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Design and construction of a permanent shotcrete lining— The A3 Hindhead Project, UK

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ABSTRACT: This paper discusses the design and construction of a permanent shotcrete lining for the 1.8 km long A3 Hindhead twin bore highway tunnel that commenced construction in January 2008 and for which excavation and support was completed in March 2009. The tunnel is excavated within the Hythe Beds—a 90 m thick sequence within the Lower Greensand Series formation consisting of 'weak, to very weak sandstone with occasional thin beds of fine sand'. The proposed construction methodology was progressive mechanical excavation of a horseshoe shaped tunnel with shotcrete installed in each excavation cycle using robotic spraying equipment. This paper describes the development of the innovative 'permanent' primary lining design including the modelling of the lining, specification of a durable shotcrete mix including early strength requirements, as well as details of the construction. This construction technique has proved to be very successful in 'soft' rock, and has local applicability to Auckland's bedrock material, East Coast Bays Formation.

1 INTRODUCTION

The A3 Hindhead project located in Surrey is the UK's latest bored highway tunnel. The development of the design has incorporated recent advances in tunnelling technology resulting in an innovative approach to many aspects of the tunnel support systems, and in particular the shotcrete lining.

The project is a 6.7 km dual carriageway trunk road that includes a 1.83 km tunnel being delivered under a UK Highways Agency Early Contractor Involvement (ECI) contract. The tunnel has been constructed using a sequential excavation method whereby shotcrete was sprayed at the face following each advance by a tunnel excavator, also known as Sprayed Concrete Lining (SCL) in the UK and as the New Austrian Tunnelling Method (NATM) in Europe.

This paper describes the development of the innovative 'permanent' primary lining design including: the modelling of the lining; specification of a durable shotcrete mix including early strength requirements; and testing and construction requirements. The use of a sprayed applied membrane has also enabled the use of a sprayed shotcrete secondary lining. This construction technique has proved to be very successful in 'soft' rock, and has local applicability to Auckland's bedrock material, East Coast Bays Formation.

2 BACKGROUND

The A3 Hindhead project is one of the schemes in the UK Government's Targeted Programme of Trunk Road Improvements. The project will complete the dual carriageway link between London and Portsmouth and remove a major source of congestion, particularly around the A3/A287 traffic signal controlled crossroads. Refer to Figure 1 for location details.

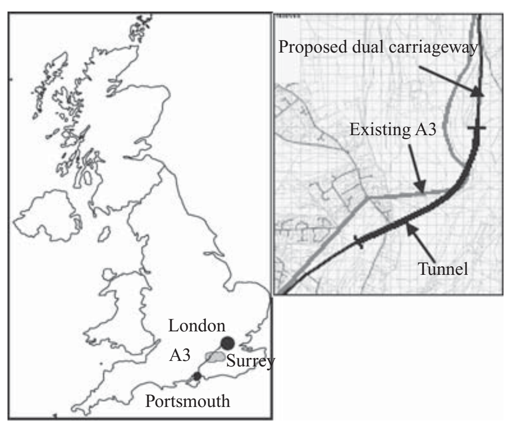

Figure 1. Project location.

The project will deliver quicker, more reliable journeys on a safer road, and remove much of the present peak time "rat-running" traffic from unsuitable country roads around Hindhead. The centre of Hindhead will be freed from the daily gridlock that blights the area, with the result that the project will bring benefits to road users, local residents, and the highly prized local environment. Construction of the project commenced in January 2007, and the tunnelling commence in January 2008, with excavation and support completed in March 2009. The tunnel is planned to be opened in 2011, with the scheme completed in March 2012.

2.1 *Early Contractor Involvement (ECI) Contract*

The Highways Agency (HA) introduced the principle of "Early Contractor Involvement" in 2001. This new form of procurement is concerned with bringing suppliers and designers together much earlier in scheme conception than previously occurred, allowing them to work together more closely. This allows more scope for innovation, improved risk management, better forward planning of resource requirements and minimisation of long term environmental impacts, improved consideration of buildability and health and safety, shorter construction periods and reduced environmental impacts during construction.

Overall, the HA considers that the early creation of delivery teams offers the opportunity for better value and improved performance.

3 GEOLOGY

3.1 *Overview*

The geology of the Hindhead area comprises a sequence of fine grained sedimentary deposits laid down during the Lower Cretaceous period in near shore transgressive marine conditions on the margins of the subsiding Weald Basin. The tunnel is within the Hythe Beds—a 90 m thick sequence within the Lower Greensand Series formation.

The Hythe beds are variably sorted, highly glauconitic, variably bioturbated and cross-bedded sands and sandstones. The Hythe bed unit is divided into six litho-stratigraphic subdivisions, through four of which the tunnel passes.

3.2 *Tunnelling conditions*

The tunnel at the southern end passes through units Upper Hythe A and B which are similar units with an increasing number of sandstone bands with depth, described as 'medium dense thinly bedded and thinly laminated, clean to silty and clayey fine and medium sand with subordinate weak to strong sandstone, cherty sandstone and chert'.

