REINFORCEMENT SUPPORT OF CONCRETE SHAFT LINING IN 
WESTONARIA FORMATION LAVA IN THE EZULWINI VERTICAL 
SHAFT SYSTEM

W D Ortlepp* W C Joughin*, A K Ward† and J Thompson
* SRK Consulting and † Seismogen

Abstract

Monolithic concrete-lined circular vertical shafts have been the preferred form of access to the deep ore-bodies of the South African gold mines for more than fifty years. Without any steel reinforcement the linings, almost without exception, have proved adequate in keeping shafts operating satisfactorily throughout the working life of the mines.

The 1100m deep, 45 year old main shaft and its companion ventilation shaft at Ezulwini Mine (originally Western Areas Gold Mine) had both experienced some damage where they passed through the weak Westonaria Formation (WAF) horizon of the Venterdorp Lava Group, during the 1980’s. This damage had required extensive repair work which appeared to have been adequate until the mine closed in 1999.

In 2006, preparations for the extraction of the shaft pillar commenced and it was decided that the shaft should be rehabilitated to a fully-operational hoisting shaft with a planned life of 40 years. Shortly before, on a nearby mining operation, a sub-vertical shaft with a similar history of damage and repair was completely lost when weak lava from the same geological horizon broke through the concrete lining. The circumstances and timing of this event had a profound influence on the approach of the new owners toward the design and execution of the rehabilitation of the shaft.

This paper discusses the new design philosophy and describes, in some detail, the innovative support technology that was consequently developed, tested and implemented.

The main change in approach was to ensure that the reinforcing cable anchors and the containment cladding would resist long-term creep, survive any large displacement strains beyond the conventional elastic design limit and possess significant residual dynamic capacity to ensure that even a large seismic event could not cause significant damage to the shaft infrastructure.

1 Introduction

Ezulwini Mining Company is in the process of preparing for a shaft pillar extraction at a depth of between 900 m and 1000 m below surface. While the stress levels are not excessive, the presence of the weak Westonaria formation lavas (WAF) in the shaft barrel poses a severe hazard, which must be managed through support design and mining layout. The design of mining layout and sequence is critical to the success of the shaft pillar extraction, but does not form part of the discussion here. This paper focuses on the design of the reinforcement support of the shaft concrete lining.

The existence of the very weak WAF at the base of the Venterdorp Lavas has caused severe ground control problems where it has been previously intersected in tunnels and shafts on Ezulwini and on neighbouring mines. Significant damage has been experienced in the
concrete linings of vertical shafts that have intersected this horizon in Ezulwini shaft itself and in shafts on neighbouring properties.

The most dramatic example was that of South Deep’s SV1 sub-vertical shaft which was completely destroyed below the reef intersection by a sudden major collapse of the concrete lining and the weak rock behind. Since there had never been any stoping anywhere within 200m of the shaft and the mined-out span beyond was limited to about 800 m, it had to be concluded that the failure was driven largely by time-dependent ‘creep’ processes rather than by stress-induced fracturing. Significantly also, the collapse occurred suddenly during rehabilitation operations that might have removed some of the small (but possibly crucially-important) confinement afforded by the damaged portion of the concrete lining of the shaft.

The Ezulwini main shaft barrel itself had given cause for concern during its original service life and some moiling of the damaged concrete and support of the lining had been done. A certain number of cable anchors had also been installed. Because of lack of documentation it is difficult to determine exactly what was installed and it is not possible to determine their present condition. In the ventilation shaft a more visible and elaborate system of chains and wire ropes has been loosely installed across the WAF section. It is not possible to ascertain the condition of the ‘shepherds crook’ anchors which secure this network.

The fact that South Deep’s SV1 suffered a disastrous failure with a mining-induced stress change of nearly zero, suggested that a conventional approach to the present Ezulwini problem would be totally inappropriate. Because of this failure and the indisputable weakness of the WAF formation together with its erratic, unpredictable distribution, a particularly strict and conservative engineering approach would be adopted.

Since there was no nearby mining and therefore no mining-induced stress, the classical engineering design approach of matching the strength capacity to the anticipated load demand and moderating this with an assumed factor of safety, is considered to be not appropriate here. While it is relatively simple to estimate the capacity of the steel support components that will be installed, it is not possible to make a reliable estimate of the load or stress demand.

