Design and construction of cast in-situ steel fibre reinforced concrete headrace tunnels for the Neelum Jhelum Hydroelectric project

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ABSTRACT: Twin 10 km long parallel headrace tunnels were excavated as part of the headrace tunnel system for the Neelum Jhelum Hydroelectric Project, using two Tunnel Boring Machines (TBMs). The permanent support normally comprised a shotcrete lining applied over initial support elements. However, concrete lining was required in certain areas of extremely poor ground conditions. Once placement of conventionally reinforced concrete commenced, it was found to be taking longer than anticipated, prompting a search for alternative solutions. Steel Fibre Reinforced Concrete (SFRC) proved to be the most viable option for accelerating the concrete lining programme. This paper briefly outlines the requirement for a conventionally reinforced concrete lining versus an SFRC lining, the design basis, and the actual design itself. A comparison of the costs and durations to install the two types of lining is also presented.

1 PROJECT DESCRIPTION AND OVERVIEW

The Neelum Jhelum hydroelectric project is located in the Muzaffarabad district of Azad Jammu & Kashmir (AJK), in northeastern Pakistan. Geographically, the area consists of rugged terrain between 500 and 3,200 m in elevation within the Himalayan foothill zone known as the Sub-Himalayan Range.

The project is a run-of-river one, employing 28.6 km long headrace and 3.6 km long tailrace tunnels to cut off a major loop in the river system, transferring the waters of the Neelum River into the Jhelum River, for a total head gain of 420 m (Figure 1). The headrace tunnels comprise both twin (69 %) and single (31 %) tunnels, while the tailrace tunnel consists of a single tunnel. Design capacity of the waterway system is 283 cumecs.

The project, which was completed in 2018, has an installed capacity of 969 MW, generated by four Francis-type turbines located in an underground powerhouse.

At commencement of construction in 2008, all tunnels were to be excavated using conventional drill & blast techniques. However, it soon became apparent that with the equipment being employed, a 13.5 km long section of the headrace twin tunnels underlain by high terrain that precludes construction of additional access adits, would take too long to excavate.

Consequently, the construction contract was amended to allow the operation of two 8.5 m diameter open gripper hard rock TBMs to each excavate some 10 km of the twin headrace tunnels (Figure 1), with an initial centre-tocentre lateral spacing of 33 m, later increased to 55.5 m.

The tunnel excavation diameter was 8.5 m diameter giving a total face area of 56.75 m². Excavation direction was upstream to promote drainage, with a typical gradient of 0.8 %.
The open gripper design was selected to give the most flexibility for the expected conditions – possible squeezing ground given the relatively weak rock mass and overburden up to 1 870 m, and the potential for rockbursts in the stronger beds. Excavation of the TBM tunnels commenced in January 2013 and was completed in May 2017.

The initial design for the entire project had employed a shotcrete lining throughout, but with short sections of full concrete lining in zones of poor ground, estimated during the feasibility study to add up to about 10 % of the total length.

However, shortly after commencement of the project, the change was made to a full concrete lining for all headrace tunnels, with the exception of the TBM portion, which retained a shotcrete lining for a number of reasons. Specifically, it was judged that the TBM excavation method had several advantages over the Drill & Blast method that resulted in a significantly less-disturbed rock mass, a much smoother tunnel profile, a circular rather than horseshoe shape, and the ability to spray a much more uniform shotcrete layer.

Nevertheless, it was recognized that some sections of concrete lining placed over the shotcrete lining would be required in zones of poor ground, albeit with a shorter aggregate length than the original 10 % estimate, judging by conditions in the early tunnels. It must also be stated that the importance of such local reinforcements had been unavoidably highlighted by the issues encountered on the Glendoe project in Scotland a few years earlier.

It is these sections of concrete lining, which of necessity had to be mostly completed before tunnel excavation had finished, that are the subject of this paper.

1.1 Geological setting

The entire project was excavated in the molasse-type sedimentary rocks of the Murree Formation, which is of Eocene to Miocene age. The succession comprises intercalated beds of
sandstone, siltstone and mudstone that have been tightly folded and tectonized, with generally steep bedding dips and a northwesterly regional bedding strike, rarely far from perpendicular to the tunnel azimuth. Weakness zones and local faults were commonly observed, and were invariably oriented parallel to the regional bedding strike.

