

# Design of Parallel-Sided Haulage Plugs at a Gold Mine in South Africa

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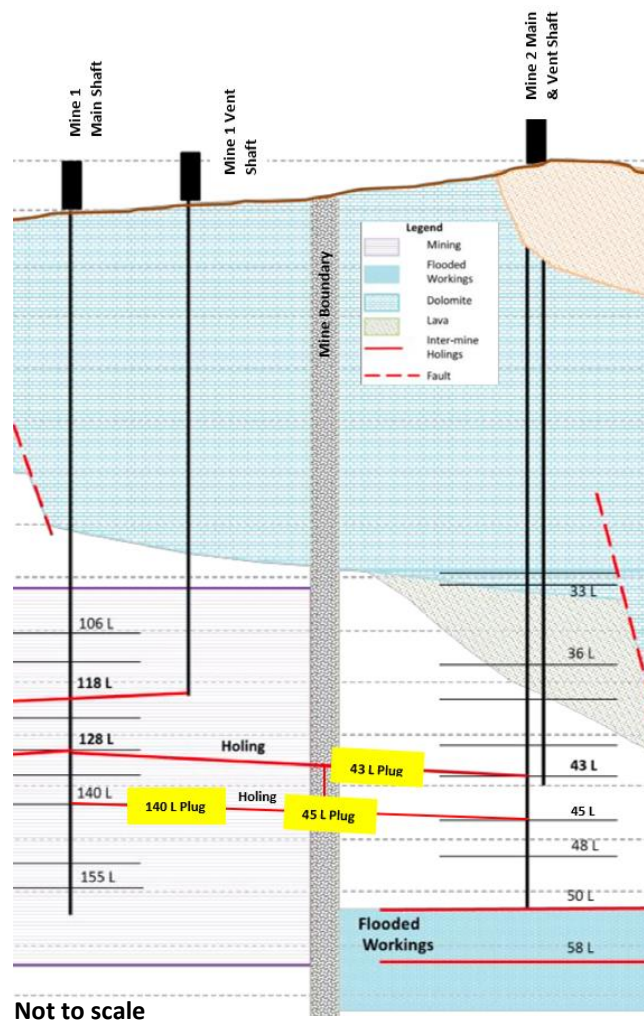
## ABSTRACT

Due to the possibility of pumping operations ceasing at a deep level gold mine in South Africa, which would result in the possible re-watering of the mine, underground plugs were required to prevent flooding of the neighboring mine. Three 25 meter long parallel-sided haulage plugs have been designed to safely resist hydrostatic heads of 1 100 m (45 level and 140 level plugs) and 1 181 m (43 level plug) respectively. This paper details the design of the plugs which considered shear failure along the rock-concrete interface, excessive seepage around the plugs, and long-term disintegration of the concrete. Site investigations comprising geological mapping and Lugeon testing, and site preparation consisting of pre-grouting of the surrounding rock mass are discussed. The plug design addresses the constituent materials and components of the plugs, the inclusion of a lime deposit to neutralize acidic mine water in the immediate vicinity of the plugs, and a bentonite geotextile to serve as a buffer to resist chemically aggressive mine water. A Monte Carlo simulation has been carried out in order to predict the probability of failure of the plugs.

For the sake of providing a complete overview of the project, aspects of plug construction are discussed concisely, including aggregate preparation, mortar intrusion, mortar quality control, and plug tightening. A description of both the short-term (during construction) and long-term plug performance monitoring requirements is also included. At the time of writing, the construction of the plugs was approximately 70 % complete.

**INTRODUCTION**

Underground plugs are required to prevent flooding of a neighboring mine as a result of ceased dewatering pumping operations at a deep level gold mine in South Africa. It is expected that the underground tunnels of the mine will flood with groundwater once the active dewatering ceases. The mine is currently in the process of obtaining environmental authorization for terminating underground mining operations, in accordance with the National Environmental Management Act and Environmental Impact Regulations, from the Department of Mineral Resources. After a site investigation and inspection of the mine layout and the connectivity between various levels, three locations for plugs were identified, located on 43 level, 45 level and 140 level (of the neighboring mine) such that the neighboring mine would be protected. The plugs were sited in the vicinity of the boundary between the two mines as shown in Figure 1. Each plug was required to withstand both hydrostatic – and seismic loading. The rock mass at the plug sites were found to be competent impermeable quartzites and conglomerates of the Middle Elsburg Reef hosted in the Witwatersrand Supergroup.



**Figure 1** Mine plan sketch indicating the locations of each plug

## DESIGN

### Design Criteria

Three main potential failure modes were identified for parallel-sided plugs (American Concrete Institute, 1972; Chamber of Mines of South Africa, 1983; Garrett & Campbell Pitt, 1958; Jager & Ryder, 1999; Lang, 1999; Littlejohn & Swart, 2006; Ryder & Jager, 2001; Westergaard, 1931). Additional design considerations are in accordance with the South African National Standard for the design and construction of underground high pressure bulkheads (SANS 536:200X, 2007). A summary of the failure modes and corresponding design criteria used in the design of the plugs is presented in Table 1 below.