The rock mass ‘strength’ of the weakest portions of the WAF can only be estimated. While the theoretical stress changes due to the planned shaft pillar extraction can be determined by computer modelling, there is no way of reliably determining the effects that the stress change would have on the surrounding rock mass. In particular the volume of WAF that might fail and be mobilized to squeeze or flow cannot be calculated with any degree of confidence.

Also, the possibility that fault-related seismic events of significant magnitude might occur during the mining of the shaft pillar, cannot be excluded.

To overcome the uncertainty, it is accepted, as a basic imperative, that the importance of the shaft is so crucial that the over-riding principle should be to install as much capacity as practically possible, in as dense a pattern of holes as can be drilled economically. Overall effectiveness rather than optimum efficiency or initial cost economy should be the target. Factor of safety is not a meaningful or useful concept in the context of ensuring the long term operational stability of the Ezulwini shafts.
The Ventersdorp contact Reef (VCR) and Upper Elsburg conglomerates, which contain the gold bearing strata, are overlain by the Ventersdorp lava, which comprises WAF and the Alberton lavas. The WAF is the most problematic material within the shaft pillar and was intersected by the two shafts between 1030 m and 1060 m below surface (a few metres above 38 level). It is texturally, mineralogically and geochemically different from the Alberton formation, which typically overlies it. The Alberton formation is typically very strong (>200 MPa), with few joints, forming a very competent rock mass. The WAF, which is invariably weaker (<100 MPa) and often contains very weak zones, is usually immediately above the VCR. A study on the mineralogical and geochemical characteristics of the WAF (Swartz et al 1994) revealed four distinct units within the WAF and these are listed in Table 2.1. The presence of volcaniclastic sediments contributes to the weakness of this material, but the major concern is the presence of talc. Talc is an extremely soft mineral (1 on Moh’s scale of hardness) and breaks down when exposed to water. The W1b unit can contain as much as 50% talc and represents the greatest concern.

### Table 2.1: Lithostratigraphy of the Klipriviersberg group lavas (from Swartz et. al 1994)

<table>
<thead>
<tr>
<th>Formation</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alberton</td>
<td>W2</td>
<td>Dark grey amorphous massive horizon with distinct porphyritic texture</td>
</tr>
<tr>
<td>Westonaria</td>
<td>W2b</td>
<td>Sequence of ’high magnesium basalt’ and ‘olivine cumulate’ zones. No volcaniclastic sediments.</td>
</tr>
<tr>
<td>Westonaria</td>
<td>W2a</td>
<td>Thick olivine cumulate texture unit with massive and amygdaloidal intervals. Chlorite, quartz and carbonate. Includes volcaniclastic breccias, sulphides</td>
</tr>
<tr>
<td>Westonaria</td>
<td>W1b</td>
<td>Massive talcose, high magnesium basalt. Talc and chlorite. Fine grained black with carbonate veining, particularly in contorted horizons. Includes volcaniclastic sediments</td>
</tr>
<tr>
<td>Westonaria</td>
<td>W1a</td>
<td>Massive and amygdaloidal basalt. Chlorite and quartz with some talc. Includes volcaniclastic sediments</td>
</tr>
</tbody>
</table>

Geotechnical borehole logging and laboratory tests were carried out on exploration borehole core. This revealed an average RMR of 45 and Q of 2 for the WAF lava. The uniaxial compressive strength ranged from 45 MPa to 138 MPa, with an average of 92 MPa. Slake durability tests carried out on WAF material yielded very high durability results (>98% after 4 cycles).

However, significant deterioration of the WAF had been observed in the shafts and in tunnels, which are more than 30 years old, it was felt that the average RMR and Q values obtained from the borehole logging were in fact too high for the WAF lava immediately around existing excavations. Further investigation revealed that boreholes, which started in or near the WAF, yielded the lower RMR values. It was evident that the WAF deteriorates over many years due to exposure to air and water.

Four boreholes (38-20, 38-21, 38-22 and 38-23) were drilled up from 38 level, near the Main shaft, to obtain more information on the WAF in the shaft barrel (Figure 2.1). Three of the boreholes were drilled from the catwalk around the shaft (38-20, 38-21 and 38-22) and therefore provide a good indication of the condition of the WAF in the shaft barrel.
These holes were drilled using typical rotary core drilling and no attempt was made to preserve the WAF. The core loss was very high, making it impossible to carry out any meaningful geotechnical logging. Core recoveries in the first 10 m were less than 20% and gradually improved along the length of the boreholes. The available core had been significantly ground down during drilling and was soft enough to be scratched with a knife and sometimes even with a fingernail. This indicates very low strength and durability for WAF, which has been altered after exposure to air and water over many years.