The majority of the tunnel passes through the more competent Upper Hythe C and D, and Lower Hythe A units, described as '*Weak, locally very weak to moderately strong, slightly clayey fine to medium SANDSTONE with occasional thin beds of clayey/silty fine sand*'. The remaining unit is Lower Hythe B which has been avoided by the tunnel as clays and sand become dominant in the lower half of the unit.

The sandstone within Upper Hythe C/D and Lower Hythe A has typical UCS values of between 2 and 5 MPa and is heavily fractured with six joint sets including the sub-horizontal bedding with mean fracture centres varying between 190 and 815 mm. The tunnel also intersects a small number of discrete thin bentonitic Fuller's Earth Beds, formed from air- and water-borne volcanic ash, including crystal and lithic tuffs. The tunnel is above the historically observed water table, with the maximum predicted water table exceeding the invert level in only one location by a depth of less than 1 m. Refer to Figure 2 for a geological longsection.

3.3 *Ground behavior model*

A challenge for this project was to define a ground behavior model in an unusual material that had not been tunnelled previously. The difficulties in interpretation stem from the weak to very weak nature of the sandstone material in combination with the content of up to 20% interbedded soil layers. A recurring challenge in tunnelling is to determine the rock mass strength and stiffness, with empirical methods such as RMR, Q-method (Bieniawski, 1984), or GSI (Marinos and Hoek, 2000) often used. The strength and stiffness relationships that are the cornerstone of these methods are generally determined from data for significantly stronger rocks than the 2–5 MPa sandstone and do not take account of the influence of the soil layers leading to a significant over-estimate of stiffness.

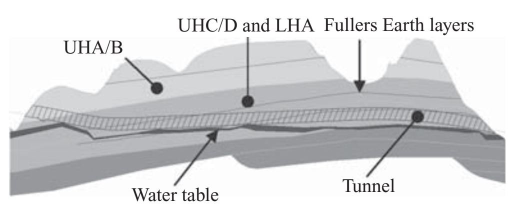

Figure 2. Geological longsection.

An extensive geotechnical investigation was undertaken with sonic testing, pressuremeters and triaxial testing all used to determine the Elastic Modulus of the rock mass. Pressuremeter testing was found to be the most reliable with the sonic testing over-estimating the stiffness, and the triaxial testing surprisingly under-estimating the stiffness in a number of cases. This was thought due to the difficultly in finding a 300 mm long specimen for testing in a material with 6 joints sets and an average bedding spacing of 190 mm. Fortunately some good quality rock joint shear box testing was undertaken that provided a lower bound for strength and stiffness interpretations. The interpreted model used in design was a small strain stiffness model ($E_{(\varepsilon a)}$) that varied the stiffness of the rock mass with strain, and a Mohr-Coulomb strain softening model used for strength.

4 TUNNEL CROSS SECTION

The Hindhead tunnel layout comprises twin 2-Lane bores with cross passages at 100 m nominal centres. Refer to the typical cross section in Figure 3 below. Each bore has two 3.65 m lanes, with full batter curbs and 1.2 m wide verges on each side of the tunnel. The verge width is sufficient to allow for sight-lines due to the horizontal curvature of the tunnel, to accommodate electrical services and also to provide wheelchair access to the cross passages and emergency points at 100 m nominal centres along the tunnel. The vertical traffic gauge provided is 5.03 m with an additional clearance of 250 mm to the Equipment Gauge to allow for flapping tarpaulins and other transitory gauge infringements.

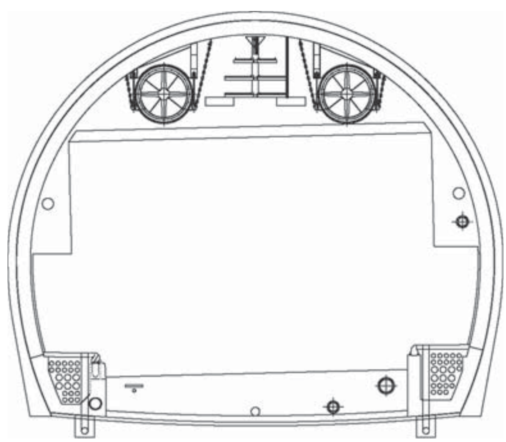

Figure 3. Typical tunnel cross section.

5 TUNNEL EXCAVATION AND SUPPORT TYPES

The presence of the sand layers, in one location up to 2 m thick, led to the selection of a sequential excavation method whereby Fibre Reinforced Shotcrete (FRS) is sprayed at the face following each excavation advance, also known as Sprayed Concrete Lining (SCL) in the UK and as the New Austrian Tunnelling Method (NATM) in Europe. Standard hard rock tunnel support techniques such as pattern bolting were not considered suitable due to the sand layers and the very low bond stress negatively impacting the effectiveness of rock reinforcement.

