PERMANENT SEALING OF TUNNELS TO RETAIN TAILINGS OR ACID ROCK DRAINAGE

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ABSTRACT

Many mining operations, particulary those in mountainous terrain, rely on tunnel plugs to permanently seal mine adits and to flood (at least in part) the mine workings upon closure. The success of these tunnel seals is a function of the design criteria established by the owner or regulator, the plug design, and the quality of construction. It is generally accepted that the design criteria for permanent mine closure tunnel plugs should be stricter than those used during mine operations, particulary if the plug is used to impound acid rock drainage. In most cases, it is the allowable seepage/gradient rather than the shear strength of the rock or concrete that controls the length of the plug. This paper will review design guidelines for permanent tunnel plugs. A number of cases histories are presented that illustrate how a tunnel sealing project can fail at either the design or construction stages. A number of sucessful tunnel sealing projects are also presented design or construction stages. A number of sucessful tunnel sealing projects are also presented wich demonstrate the key elements that should be part of the design and construction of tunnel plugs.

INTRODUCTION

Permanent sealing of underground workings is an increasingly important part of underground mine closure programs. Upon mine closure, many operators wish to re-establish the pre-mining groundwater profile (to the extent possible) in order to flood the mine workings and prevent acid rock drainage (ARD). Where the mine workings are located within mountainous terrain, it has generally not been possible to completely flood all of the mine workings due to seepage rates exceeding the rate of mine water inflow. Most seepage from flooded underground mines occurs from adits and other underground workings that intersect topography, from diamond drill holes and from faults and other major geological structures. Flooding of underground mine workings has the following environmental benefits:

- the oxygen required for the ARD reaction is reduced; and,
- the net groundwater inflow to the underground workings is reduced.

Constructing plugs in the tunnel using concrete or other materials can reduce drainage from adits. The drainage will never be totally eliminated since seepage will always occur to some degree through jointing in the rock mass surrounding the plug, even if the rock mass has been grouted.

Tunnel plugs can also be used to help impound tailings. Prior to building a large valley-fill tailings dam for example, the existing flow may be diverted through tunnels around the dam site. When the dam construction is completed, the diversion tunnels must be sealed with concrete plugs.

Tunnel plugs are significant engineering structures. The head acting on tunnel plugs can often exceed the head on the highest dams in the world (Figure 1). In addition, tunnel plugs used for mine closure are often exposed to aggressive water (low pH, high sulphate) that can significantly reduce their service life. Another challenge for the plug designer is the mine operator's frequent desire for a "walk-away" structure that will facilitate permanent mine. This is significantly different from the

design of concrete dams for example, which are designed for a service life in the order of 100 years, and which undergo regular monitoring and maintenance.
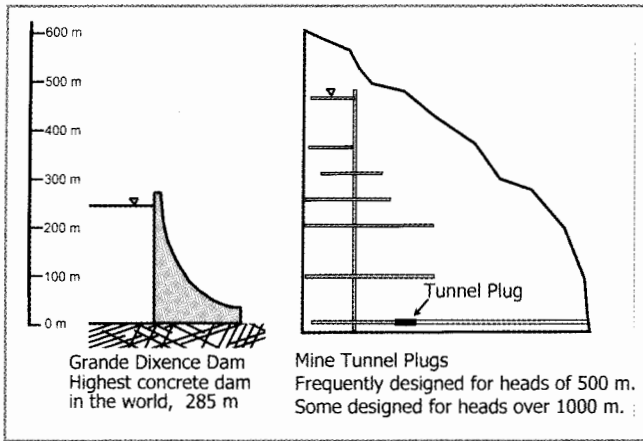

Figure 1. Comparison of Highest Dam to High Head Plugs.