The boreholes 38-20, 38-21 and 38-23 were all inspected with a borehole camera and these inspections were recorded to video. The brow concrete, conglomerate and lava boundaries were clearly visible in each of the boreholes. It was also possible to identify the extent of borehole breakout at different positions within the borehole (Figure 2.2). The amount of borehole breakout was used to identify the very poor lava, poor lava and good lava zones indicated in Figure 2.1. These geotechnical zones compare favourably with geotechnical borehole logging carried out by Burke and James (1990) during an investigation into the stability of the shaft carried out during that time.

Geotechnical mapping was carried out in WAF exposure on 38 level and an RMR of 33 and a Q value of 0.27 were obtained. This is more representative of the material observed in the shaft boreholes and in excavations in WAF.
Figure 2.2: Borehole breakout “dog-earing” observed in borehole 38-20.

3 Historic rehabilitation in WAF material in the Ezulwini Main and Ventilation shafts

The instability problems associated with Ezulwini Main shaft and remediation measures undertaken are described in White (1988) and Lourens and White (1988). The sinking of the 6.7 m diameter ventilation shaft commenced on 20 January 1960 and was completed to the final depth of 1102 m below collar on the 29 August 1960. Sinking of the main shaft (7.925 m diameter) was started on 13 March 1960 and the final depth of 1518 m below collar was reached on 3 March 1961.

The problems were first highlighted in September 1982, when investigations showed that the reef pass, between 36 and 38 levels, had scaled to within 8 m of the Main shaft. The reef pass was subsequently waste and grout filled to stabilise the area.

Following further instability during the first 6 months of 1984, 100 ton pre-stressed grouted anchors were installed to stabilise the 38 level station brow and shaft barrel. The shaft steelwork was also modified during this time. Five sets of buntons were cut away from the barrel and suspended from above.

During December 1986, 6 displacement transducers were installed at bunton levels 220 and 221 (±4 m above the 38 station brow). Closure displacements of up to 50 mm and 19 mm were recorded in the North East and South East compartments in 1988. The high closure displacements correlated with the area then most affected by cracking of the concrete lining. During most of the operational life of the shaft, it had been necessary to moil the East and West sides of the shaft in the affected area on a number of occasions to provide clearance for the skips.

The deformation was considered to be of a time dependant nature and directly associated with the WAF.
There are no records of deformation available since this time, but it appears that with support installed and limited mining taking place subsequently, the shaft has not experienced significant further deformation. The conditions and remediation measures observed in the Main and Ventilation shafts are shown schematically in Figure 3.1 and Figure 3.2.

The support installed prior to 1988 may have corroded and its performance cannot be guaranteed. Corrosion is evident on the exposed portions of the support units and some have failed.

Figure 3.1: Historical support and remediation measures observed in the main shaft
4 Design philosophy

The main purpose of the design is to ensure safe, efficient and continuous use of the shafts for the extraction of the multi-layered and sometimes massive gold-bearing strata within the shaft pillar area. This requires that adequate provision be made for inevitable convergence and ride movements that will be considerably greater than is usually experienced in shaft pillar mining, and the possibility of significant seismic activity.

The fact that there is no other access shaft on the property dictates that there is no viable way to access the pillar area other than via the Ezulwini shaft itself. Long term requirements may justify another shaft at a different location but prevailing economic circumstances leave no alternative, other than rehabilitation of the existing infrastructure.

The identification of the mechanisms of deformation and failure of the weak WAF mass is of key importance in the process. As there is only one previous case (South Deep SV1 shaft) where complete failure has occurred it is not possible to understand what the actual failure process was. A multi-disciplinary design workshop team, comprising SRK rock engineers, RSV project engineers, support contractors and mine production personnel was established. The team has used collective experience to visualize a range of possible failure modes and then assess how the damage could be contained by a reinforcing/supporting system that would fit within the operational constraints of a main hoisting shaft.