Four basic support types were designed for the standard tunnel cross sections with minor variations required at cross passage junctions and Emergency Point niches. There is one main support type for the sandstone section, with three support types covering the section through sand and the transition from sand to sandstone.

Excavation and support types were specified based on tunnel chainage and have been designed to cover all expected ground conditions. It was not proposed that support types be selected based on geological inspection. The Hythe beds had 6 joint sets and an average joint spacing of less than 200 mm. The UCS values were typically 2–5 MPa. This material was expected to act as a continuum, and given the heavily fractured nature of the material, and presence of sand layers, meaningful variations in rock quality were expected to be difficult to detect. A suite of 'additional measures' discussed below were designed to account for any local stability issues. Geological inspection and mapping of the open excavation, and monitoring results, were used to determine the advance length that may have varied between 1 m and 2 m, with 1 m advances specified at critical locations such as beneath surface structures and roads.

5.1 *Support Type 1*

At the northern end of the tunnel (Chainage 3120 to 4650), excavation was in rock (UHC/D, LHA) and Support Type 1 was specified throughout. The tunnel was generally excavated with a full face heading followed at a distance by the bench excavation. Due to the generally stable nature of the ground and tunnel location above the water table, a closed invert was not required and the horse-shoe shaped primary lining was supported on elephant's feet. Refer to Figure 4 for details of the primary lining in the sandstone section.

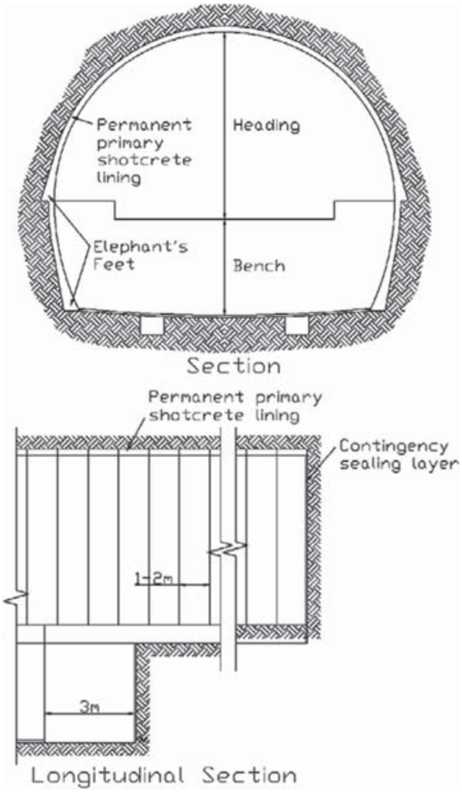

Figure 4. Support Type 1 in the SANDSTONE section.

5.1.1 *Additional support measures*

In addition to the sequences and support requirements for each of the support types, contingency 'additional support measures' were specified. The requirement for the contingency measures was triggered by geological inspection and mapping, and monitoring results, and included:

Spiling. Self drilling GRP tubular spiles were installed when ground conditions resulted in excessive overbreak, or instability in the crown. Spiling was detailed as mandatory for some of the Support Type 1 excavation, in areas of known potential crown overbreak such as where the tunnel crosses Fullers Earth bands, and where a 2 m thick layer of sand and shattered rock intersected the crown. During construction it was found that there were less areas of instability and only 26% of the tunnel required forward support compared to the initially anticipated 39%.

Additional face support measures. Geological inspections informed the selection of additional face support measures such as sealing layers, face support by the later excavation of a central support wedge, or face dowels.

Probe Drilling. Probe holes drilled ahead of the face were specified for the entire length of Support Type 1 to relieve any potential hydrostatic pressure from perched water ahead of the face. If water was detected in the probe holes then additional holes were drilled to drain any water.

Grout Stabilisation. Microfine cement or chemical grouts were specified to stabilise running sand bands, or other local areas of instability caused by sandy layers. In the event this was not required.

Invert strut. Convergence monitoring was undertaken during construction with a three stage trigger limit system implemented. Unplanned convergence resulting from worse than expected ground conditions was addressed by installation of an invert strut at bench level.

5.2 *Support Types 2–4*

At the South end of the tunnel (Chainage 2880 to 3120 (m001)), excavation was in sand (UHA/B) and support types 2, 3 and 4 were specified. Refer to Figure 5 below for details. The excavation was

