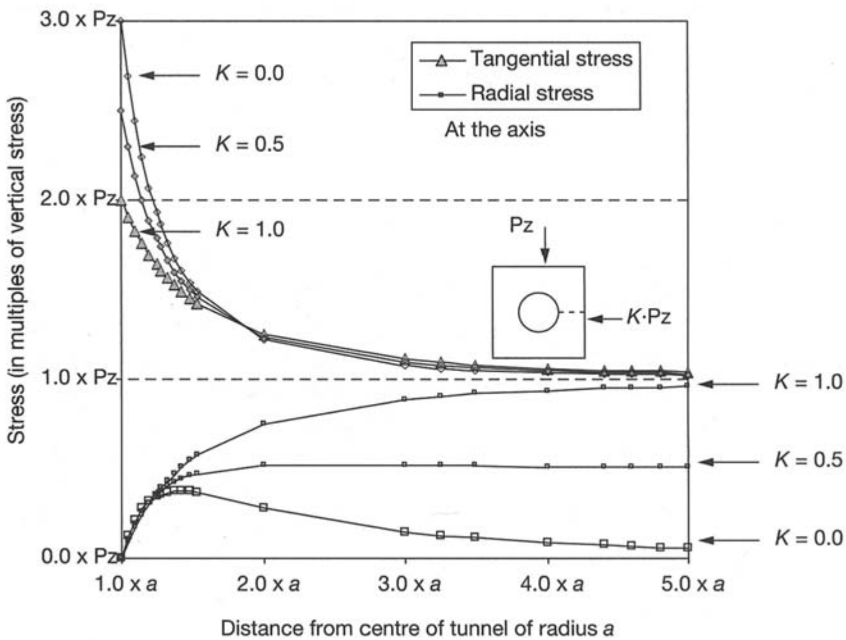


Figure 6.7 Stress distribution around a hole in an elastic medium under applied stresses Pz and *K*·Pz

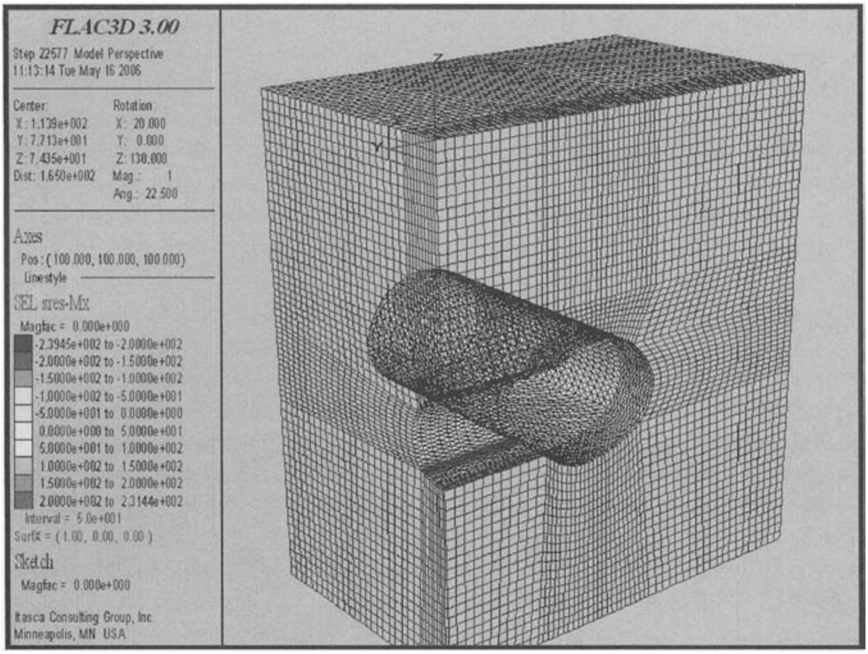


Figure 6.8 3D numerical model of a tunnel junction

loads than normal. Also, the first tunnel may be loaded as the ground stresses arch around the new tunnel. The exact nature of the interaction depends on the stiffness and strength of the ground. Generally in soft ground, if the separation between the centrelines of the two tunnels is more than twice the largest diameter of the tunnels, then the interaction will be a minor effect. Figure 6.7 shows one simple method of estimating the impact of the interaction, although it is arguably more realistic to use the plasticity solution for this case (see Figure 4.2). Tunnels can be built very close to each other even in soft ground (e.g. JLE London Bridge, UK), provided that the linings are designed to cope with the increased loads. When tunnelling in close proximity, the excavation sequence should be considered carefully.

6.7 Portals

Due to the low cover, the ability of the ground to arch and redistribute the *in situ* stresses is limited. The strength of the ground may be lower than normal due to weathering, slope instability or the lack of confinement. Therefore additional reinforcement of the ground and/or lining is required. Otherwise the principles of the design are the same. The spatial arrangement of the portal and support measures must be considered. The lining at a portal may be subjected to pronounced asymmetric loading, if it intersects a slope or rock head obliquely. Since they are important structures, portals may warrant the use of numerical modelling, ideally in 3D.

It is prudent to specify more conservative excavation sequences, firstly to prevent instability but also because the portal is built at the start of tunnelling while the construction team is still in the learning phase. Additional support measures such as steel arches, spiling or canopy tubes are commonly used at portals.

6.8 Special cases

When confronted with a special case in design a common response is first to scour the published literature to find similar case histories and, ideally, guidance or rules of thumb. Combined with the basic principles this information can be used to identify an outline design solution, which is then further developed with the help of numerical modelling. In the sections below some special cases are considered briefly. The comments are not meant to be comprehensive but rather cover only the impact on the design of the sprayed concrete lining.

6.8.1 Seismic design

Tunnels generally perform well in earthquakes. They are flexible enough to move with the ground so unlike structures above ground, they are much less vulnerable to the effects of their own inertia. The risk of damage decreases as the height of overburden increases (Hashash *et al.* 2001). Although the tunnel may not suffer major structural damage, the contents of the tunnel might not fare so well. The worst cases for a tunnel are when it intersects a fault or where there are substantial changes of stiffness in the structure (e.g. at tunnel junctions and portals). In both cases the lining and/or the waterproofing could sustain major damage. A detailed review of the effects of seismic events on tunnels in general can be found in Hashash *et al.* (2001).

In short, a SCL tunnel is likely to be subjected to three types of displacement:

- axial compression and extension – due to seismic waves running parallel to the tunnel;
- longitudinal bending – due to seismic waves which move the ground in a direction perpendicular to the tunnel axis;
- racking/ovalisation – due to shear waves normal to the tunnel.

A simple solution to the first case is to build an enlargement at the intersection (e.g. Los Angeles Metro and San Francisco BART metro). If there is movement at the fault, there will still be enough space for the tunnel to remain operational, although the internal works will require adjustments, for example, re-aligning the track in a rail tunnel. A similar approach was adopted on the Channel Tunnel at Castle Hill (Penny *et al.* 1991). The tunnel diameter was increased by 1.22 m, to allow for possible movement along the line of a landslip. Movement joints were installed and the lining reinforced near the failure plane.

Where there is a large difference in stiffness, there is potential for damaging differential movement. Movement joints are installed to accommodate this (e.g. see Figure 6.9). Standard texts on seismic design should be consulted for more information on such movement joints.

As for the lining, Hendron and Fernandez (1983) proposed a simple check to determine whether or not the tunnel is substantially stiffer than the ground in terms of its flexibility, F (see Equation 6.1):

$$F = \frac{2E_g(1 - v)R}{E(1 + v_g)t} \tag{6.1}$$

where E_g and v_g are the elastic modulus and Poisson's ratio of the ground, E and v are the elastic modulus and Poisson's ratio of the lining, t is the lining thickness and R is the radius of the tunnel. If F is greater than 20, the lining can be deemed to be perfectly flexible compared to the ground and it should not experience damage.

A more detailed analytical approach, originally proposed by Wang, is reproduced as a worked example in Hashash *et al.* (2001). For more detailed

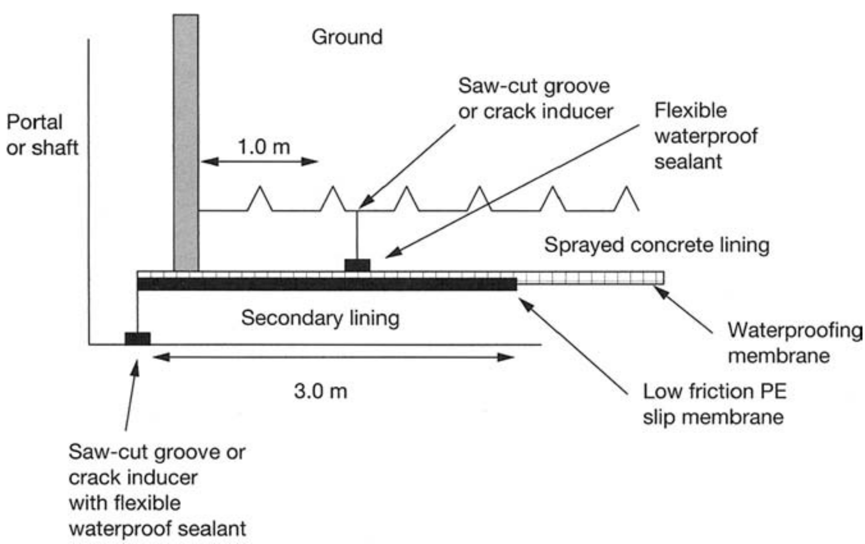


Figure 6.9 Cross-section through movement joint

analyses, there are commercial numerical modelling programs, which can simulate dynamic behaviour.

6.8.2 Squeezing ground

Squeezing ground (like swelling, creep and rockburst) essentially presents an additional form of loading. Hoek and Marinos (2000) described squeezing in detail and proposed an equation of estimating squeezing potential (see Equation 6.2):

$$\varepsilon = 100\left(0.002 - 0.0025\frac{p_i}{p_o}\right)\frac{\sigma_{cm}}{p_o}^{\left(2.4\frac{p_i}{p_o}-2\right)} \tag{6.2}$$

where ε is maximum strain in the ground (defined as tunnel convergence/tunnel diameter), p_i is internal pressure/resistance provided by the lining, p_o is *in situ* stress and σ_{cm} is the uniaxial compressive strength of the rock.