In addition the experience and insights of others who were involved with the investigation of the SDSV1 were sought. In particular the view of T Rangasamy was accepted, namely that
“….. the WAF should be characterized as a very weak squeezing material with a strong creep potential….”. Enquiries were also pursued into certain instances where problems of tunnelling and stoping through WAF on neighbouring mines had been successfully overcome.

Two things appeared to be widely accepted. The practice of injecting a two-component resin at low pressure into the rock surrounding excavations, was usually very beneficial in improving the condition of the WAF. Where collapses had occurred there had usually been some indication of impending danger with preliminary symptoms developing with time. This suggested that ‘best practice’ would certainly include the use of resin injection wherever possible and the use of sophisticated automated monitoring technology in affected areas.

Due to the complex creep behaviour of the WAF rock mass, it is not practical to set numerical values for the design criteria applicable to the several components of the support system. Therefore, a “multi barrier” design was adopted. This entails the application of several support systems, each providing a different support function. This means that for the most part, a considerable degree of ‘back-up’ capability is provided. If deformation or displacement exceeds an initial critical level and some failure occurs in one support element then a second set of elements will come into play. All support systems will need to fail before a major failure occurs.

Essentially, the over-riding requirement is to provide the maximum possible long-term resistance to movement of the weak rock mass together with the ability to survive possible extreme over-loading of short duration such as might result from a large seismic event. Obviously these requirements must be provided in a practicable way that fits in with the tight clearances and operational constraints of a normally functioning hoisting shaft.

In broad terms therefore, the following requirements were established as being necessary to minimise the possibility of significant damage occurring during the life of the shaft.

- The existing confinement of the weak rock behind the concrete lining should be maintained intact as far as possible. Some moiling will be necessary where excessive bulking has taken place and cage clearance is compromised.
- The ‘hoop stress’ capacity of the lining must be supplemented by a stiff, strong cladding.
- The weak-rock ‘squeezing ground’ in the cylindrical annulus beyond the shaft barrel must be consolidated and strengthened by grout injection
- During consolidation, the cylindrical annulus should be reinforced with a dense array of stiff, strong tendons.
- The resulting reinforced thick ‘shell’ of cladded concrete and strengthened rock must be tied or anchored back to the surrounding rock mass with the maximum possible retaining force by means of long anchors.
These anchors must be able to survive large amounts of elongation and high ground accelerations due to seismic events without compromising their initial “stiffness”.

The combined cladding, which consists of the original lining strengthened by a strong, shotcrete-stiffened steel screen, will form a load-distribution diaphragm that will spread the point loading of the anchors and injection rods more uniformly over the weak WAF layer.

The long-term serviceability of all the steel components in respect to their extension capacity, their rockburst resistance and their corrosion life must be specifically considered and optimized in the total design.

5 Method of Shaft Barrel Support

It is assumed that the concrete lining, normally of 30 MPa compressive strength, has zero ‘hoop strength’ over the WAF intersection because it has already failed or been moiled away in places.

The main thrusts in the support strategy are:

i) rock consolidation and reinforcement by resin injection

ii) construction of strong cladding (surface support in the form of steel screen panels and shotcrete)

iii) ‘tie back’ anchoring of the cladded, reinforced rock annulus by means of cable anchors that possess considerable dynamic capability (via Duracables) and are pre-stressed on installation

iv) Yielding straps to prevent any large blocks from falling into the shaft.

5.1 Rock consolidation and reinforcement

The Wimico system of resin-reinforcement was adopted because of its history of success in horizontal workings in neighbouring mines. Holes are drilled by pneumatic percussion drills (S36) at a grid-spacing of 0.95 m x 0.95 m using Wilborex 30/11 injection rods as the drilling rods. The length of the Wilborex is, alternately, 6 m and 4 m and the hollow bar is left in the completed hole to serve as the conduit for injection of the two-component liquid resin. The continuous (left-hand) thread of the Wilborex ensures that the tensile strength of the steel bar is closely coupled to the concrete lining and the jointed rock continuously along its full length.

The strength of the bar is nominally 320 kN. These are installed in 0.95 m x 0.95 m pattern. Theoretically, the radial confinement \( \sigma_{hr} \) on the weak WAF immediately behind the concrete lining would be 350 kN/m\(^2\).
The support resistance provided by the Wilborex rods is passive, requiring dilation (volume increase) of the WAF before the resistance is activated. With effective full-column grouting, the high stiffness of the rods would ensure that the required dilation is small (of the order of a few milli-strains). The load in the bar is transferred to the concrete lining through a 20 mm thick steel plate of 200 mm diameter.