1.2 TBM configuration and rock support installation

The two TBMs were conventional in their layout, and were based on the successful Gotthard Base TBMs. Nearly all rock support elements, including rock bolts, mesh, channel sections and TH ring beams, but with shotcrete application limited to the maximum extent possible, were installed in the so-called L1 zone immediately behind the shield.

The shotcreted tunnel invert was installed between the L1 and L2 zones, while the majority of shotcrete was sprayed in the L2 zone, some 60 m behind the face, using robots installed outside a cylindrical shield that kept workers and equipment free of overspray and rebound.

2 TBM EXCAVATION

2.1 Progress

Both TBMs started headrace tunnel excavation in early 2013 with completion by the first TBM in October 2016 and the second TBM in May 2017. The lengths of the left and right tunnels, respectively (looking downstream), were 10.428 km and 9.893 km, giving an average daily excavation rate of 8.02 m and 6.37 m. For simplicity, the chainages used in this report start at zero where TBM excavation commenced, and increase in the direction of advance. (In practice, construction records reflect actual chainages, which decreased with upstream advance, and which took into account a section of drill & blast tunnel upstream.) Over most of the excavation programme, the left TBM was generally the lead TBM.

2.2 Encountered ground conditions

Overall, encountered ground conditions were better than expected, in that the squeezing conditions anticipated in the weaker mudrocks were never encountered, despite an overburden of up to 1 870 m. The most likely explanation is that the closely intercalated nature of weak and strong lithologies meant that there was always a ‘skeleton’ of stronger sandstones and siltstones that provided support for the excavation. Also of benefit to the excavation was the almost complete absence of groundwater ingress.

Furthermore, with the exception of a single major fault, encountered some 1.5 km into the drives, and described below, no other major faults were encountered. Small-scale faulting was common, as were shear zones along beds of weak mudstone, but the strike of most of these features followed the regional bedding strike, which the alignment usually beneficially intersected close to perpendicularly (with the notable exception of the location of the largest rockburst).

On the negative side, however, the incidence of rockbursts was significantly higher than had been anticipated, primarily it is thought because of the existence of elevated horizontal stresses that were unanticipated. It is these rockbursts that primarily contributed to the overall low daily production rates. The left tunnel experienced 937 documented rockbursts, ranging from small to major violent events, while the (usually trailing) right tunnel experienced 590 rockbursts. The zone of the most intense rockbursts persisted for approximately 3 km, before diminishing relatively abruptly.

The two longest sections that required concrete lining were the major fault encountered in both drives, and the location of the largest rockburst experienced on the project, which was named ‘5/31’ after the day on which it occurred, May 31st, 2015. Both features, which are described in more detail below, necessitated construction of concrete lining in both tunnels for over 120 m.
The junction with an access adit, A2, required 48 m of concrete lining in the left tunnel to ensure stability and improve hydraulics. Additional, shorter sections of concrete lining were required in the left tunnel only. Two of these sections were in sheared mudstone that was judged to require additional support, and one was in an area of badly delaminated shotcrete lining. The sections of concrete lining that were placed in the TBM tunnels are shown in Table 1. They amount to 612 m of tunnel, or 3.0%, significantly lower than the approximately 10% estimated during the feasibility study.

2.3 Encountered geological features requiring extensive concrete lining

Two geological features required concrete lining in both tunnels in excess of 100 m length. The first is a 95-110 m wide zone of highly sheared mudstone that unusually was associated with groundwater inflows of up to 10 L/min. During the initial encounter by the left TBM, a cavity formed above and ahead of the cutterhead, and the TBM subsequently became jammed at Ch. 1+430 m by the pressure of the collapsed fault gouge on the shield. Stabilizing the excavation and freeing the TBM required installation of a pipe roof canopy, followed by excavation of a top-heading above the cutterhead, and extensive chemical and cement grouting to consolidate the ground ahead of the TBM.

The second feature was the 5/31 rockburst that occurred in the right TBM tunnel, which at the time was trailing the left TBM by 180 m, separated by a 24.5 m wide pillar. The event had a calculated energy release equivalent to a Richter magnitude 2.4 earthquake, causing extensive damage to the TBM, ancillary equipment and rock support over a 60 m section of tunnel as well as significant damage to the tunnel lining of the already-excavated neighbouring TBM tunnel. It disabled the TBM for 7.5 months.