They suggest that in unlined tunnels squeezing begins to pose problems when the *in situ* stress exceeds 2.25 times the rock mass strength. At stresses above 3.5 times the rock mass strength, severe squeezing may occur, with the convergence of the tunnel lining potentially exceeding 2.5% of the diameter, i.e. 250 mm convergence in a 10 m diameter tunnel. This represents huge forces which can easily destroy a stiff lining that is placed in the tunnel. Indeed it is just these conditions that led Rabcewicz to formulate the NATM approach.

Rather than trying to resist the forces of nature, he advocated installing a lining that was flexible enough to absorb these large deformations and would bring them to a halt in a controlled manner. Sprayed concrete with its pronounced time-dependent behaviour works well in these situations when used in conjunction with rockbolting which reinforces the rock mass around the tunnel. Installation of the secondary lining is delayed until the deformations have stabilised. However, in the worst cases, this may not be enough and slots are left in the lining to permit the large convergences to occur without wholesale destruction of the sprayed concrete lining. As described earlier (see Section 3.3.3), the slots may be left empty or yielding supports can be inserted to help control the deformation. Slots or yielding supports can be modelled in numerical simulations by gaps in the lining or elements with a low stiffness (e.g. see Arlberg tunnel in John (1978) and the fault zone in the Sedrun section on the Gotthard Base Tunnel (Henke and Fabbri (2004)).

6.8.3 Swelling ground

Swelling is associated with marls, anhydrite, certain basalts and clay minerals such as corrensite and montmorillonite. Swelling is a stress-dependent process. It can be minimised by limiting the exposure of the ground to water and maintaining confinement. For example, the swelling of anhydrite is caused by the absorption of water and the stress relief around a tunnel causes fissures to open and the permeability of the rock mass to increase, thereby letting water into the anhydrite. The Huder–Amberg test can be used to determine the stress-strain relationship for the ground and this can be used to estimate the swelling loads on the lining. However, it is best to validate these estimates with measurements from a real tunnel as the behaviour may differ. For example, there is some evidence that the swelling may cause 'self-healing' as it reduces the permeability of the rock mass and so the penetration of water. The website of Prof. W. Wittke (www.wbionline.de) is a useful source of information and references. Examples of tunnels in swelling ground include the Lyon–Turin Base Tunnel (Triclot *et al.* 2007) and the Freudenstein experimental gallery (www.wbionline.de).

As in the case of squeezing ground, here the installation of the secondary lining is often delayed until the deformations have stabilised. Convergences can be as large as 1 or 2 m, with the result that the tunnel has to be reprofiled (Triclot *et al.* 2007). Even so, a thick and heavily reinforced secondary lining may be required to resist the residual swelling pressures. Alternatively, in more extreme cases, a yielding support may be installed. This can include yielding arches in the primary lining (Triclot *et al.* 2007) or a compressible invert of the secondary lining (Wittke 2007), in addition to deep rockbolt reinforcement. A ring of face dowels reinforcing the ground ahead has also be found to be beneficial (Triclot *et al.* 2007).

6.8.4 Creeping ground

Rocks such as rock salt, chalk, coal and marl may exhibit creep with the result that over time the ground will continue to deform and add load to the tunnel lining. Obviously the extent of creep in the ground is also heavily dependent on the stresses in the ground.

Such behaviour should be considered in the design calculations. Many commercial numerical modelling programs offer constitutive models for creep behaviour. As with all advanced numerical modelling, where possible the components of the model should be validated. For example, the suitability of the creep model of the rock could be checked by back-analysing laboratory creep tests or field data and comparing the predictions with the test results (e.g. Watson *et al.* 1999). The creep of the ground will interact with the creep of the sprayed concrete lining. Alkhaimi 1995 noted that in this case the creep of the sprayed concrete may exacerbate the situation rather than be beneficial. Hellmich *et al.* (1999c) found that the loads on the lining will increase as the rate of creep in the ground decreases relative to the rate of hydration, as more load is being added to the lining at times when it is stiffer.

As in the case of squeezing ground, here the installation of the secondary lining is delayed until the deformations have stabilised. However, the creep may continue over a long period so a thick and heavily reinforced secondary lining may be required to resist the residual pressures. The Channel Tunnel Crossover is an example of a SCL tunnel built in creeping ground (Hawley and Pöttler 1991).

6.8.5 Rockburst

Rockburst is the sudden failure of rock due to overstressing. It tends to occur soon after excavation but it may continue over a long period. Rockburst can be very dangerous, not just because of the violence of the spalling but also because it is very unpredictable. Grimstad (1999) suggests that rockburst is worst in stiff, strong rocks which tend to behave more brittlely than weaker ones. Hoek and Brown (1980) contains a detailed description of the phenomenon and countermeasures. The Laerdal Tunnel in Norway is one example of rockburst during tunnelling (Grimstad 1999) and includes the recommendation to install the rockbolts after first spraying the concrete. Steel fibre and synthetic fibre sprayed concrete, in combination with rockbolts, has been found to perform well in this extreme situation because it has a high capacity for absorbing energy during deformation and because it can withstand larger deformations than steel mesh either on its own or embedded in sprayed concrete (see Grimstad 1999 and ITA 2006 (report from South Africa) for further information).

6.8.6 Compressed air tunnelling

Occasionally sprayed concrete has been used in the construction of tunnels under compressed air (e.g. Munich and Vienna Metros in 1980s – Strobl 1991). In itself this construction method does not really affect the design of the sprayed concrete lining. In practical terms air loss through the sprayed concrete is a concern for the construction team. Compressed air is usually employed in water-bearing granular material so the more general questions of the suitability of sprayed concrete in these cases also apply. The compressed air will keep the groundwater out but the effectiveness of the lining will depend on how easy it is to spray onto the ground.

As a whole there may be an increased risk to the tunnel as the system is not fail-safe. If there is a massive loss of air, for example, through an area of loose ground, the pressure will drop and water will flow in, further weakening the ground. Spraying concrete on the face may not be able to stabilise the ravelling ground.

Loss of the air through the ground and the lining increases the costs of compressed air tunnelling. This can be reduced by treating the ground (e.g. permeation grouting or spraying a sealing coat of concrete on the face) or improving the quality of the sprayed concrete (e.g. by adding microsilica, increasing early-age strength, curing and reducing the water/cement ratio) (Bertsch 1992).

The air inside a tunnel under compressed air tends to be quite humid which can pose problems for dry mix sprayed concrete as it may start to hydrate before it is sprayed (ÖBV 1998).

6.8.7 Frozen ground and cold weather

A cold environment slows the hydration but the final strength is not usually reduced by much so long as the concrete does not freeze while hydrating. The simple remedy is to heat the tunnel and the rock surface so that the concrete gains strength sufficiently quickly. Figure 6.10 shows the use of heaters and plastic sheeting during curing. The temperature of sprayed concrete should not be allowed to drop close to freezing during curing as this could stall the hydration process and the formation of ice would disrupt the structure of the young concrete. Once the concrete has reached a strength of about 5 MPa, such protective measures can be stopped.

Sprayed concrete is not often exposed to cold temperatures in a tunnel unless ground freezing is being used to stabilise the ground. In that case the early-age strength of the concrete is less important structurally as the freezing helps to support the ground but the concrete is accelerated to speed up hydration and to reduce the risk of freezing during this sensitive period. Sometimes it is necessary to pre-heat the ingredients too. Klados (2002)

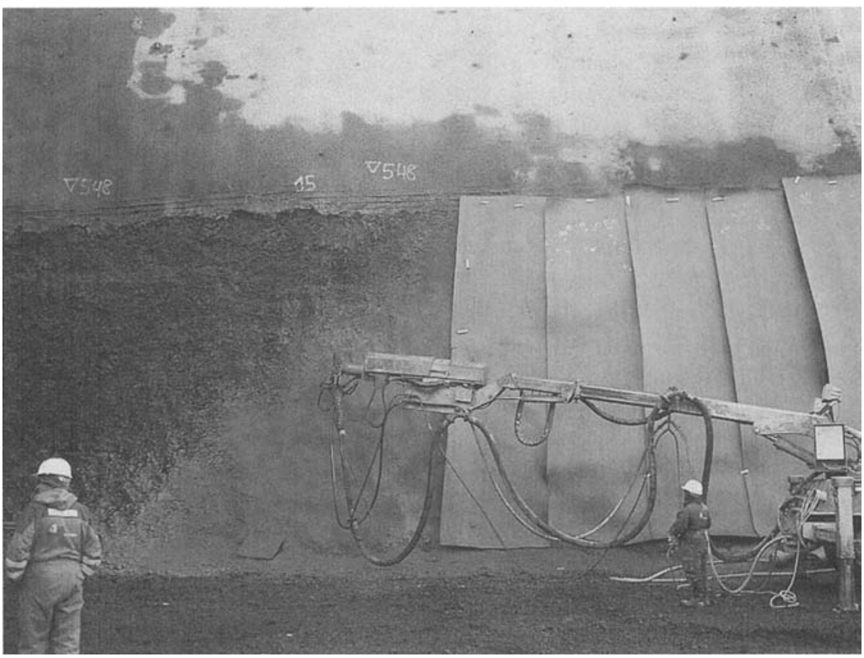


Figure 6.10 Weather proofing in icy conditions

provides a detailed description of one case study, including the mix design and profiles of temperature in the lining.

The tunnel lining within about 200 to 400 m of the portals can be exposed to freeze-thaw cycles (ITA 1993). A higher grade of concrete is required to cope with exposure to freeze-thaw cycles and specifications give guidance on what is needed (e.g. see ÖBV 1998).

6.8.8 Hot ground and hot weather

The temperature of the ground increases with depth, at a rate of about 20°C per 1000 m. So, in extreme cases, the rock itself can be hot. For example, rock temperatures of up to 45°C were encountered on the Gotthard Base Tunnel project (T&TI 2000). This can reduce the strength of the sprayed concrete. An equation for predicting the reduction due to high temperatures is given in Section 2.2.1. Although the temperature in linings in shallow tunnels rises to more than 40°C during hydration, this rise is short-lived. This is less onerous than the prolonged curing at elevated temperatures in deep tunnels. Therefore thermal damage in SCL tunnels is probably negligible, except in the extreme cases of very hot ground.