Protection against corrosion will be provided by the resin and a 20 mm layer of dense shotcrete.

5.2 Steel screen panels and shotcrete

The concrete linings of both the main shaft and the ventilation shaft have failed in places across the respective WAF intersections. In order to prevent bulging of the low strength ‘mobile’ lava through the failed or weakened lining between the 0.95 m spaced Wilborex injection rods, the lining needs to be strengthened. Pre-formed, curved panels of heavy metallurgical screen provide a cladding layer that possesses multi-directional strength and a degree of dynamic ductility. The ductility reduces the risk of ejection of blocks of concrete that might result from a seismic event. A high strength, high density shotcrete, sprayed over the screen panels, improves the stiffness of the cladding and provides additional corrosion protection.

5.2.1 Dimensions and pattern

The screen is woven from 10 mm diameter crimped wires at 110 mm centre spacing in both weft and warp directions. To ensure continuity of strength between adjoining sections of cladding the screen is formed into panels of 1.87 m x 1.87 m by welding the wire ends to a surrounding frame of 22 mm diameter round mild steel bar, Figure 5.1.
Figure 5.1: Steel screen panels

5.2.2 Strength

The load extension characteristic of the screen wire is given in Figure 5.2 with an ultimate tensile strength (UTS) of 44 kN per wire. The total tensile strength in-plane, in both vertical and horizontal directions, is 1500 kN.

A second function of the panels is to provide improved area load distribution of the ‘point-loading’ from the Wilborex rods. This will require the woven screen panels to be stiffened and closely-coupled to the existing concrete lining by means of ultra high quality (60 MPa) shotcrete. This will transfer and distribute the 500 kN ‘point load’ of the twin 18 mm diameter Duracable anchors into a more uniform restraint which will contain any bulging of possible weak patches in the lining and will effectively prevent any possibility of breakthrough collapse of WAF through the 1.9 m spacing between anchors. Proper bonding of shotcrete to steel to lining will contrive a ‘composite’ effect to the combined concrete lining/steel screen cladding.
5.2.3 Surface testing

A series of surface tests was conducted on screen panels at the Savuka drop test facility near Carletonville. A total of 7 tests was conducted of which 4 were statically loaded and 3 were dynamically loaded. Two of the seven screen panels were sprayed with a 70 mm layer of 60 MPa shotcrete. In each test, the screen panel was bolted to a square steel frame (Figure 5.3). The bolting positions represent the four Wilborex bolts in the actual underground design. Steel and wooden blocks were placed on top of the screen panel in between the four bolts to simulate a punch load.

During static loading, a 10.3 tonne weight was then lowered onto the blocks on the screen panel. The static tests comprise a loading phase, an unloading phase and a reloading phase. The test arrangement allowed the screen panels to be deflected more than 300 mm, before the weight was arrested on the frame. The applied loads were measured using a suspended tensile load cell and induced deflections were measured using measuring tapes and a dumpy level. The weight was stopped at regular intervals to record measurements. During the reloading phase the weight was not stopped to take measurements, to simulate a higher rate of loading.
The test arrangement was modified for dynamic load testing. The maximum deflection had to be limited in order to stabilise the arrangement. Durapack blocks were also placed on top of the steel frame to cushion the steel frame when it arrested the steel weight. This further reduced the available deflection. Two 2 MN compressive load cells were installed, below the steel mounting frame, to measure the impact loads and replace the suspended tensile load cell. These provide a fairly rudimentary means of estimating the impact loads. A 200 mm travel Linear Variable Displacement Transducer (LVDT) was used to measure deflection. A spider 8 data logger allowed data capture at a frequency of 2400 Hz. The 10.3 tonne weight is dropped from a suspended height of 100 mm to provide an impact velocity of 1.4 m/s.

A comparison of the load/deflection characteristics for different cases is shown in Figure 5.4.

Under static loading, screen panels (without shotcrete) strain harden and become stiffer with increased deflection during the first phase. After the initial loading and unloading phases, the screen panels maintain a residual deflection of 215 mm, which is 70% of the maximum deflection during the loading phase. During the reloading phase, the stiffness of the screen panels was much greater (0.49 MN/m) than during the initial loading phase (0.114 MN/m). The residual deflection and increased stiffness is mainly due to straightening of the crimped wires.