2.4 Requirement for rapid construction of concrete lining

Given the obstacles posed by everyday rockbursts, not to mention the catastrophic 5/31 event, it is perhaps not surprising that the tunnel excavation programme slipped significantly behind schedule, and that as the commissioning date approached, it had become one of the project structures (but by no means the only one) on the critical path.

To meet project deadlines, as much of the concrete lining as possible had to be placed within each tunnel while the TBMs were still operational. Failing that, remaining installation had to proceed after completion of the excavation, but while parts of the TBMs were being removed and other tunnel finishing works, such as grouting, were being completed.

Throughout the lining installation, full tunnel logistical access by way of the rail track had to be maintained. These factors significantly influenced the methodology selected. Principally, the solution was to install the additional concrete lining in two stages. Stage one consisted of the installation of the upper 270° of the concrete lining, while maintaining full tunnel access

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Chainages</th>
<th>Length (m)</th>
<th>Lining Type</th>
<th>Reason for Lining</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left</td>
<td>6+022 to 5+854</td>
<td>168</td>
<td>Conventional RC</td>
<td>5/31 rockburst</td>
</tr>
<tr>
<td>Right</td>
<td>5+818 to 5+698</td>
<td>120</td>
<td>Conventional RC</td>
<td></td>
</tr>
<tr>
<td>Left</td>
<td>1+649 to 1+553</td>
<td>96</td>
<td>Conventional RC</td>
<td>Major Fault</td>
</tr>
<tr>
<td>Right</td>
<td>1+512 to 1+404</td>
<td>108</td>
<td>Conventional RC</td>
<td></td>
</tr>
<tr>
<td>Left (U/S)</td>
<td>1+710 to 1+698</td>
<td>12</td>
<td>SFRC</td>
<td>Adit A2 Hydraulic Improvements</td>
</tr>
<tr>
<td>Left (D/S)</td>
<td>1+698 to 1+662</td>
<td>36</td>
<td>SFRC</td>
<td></td>
</tr>
<tr>
<td>Left</td>
<td>1+327 to 1+303</td>
<td>24</td>
<td>SFRC</td>
<td>Sheared mudstone 1</td>
</tr>
<tr>
<td>Left</td>
<td>1+188 to 1+164</td>
<td>24</td>
<td>SFRC</td>
<td>Sheared mudstone 2</td>
</tr>
<tr>
<td>Left</td>
<td>0+466 to 0+442</td>
<td>24</td>
<td>SFRC</td>
<td>Delaminated shotcrete</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>612</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
for other tunnel related work. Stage two consisted of the installation of the lower 90°, after the tunnel rail track had been removed and was no longer required.

2.5 Locations of additional concrete lining

Concrete lining works commenced at the upstream end of the tunnel and worked backwards downstream. However, it gradually became apparent that the logistical complexity of the operation, with the various concrete lining sections distributed in discrete zones often hundreds of metres apart, all the while maintaining rail access, was delaying completion more than anticipated. One way of streamlining the process was to prioritise construction of the remaining lining sections further downstream initially. This became the driver for assessing and developing alternative concrete lining methodologies.

A major task in conventionally reinforced concrete lining construction involves the careful installation of reinforcing bars at their required locations. It was realized that considerable time could be saved by the implementation of a SFRC lining, and this technique rapidly became the preferred acceleration option.

A comprehensive design review was initiated for the remaining concrete lining locations, and the computations showed that SFRC met all the required specifications, so much so, that its use for most of the required concrete lining sections would have been possible from the start.

The design methodology used for the SFRC lining is presented in detail below.