Obviously, hot weather is unlikely to affect tunnelling works, except for exposed areas of the portal. In hot climes, the standard procedures should be adopted to avoid damage to the concrete due to excessive shrinkage. Care should be taken during the batching and delivery of the concrete to ensure that it does not start to hydrate before it reaches the tunnel. Finally, in hot countries, the normally available cements may have a reduced proportion of tricalcium aluminate or lower Blaine value to slow the hydration process for outdoor applications. This would result in a slow setting sprayed concrete and could cause problems with low early strengths. However, the normal pre-construction testing should identify any potential problems (see Section 7.1.1).

6.8.9 Fire resistance

The subject of fire resistance of tunnel linings is an extensive and growing subject, spurred on by the consequences of recent major fires in tunnels. It is also intertwined with the fire-life-safety strategy for the tunnel operation as a whole. Guidance on the subject has been produced by the ITA (ITA 2004), albeit only for road tunnels at this time. Only brief comments are provided here by way of introduction to the subject.

The principles behind design for fire resistance demand, in the first instance, that the tunnel lining can withstand a fire for a certain time so that the people in the tunnel can be evacuated, typically 60 to 120 minutes. In addition, if the tunnel passes under high risk structures such as a railway line or is located under the sea or in unstable ground, the consequences of a lining collapse are unacceptable so the lining must also be able to survive the fire without major damage. The author is not aware of any example of a fire in a public tunnel that has caused it to collapse, although there are examples of fires in mines with have caused local collapses. The ITA guidelines propose a range of design criteria based on the type of tunnel and the traffic within it. For the design analyses, sprayed concrete can be assumed to behave in the same way as normal concrete. Winterberg and Dietze (2004) describe the mechanisms that cause damage to linings during fires as well as reviewing the assumed fire curves that can be used for design.

In a few recent tunnels where sprayed concrete forms the permanent lining, fine polypropylene fibres have been added (typically 1 to 2 kg/m^3) to enhance the fire resistance (e.g. Heathrow Terminal 5 – Hilar and Thomas 2005). In the event of a fire the fibres melt and let the steam which generated inside the lining by the heat of the fire to escape without causing explosive spalling (ITA 2004, Winterberg and Dietze 2004). Initially this method was used for tunnel lining segments and has been found to perform well in fire tests (see also Section 2.2.8). Coarse polypropylene fibres or structural synthetic fibres do not exhibit this beneficial effect.

6.9 Specifications

Specification of sprayed concrete works is straightforward since there are several published guides (see Table 6.1) that can be used as a basis for a project's specification. A good specification should be comprehensive yet concise and unambiguous. Typically it defines the inputs required (i.e. materials and competences of key staff), methods (both how to build the structure and the management processes) and the quality of the final product (e.g. strengths, geometric tolerances or watertightness). Because sprayed concrete linings are formed *in situ*, not in the controlled environment of a factory, there tends to be more detail in a specification. The form and detail of a specification is governed by the unique needs of each project. However, some general comments have been included below.

Target strengths should always be specified for the early-age period (i.e. $t < 24$ hours), for example, using the ÖBV's J-curves. The J2 curve is the minimum criteria for structural sprayed concrete. J1 refers to concrete that has no special load-bearing requirements during the first 24 hours while J3 refers to special cases where a very rapid set is needed (e.g. to control strong water ingress). It is vital that the sprayed concrete gains sufficient strength to carry the anticipated loads at all ages. Steel fibre sprayed concrete is addressed in detail in EN 14487 (2005) as well as ÖBV 1998 and EFNARC 1996.

In addition to the engineers and foremen, the key staff for a SCL tunnel are the nozzlemen (and, albeit to a much lesser extent, the pump operator). The competence of the engineering staff is also important since the effectiveness of decision making (either at the face or in Daily Review Meetings) is governed by their judgements. Many of the failures of SCL tunnels stem

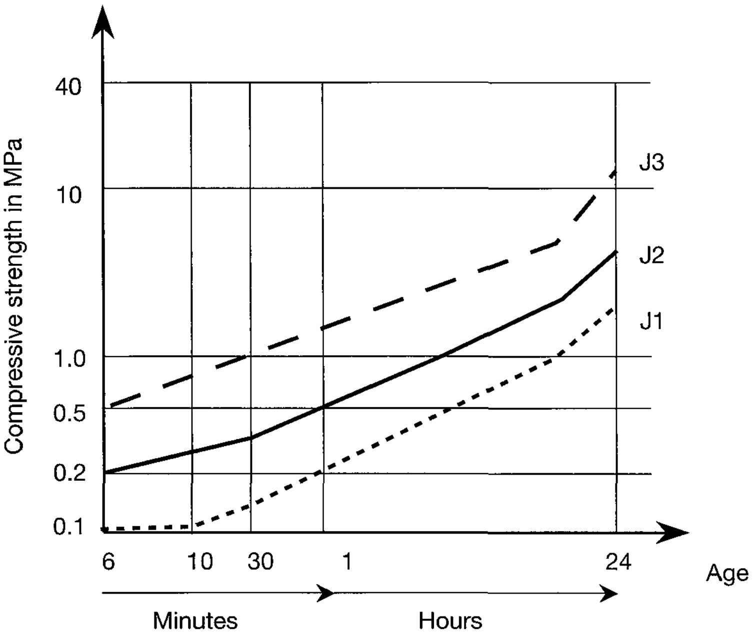


Figure 6.11 Early-age strength criteria (ÖBV 1998)

in part from the inexperience or poor judgements of engineering staff. For SCL tunnels it is important that the site team understands the methods in use, the limits of the design and how the tunnel is intended to behave. Setting criteria for the competence of the site team goes some way to achieving this but to complete it there must be good communication between the designers and the site team. This can be aided greatly by having a design representative on site (see Section 7.3).

The quality control test regime stipulated in the specification should be appropriate to the scale of the works (see Tables 11.1 and 11.2 in ÖBV 1998 – see also Section 7.1).

Table 6.1 Common specifications

Specifications	*Country*
EN 14487 (2005) and EN 14488 (2005) Eurocodes	EU
ACI 506.2 (1995) American Concrete Institute	USA
British Tunnelling Society – BTS Tunnel Specification (2000)	UK
EFNARC (1996)	Europe
ÖBV (1998)	Austria
NCA (1993)	Norway

Specifications can be made more user-friendly by adding in tables to list the required tests and their frequency as well as a table of submittals and their timing.

Despite its age, the ITA review (ITA 1993) still remains a useful overview of the existing specifications. Of all the current specifications, the ÖBV document probably remains the most comprehensive and it even includes recommendations on one pass linings.

6.10 Detailing

6.10.1 Steel reinforcement

To reduce the risk of shadowing, the minimum spacing between bars is typically 150 mm. Bars should not be placed closer than 10 mm to the substrate so that the concrete can be sprayed behind the bars. Some guides put limits on the maximum diameters of bars to be used (e.g. 14 mm (ÖBV 1998), 12 mm (Holmgren 2004)) but in practice concrete can be sprayed around bars of any size so long as they are not too densely spaced.

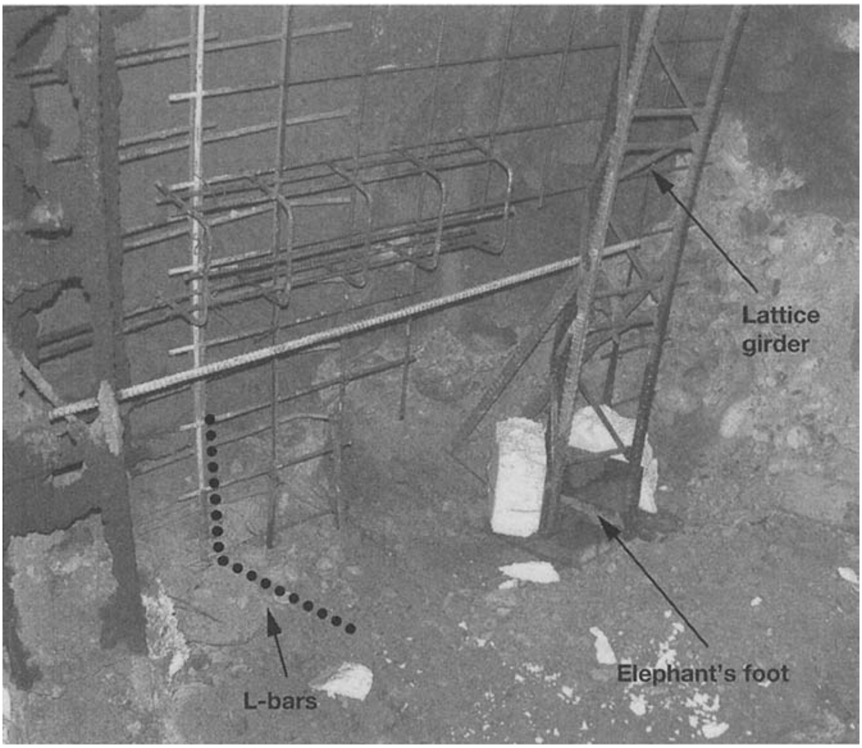


Figure 6.12 Connecting reinforcing bars at a footing joint

Obviously the thicker the bars the harder they are to place in the tunnel because they are heavier and more difficult to bend.

6.10.2 Structural continuity at joints

The normal rules for structural continuity of reinforcement apply and lap lengths can be calculated from standards for cast *in situ* concrete. Usually more informal rules are used. For example, the overlap of mesh reinforcement is normally expressed as a multiple of 'squares' of the mesh – typically two squares. This is an easy rule of thumb for miners to remember.