**Table 1** Failure modes and design criteria

Failure mode	Design criteria
Punching shear failure along the rock-concrete interface or through rock mass	Factor of safety > 2 (normal conditions)
	Factor of safety > 1.8 (inundation up to ground level)
	Factor of safety > 1.5 (earthquake conditions)
	Sufficient shear strength of rock mass
	Sufficient shear strength of concrete
Excessive seepage around the plug and possible downstream erosion	Ensure watertightness
	Limit the hydraulic gradient to 50
Long term disintegration of concrete	Minimum 28-day compressive strength of 25 MPa
	Ensure an appropriate and robust concrete mix design

## Design approach

### *Punching shear failure prevention design considerations*

In the design against punching shear failure along the rock-concrete interface or through the rock mass, the following load cases were considered:

- Load case 1: Hydrostatic heads equivalent to the height of the natural groundwater table above the plugs.
- Load case 2: Hydrostatic heads equivalent to the height of the natural groundwater table above the plugs with additional hydrostatic head due to seismic activity.
- Load case 3: Hydrostatic head equivalent to the height of water if total inundation up to ground level were to occur.
- Load case 4: Hydrostatic head equivalent to the height of water if inundation up to ground level were to occur with additional hydrostatic head due to seismic activity.

For static equilibrium, the shear resistance along the outer surface area of a plug (the resisting force) is required to equal the pressure head acting on a plug (the driving force) as shown in Eq. 1 (Lang, 1999) below.

$$\frac{\text{Resisting force}}{\text{Driving force}} = \frac{2(h+w)LU}{\rho_w g H w h} = 1 \quad (1)$$

The resisting force is calculated by multiplying the outer surface area of a plug in contact with the haulage with the allowable shear strength of the rock mass  $U$ .  $U$  should be substituted with the allowable shear strength of the concrete  $f_s$  if  $f_s$  is lower. The driving force is a function of the density of water  $\rho_w$ , gravitational acceleration  $g$ , head of water on a plug  $H$  in meter, and the haulage dimensions.

The recommended design shear strength for a Council for Scientific and Industrial Research (CSIR) rock mass rating of 'very good' is 1.5 MPa (Lang, 1999). The Chamber of Mines of South Africa's code of practice for the construction of underground plugs (1983), states that a safe uniform shear stress of 0.83 MPa can be assumed for plugs constructed in quartzites of the Witwatersrand Supergroup. This figure has been established experimentally and contains a large safety factor. If the allowable shear stress in the concrete is lower than that of the rock, punching shear failure will be governed by the allowable shear stress of the concrete and therefore the lower value of 0.83 MPa was used in the design calculations.

Lang (1999) states that if a plug is installed in a haulage and a seismic event occurs, it may cause differential movement of the water and the surrounding rock to induce water hammer. The additional pressure  $P_H$  in kPa which results from the water hammer is a function of the acoustical velocity of water  $c$ , the ground velocity  $v$  and the density of water  $\rho_w$  as given by Eq. 2 (Westergaard, 1931).

$$P_H = cv\rho_w \quad (2)$$

Considering the additional pressure as a result of water hammer and the pressure head acting on a plug, the driving force was calculated by modifying Eq. 1 as shown in Eq. 3.

$$\frac{\text{Resisting force}}{\text{Driving force}} = \frac{2(h+w)LU}{\rho_w wh(gH + cv)} = 1 \quad (3)$$

An extreme case of 3 m/s was selected for the ground velocity  $v$  based on recommendations for underground dynamic support design specified by Jager and Ryder (1999) and Ryder and Jager (2001).

Considering all load cases for each haulage, a required structural design length  $L$  of 25 m was determined for the three plugs. This length was deemed acceptable as it yields a higher factor of safety than required for all load cases. Additionally, from the surveys and site investigation, the rock surface of the haulages were found to be irregular. The rough and uneven surface provides additional interlock and improves the frictional resistance between the concrete and the rock surface against shear failure.

### *Hydraulic gradient*

Under the hydrostatic head and the associated hydraulic gradient (defined as the design head of water divided by the length of the plug), and due to possible fractures in the rock mass, dissolution of the grout is possible at the rock-concrete plug interface. To reduce the flow of groundwater past the plugs, the hydraulic gradient was limited to 50 (Garrett & Campbell Pitt, 1958). After taking the water tightness requirements into consideration, a length of 25 m for each plug was deemed adequate. In addition to limiting the hydraulic gradient, additional precautionary measures were included - the rock profile was pre-grouted prior to construction and the plugs will be re-grouted after construction to ensure water tightness.