The shotcrete screen panel displayed a substantially stiffer response of 1.22 MN/m until cracking of the shotcrete occurred and reduced to 3.67 MN/m during strain hardening. The 10 tonne weight is supported at maximum deflection. This shows the importance of applying the shotcrete.

The dynamic test results in Figure 5.4 represent the responses of two screen panels (one plain and one with shotcrete) under dynamic loading conditions (1.4 m/s). During the shotcreted screen panel test, the shotcrete provided a high initial stiffness, until it failed. The screen panel continued to deform until the weight was fully arrested by the screen panel and...
durapack blocks. No failure of the steel screen panel occurred. The LVDT, which was fixed to the shotcrete, became detached when the shotcrete failed. The curve in Figure 5.4 is cut off where durapack blocks begin to influence the test results. The maximum deformation, measured physically, was 150 mm. The plain screen panel deforms at substantially lower stiffness and is allowed to deform more due to the test arrangement. The energy absorption can be determined from the area under the load deflection curve. The addition of shotcrete increases the energy absorption capability from about 0.8 kJ to 9.8 kJ over the first 80 mm of deflection.

None of the screen panels was destroyed during testing. The test arrangement did not allow the point of failure to be determined and the energy absorption capacity of the screen panels is probably much greater than that recorded. In the test arrangement, the edges of the screen panels are free, while they are connected by clamps and continuous shotcrete in-situ. A stiffer response is expected in-situ.

**5.2.4 Underground Testing**

An underground test site was prepared where two panels, connected by U-bolts, have been anchored to the sidewall on 38 level. Measuring the deflection produced by hydraulically inflating flat-jacks sandwiched between the side-wall and the screen will define the actual load/deflection characteristic of the total support system. This will be done both before and after the application of a 70 mm layer of shotcrete, enabling determination of the strengthening effects of the steel-shotcrete composite. Preliminary tests on the unsprayed installation have been conducted but, because of temporary difficulty of access to the area, the tests have yet to be completed for the sprayed case.
5.3 Duracable tie-back anchors

The possible occurrence of significant seismic activity presents a threat of dynamic damage to the shaft barrel particularly where the lining may already be weakened by pressure from the WAF. Yielding cable anchors do not fail when their peak load capacity is exceeded, but allow controlled deformation to take place with a constant residual resistance. This enables the cables to absorb the energy of an ejected rock block and to control ongoing squeezing of the rock mass, preventing rocks from being dislodged into the shaft.

5.3.1 Pattern

The installation of 8 m long twin 18 mm diameter Duracables at the centre of each panel screen (ie. at 1.9 m centres) counters this risk of seismic damage by providing a substantial degree of dynamic ductility.

5.3.2 Capacity

Without considering the beneficial effects of cohesion and friction, a uniform thickness of 1.0 m of lining and rock should, theoretically, be arrested within 60 mm of displacement if it were ejected at an initial velocity of 2.4 m/s.

This dynamic capacity assumes that the ductile resistance of the twin Duracable is adequately distributed across the area between anchors by the 25 mm thick base-plate and the screen/shotcrete-cladded lining.

5.3.3 Testing

Double-embedment tests have been carried out in the laboratory on individual Duracable anchors where the crinkled anchor length has been equipped with ‘waves’ of varying amplitude and number. The intention was to determine the amplitude that would give optimum resistance with four or five waves (anchor length of 360-450 mm and length of yield of 400 mm).

The results indicated in Figure 5.5, show four test results with a four wave configuration with an amplitude of 14 mm and the results of a test with a six wave configuration with and amplitude of 13 mm. These results are compared with a typical 18 mm cable. Additional tests were conducted with configurations with five and six waves with amplitudes up to 16 mm, which proved to be too constricting and the cables failed during testing. The four wave, 12 mm amplitude was selected.

Three twin Duracable anchors have been installed in the WAF lava at the 38 level test site referred to above. One pull test has been performed on one leg of an 8.0 m long anchor installed in WAF on 38 level. Its yield value of 250 kN is considered satisfactory. This result is also shown on Figure 5.5. Further testing is planned when the site becomes available again, including dynamic testing by means of controlled use of explosives.