At some joints (e.g. the footings of headings) it is not possible to lap both layers of mesh directly. Traditionally, L-shaped bars are used at joints to provide an overlap. When the new section of the tunnel is excavated, the L-bars are exposed and bent straight. The disadvantage of this system is that the placement of the individual bars is time-consuming and it can be hard to locate them later. To overcome this, pre-fabricated strips can be used – for example, KWIK-A-STRIP – see Figure 6.13. They are quick and simple to use so the quality is better at the joints.

The bond of the concrete too is important at joints for structural continuity. The geometry of the joints should be designed so that they are easy to spray but also do not introduce a potential plane of weakness into the lining. On one recent project the ambitious target was set that the joints in the steel

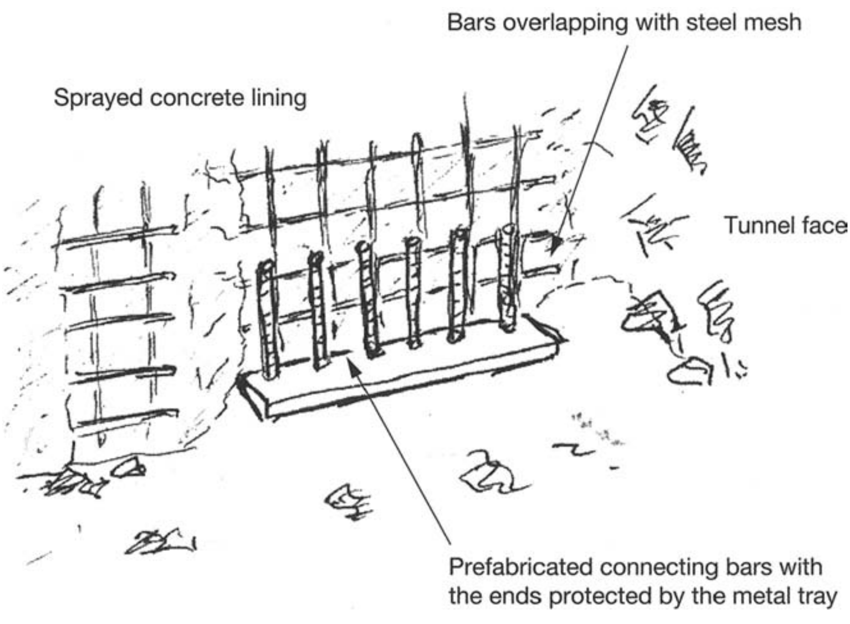


Figure 6.13 KWIK-A-STRIP at a joint in the lining

fibre sprayed concrete lining should have the same tensile capacity as any other part of the lining. Tests proved that this was achievable (Hilar *et al.* 2005). Similarly Trottier *et al.* (2002) found that construction joints did not detrimentally affect the flexural capacity of test panels of plain, synthetic or steel-fibre reinforced sprayed concrete. The initial load capacity (i.e. load at first crack) was lower for mesh reinforced panels.

6.10.3 Waterproofing at joints

Waterproofing is a huge subject in its own right. It is discussed in part in Section 4.2.3. This section covers only waterproofing details for joints in sprayed concrete.

Re-injectable grout tubes have become popular in recent years as a joint sealing measure at major joints (such as at junctions – see Figure 4.11). A big advantage is that they can be used repeatedly, provided that the tubes are flushed clean after each injection of grout. This is useful where differential movement may occur over a prolonged period of time.

Hydrophilic strips are also sometimes used at major joints. In theory, since they are confined within the concrete, when water meets them, they will swell and seal the water path. The strips must be kept dry until covered in concrete to prevent premature expansion.

Both of the above may be used with crack inducers to improve their effectiveness. The traditional water-bars used for cast *in situ* concrete are not suitable for sprayed concrete because it is too difficult to spray the concrete around them.

In some cases linings have been designed to rely solely on the water-tightness of the sprayed concrete (e.g. tunnels at Heathrow Terminal 5 – see Hilar and Thomas 2005). This is only suitable when the ground itself is largely impermeable. It is difficult to ensure that there is no cracking or that a good bond it always achieved at the many joints in a sprayed concrete lining. However, any residual leakage can be countered with conventional injection techniques for sealing leaks in concrete.

Note

1 Eurocode 2 (2004) Cl. 6.1(4) stated 0.033 × the lining thickness but not less than 20 mm. Linings are less than 600 mm so in practice this means 20 mm.

7 Construction management

Because a sprayed concrete lining is manufactured in the tunnel and because of the opportunity to vary elements of the design, rigorous management of construction assumes an even more important role than in other types of tunnels, such as a segmentally lined TBM tunnel, where the segments are manufactured in a factory and installed within the safety of a TBM shield. Construction management includes the control of quality and monitoring to verify that the SCL tunnel is behaving as the designer had intended. These topics will be discussed below.

7.1 Quality control

The quality control regime for a tunnel is defined in the specification. Several countries have produced specifications for SCL tunnels (see Section 6.9) which can be used like any other civil engineering specification. Full details of the test methods can be found in some these specifications (e.g. ÖBV 1998 and EFNARC 1996). One peculiarity of SCL tunnels is that there are two distinct phases of quality testing – pre-construction testing and testing during construction.

7.1.1 Pre-construction testing

The purpose of pre-construction tests is simply to verify the suitability of the mix design and equipment before it is used in anger. Typical tests are described in EN 14487 (2005) and Table 7.1. Durability tests such as water penetration or freeze-thaw should also be included when relevant.

These tests should be completed at least one month before tunnelling starts so that it can be confirmed that the 28-day results meet the specified requirements. If a substantial change to the mix is planned during production it will be necessary to repeat the pre-construction tests.

Pre-construction tests also provide an opportunity to examine the competence of the construction crew. Despite the increasing automation, SCL tunnelling still relies heavily on the skill of key workers such as nozzlemen. Several international standards exist for assessing the competence

Table 7.1 Pre-construction tests (EN 14487 2005)

Type of test	*Category 2 Temporary support*	*Category 3 Permanent support*
Consistency of the wet mix	Yes	Yes
Early-age strength development	Yes	Yes
Compressive strength	Yes	Yes
Modulus of elasticity	Optional	
Bond to substrate	Optional	
Maximum chloride content	Optional	
And if using fibres		
Residual strength or energy absorption capacity	Yes	Yes
Ultimate flexural strength	No	Yes
First peak flexural strength	No	Yes

of nozzlemen (e.g. ACI-C660 (2002) for dry mix and the forthcoming EFNARC scheme for wet mix). While the focus traditionally has been on nozzlemen, pump operators too are important as they can influence the efficiency of spraying and the quality of the end-product.

This phase is a valuable opportunity to identify any weaknesses and remove them. The value of pre-construction testing cannot be over-emphasised. This effort can save much wasted time and money during construction.

7.1.2 Testing during construction

This section contains brief comments on the common tests. Full details of each test can be found in the references given. The frequency of testing should be commensurate with the scale of the works (e.g. EFNARC 1996 or see Tables 11.1 and 11.2 in ÖBV 1998, abbreviated in Appendix G, or EN 14487 Tables 9 to 12). The European standard, EN 14487, recommends minimum testing frequencies of inspection category 2 or 3 during normal production in tunnelling works (see Table 7.2).

A refinement of the regimes above could be to give the site team the option to reduce the frequency of testing if the results are consistently above the required values. Should the results deteriorate, the frequency could be increased again. In fact EN 14487 recommends testing at four times the minimum frequencies stated above at the start of works or at critical sections.

SLUMP

This simple test gives an instant indication of whether the concrete is too stiff or too fluid. If the concrete is too stiff it may not be possible to pump

Table 7.2 Control of sprayed concrete properties (EN 14487 2005)

	Type of test	*Method*	*Category 2 Temporary support*	*Category 3 Permanent support*
Control of fresh concrete				
1	w/c ratio	By calculation or test method		Daily
2	Accelerator	From quantity added		Daily
3	Fibre content	prEN 14488-7	1/200 m^3 or 1/1000 m^2	1/100 m^3 or 1/500 m^2
Control of hardened concrete				
4	Early-age strength	prEN 14488-2	1/2500 m^2 or once per month	1/1250 m^2 or twice per month
5	Compressive strength	EN 12504-1	1/500 m^3 or 1/2500 m^2	1/250 m^3 or 1/1250 m^2
6	Density	EN 12390-7	When testing compressive strength	
7	Bond strength	EN 14488-7	1/2500 m^2	1/1250 m^2
Control of fibre reinforced concrete				
8	Residual strength or energy absorption capacity	prEN 14488–3 or prEN 14488-5	1/400 m^3 or 1/2000 m^2	1/100 m^3 or 1/500 m^2
9	Ultimate flexural strength	prEN 14488-3	When testing residual strength	
10	First peak flexural strength	prEN 14488-3	When testing residual strength	

Fibre content can be measured when testing residual strength if not done already.

it. Excessive fluidity may indicate that the water content is too high and the concrete may not adhere when sprayed. For more fluid mixes, a flow table can be used instead.

COMPRESSIVE STRENGTH – EARLY AGE

Typically the early-age strength of sprayed concrete is tested at regular intervals (e.g. after 1, 6 and 24 hours). Table 7.3 lists suitable test methods and more information can be found in EN 14487 (2005).

Clements (2004) cautioned against the use of standard soil penetrometers for early-age testing as they are prone to overestimating the strength. The cross-sectional area of the needle should be less than 10 mm^2.

Table 7.3 Test methods for sprayed concrete (ÖBV 1998)

Strength in MPa	*Test method*	*Reference*
0–1.2	Meyco or Proctor penetration needle	ASTM C403M (2006)
1.0–8.0	Hilti gun penetration test	ÖBV 1998
3.0–16.0	Pull-out test on bolts	ÖBV 1998
16.0–56.0	Pull-out test on bolts	ÖBV 1998
>10.0	Drilled cores	ÖBV 1998

COMPRESSIVE STRENGTH – MATURE

These test are normally performed on cores from the lining itself or test panels. It may be necessary to repair coreholes in the lining with a non-shrink mortar. Coring should not be used in secondary linings if there is a risk of puncturing a waterproofing membrane. Generally cores can be taken once the sprayed concrete has reached a strength of 10 MPa (ÖBV 1998), although it may be possible to core at strengths as low as 5 MPa.