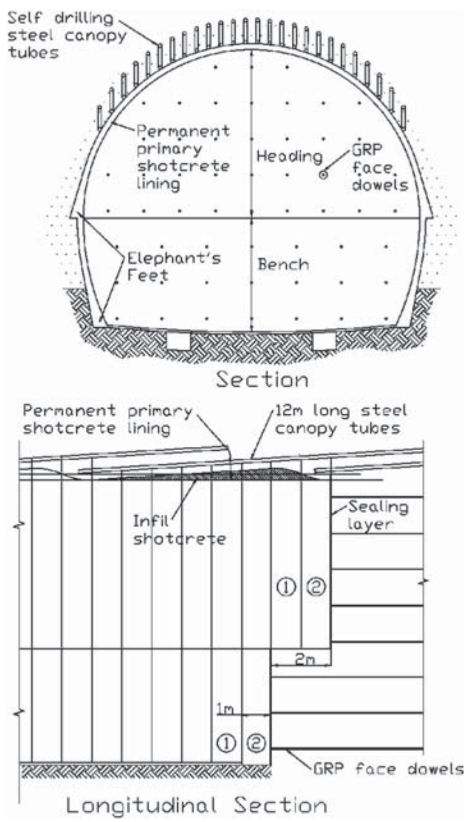

Figure 5. Support Type 3 which is typical for the SAND section.

carried out on dayshift only due to constraints on working hours and was made stable with the use of a steel pipe umbrella and face dowels. The pipe canopy comprised 12 m long 139.7 mm diameter tubes at 425 mm centers with an overlap of 4 m. The advance length was a maximum of 1 m for these support types.

Support Type 2 had sandy material (UHA/B) in the heading only, with the heading elephant's feet supported on the sandstone material (UHC/D). This meant GRP face dowels were required in the heading only, and the heading could advance ahead of the bench. The face dowels were 12 m long with a 4 m overlap and were installed with the same drill jumbo used to install the pipe canopy.

Support Type 3 had a full face of sandy material with the elephant's feet of the bench supported on the sandstone material. As the heading elephant's feet were not supported on sound material, the heading was advanced with the bench, with a 2 m separation provided to maintain face stability. Face dowels were required for both the heading and the bench. Support Type 4 had a full face of sandy material that extended below the tunnel, and therefore a closed invert was required. The heading had to be advanced with the bench, with the invert closed at a maximum of 6 m behind the face.

6 DESIGN APPROACH AND METHODOLOGY

The excavation sequences outlined above were designed to control strains in the ground so that as much as possible of the ground load bearing capacity was used and the strains maintained at levels that minimised yielding.

6.1 *Analysis*

The main design approach was to utilise numerical methods of analysis (FLAC Version 5.0) to model in 2-D the different excavation stages and predict the performance of the linings in terms of the stresses and corresponding deformations. The model included the non-linear small strain stiffness material model and Mohr-Coulomb failure criterion with strain softening to residual strength parameters. The models included the insitu stress regime with the horizontal stress modelled between 0.7 and 1.15 of the vertical stress, with 0.5 also used in the lower cover regions.

A modelling simplification inherent in 2-D modelling is the relaxation of the ground ahead of an excavation face, a largely 3-D effect, and the time-dependent development of lining strength and stiffness. In order to determine the appropriate value for these ground conditions a 3-D numerical model (FLAC Version 3.0) was utilised, which can accurately model the ground relaxation based on the proposed construction sequence and also the time-dependent strength and stiffness development of the shotcrete. For the 2-D model, the 28 day stiffness value was used in accordance with Eurocode 2, divided by a creep and shrinkage factor of 2. This factor reduces the stiffness of the spayed concrete lining to take account of the creep and relaxation that occurs when loading during early age (John and Mattle, 2003). The 2-D model was then calibrated against the 3-D modelling to determine an equivalent relaxation of 60%. In comparing the 2-D analysis with the 3-D analysis it was found that moments were over-estimated in 2-D at the heading stage and slightly under-estimated at full excavation stage. Axial loads were consistently over-estimated in 2-D everywhere except at the elephant's feet, however this was due to the increase in mesh fineness at the elephant's feet locations used in the 2-D analysis.

In addition to providing calibration of the relaxation figure, the 3-D FLAC modelling was also used to model stability of the face, early age capacity of sprayed concrete lining near the face, and the effect of variations in advance length.

6.1.1 *Shotcrete strength gain*

There are various different models on which to base the strength gain properties of the sprayed concrete lining. The J2 and J3 curves from the Austrian Sprayed Concrete curves (Osterreichischer Betonverein, 1999), and an alternative model for tunnel shotcrete (Chang & Stille, 1993), where the properties (Y) of young shotcrete are estimated from 28 day values (Y_{28}) with the expression:

$$Y = a \cdot Y_{28} \cdot e^{(c/t)\wedge 0.7} \qquad (1)$$

where a and c are constants that vary for each property (values for which are provided in the paper); and t = time in days. This equation indicates lower shotcrete strengths than the J curves during the first 3 hours of shotcrete life, but provides a strength gain curve for longer than the first 24 hours of the concrete's behaviour.

For the A3 Hindhead project tunnel shotcrete, an upper J2 curve which is half way between the J2 and J3 curves, was specified. In order to test the sensitivity of the design to the strength gain, modelling was undertaken using both the Chang & Stille shotcrete strength model and the Upper J2 curve up until 24 hours followed by the Chang & Stille relationship. For details of the differences between the two strength models refer to Figure 6.