### *Concrete strength*

Considering the strength of the surrounding rock mass, the concrete is the weakest element in the design of the plugs. The safe uniform shear stress for the concrete of 0.83 MPa, is equivalent to a uniaxial compressive strength (UCS) of 25 MPa (American Concrete Institute, 1972) and therefore, a minimum 28-day UCS of 25 MPa was specified for the concrete. The specified concrete mix included pozzolanic cement to provide resistance to chemical attack by the acid generating mine water.

## **SENSITIVITY ANALYSIS**

To estimate the probability of any of the plugs failing, a Monte Carlo simulation was carried out by varying haulage dimensions, ground velocity during a seismic event, mortar strength and possible voids due to ineffective high point tightening. The simulation utilizes a different set of random input values constrained by the input distributions for each factor of safety calculation. A total of 300 000 iterations were carried out to obtain the frequency distribution of the factors of safety. The probability of failure after 28 days for the three plugs, which is when the concrete is expected to reach its design strength, was determined to be less than one in ten million.

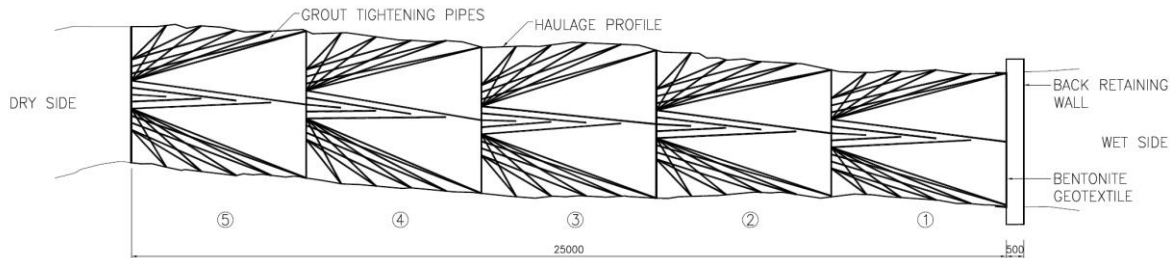
## SITE INVESTIGATIONS

The three plug locations were structurally and geologically mapped to assess the rock mass conditions at the different sites. The pattern of rock deformation at the plug sites was generally found to be relatively homogeneous, and was therefore determined to be relatively impermeable. After it was established that the rock mass at the plug sites are not likely to be very permeable, Lugeon testing was carried out to determine the intensity of pre-grouting required. Lugeon testing is an in-situ testing method used to estimate the average permeability of a rock mass in terms of a Lugeon value, where 1 Lugeon is equal to 1 liter of water flow per meter of borehole under a pressure of 1 MPa. Considering a homogeneous and isotropic condition, 1 Lugeon is equivalent to  $1.3 \times 10^{-7}$  m/s. A maximum residual water tightness of 3 Lugeon was specified (Littlejohn & Swart, 2006). The results of the Lugeon testing indicated values of 1.7 or less for all the plug sites, confirming that the rock mass is relatively impermeable to the natural flow of water.

## CONSTRUCTION APPROACH

The low permeability of the rock mass at the plug sites required only nominal pre-grouting in the plug designs – two rings of eight meter deep pre-grouting boreholes per segment, grouted up with 10 MPa strength cement grout under a pressure of 7 MPa – essentially forming impermeable grout curtains. The plugs will be constructed in five segments of 5 m each, starting from the wet side (the side that will face the re-watered side of the mine) to the dry side (the accessible side) as illustrated in Figure 2. Before construction commences, 1 000 kg of lime (composition 60 % calcium hydroxide and 40 % sodium carbonate) will be deposited onto the footwall on the wet side of the plug sites. The purpose of the lime deposit is to neutralize any acidic water that might occur in the immediate vicinity of the wet face of the plugs. A 500 mm thick reinforced concrete back retaining wall, which acts as a sacrificial shuttering, is constructed on the wet side of each plug and keyed-in to the haulage perimeter by 500 mm. The back retaining wall consists of 25 MPa concrete and is reinforced with nominal reinforcing, and was designed as a two-way spanning slab as per Eurocode 2 (Mosley, Bungey & Hulse, 2012). The reinforcement in the back retaining wall was sized such that there is sufficient resistance against bending and shear on the inside face of the concrete plugs during construction. Once the first segment of the plugs has set, the back retaining wall is considered to have served its main purpose. On the dry side of the back retaining wall, four 5 mm thick layers of self-sealing bentonite impregnated geotextile are placed over the full face area of the plugs with an additional overlap of 300 mm on the rock perimeter. The geotextile joint with the rock is sealed with bentonite paste. The bentonite geotextile serves as a buffer to resist aggressive mine water that could cause dissolution of the cementitious material in the plugs. The geotextile expands on contact with water and has a low permeability ( $1 \times 10^{-11}$  m/s). Mass concrete, rather than reinforced concrete, was specified for the plugs, because the plugs do not require tensile resistance or resistance against bending. Rather, the mass and length of the plugs contribute towards the resistance against sliding and punching that may occur due to the total pressure head acting on the plug. Stainless steel tie bars are placed between segments where cold joints will form. The specified mortar strength is 25 MPa with quartzite aggregate ranging from 75 mm to 300 mm. The aggregate is high pressure jet sprayed prior to being sent underground, and double washed and scrubbed free of grit at the plug sites. Due to the large size of the aggregate, the aggregate is preplaced and the mortar is intruded through mortar intrusion pipes. Approximately 12 hours after mortar intrusion is completed, high point grouting is carried out to fill any remaining voids between the concrete and the rock mass. After three