If necessary the strengths measured from cored cylinders can be converted in cube strengths using established conversion factors. Typically the cylinder strengths are around 80% of the cube strengths. In addition, some have proposed a correction factor to allow for the disturbance caused by drilling. The Norwegian Concrete Association (NCA 1993) suggested the strength is reduced by a factor of 0.8 compared to cast cubes.

TENSILE STRENGTH

Tests on the direct tensile strength of sprayed concrete are rarely specified.

FLEXURAL STRENGTH

A variety of test methods exist, using beams, round panels or square plates to measure the flexural strength. Unlike most of the other tests, these require special samples to be sprayed (rather than using samples from the lining) and the size of the samples is quite large. This increases the cost of testing. Concern has been voiced that there is an excessive variability of results from tests within one batch of beams (e.g. Collis and Mueller 2004). Round panel tests have been promoted as a more consistent alternative (Bernard 2004c).

BOND STRENGTH

Tests can be performed in accordance with EN 14488-4.

REBOUND

The simplest method of measuring rebound is to lay a plastic sheet on the ground to collect the rebound as it falls. Knowing how much concrete was sprayed, the percentage of rebound can then be calculated. Typically rebound will be around 10 to 16% for wet mix and 21 to 37% for dry mix (Lukas *et al.* 1998).

THICKNESS

Traditionally thickness has been checked using markers such as thickness pins or by drilling holes into the lining. Now non-invasive methods are being used too, such as the DIBIT system (which takes 3D photographic images of surfaces) and the TunnelBeamer (which takes spot readings with a laser distometer).

Where thin linings are in use it may not be possible to obtain cores from the lining. In these cases an area can be oversprayed to provide sufficient thickness for cores (typically more than 100 mm thick) or cores can be taken from test panels.

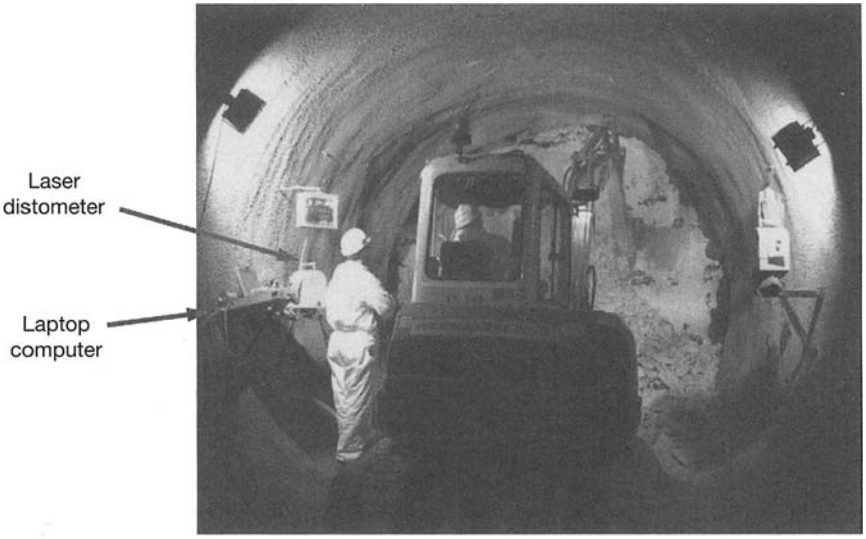


Figure 7.1
Laser-guided profile control

DURABILITY

Various tests for durability may also be required depending on the purpose of the sprayed concrete – for example, water penetration (EN 12390-8) or permeability tests, freeze-thaw resistance.

When testing it is important to use representative samples. It is tempting to just perform tests where it is easy to do them (e.g. at the sidewall). Testing should be targeted to the areas where quality is most at risk (e.g. the crown where spraying overhead is more difficult or at joints where rebound may become trapped).

While it is undoubtedly important to perform quality control tests, designers should beware of specifying an unduly strict regime. As the construction proceeds and the spraying operation emerges from the initial learning phase, there may be scope to reduce the testing frequency, if the specification is being consistently met.

7.2 Instrumentation and monitoring

During construction of a SCL tunnel it is normal to monitor the performance of the ground around the tunnel and the tunnel lining to ensure that it is behaving as expected. The review of monitoring data represents the umbilical cord that connects the growing construction with its design. In certain cases visual observation augmented with a few simple monitoring sections may suffice (e.g. a simple tunnel in hard rock under an uninhabited area). In other cases an extensive system of instrumentation may be installed in the ground and the tunnel (e.g. a metro station in soft ground under a city). Critical sections such as junctions should always be monitored carefully and the designer should be involved in the review of the monitoring.

When specifying the monitoring regime, the frequency and sophistication of the monitoring should be appropriate to the scale and importance of the tunnel. However, the designer should also build in redundancy into the system since some instrumentation will inevitably be damaged or become unusable. All instruments should provide relevant information. Trigger values should be specified for the monitoring, based on the design. Figure 7.2 contains an example of a monitoring regime for a shallow urban tunnel – i.e. at the more rigorous end of the spectrum.

For certain instruments it is important to obtain a set of baseline readings before the object in question is affected by the tunnel. This means that the effect of the tunnel can be clearly separated from seasonal effects or other influences.

7.2.1 Instrumentation

This section will focus on the monitoring of the sprayed concrete lining itself. An overview of instrumentation for the ground can be found in BTS

Distance from face	Frequency	Distance from face	Frequency
Surface settlement points		Inclinometers and extensometers	
–30 to 0 m	Daily	–30 to –15 m	Twice weekly
0 to +30 m	Daily	–15 to 0 m	Daily
+30 to +60 m	Twice weekly	0 to +30 m	Daily
> +60 m	Weekly	+30 to +60 m	Weekly
		> +60 m	Weekly
Lining convergence points		Piezometers	
0 to +30 m	Daily	General	Weekly
+30 to +60 m	Twice weekly	–15 to 0 m	Daily
> +60 m	Weekly	0 to +30 m	Daily
		> +30 m	Weekly

For a 10 m diameter tunnel in soft ground; excluding baseline readings

Figure 7.2 Monitoring regime for a shallow urban tunnel

Table 7.4 Instruments for monitoring ground behaviour

Settlement pins (surface)	Inclinometers	Earth pressure cells
Settlement pins (deep level)	Deflectometers	Piezometers
Electrolevels	Extensometers	

Table 7.5 Instruments for monitoring lining performance

Instrument	*Typical range and accuracy*
Convergence pins	(with tape extensometers) Up to 30 m and ±0.003 to 0.5 mm (with 3D optical survey) ~100 m and ±0.5 to 2.0 mm
Radial (Earth) pressure cells	0.35 to 5 MPa and ±0.1% see Geokon website[a]
Tangential (Shotcrete) pressure cells	2 to 20 MPa and ±0.1 to 2.0%[a]
Vibrating wire strain gauges	Up to 3000 $\mu\varepsilon$ and ±1 to 4 $\mu\varepsilon$

Note:
a This is the manufacturer's stated accuracy for the instrument alone; the accuracy when embedded in the lining is different – see Jones 2007.

Lining Design Guide (2004) and van der Berg (1999). A good description of monitoring of adjacent structures can be found in the compendium of papers on London's Jubilee Line Extension project (Burland *et al.* 2001). Table 7.4 lists some of the instruments used on tunnelling projects.

The normal hierarchy of monitoring is: in-tunnel convergence; surface settlement; subsurface instruments (e.g. inclinometers, extensometers, piezometers); in-tunnel stress-strain measurements. In other words, most weight is placed on the measurements of lining deformations, then surface movements, and so on.

In the sections below there are a few brief comments on the instruments and their usage.

Convergence monitoring

Tape extensometers are more accurate that optical surveying methods but taking measurements with the tape can be more disruptive since it blocks traffic in the tunnel during the measuring. For this reason optical surveying has largely replaced tape extensometers.

When interpreting the monitoring by 3D optical measurements, it is important to remember that the readings can be affected by changes in the atmosphere in the tunnel and the accidental disturbance of targets. Sudden changes in readings may not actually indicate a change in deformation but merely that someone has knocked one of the pins.

Traditionally, convergence monitoring has focused on the inward movement of the lining. However, recent research indicated that the longitudinal movements can provide useful information too and they can be used to predict changes in rock conditions (Steindorfer 1997).

Stress monitoring

Pressure cells are the traditional method of measuring stresses in SCL tunnels, although there are other methods (e.g. slot cutting and overcoring – see Jones 2007 for a full review). Many authors have questioned the reliability of pressure cells in sprayed concrete (e.g. Golser *et al.* 1989, Golser and Kienberger 1997, Mair 1998, Kuwajima 1999, Clayton *et al.* 2000), for the following reasons:

- The physical size of the cells (100 mm wide) may lead to shadowing. Incomplete encasement would lead to under-reading of stress.
- During the rapid hydration, the cell may expand and on cooling leave a gap between itself and the concrete, again leading to under-reading (Golser *et al.* 1989).
- The increase in readings of tangential cells due to thermal effects has been calculated as 0.10 MPa/°C for mercury-filled cells and 0.15 MPa/°C for oil-filled cells (Clayton *et al.* 2000).[1]

- The increase in readings of radial cells due to thermal effects has been calculated as about 0.06 MPa/°C for oil-filled cells (Jones 2007).
- Shrinkage can also induce stresses into the pressure cells (Clayton *et al.* 2000).
- The stiffness of the cell is different from the surrounding lining. If there is no difference the Cell Action Factor (CAF)[2] is 1.0. The CAF is often close but lower than 1 (Clayton *et al.* 2000, Jones 2005), leading to under-reading slightly.

The results may also depend on the measuring system. Pressure cells with a hydraulic measuring system yielded readings that were about 80 kPa higher than vibrating wire cells (Bonapace 1997). Clayton *et al.* (2000) and Aldrian and Kattinger (1997) suggest that tangential cells record changes in stress accurately but should not be assumed to be recording the correct absolute values. The standard deviation in readings is often almost as large as the average readings themselves. On the Jubilee Line Project (JLE) the tangential stress was on average 2.0 MPa after 3 months (corresponding to about 25% of the full overburden pressure (FOB)) but ranged from 0.0 to 7.0 (Bonapace 1997).