3 DESIGN

3.1 Introduction

Although the majority of the reinforced concrete tunnel linings on the Neelum Jhelum project had been done so with conventional steel bars, steel fibres were considered as a suitable alternative for certain sections within the TBM tunnels since:

![Cross-section of concrete lining](image-url)

Figure 2. Cross-section of concrete lining.
The lining shape was circular (as opposed to the horseshoe shape of the portion of the tunnels excavated by Drill & Blast)

- High compressive axial forces would act on the lining with low bending moments
- Steel fibres work to prevent the formation and widening of cracks

3.2 **Geometry**

As previously mentioned, the excavated diameter of the TBM tunnels was 8.53 m. Due to the severity of some of the ground conditions encountered, large quantities of initial support had been installed in sections where a permanent concrete lining was required, e.g. heavy steel ring beams at 700 mm centres longitudinally along the tunnel and thick layers of shotcrete. A generous allowance of 350 mm had therefore been stipulated for the initial support and any convergence. The minimum design concrete lining thickness was specified as 350 mm. This thickness was a compromise between finding a constructible solution that achieved the design intent, whilst reducing the cross-sectional area of the waterway as little as possible in order to minimize the head loss and thereby the hydraulic penalty.

3.3 **Design loads and tunnel design cases**

The design loads and tunnel design cases applied in the design are summarised in Table 2.

<table>
<thead>
<tr>
<th>Tunnel Design Cases</th>
<th>Construction Case</th>
<th>Filling Case</th>
<th>Operational Static Case</th>
<th>Transient Water Hammer Case</th>
<th>Dewatering Case</th>
<th>Faulted Ground</th>
<th>Special Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>1.1</td>
<td>1.1</td>
<td>1.3</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>Contact Grouting Pressure</td>
<td>1.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>External Water Pressure</td>
<td>1.4</td>
<td>1.4</td>
<td></td>
<td></td>
<td>1.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transient Water Pressure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.1</td>
<td></td>
<td>1.1</td>
</tr>
<tr>
<td>Wedge Failure Of Rock</td>
<td>1.2</td>
<td>1.4</td>
<td>1.4</td>
<td></td>
<td>1.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Faulted Ground</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
</tr>
</tbody>
</table>

3.4 **Structural analysis of the lining and design assumptions**

Elastic continuum, closed form analysis was used to determine the stresses acting on the lining. The analysis is based on excavation and lining of a hole in a stressed isotropic and homogeneous elastic medium.

Once bending moment and ring thrust in a lining have been determined, or a lining distortion estimated based on rock-structure interaction, the lining must be designed to achieve acceptable performance. Since the lining is subjected to combined normal force and bending, the analysis is carried out using the moment-axial force capacity curve, (U.S. Army Corps of Engineers, 1997).

Due to local areas of overbreak or variations in the convergence and thickness of initial support, the concrete lining may in reality deviate slightly from the actual design value. As well as obviously affecting the centroid radius value of the tunnel, the thickness of the lining affects the second moment of area, \( I \), of the lining. The second moment of area is also affected by the reduced stiffness effect of having joints in the lining (the upper 270° is placed first and then the lower 90° is cast below the advancing formwork). In this analysis, as well as calculating the standard value of \( I \) based on the thickness of the lining, a sensitivity analysis with
different scenarios was also performed where upper and lower bound values were calculated by taking a percentage variation of the ‘standard’ value.

3.5 Design of conventionally reinforced and fibre reinforced lining

3.5.1 Material properties

The material properties employed in the design are presented in Table 3:

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Specified compressive strength (MPa)</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>Modulus of elasticity of concrete (GPa)</td>
<td>25.74</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio</td>
<td>0.15</td>
</tr>
<tr>
<td>Steel reinforcement bars</td>
<td>Specified yield strength (MPa)</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td>Tensile strength of reinforcement (MPa)</td>
<td>620</td>
</tr>
<tr>
<td></td>
<td>Elastic Modulus of steel (GPa)</td>
<td>200</td>
</tr>
<tr>
<td>Steel fibres</td>
<td>Fibre length (mm)</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Fibre diameter (mm)</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Aspect ratio l/d</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>Tensile strength (MPa)</td>
<td>1345</td>
</tr>
<tr>
<td></td>
<td>Young’s modulus (GPa)</td>
<td>210</td>
</tr>
<tr>
<td></td>
<td>Minimum Dosage (kg/m³)</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>CMOD 0.5mm, fR1,m (MPa)</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>CMOD 1.5mm, fR2,m (MPa)</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>CMOD 2.5mm, fR3,m (MPa)</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>CMOD 3.5mm, fR4,m (MPa)</td>
<td>3.2</td>
</tr>
</tbody>
</table>