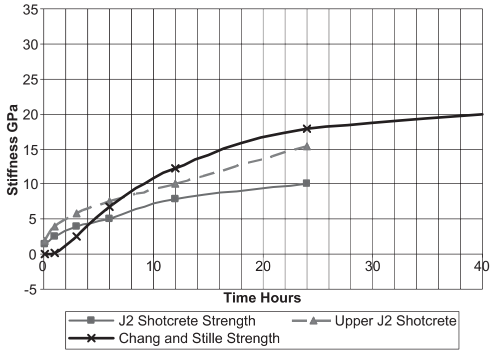

Figure 6. Shotcrete stiffness vs time relationships.

The stiffness of the concrete was also based upon an equation from Chang & Stille:

$$E = 3.86\ \sigma_c^{0.60} \quad (2)$$

where σ_c = compressive strength of the concrete at the point in time at which the concrete stiffness is being calculated. Stiffness develops at a faster rate than strength, which can result in a high loading rate on the shotcrete. Eurocode 2 defines the concrete stiffness as:

$$E_{cm}\ (\text{GPa}) = 22[(f_{cm})/10]^{\ 0.3} \quad (3)$$

where f_{cm} = mean compressive strength of the concrete at 28 days. This is then reduced by 10% due to the inclusion of Limestone in the concrete aggregate. In order to model how the stiffness of the concrete changes with age the equation used is:

$$E_{cm}(\text{t}) = (f_{cm}(t)\ /\ f_{cm})^{0.3}\ E_{cm} \quad (4)$$

The Eurocode equation does not allow for as much creep of the young concrete, which is important in the behaviour of shotcrete. Modelling was undertaken using the Chang & Stille equation as it was proposed particularly with sprayed concrete in mind. The Chang & Stille stiffness relationship may still not be entirely suitable for very early age shotcrete and is probably still an over-estimation of stiffness. In order not to design over-conservatively for very young concrete the ULS has not been used for concrete less than 2 days old, and a reduced factor of safety was considered acceptable.

The analysis indicated that for the heading excavation the Upper J2 strength curve, which gains strength and stiffness fastest, attracts more axial load. The Chang & Stille model starts as the weakest so has the lowest axial load at the lining. The moments are generally higher for Upper J2 but the Chang & Stille model results in the peak moment due to greater deflections resulting from a lower stiffness lining.

The choice of strength gain curve had the greatest impact for the shotcrete advances just behind the face. If the strength of the concrete does not increase quickly enough the excavation rate may have to slow. Having a rate fast enough is particularly important at the south portal where work can only take place during the day. For this reason, despite indications that the lower stiffness at early age of the J2 curve results in lower final axial loads and moments in the lining, it does not gain strength quickly enough to progress at a fast enough pace at the face. In the event during construction the contractor elected to implement a 3 m exclusion zone at the face, and as the early strength was not as critical the J2 strength gain curve could be used thereby saving the need for the additional additives required to achieve the higher strengths at very early age.

6.1.2 *Heading advance length analysis*

In order to evaluate the effect of advance length on load development in the shotcrete lining, analyses were carried out with a variation in excavation step size of 1, 1.5 and 2 metres.

For the 1.0 m heading excavation step, the loads on the lining were found to have a short term safety factor of around 1.4 at 3 m back from the lining face. This meant that the span that needed to be self-supporting was 4 m. Using typical stand-up time curves against RMR (Bieniawski, 1984) which plots roof span against stand-up time in hours, a rock mass rating (RMR) of 40 or greater would be needed. This meant that the tunnel would be self supporting for the 32 hours required for the concrete to gain enough strength to provide support, assuming 4 hours per excavation step with 50% contingency. If the RMR was found to be less than 40 spiles should be inserted to add support to the excavation. For the 1.5 m and 2.0 m heading excavation steps, to achieve the equivalent safety factor, a rock mass rating (RMR) of 45 and 50, respectively, or greater, was required for these advance lengths.

6.2 *Design*

The primary lining for the main bores and cross passages was permanent 32 MPa steel Fibre Reinforced Shotcrete and was designed to resist all ground loading. The approach to the design of the primary lining is summarised as follows:

- Primary lining was designed structurally as plain concrete
- No flexural capacity of steel fibres was used in the design

- The selected shape of the primary lining results in no tension in the lining (bending moments are resisted by axial forces in the lining)
- Areas of the lining with very low axial loads utilise the allowable tensile capacity of the concrete in accordance with Eurocode 2 to resist bending moments.

Fibres are included in the sprayed concrete due to the selected excavation and ground support technique. In particular fibre reinforcement allows the primary lining to be installed safely whereas the installation of steel mesh would require personnel to stand and operate under unsupported ground as well as introducing implications relating to manual handling regulations. The fibre reinforcement was used for improved constructability of the SCL lining by improving the initial behaviour of the shotcrete and preventing slumping or tearing of the material as it was placed ensuring a homogeneous lining was achieved.