Radial (Earth) pressure cells are believed to be more reliable because they are easier to install and the cell stiffness and the behaviour of the sprayed concrete have less influence on the readings (Clayton *et al.* 2000). However, like tangential cells, even when the results from a large number of cells are examined, there is usually considerable scatter in the results from radial cells (Bonapace 1997).

However, Jones (2007) has described how the errors listed above can be quantified and removed from pressure cell readings. The key steps are:

- Install the pressure cells as soon as possible after excavation and minimise shadowing around the cells.
- Spray two tangential cells in a test panel at the same time as the main array, leaving the test panel to cure in the tunnel (i.e. in the same environment).
- Record readings at very frequent intervals during early hydration using a datalogger (e.g. 10 minutes during first 7 days and hourly thereafter).
- Crimp to ensure that the cells remain in contact with the concrete.

Crimping not only ensures that contact is maintained with the concrete but can also provide an indication of the quality of installation. Temperature sensitivity can also be used for this purpose. The post-processing of the readings requires the following procedure.

For tangential cells:

- Make adjustment for temperature sensitivity of the vibrating wire transducer (using manufacturer's calibration).

- Remove any zero offset.
- Remove any crimping offset.
- Check for lost pressures if the pressure cell has at any time lost contact with the sprayed concrete.
- Estimate cell restraint temperature sensitivity from test panel data and estimate its variation with time during early age. Apply the correction for this.
- Estimate shrinkage pressure development with time from the test panel data and subtract from readings.

For radial pressure cells:

- Make adjustment for temperature sensitivity of the vibrating wire transducer (using manufacturer's calibration).
- Remove any zero offset.
- Remove any crimping offset.
- Check for lost pressures if the pressure cell has at any time lost contact with the sprayed concrete or ground. This is unlikely in the case of radial cells.

The correction for the overall temperature sensitivity for the embedded cell was found to be particularly important for tangential cells where even slight variations in ambient temperatures could induce changes in stress. This sensitivity explains some of the fluctuations in pressure cell readings. The correction for temperature sensitivity is complicated by the fact that the coefficient of thermal expansion changes with the age of the concrete (Jones *et al.* 2005). The apparent superior reliability of radial cells may simply be because they are not so prone to the effects of temperature and shrinkage.

This research seems to offer the important reassurance that – if the proper measures are taken – pressure cells can be used to obtain continuous and reliable measurement of the stresses in SCL tunnels. This enables the direct calculation of the factor of safety.

Finally, on the subject of stress measurements, one recent innovation is the use of software which estimates lining strains or stresses from lining deformations (e.g. Rokahr and Zachow 1997). While interesting, it should be noted that there are some fundamental theoretical limitations in these methods (e.g. they are based on 1D constitutive models or they assume that there is no bending in the lining). Also these calculations have rarely been calibrated against actual measurements of lining strains or stresses. Furthermore, Stark *et al.* (2002) note that consideration of the estimated stress intensity in the upper part of the lining can provide no warning of a failure in the invert, where deformation measurements are rarely taken. Also the variability in the readings of lining deformations (often ±2 or 3 mm) and the relative infrequency of the measurements hamper these methods too (Jones *et al.* 2005). Consequently it is arguable that, despite its increasing use, this system has yet to be fully proven.

7.2.2 Trigger values

A system of trigger values is used to assess the performance of the tunnel. Many different approaches exist so this section will describe one that is commonly used.

Typically there are three trigger values – a green, amber and red limit. The green limit marks the boundary of normal behaviour. The amber marks the boundary of serviceability while the red trigger should be set below the ultimate capacity of the lining. The contractor's Action Plan should include pre-planned contingency measures that can be taken if a trigger value is exceeded – see also Powell *et al.* (1997) and BTS Lining Design Guide (BTS 2004).

The estimation of the trigger values can be summarised in the following procedure (see Figure 7.4).

In the absence of published guidance there are many alternative ways to set the trigger values. For example, one major project set the amber trigger at 75% of the predicted settlement and the red trigger at 125%.

Sometimes a hierarchy is specified for the interpretation of monitoring (e.g. see Section 7.2.1). Table 7.6 shows a typical hierarchy with the deformation of the lining at the top of it. The order of the ranking should reflect the key concerns of each project, as well as the reliability of the instruments. For example, if a project is installing pressure cells in the tunnel lining and believes that good results can be obtained from them, then arguably pressure cells should feature higher up in the ranking since they give direct information on the factor of safety of the lining.

Due to the accuracy of the instrumentation, a fluctuation in readings can be expected. Therefore, if an individual reading reaches a trigger value, it

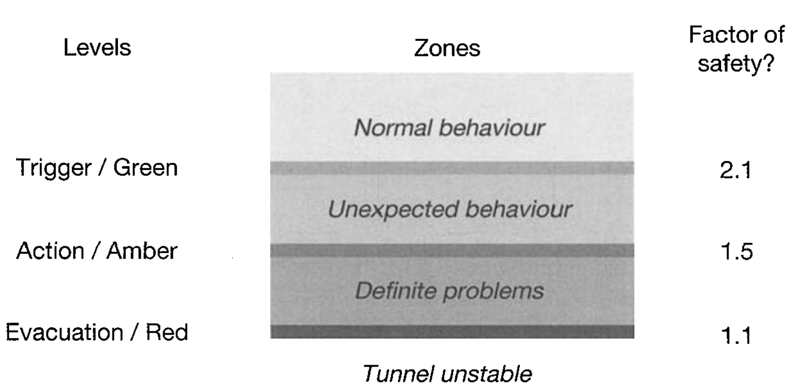


Figure 7.3 Trigger values

is not necessarily a cause for concern if the overall trend was stable; trends are more important than individual readings in determining whether trigger values had truly been exceeded.

When reviewing monitoring data, it is as important to consider the trends in the data as the absolute values themselves. For in-tunnel deformation

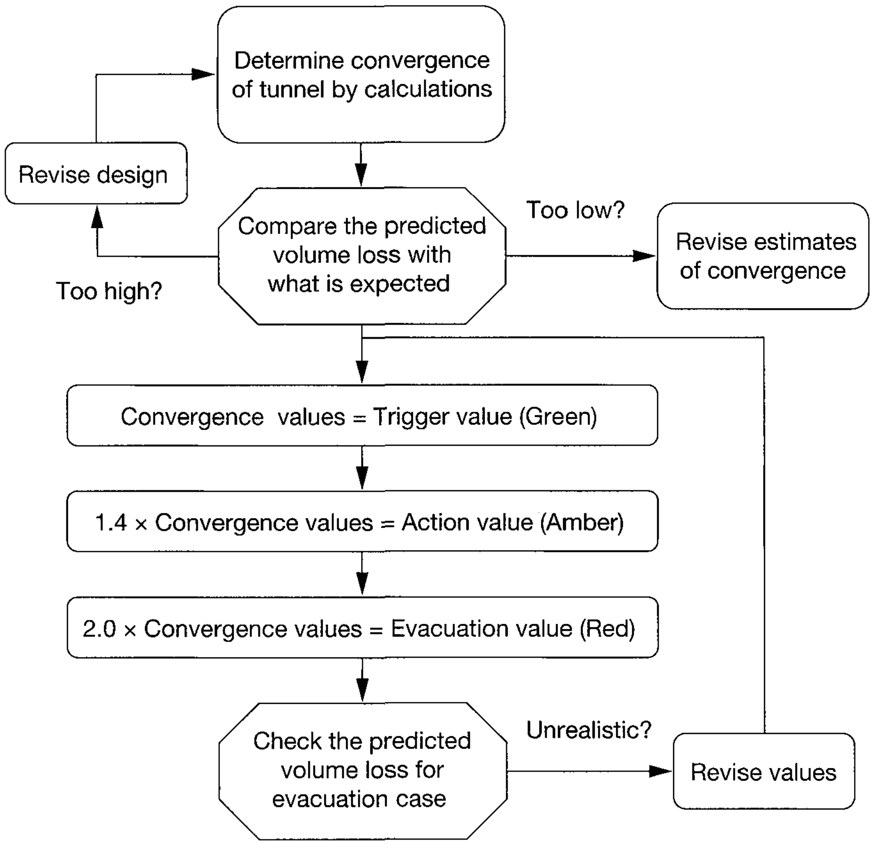


Figure 7.4 Derivation of trigger values

Table 7.6 Hierarchy for monitoring

Rank	*Instrument/parameter*
1	In-tunnel lining deformation
2	Surface settlement
3	Monitoring of adjacent structures (e.g. electrolevels, crackmeters)
4	Ground movement (e.g. inclinometers, extensometers)
5	Pressure cells (in-tunnel and in the ground)
6	Strain gauges

Table 7.7 Frequency of monitoring

Stage	*Location of tunnel face/timing*	*Frequency*
Prior to tunnelling	At least 3 months before entering Zone of Influence – ZoI	Weekly
During tunnelling	Tunnel face within ZoI	Daily
After tunnelling	Tunnel face outside the ZoI	Weekly
Until completion of monitoring	Not less than 3 months after leaving the ZoI	3 months or as instructed by the supervision team

readings, the tunnel is defined as stable when the rate of convergence has reduced to less than 2 mm per month. In high-stress environments, the rate of convergence is sometimes used to judge the best time to install the final cast *in situ* lining.

The frequency of monitoring is varied depending on how close the tunnel face is to the instruments and when the tunnelling will occur. The so-called 'Zone of Influence' – ZoI, is used to delineate the areas which require the most frequent monitoring (see Figure 7.2 and Table 7.7). In shallow, soft ground tunnels, typically the ZoI extends from three or four tunnel diameters ahead of the face to three or four tunnel diameters behind the closure of the completed ring.

The ZoI also extends laterally from the centreline of the tunnel. The lateral extent may be defined according to the predicted settlement contours – for example, set at a distance equal to the 1 mm contour.

7.3 Designer's Representative on site

The presence of a Designer's Representative on site (DR) provides continuity of the design through the construction phase. Typically the DR will work alongside the supervision team, with the objective of promoting a single team approach.