3.5.2 Serviceability Limit State (SLS) design

3.5.2.1 SERVICEABILITY LIMIT STATE (SLS) DESIGN FOR CONVENTIONALLY REINFORCED LINING

The minimum required cover according to the applicable standards was 75 mm for the structure. #8 (25 mm diameter) and #6 (19 mm diameter) reinforcement bars were specified at centres of 175 mm in the circumferential and longitudinal directions respectively. This reinforcement configuration satisfied the minimum reinforcement requirements and other durability considerations stipulated in the relevant standards such as:

- Minimum reinforcement
- Requirements for flexural crack control
- Requirements for temperature and shrinkage reinforcement

3.5.2.2 SERVICEABILITY LIMIT STATE (SLS) DESIGN FOR STEEL FIBRE REINFORCED LINING

Crack control is one of the main benefits provided by steel fibres to structural elements. If these cracks do not exceed a certain width, they are neither harmful to a structure nor to its serviceability. The limitation of crack width means that steel fibres provide a post crack strength to the concrete.

Fibres may have been originally introduced for strengthening of the matrix, without distinguishing the difference between material strength and material toughness. (Toughness is used for describing the post-peak response of structural members that quantifies the energy absorption characteristics.) The most significant effect of fibre addition to the brittle cementitious matrix is the enhancement of toughness.
One of the greatest benefits to be gained by using steel fibre reinforcement is improved long-term serviceability of a structure. SFRC in a tunnel lining offers: a) Excellent ductility, b) Reduction in the shrinkage of concrete, c) Elimination of mistakes in conventional reinforcing, d) Shorter construction periods compared to traditional ones, e) Increased tensile and flexure strengths that are equal in all directions, f) Easy crack control and high absorbed energy after matrix failure.

The distance between steel fibres is much smaller than typical spacing between traditional bars. Unlike reinforced concrete, fibres are distributed throughout the whole section. Hence, there is no concrete cover without reinforcement. Furthermore, stresses in the root of a crack can be picked up more quickly. This is why crack propagation and crack patterns change when compared to plain or even reinforced concrete (Vitt, 2005).

3.5.3 Ultimate Limit State (ULS) design
The ULS axial loads and bending moments were determined for each type of scenario for each tunnel design case. The moment – axial force capacity curve for the conventionally reinforced section was plotted assuming that there was an equivalent area of circumferential reinforcing steel of 2805 mm²/m in each face of a column under combined axial load and bending. The moment – axial force capacity curves for the SFRC lining were generated using both the Rilem Method and the method detailed in Appendix A of ACI 544.7 These curves are plotted in Figure 3. The singular points on the diagram are the results from the elastic continuum analysis for different scenarios for each of the tunnel design cases listed in Table 2. Figure 3 shows that the points fall within the capacity curves.

The shear was checked using the equation: \[ V_{\text{max}} = \frac{(M_{\text{max}} - M_{\text{min}})}{R} \]

The shear capacity of the concrete alone (without separate shear reinforcement or taking the beneficial effect of the fibres into account) was found to be easily adequate, even for the maximum factored shear force of 58 kN.

4 DURABILITY CONSIDERATIONS WHEN USING SFRC
SFRC is often used for concrete structures subject to severe exposure conditions e.g. bored tunnels exposed to saline ground water containing high levels of sulphates (Edvardsen, 2018). Two phenomena need to be examined when analyzing the durability of SFRC:
• The reinforcement has to provide good “tightness” against infiltration of water by controlling in situ crack opening.
• Corrosion of the reinforcement should not cause a notable reduction in the bearing capacity of the lining.

Two different configurations must be taken into account when analyzing the corrosion of steel fibres and its consequences:
1. The fibre does not cross a crack emerging on the surface
2. The fibre crosses a fracture crack on the surface

In the first case, apart from some stains which affect the appearance of the structures, corrosion of the fibres does not lead to any serious problems for the durability or the bearing capacity of these structures in SFRC.

In the second case, the bearing capacity of the SFRC is not reduced significantly with crack openings of 250 μm or less.

The corrosion resistance of SFRC is governed by the same factors that influence the corrosion resistance of conventionally reinforced concrete. Processes such as carbonation, penetration of chloride ions and sulphate attack are in direct proportion to the permeability of the cement matrix.