Structures that are used in underground mines to impound water, tailings, or backfill can be classified as dams, fill fences, bulkheads, or plugs (Figure 2). Plugs are considered to be the best structures for permanent tunnel sealing and, as such, are the focus of this paper. The term permanent can be controversial but, for the purposes of this paper, is defined as a minimum 100 year service life without the need for monitoring or maintenance. It is meant to distinguish such plugs from more common bulkhead designs used in everyday mining applications. A well-designed plug will likely last much longer than 100 years, however some maintenance would likely be required. The author is currently undertaking research at the University of British Columbia involving the design and testing of a compacted bentonite/sand tunnel plug with an expected 1000+ year service life.

Dams

Dams are generally used in underground mines to store water for drilling purposes or for settling sumps. They are typically no more than a couple of metres in height and are free to overflow if the water height exceeds the height of the dam. Dams are generally constructed of concrete but can also be made of timber or sand/cement filled sandbags.

Fill Fences

Fill fences are normally used for retaining backfill in mine stopes. They are defined as structures with design heads not exceeding 100 kPa (about 10 metres of water or 5 metres of liquefied tailings). They can be constructed of waste rock, shotcrete, timber, cable slings and wire mesh, or a combination of the above.

Bulkheads

Bulkheads are typically constructed underground for low head (100-1000 kPa) conditions. They may be used at the base of open stopes to retain backfill, to seal off water from a

part of the mine, or to retain solution in an in-situ leaching stope. Concrete bulkheads are typically designed as reinforced concrete plates supported on four sides, that must be designed against flexural failure. In rare cases, the bulkhead may be designed as an unreinforced concrete arch plug.

Bulkheads for retaining backfill can also be constructed of waste rock, shotcrete, concrete, timber, cable slings, or a combination of the above. These structures are designed to be free draining or have drainage/decant systems installed to prevent high hydrostatic heads. An important distinction in the case of backfill bulkheads is that the maximum design loading condition is temporary and normally occurs only during the backfill pour.
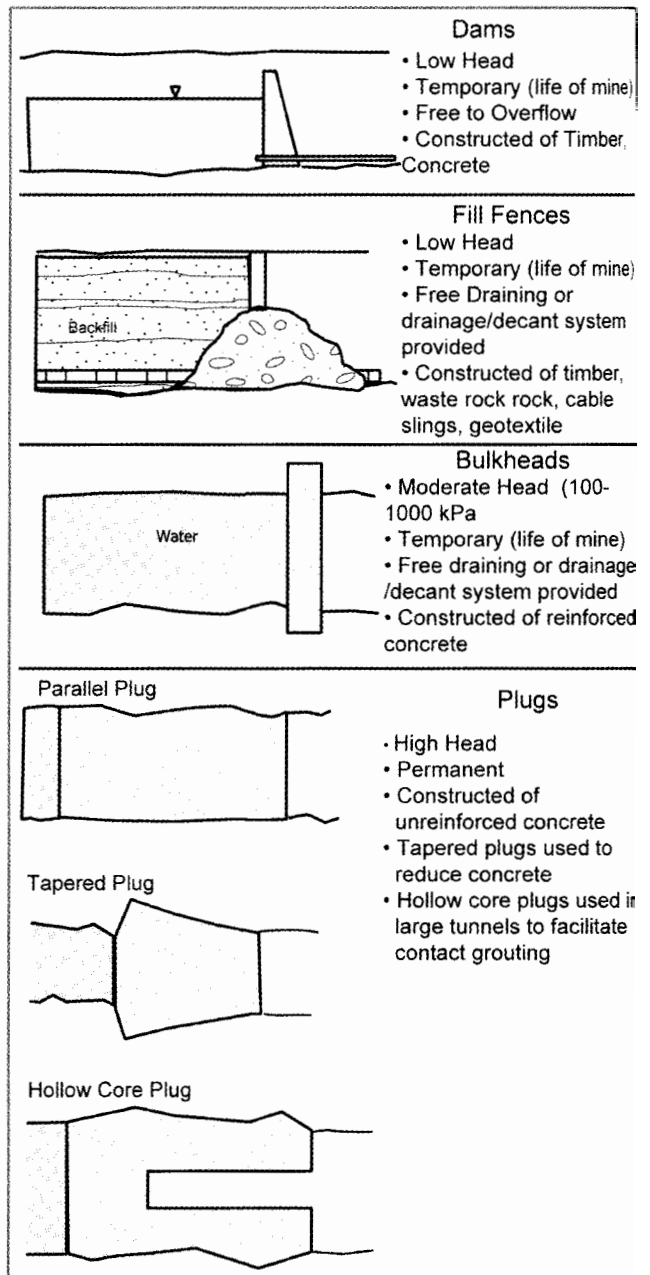