Whilst the flexural capacity of the steel fibres was not be used in the design, RILEM TC 162-TDF: 'Test and design methods for steel fibre reinforced concrete' was required for design of the main bore primary lining sections, to provide suitable partial factors applied to the compressive stress block of fibre reinforced sprayed concrete. The proposed partial factor outlined below is the same as for reinforced concrete to reflect the increased ductility provided by fibre reinforced sprayed concrete when compared to plain concrete. The design compressive strength of the fibre reinforced concrete ($f_{cd\text{-}sfr}$) was derived in accordance with the equation in Clause 3.1.6 (1) of Eurocode 2 as below:

$$f_{cd\text{-}sfr} = \alpha_{cc} f_{ck\text{-}sfr} / \gamma_c \tag{5}$$

where α_{cc} = coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied (a value of 0.85 was used in accordance with the UK National Annex); $f_{ck\text{-}sfr}$ = characteristic compressive cylinder strength at 28 days; γ_c = partial safety factor for concrete, taken from Table 2.1 (1.5 for Persistent and Transient loading in accordance with the UK National Annex).

For all permanent loads, the primary lining will experience no tension. With the section fully under hoop compression the tensile capacity of the fibre reinforced sprayed concrete is not used in the design. The lining design was undertaken at the Ultimate Limit State (ULS) whereby the design value of the load in the lining had a load factor of 1.35 applied, as given in BS EN 1990 Eurocode—Basis of Structural Design.

7 DESIGN INNOVATION—PERMANENT PRIMARY LINING DESIGN

The principal innovation with the support measures is the design of the primary lining as permanent. This was possible due to a number of advances in tunnelling technology in recent years. Firstly, 'alkali free' set accelerators are now available with no loss in shotcrete strength with time. A recent innovation is the use of 3-D scanning survey equipment that provides excellent shape control for both excavation and spraying, and allows shotcrete lined tunnels to be constructed without lattice girders. This technique has been recently used successfully for the Heathrow T5 project (Williams et al, 2004).

Historically the inclusion of lattice girders meant the primary lining had to be considered temporary due to the corrosion potential of the steel lattice girder within the primary lining. Spiling is envisioned in several locations due to adverse soil layers. This will be carried out with self-drilling Glass Reinforced Plastic (GRP) dowels, again with no adverse durability issues. Some of the key aspects of a permanent primary lining relate to construction techniques and workmanship, such as the shotcrete mix design and accelerator selection and use of robotic spraying equipment.

The ECI project delivery process allowed the designer to work hand in hand with the contractor during the design development phase, thereby ensuring all the necessary issues were addressed.

8 SHOTCRETE MIX SPECIFICATION

In order to achieve a durable shotcrete mix suitable for use as a permanent lining the specification included requirements for the base mix concrete, shown in Table 1, as well as specific fibre reinforced shotcrete requirements, included

Table 1. Base mix design requirements.

Requirement	Unit	Specified value
Intended working life of structure	yrs	120
Compressive strength (f'_c)	MPa	32
Minimum cementitious content	Kg/m^3	360
Maximum w/c ratio	–	0.45
Maximum aggregate size	mm	14
Chloride content class	–	Cl, 0.3
Maximum cementitious content	Kg/m^3	450
Maximum temp. of fresh concrete	°C	20
Water penetration to BS EN12390-8	mm	50

in Table 2. The other critical element included in the specification was the requirement for robotic spraying of the shotcrete. The durability of the base mix was assured through the specification of a maximum w/c ratio of 0.45 along with a water penetration requirement of less than 50 mm.

The performance specification of the shotcrete was not particularly onerous as the design approach did not utilise the flexural toughness of the steel fibre reinforced shotcrete.

The tests specified for pre-production mix confirmation as well as production control are included in Table 3. The aggregate grading requirements are shown in Figure 7.

Table 2. Fibre reinforced shotcrete requirements.

Structural performance requirement	Time	Specified value
Compressive strength	1 hr	Upper J2
	8–16 hrs	Upper J2
	24 hrs	10 MPa
	7 days	27 MPa
	28 days	32 MPa
Energy absorbtion (Plate test to EN14488-5)	28 days	700 J
Drying shrinkage to ASTM C157	28 days	<0.03%
Maximum w/c ratio	–	0.45
Maximum aggregate size	mm	14
Chloride content class	–	Cl, 0.3
Maximum cementitious content	Kg/m^3	450
Maximum temp. of fresh concrete	°C	20

Table 3. Shotcrete testing requirements.