The exact relationship depends on the contractual environment of the project. The DR can assist to:

- review and approve the contractor's action plans (which defined the monitoring regime, contingency measures and the criteria for implementing changes to support);
- review and interpret monitoring data on a daily basis;
- review and approve changes in support and excavation sequences.

When required, the supervision team may issue instructions to the contractor on the basis of the DR's recommendations.

A single team approach, where key engineering decisions are made through assessment and evaluation of information on a routine basis during construction, has clear benefits in terms of reducing risks related to safety and quality control. The single team approach does not, however, mean that lines of responsibility are blurred as each organisation is required to appoint experienced staff to understand the engineering as well as contractual risks. This is seen as almost essential for the SCL approach where modifications and adaptations of the design within a specified framework are an inherent part of the design and construction process.

As an example, in one case the cost/benefit ratio of the presence of a Designer's Representative was estimated at more than 3:1. The engineers helped to achieve cost savings of more than $300,000 (Thomas *et al.* 2003).

For small projects a permanent presence on site may not be warranted but the designer should visit regularly and be involved in reviewing monitoring data. This is easy to achieve remotely with modern communications. Alternatively another way to communicate the design intent is to hold a workshop between the construction team and the designers before the works commence.

7.4 Daily Review Meetings (DRM)

Monitoring alone serves no useful purpose. The construction of SCL tunnels is controlled by the process of reviewing the progress and monitoring data

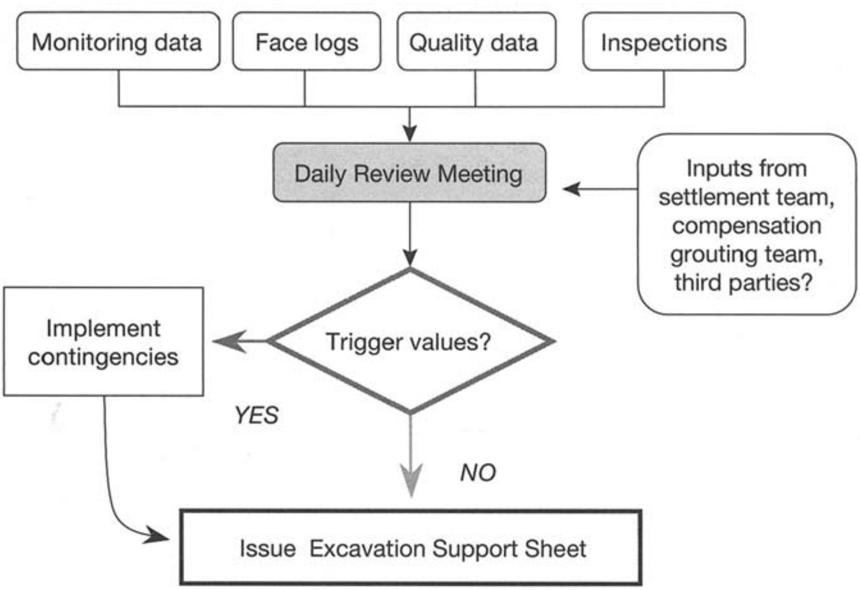


Figure 7.5 Information flow at the Daily Review Meeting

and taking actions based on that information, guided by criteria such as trigger values on deformations. Figure 7.5 illustrates the types of information that is fed into the review process. Daily Review Meetings (DRM) are standard practice. All parties actively involved with a tunnel should be represented at the DRM. For example, if compensation grouting is being performed nearby, the grouting team should be present too since their works affect the tunnel. The DRMs are augmented with Weekly and Monthly Review Meetings on more complex projects so that higher level staff can be kept informed of the overall performance of the tunnels. The data should be presented clearly at the meeting, ideally in graphs showing the absolute values and trends.

The outputs from the DRM are the instructions for the next day's tunnelling and any additional measures required to counter adverse trends in

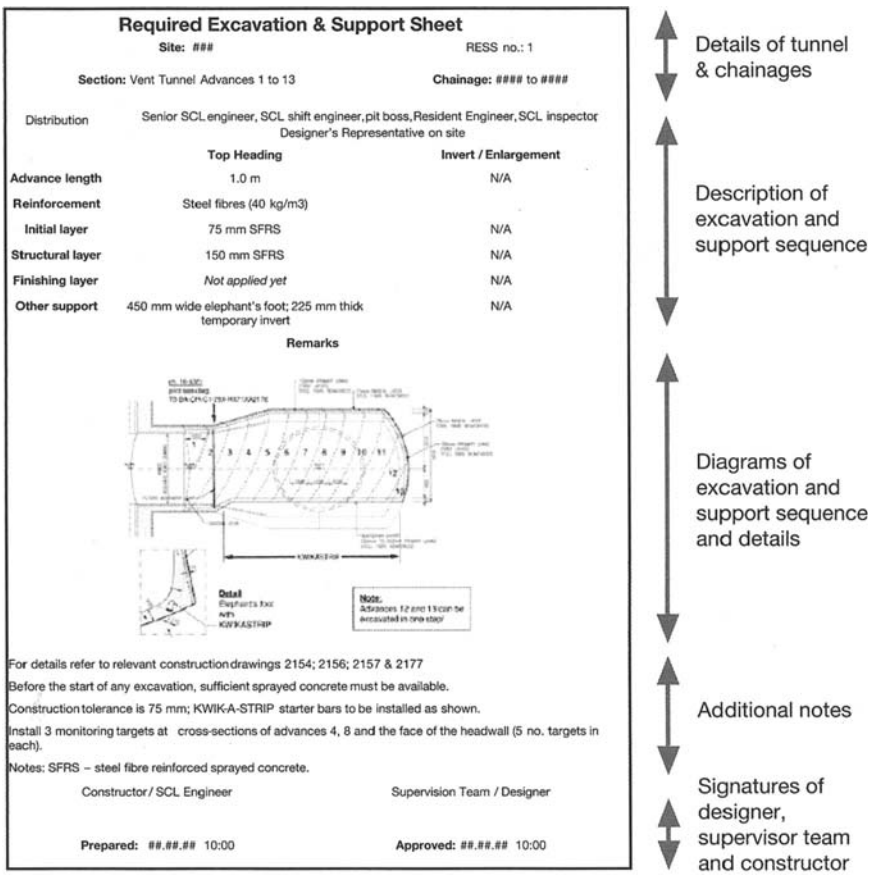


Figure 7.6 Excavation and Support Sheet

behaviour of the tunnels. An Excavation and Support Sheet is a simple tool to record these instructions and to communicate them to everyone concerned, from managers to the foreman at the tunnel face (see Figure 7.6).

If a trigger value is reached, first the site team should check that the reading is correct and consistent with the readings from other instruments. If the trigger has really been breached, then contingency measures will be instigated, in accordance with a pre-defined Action Plan and as directed in the DRM. The contingency measures are designed to correct any anomalous behaviour. They range from increasing the frequency of monitoring and inspections (to gain a better understanding of what is happening), amending the excavation sequence, increasing the support measures or adding new support measures to, in the worst case, backfilling the tunnel. However, on a happier note, if these simple construction management techniques are followed, such drastic situations can be avoided.

Notes

1 The temperature corrections quoted by the manufacturers of pressure cells normally refer to the transducer only and not the whole cell. The peak in recorded pressures often coincides with the peak in the temperature of the concrete (Clayton *et al.* 2000).

2 The Cell Action Factor (CAF) is the ratio of recorded pressure to actual pressure.

Abbreviations and symbols

α	utilisation factor = stress/strength or r/yield strength
acc.	according to
agg.	aggregate
ACI	American Concrete Institute
B	relaxation time in Kelvin creep model – see 5.6.2
BTS	British Tunnelling Society
bwc	by weight of cement and microsilica
ξ	degree of hydration, except in Equation 4.1 where it is the 'skin factor'
c/c	centre to centre
CCM	Convergence Confinement Method
C/D	cover (depth from ground surface to tunnel axis)/tunnel diameter
C_u	undrained shear strength
δ_v	vertical deformation
ε	strain
ε_{dev}	deviatoric strain
E	Young's modulus of elasticity
E_a/R	activation energy = 4000 K – see Section 5.7
E_{max}	maximum value of the elastic modulus
E_0	initial tangent modulus
E_{tan}	tangent elastic modulus
e_{ij}	deviatoric strain rate
$\dot{e}_{ij} = \sqrt{(2 \cdot \dot{J}_2')}$	deviatoric strain rate
est.	estimated
f_c	strength – f_{cu} or f_{cyl}
f_{cu}	uniaxial compressive cube strength
f_{cyl}	uniaxial compressive strength (from tests on cylinders)
FLAC	FLAC3D and FLAC (2D) finite difference program by Itasca
FOB	full overburden pressure
G	elastic shear modulus
G_{vh}	independent shear modulus

GGBFS	Ground Granulated Blast Furnace Slag
γ_f	partial factor of safety for loads (see BS8110)
γ_m	partial factor of safety for materials (see BS8110)
GPa	giga Pascals
h	depth below groundwater level
HEX	Heathrow Express project
HME	Hypothetical Modulus of Elasticity
HSE	UK Health and Safety Executive
I_{10}	SFRS toughness index
ICE	UK Institution of Civil Engineers
ITA	International Tunnelling Association
JLE	Jubilee Line Extension project
J_2	second deviatoric invariant of principal stresses $J_2 = 1/6((\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2)$
k	permeability
K	elastic bulk modulus
K_0	ratio of horizontal effective stress to vertical effective stress
kPa	kilo Pascals
long.	longitudinal
λ	stress relaxation factor
max.	maximum
MPa	mega Pascals
η	viscosity, except in Equation 2.2 where it is a constant
NATM	New Austrian Tunnelling Method
OCR	overconsolidation ratio
OPC	ordinary Portland cement
p	mean total stress
p'	mean effective stress
PFA	Pulverised Fly Ash
pts	points
Q	water flow
r	tunnel radius (e.g. Equation 4.1)
r	deviatoric stress $= 2(J_2)^{0.5}$
R	tunnel radius
R	universal constant for ideal gas – see E_a/R above
RH	Relative Humidity
σ	stress (compression is taken to be negative)
σ_c	uniaxial compressive strength
σ_v	vertical stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses
SCL	Sprayed Concrete Lined/Lining
SFRS	Steel Fibre Reinforced Shotcrete
str	strength