The ability of the Duracable to retain its loads over the course of time (creep) has been checked in the laboratory with four grouted specimens, each monitored in a different way. After an initial bedding-in loss in load, acceptable load retention was displayed during the following two weeks.
Figure 5.5: 18 mm duracable laboratory and underground tests
5.4 Duracable straps and connectors

The twin Duracable straps which connect the 8 m long tie-back anchors in the vertical direction, represent the ‘last line of defence’ for the shaft. A consideration of the geometry of the forces involved will show that transversely arranged straps can have no initial effect on resisting any bulging of the lining. Vertically arranged straps should be taut from the time of installation but no significant additional benefit would result from additional tension being applied to them. Their purpose would be to prevent large portions of the shaft wall from falling free into the shaft.

The fact that the straps have the same stiffness, peak strength and residual resistance as the tie-back anchors will provide a very considerable damage-limiting benefit in the event of an otherwise catastrophic failure that could occur with conventional (non-ductile) support systems. Continuity of strength and of ductility is ensured by the novel connector which has been custom-designed for this application.

5.4.1 Pattern

The straps will be installed as five modules each 6 m long requiring 4 connectors and special anchoring at the top and bottom which should be independent of the tie-back anchors.

The base-plates of the tie-back anchors (Figure 5.6) have been designed to lightly clamp the strap but permit sliding under small loads, so that transverse bulging forces that might be localized at some point of intense damage would be transferred along the 6 m length of each module of duracable strapping.
5.4.2 Strength

The ends of the 6 m long, 18 mm diameter cable that constitutes one-half of the Durastrap are crinkled in exactly the same way as the anchor end of the Duracable so its strength/yield response will be similar. Since the connector is a mechanical contrivance the crinkle will tend to flatten slightly compared with the confined grouted anchor and controlled sliding at the connector will occur at a slightly lower value (peak and residual) than the values of 280 kN and 175 kN indicated in Figure 5.5 for the duracable performance.

5.4.3 Testing

Tests have been carried out in the laboratory on a preliminary design of the connector that have confirmed this expectation.

In-situ testing will be done at the test site in 38 level cross-cut as part of the overall stiffness testing of the completed cladding. Because of inherent difficulties in applying measurement devices the results are likely to be qualitative and confirmatory rather than quantitative.
6 Monitoring

Monitoring of the shaft barrel in the WAF zone is crucial to the success of the support strategy. The deformation of the WAF must be monitored and recorded continuously. This should be done in real time and transmitted directly to the control room on surface. The onset of deformation can be determined. From then on the rate of displacement will need to be determined. A significant increase in the rate of displacement will indicate that the rock mass is becoming unstable. The amount of displacement will indicate whether the steelwork or the safe passage of conveyances is at risk. It will also provide an indication of the status of the Wilborex and duracable anchors installed in the WAF. The amount of deformation can be compared with the load deformation characteristics of the support units.

The following instrumentation will be installed in the main shaft barrel:

- A ring of 3 three point extensometers each 9 m long
- A ring of 3 six point extensometers each 12 m long
- Laser targets to be monitored with a laser distomat.

A ring of three point extensometers and laser targets will also be installed in the ventilation shaft barrel.

7 Implementation of the design

The rock consolidation and reinforcement has been completed in both the main and ventilation shafts. Resin volumes were recorded for every drill hole. The average resin volume was 45 l per hole, however some holes required in excess of 200 l.

The original concrete lining had deformed so much in places in the main shaft that it was necessary to moil parts of the lining to ensure safe passage of the conveyances. This was done only after the resin consolidation and reinforcement was completed and was carried out in small increments to prevent unravelling of the exposed WAF material. This was completed successfully.

The steel screen panels have been installed in the main shaft and are being installed in the ventilation shaft. The shotcreting of the screen panels has been completed in the main shaft with 60 MPa material. The voids formed behind the screen panels, where the concrete lining had deformed or been removed were filled with shotcrete.

The duracables still need to be installed. One of the concerns raised was that the duracables must be tensioned during the shift and the ends should be cut off to allow the conveyances to pass unhindered. A quick setting grout was used, which allows tensioning to 20 tonnes after 2 hours. Trial installations were successfully conducted on 38 level in WAF, including 2 hour pull tests.

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9 References