Requirement	Pre-production testing	Production testing frequency
Compressive strength	Yes	Every advance (1, 8, and 24 hr)
		1 set/100 m^3 (28 and 90 days)
Water penetration	Yes	1 set/month
Homegenity	Yes	1 test/week
Energy absorption	Yes	1 set/month
Flexural strength & toughness	Yes	–
Shrinkage	Yes	–
Density		1 set/100 m^3
Thickness		5 /shift

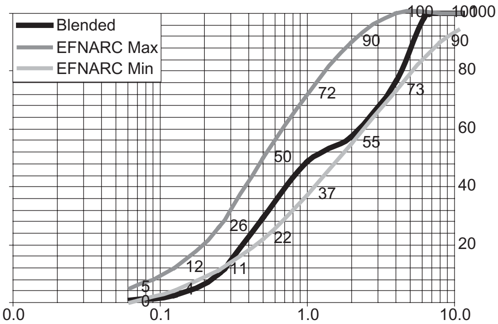

Figure 7. Aggregate grading curve.

9 CONSTRUCTION

The sprayed concrete lining was sprayed robotically using a BASF Meyco Potenza chassis with Logica laser scanning systems. The maximum throughput was 25 m^3 per hour.

The sprayed profile was very accurate. An as-built full laser survey of the tunnels was completed and the shotcrete profile was generally within +/–50 mm of design. The profile control was carried out using reflectorless measurement of the shotcrete when sprayed. The profile was excavated 100 mm bigger than design and the target was to spray 50 mm bigger than design. This was largely achieved although there was some overbreak in areas of instability. The following sections describe the details of the construction.

9.1 *Shotcrete mix trials*

Trials were carried out onsite over the winter of 2007/08 to verify that the sprayed concrete met the specification requirements prior to being placed in the permanent works. These trials consisted of spraying panels (both vertical and overhead) to observe strength gain, 28 day strength and workability. Concurrently with these tests, samples were sent away for permeability and flexural strength testing.

The conditions during the testing were onerous as low ambient temperatures were experienced. This lead to some challenges in obtaining the required early age strength. An oil-burning water heater was used to raise the water temp at the batcher to around 50ºC. This resulted in a batched shotcrete temperature of around 20ºC. This was instrumental in achieving the J2 early age strength curve and this temperature was achieved, even when batching with ambient temperatures below zero Once inside the tunnel, the consistently higher ambient temperatures helped to achieve the required strengths at early age. There were no problems in achieving the required long term strength. There were no problems in attaining the permeability requirements (tested using the depth of penetration under water pressure method). Energy absorption testing was to EN14488-5 (former EFNARC panel test). Panels were sent away for testing and the optimum fibre dosage selected based on the results. The shotcrete mix design is shown in Table 4 below, with the aggregate grading curve included in Figure 7.

The shotcrete mix initially included 40 kg/m^3 steel fibres but was reduced to 30 kg/m^3. Alternative steel fibres at 35 kg/m^3 were also approved. Additional trials proved that macro-synthetic polyproplyene fibres at 7 kg/m^3 met the performance specification requirements and subsequent testing allowed a reduction to 6 kg/m^3. Alternative macro-synthetic polypropylene fibres were also approved at 6 kg. The macro-synthetic polypropylene fibres were 55 mm long and were sourced from various suppliers (Bekaert, Barchip and Propex). The shotcrete mixes were easier to apply with steel fibres but the supply proved unreliable and more expensive than polypropylene.

9.2 *Progress rates*

Tunnelling was undertaken concurrently in each bore from the northern portal on a 24-hour basis. Tunnelling was undertaken from the southern portal on day shift only, with crews alternating between bores for the canopy tube installation, and the excavation and spraying. Tunnelling commenced in February 2008, and both headings broke through on 26 February 2009. Peak combined progress on all faces exceeded 100 m per week. Table 5 includes the sprayed concrete lining production rates achieved.

9.3 *Survey control*

Excavation profile during excavation was maintained by reflectorless measurement from total stations. Target arrays were installed in the primary lining at 25 m centres and surveyed back to control stations outside the tunnel portal. These target arrays were used to position the total stations. Primary lining profile control was also carried out using reflectorless measurement from total stations. The initial layers were placed using the previous lining as a guide, with the total station being brought in for guidance of the final passes. It is worth noting that whilst the reflectorless measurement technique allows points to be measured close to the face, it is susceptible to dust and overspray (causing apparent crown level drops) and blocking by plant/equipment.

Table 4. Shotcrete mix design.

Component	Quantity/m^3 or product
OPC cement	450 kg
Microsilica	40 L
Coarse aggregate 2/6	413 kg
Fine aggregate	1240 kg
Water	0.38 w/c
Superplasticiser	Glenium 51
Stabiliser	Delcocrete
Accelerator	Meyco SA 160
Fibres	30 kg steel fibres or 6 kg structural fibres

Table 5. Sprayed concrete production rates.