θ	Lode angle (or angle of similarity) where, $\cos\theta = \dfrac{2\sigma_1 - \sigma_2 - \sigma_3}{2\sqrt{3J_2}}$; $\theta = 0°$ corresponds to the tensile meridian and $\theta = 60°$ corresponds to the compressive meridian
t	time or age, except in Equations 4.1 and 6.1 where it denotes thickness
v, u	Poisson's ratio
w/c	water/cement ratio
w.r.t.	with respect to
2D	two dimensional
3D	three dimensional

Symbols

A_1	in the first principal stress/strain direction
A_c	compressive
A_g	related to the ground
A_h	in the horizontal plane/direction
A_{hh}	in the horizontal plane/direction
A_{ij}	in principal stress/strain directions where i and j can be 1, 2 or 3
A_0	initial value (e.g. value of modulus at strain is zero)
A_t	time-dependent value
A_{tan}	tangential (e.g. tangential elastic modulus)
A_u	undrained (in context of geotechnical parameters)
A_v	in the vertical plane/direction
A_{xx}	in the direction of x-axis
A_{yy}	in the direction of y-axis
A_{zz}	in the direction of z-axis
A_{28}	value at an age of 28 days

Sprayed Concrete Lined Tunnels

Sprayed concrete lined (SCL) tunnels are rapidly growing in popularity due to their versatility. The design and construction of both hard rock and soft ground tunnels has been revolutionised by the advent of the SCL method and now the use of permanent sprayed concrete linings has unlocked the true potential of the method to minimise construction costs and times. Yet the complex early-age behaviour of the sprayed concrete makes the design difficult and requires a robust management system during construction. Consequently the great advantages of the method must be balanced against the large risks, as illustrated by recent high-profile tunnel collapses.

Practising engineers on site, in the design office or in client organisations will find this book an excellent introduction. It covers all aspects of SCL tunnelling – from the constituents of sprayed concrete to detailed design and management during construction. Although there is a close interdependence between all the facets of sprayed concrete, few engineers have the right breadth of experience and expertise to cover all of them. This urgently needs to be transferred to the wider engineering community as SCL tunnels play an important role in the delivery of the underground infrastructure which modern urban life demands.

Alun Thomas is a senior engineer at Mott MacDonald Ltd. He has been involved in many of the recent major UK tunnelling projects such as the Jubilee Line Extension, Heathrow Express, Terminal 5 and Crossrail, as well as working on the design and construction of tunnels internationally.

Illustrations

Figures

Tables

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Appendices

Appendix A The evolution of mechanical properties of sprayed concrete with time

Property and author	*Equations*	*Constants*	*Range of applicability*	*Age range*	*References*
Elastic modulus – Weber	$E_b = E_b^{28} \cdot a_e \cdot e^C$ $C = C_e/t^{0.60}$ t = age in days	$a_e = 1.084$ $C_e = -0.596$ $E^{28} = 30$ GPa; $a_e = 1.132$ $C_e = -0.916$ $A = 1.084$; $C_e = -0.916$ $E^{28} = 26$ GPa – Yin; $a_e = 1.084$; $C_e = -0.196$ – Huang $a_e = 1.084$; $C_e = -0.596$ – Schropfer, Pöttler	Z35F/45F cement Normal concrete Z25, 35L, 45L(Pöttler) Used by Yin, Huang, Pöttler and Schropfer		Alkhiami 1995; Yin 1996; Huang 1991; Schropfer 1995; Pöttler 1990
Compressive strength – Weber	$\beta_b = \beta_b^{28} \cdot a_\beta \cdot e^C$ $C = C_\beta/t^{0.55}$ t = age in days	$a_\beta = 1.27$ $C_\beta = -1.49$	Z35F/45F cement Normal concrete (Yin)		Alkhiami 1995; Yin 1996
Elastic modulus – Chang	$E = E^{28} \cdot a \cdot e^C$ $C = c/t^{0.70}$ t = age in days	$a = 1.062$ $c = -0.446$ $E = 3.86\sigma_c^{0.60}$	Based on literature review	$E - \sigma_c$ valid for 0–7 days; E valid for 4 hours to 28 days; $\alpha < 0.70$	Chang and Stille 1993; Chang 1994
Compressive strength – Chang	$\sigma_c = \sigma_c^{28} \cdot a.e^C$ $C = c/t^{0.70}$ t = age in days	$a = 1.105$ $c = -0.743$ $\sigma_c = 0.105\ E^{1.667}$	Based on literature review	$E - \sigma_c$ valid for 0–7 days; E valid for 4 hours to 28 days; $\alpha < 0.70$	Chang and Stille 1993; Chang 1994

Property and author	*Equations*	*Constants*	*Range of applicability*	*Age range*	*References*
Compressive strength – modified Byfors	$f^t_{cc} = 0.01 \times (A_1 t^{B1}/X) \cdot f^{28}_{cc}$ in % $X = 1 + (A_1/A_2) \cdot t^{(A1-B2)}$	Cylinder str in MPa, t = age in hours, For w/c = 0.4 – 0.5, A1 = 0.300%; A2 = 29.446% B1 = 2.676; B2 = 0.188 f^{28}_{cc} = 29.022 MPa	Back-calc from cores on panels sprayed *in situ*		Kuwajima 1999
Elastic modulus – modified Byfors	$E_c = c_1 f^{d1}_{cc}/X$ in GPa $X = 1 + (c_1/c_2) \cdot f^{(d1+d2)}_{cc}$	C_1 = 1306 C_2 = 7194 D_1 = 1.920; D_2 = 0.363	Wide scatter for $t < 10$ hrs		Kuwajima 1999
Elastic modulus – modified Byfors	$E_{cc} = 7.194 f_{cc}^{0.363}$		Strengths > 1 MPa		Kuwajima 1999
υ Poisson's ratio	$\nu_{cc} = a f^b_{cc}$ $r = 0.847$,	a = 0.128, b = 0.192, n = 52 and $f_{cc}/f^{28d}_{cc} = at^b$	Strengths > 1 MPa		Byfors 1980
Elastic modulus – Golser	$E_t = E_{28} \cdot ([a+bt]/t)^{-0.5}$	a = 4.2; b = 0.85; t in days			Yin 1996

Appendix A *continued*

Property and author	*Equations*	*Constants*	*Range of applicability*	*Age range*	*References*
Compressive strength – Aldrian	$\beta_t = \beta_{28} \cdot 0.03t$ $\beta_t = \beta_{28} \cdot [(t-5)/(45+0.975t)]^{0.5}$	$t \leqslant 8$ hours $t > 8$ hours	For prisms; $\beta_{cube} = 1.07.\beta_{prism}$ for the sizes used by Aldrian		Aldrian 1991
Elastic modulus – Aydan	$E_t = A(1 - e^{Bt})$	$A = 5000$, $B = -0.42$	Wet mix, loaded at right angles to spraying	Age = 3 hrs to 28 days	Yin 1996; Aydan *et al.* 1992
υ – Poisson's ratio – Sezaki	$\upsilon_t = a + b \cdot e^{ct}$	$a = 0.18$, $b = 0.32$, $c = -5.6$	Wet mix, loaded at right angles to spraying	Age = 3 hrs to 28 days	Yin 1996; Aydan *et al.* 1992
Compressive strength – Meschke	$f_{cu(t)} = f_{cu(1)} \cdot [(t+0.12)/24]^{0.72453}$ $f_{cu(t)} = ac\ e^{(-bc/t)}$ $ac = 1.027 f_{cu28}$ $bc = 17.80$	for $t < 24$ hours for $t > 24$ hours, where $\kappa = 0.489 = f_{cu1}/f_{cu28}$; ac and bc are functions of κ	Austrian shotcrete guidelines for $t < 24$ hours		Meschke 1996; Kropik 1994
Elastic modulus – Meschke	$E_t = \beta_{E\tau} \cdot E_{28}$ $\beta_{Et} = 0.0468t - 0.00211t^2$	$t < 8$ hours	Modified CEB-FIP	Age = 0 to 28 days	Meschke 1996; Kropik 1994

Property and author	*Equations*	*Constants*	*Range of applicability*	*Age range*	*References*
	$\beta_{Et} = (0.9506 + 32.89/[t - 6])^{-0.5}$	$8 < t < 672$ hours			
Compressive strength – Pöttler	$\beta_t = \beta_1 t^{0.72453}$ or $\beta_t = \beta_1(1 + 4t)/5$	$\beta_1 = 1$ day str	From cube tests	Age > 1 day	Pöttler 1993; Pöttler 1990
Compressive strength – Eierle and Schikora	$f_{cc}(\alpha)/f_{cc,\alpha=1} = [(\alpha - \alpha_0)/(1 - \alpha_0)]^{3/2}$	$f_{cc,\,\alpha=1}$ = compressive strength at complete hydration; α = degree of hydration $\alpha_0 = 1.8$ est.	For plain concrete; for tensile strength, $f_{ct}(\alpha)/f_{ct,\,\alpha=1} = (\alpha - \alpha_0)/(1 - \alpha_0)$		Eierle and Schikora 1999
Elastic modulus – Eierle and Schikora	$E(\alpha)/E_{\alpha=1} = [(\alpha - \alpha_0)/(1 - \alpha_0)]^{2/3}$	$E_{\alpha=1}$ = modulus at complete hydration; α = degree of hydration			
Compressive strength – Kusterle	Prisms $Y = 1.8171X^{0.8285}$ Cylinders $Y = 0.0521X^2 + 0.3132X + 0.9213$ Beams $Y = 0.9208X^{0.9729}$	Y = prism, cylinder or beam X = cube str in MPa Cube = 20 × 20 × 20 cm; Prism = 4 × 4 × 16 cm; Cylinder = 10 cm dia. × 10 cm; Beam = 10 × 10 × 50 cm	Str range = 1 to 12 MPa		Testor and Kusterle 1998

Note: See Appendix C Part 3 for tensile strength.

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