days, grout tightening is done by drilling holes to the concrete-rock interface, conducting pressure tests to measure water tightness and re-grouting these holes. This process is repeated for the subsequent plug segments.



**Figure 2** Typical cross-section through a plug

## MONITORING

### Construction monitoring

Throughout the construction of the plugs, several records - such as grout quantities, mortar quantities, injection pressures, mortar mixes, and quality control tests - will be kept to ensure that the construction and materials comply with the design.

During construction, temperature measurements are taken using thermocouples in order to monitor the progress of the hydration process of the cement in the mortar. Two thermocouples are installed in each plug segment, one at a depth of 2.7 m, and the other at a depth of 1.7 m into the plug segment. The thermocouple readings are taken daily during the placement of the coarse aggregate and immediately before the start of mortar intrusion. Thereafter, readings are taken hourly during the next 24 hours and daily for a period of 28 days. Readings obtained thus far, indicate that the temperature generally dissipates to between 40 °C and 50 °C.

### In-service monitoring

Long-term in-service monitoring of the plugs is essential to ensure the plugs are functioning as intended and to provide an early warning system to ensure that personnel will have sufficient time to evacuate in the event of failure. In order for monitoring and maintenance to be carried out effectively, it is essential that the access to the dry side of the plugs be kept in good condition.

The monitoring network included in the designs comprises an embedded sensor network measuring key physical parameters including strain, load and water pressure, and a video monitoring system using intelligent motion detection software to identify key indicators of seepage, leakage and structural movement. The monitoring objectives will be achieved by installing eight vibrating wire embedded sensors within each plug. A stainless steel harness has been designed to protect the sensor cables during construction. The network will be linked to a central surface computer running dashboard monitoring system and will be capable of sending notifications via e-mail and text messages to mobile phones when trigger action levels are detected.

The following instrumentation, capable of automatically detecting changes in water flow through the plug or strain-displacement changes, were included in the plug designs:

- Two embedded vibrating wire piezometers, capable of measuring pressure head to detect any seepage through the plug.
- Three embedded vibrating wire strain gauges used to measure strains in the plug.
- One standpipe piezometer installed on the wet side of the plug to measure pressure head as the water level is rising during re-watering.
- Two embedded vibrating wire pressure cells capable of measuring pressures in the plug.

The video monitoring network will comprise a single outdoor dual lens video camera at each plug with cabling, lighting, communication network to surface and the capability of sending notifications via video call.

In addition to the concrete embedded monitoring network, the overall water seepage will be monitored using a V-notch weir combined with a vibrating wire piezometer across the haulage on the dry side of each plug. Excessive seepage will require remedial grouting. Geophones have also been included at each plug to detect any seismic activity.

During the re-watering stage, while the head on the plugs is rising, readings on the instrumentation should be taken hourly. Once the hydrostatic head on the plugs has stabilized and readings on the instrumentation reach conditions of equilibrium, the frequency of readings can be reduced to a monthly basis.

## CONCLUSION

At the time of writing, construction of the plugs was approximately 70 % complete. During construction it was found that after barring and scaling, approximately 500 mm of the footwall was removed resulting in an increase in haulage perimeter. While this did not affect the design length, additional plug materials and pipes were required. The possible increase in haulage perimeter should be considered in future designs. Additionally, the logistics of a project of this scale needs to be carefully considered during planning as this is a major cost component. Upon internal and external third party review, it has been found that the design assumptions and criteria are appropriately conservative and the plugs are not expected to fail provided that they are diligently constructed and monitored according to the design.



## NOMENCLATURE

$U$	allowable shear strength of rock mass
$f_s$	allowable shear strength of concrete
$h$	haulage height
$w$	haulage width
$H$	pressure head
$\rho_w$	density of water
$g$	gravitational acceleration
$P_H$	pressure head due to water hammer
$c$	acoustical velocity of water
$v$	ground velocity
$L$	plug length

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