As long as the matrix retains its inherent alkalinity and remains intact, deterioration of SFRC is not likely to occur. It has been found that good quality SFRC, when exposed to conditions conducive to reduced alkalinity, will only carbonate to a depth of a couple of millimeters over a period of many years (Kern & Schorn 1991, Hannant & Edgington 1975).

5 COMPARISON OF COSTS AND DURATIONS

5.1 Comparison of time and cost of different methodologies

The requirement to meet a completion date necessitated the implementation of a detailed project management system to record all aspects of the concrete lining. This data was recorded by both design and supervision teams, and the key points are presented in Table 4.

The sections numbered 1 to 4 refer to additional concrete lining sections constructed using concrete with steel reinforcement bars, or what may be considered the conventional methodology. Sections numbered 5 to 9 refer to the sections of lining constructed using SFRC.

The sections numbered 1, 2, 3, 7, 8 and 9 were installed whilst maintaining full logistical access for other tunnel works by way of the rail track. Sections 4, 5 and 6 were the last sections to be installed when there was no requirements to maintain tunnel access for other works. Furthermore, they were located directly adjacent to access Adit A2, affording uninterrupted access to surface facilities. (It should be noted that while section 4 was one of the last to be placed, it was one of the first to be designed and issued to the contractor, hence the use of conventionally reinforced concrete rather than SFRC.)

5.2 Comparison by time

Table 4 presents three columns relating to time required for lining installation. “Time per Section” provides the total time for the concreting works required for that section length. “Time per Lm” is the average time taken to install one linear metre of tunnel lining. “Delays during concrete placement” presents the extent of production delays for each section and is expressed as a percentage of the total time.

5.3 Findings from time data

The data from Table 4 shows that lining installation using conventionally reinforced concrete ranged from 43 to 175 minutes per linear metre. However, section 4, as mentioned previously,
enjoyed direct access to concrete trucks via Adit A2, resulting in a far more favourable result. Consequently, section 4 should be discounted, and the results from sections 1, 2 and 3 (i.e. 134 to 175 minutes per linear metre) are adopted as a more typical time per linear metre for conventionally reinforced concrete.

The associated time delays when using the conventionally reinforced concrete varied from 9% to 35%. However, once again the result from section 4 should be ignored, and the results from sections 1, 2 and 3 (i.e. 26 to 35%) should be regarded as more representative.

The data from Table 4 shows one outlier, at 173 minutes per linear metre for SFRC. This anomaly was a result of some of the batches of SFRC being rejected and extensive stoppages due to the blockages of pipes during placement and is therefore only shown here for the sake of completeness. Excluding this outlier (section No. 8), the time taken to install the SFRC lining ranges from 62 to 98 minutes per linear metre giving an average of 77 minutes per linear metre, offering a significant time saving over conventionally reinforced lining installation.

The associated typical time delays using the SFRC lining type range from 0 – 9% (discounting the outlier).

5.4 Findings from cost data

The details of the costs for each section are presented in the last column of Table 4. Information for sections 1 and 2 has been excluded and noted as ‘special cases’ because these sections required lining as a result of the 5/31 severe rockburst, which caused near-complete destruction of the existing shotcrete lining and surrounding strata. A far more extensive effort was therefore required to repair these two sections, making a comparison with other sections misleading.

The first section of concrete lining to be completed was section 3, associated with the major fault. This has been taken as the reference point for the costs analysis, i.e. this cost is expressed as 100% and all other costs are compared to this benchmark.

The data from Table 4 shows that the lining installed using conventionally reinforced concrete ranged from 82 to 100%.

The data from Table 4 shows that the lining installed using fibre reinforced concrete ranged from 33 to 41% of the reference cost of section number 3, a very significant saving.
6 CONCLUSIONS

In summary, the adoption of SFRC over conventional reinforcement proved to be a notable success. Not only did SFRC lining meet the same design criteria as a conventionally reinforced lining, it offered the following advantages:

- Saving cost over the actual quantity of steel employed
- Saving time by being quicker to install
- Producing a lining with smaller crack widths and improved durability over the life of the structure
- Producing a lining that is more efficient at resisting stresses due to groundwater loads and ground loads than conventionally reinforced concrete

REFERENCES

American Concrete Institute. 2016. 544.7R-16 Report on Design and Construction of Fiber-Reinforced Precast Concrete Tunnel Segments. Farmington Hills: American Concrete Institute.