Production rates	South (m)	North (m)	Comb. total (m)
Average heading/day	1.3	4.3	14
Best day	4.0	16.2	20
Best week	20.1	95.8	115
Best month (Heading)			457
Best month (Bench)			1232

Balfour Beatty has been working with suppliers to investigate the use of laser-scanning guidance systems which can be linked to the both the excavator and sprayed concrete robot. These would allow control of both the excavated and sprayed profiles without the need for personnel monitoring the profile in close proximity to the face. Currently, it is not clear that the systems provide additional accuracy although there are safety benefits associated with removing personnel from the tunnel.

The use of laser scanning for control of the internal profile of the sprayed concrete primary lining was not successful, and was profile was achieved by the separate survey system described above. Where a constant thickness is required (eg. secondary lining), the laser scanning system appears more capable although variations in concrete flow can cause variations in thickness due to inherent assumptions made by the machine about concrete flow rates.

9.4 *Construction issues*

The key advantage of the sprayed concrete support design is the degree of inherent flexibility it affords. This flexibility was increased by the development of certain structural items not originally envisaged in the design. These include temporary cross passages, the ability to excavate almost all of the northbound tunnel heading-only, turning bays at full depth constructed using the bench profile and a full-closed heading developed for the sand encountered at the south end. Each of these designs were produced by adapting the flexible base design and so were developed using minimal analysis whilst maintaining a safe and optimised design. The design elements were able to be installed as required by the contractor, allowing flexibility of programme onsite.

The design was optimised with the level of ground support provided being adjusted depending on ground conditions encountered at each face. Whilst this requires careful monitoring and control, the benefits were great (tunnelling with optimised support from the north portal was around four times faster than that from the south which used full support). As the site team developed a greater knowledge of the ground and tunnel behaviour, increased optimisation of the support level was possible. This included the use and type of spiles (32 mm solid GRP spiles installed into pre-drilled holes were the preferred option), the type of face support (an inclined face was the preferred option) and the installation of an invert strut.

10 CONCLUSIONS

In the 10 years since the previous bored highway tunnel was designed and constructed in the UK, there have been several advances in tunnelling technology. The A3 Hindhead project has adopted several new approaches and details, in order to minimise both the project risk and cost including the first use of fibre reinforced shotcrete as a permanent lining in the United Kingdom. This has been possible due to the:

- Availability of 'alkali free' set accelerators,
- Use of 3-D laser scanning survey technology for control of robotic spraying equipment to achieve tight shape tolerances avoiding the need to use lattice girders,
- Use of self-drilling GRP spiles in areas of poor ground.

The analysis and design of the lining included 2-D analysis as the primary design tool, as well as using 3-D analysis for the time-related effects. Design was undertaken for the Ultimate Limit State within the Eurocode 2 framework, although a revision to the material reduction factors was required to reflect the ductile nature of fibre reinforced shotcrete. Although the durability of the lining structure is greatly enhanced by avoiding the inclusion of steel reinforcement, a durable shotcrete mix was assured through the specification of a low permeability concrete with a low water: cement ratio.

Construction proved to be very successful, with consistent high quality shotcrete provided at a rapid rate by the shotcrete robots. A combined production rate of over 100 m per week indicates that the method is very economical where multiple headings can be excavated concurrently.

Whilst sprayed concrete linings have been used typically in soft ground, this project has proved that this approach can be very economical for soft rock conditions. This lining was installed using NATM principles such that the majority of the load was taken by the ground, resulting in a relatively thin 200 mm thick lining. Where difficulties exist in developing tensile loads in bolted support systems due to low bond stress values, or in highly jointed rock masses, then SCL linings provide an economical but safe tunnel design alternative. These economics improve significantly when the primary lining is designed as a permanent lining. These conclusions are directly applicable to the soft bedrock material that exists in Auckland, the East Coast Bays Formation (ECBF).

REFERENCES

Bieniawski, Z.T. 1984. Rock Mechanics *Design in Mining and Tunnelling*. Rotterdam: A.A.Balkema.

Chang & Stille. 1993. Influence of early-age properties of shotcrete on tunnel construction sequences. *Shotcrete for Underground Support VI—Proceedings of Engineering Foundation Conference.*

John, M. & Mattle, B. 2003. Shotcrete Lining Design: Factors of Influence. *Rapid Excavation and Tunnelling Conference*, pp. 726–734.

Marinos, P.G. & Hoek, E. 2000. GSI: A geologically friendly tool for rock mass strength estimation. *Proceedings of the International Conference on Geotechnical and Geological Engineering (GeoEng 2000),* Technomic Publishing Co. Inc., pp. 1422–1440, Melbourne, Australia.

Osterreichischer Betonverein. 1999. *Sprayed Concrete Guideline—Application and Testing.* Nelkengasse. Ing. Rudolf Bankl.

Williams, I., Neumann, C., Jäger, J. & Falkner, L. 2004. Innovativer Spritzbeton-Tunelbau für den neuen Flughafenterminal T5 in London. In Proc. *Österreichisher Tunneltag* 2004, Austrian Committee of the ITA, Salzburg, pp. 41–62. Salzburg: Die SIGN Factory.