Figure 2. Types of water retaining structures in underground mines.

Plugs

Like bulkheads, tunnel plugs are structures used to impound water or tailings at pressures exceeding 100 kPa (10 m of water). The main difference is that tunnel plugs are designed to be permanent structures, not requiring maintenance or monitoring. Consequently, they will normally incorporate higher factors of safety, and meet more rigorous quality control and quality assurance specifications during construction. Tunnel plugs can be constructed as monolithic plugs or hollow core plugs depending on the tunnel size

Monolithic Concrete Plugs. These structures are constructed as a single concrete pour. They are more commonly used in small tunnels where the heat from cement hydration will not be as great as in larger tunnels. Monolithic plugs are most commonly parallel plugs; that is, there is no keying of the structure into the tunnel walls. In some cases, the plug may be constructed with a taper by enlarging the tunnel in a conical shape with the narrow end facing downstream. In general however, blasted tunnel walls will provide sufficient shear strength provided the plug length:width ratio is at least 1:1. Contact grouting is usually carried out from pre-installed injectable grouting tubes at the contact and/or from holes drilled from the downstream end of the plug.

Hollow Core Plugs. Hollow core plug designs are commonly used in large diameter tunnels such as those in water diversion schemes for dam developments (generally greater than 6 m diameter). Construction involves pouring concrete in sections and forming a gallery in the centre of the plug, which is later used for contact grouting. The grouting gallery may or may not be filled when the grouting is completed. Some operators prefer to leave the gallery unfilled in case future grouting is required. The gallery can generally be left unfilled if the plug diameter is equal to or greater than three times the width of the grouting gallery. Hollow core plugs can either be parallel or tapered.

SITE INVESTIGATION

Before constructing a tunnel plug, a suitable location must be selected within the area considered. A site investigation should be carried out to assess the geotechnical and hydrogeological characteristics of the site on which to base the design. The key questions that need to be answered by the site investigation are:

- Are there any major continuous faults, or shears that would affect the plug stability or result in excessive seepage?
- What is the shear strength of the rock mass?
- What is the hydraulic conductivity of the rock mass and how does it change with distance from the tunnel?
- Is there sufficient confining stress?
- How tight are the joints and what kind of grout can be used to grout the rock mass?
- Is additional ground support needed around the bulkhead?
- Is the rock or joint filling soluble or erodible?
- How much water is flowing in the tunnel and how will this be handled during construction?

Often, mines are remote from ready-mix concrete plants so the concrete is batched on site. For permanent concrete plugs, where concrete longevity is a priority, the site investigation should include collection of fine and coarse aggregate samples for a durability assessment and alkali-silica reactivity testing.

Failure Mode	Design Criteria
Hydraulic jacking of rock surrounding plug.	F.S. > 1.3 normal condition F.S. > 1.1 earthquake condition
Shear failure along rock/concrete contact or through rock mass.	Rock mass shear strength according to Table 2. Allowable concrete shear stress according to ACI Code. F.S. > 3.0 normal condition F.S. > 1.5 earthquake condition
Deep beam flexure	Allowable concrete tensile stress according to ACI Code.
Excessive seepage around plug and possible downstream erosion	Maximum hydraulic gradient based on Table 2. Seepage to be limited to occasional drips at plug and less than 0.5 L/s measured 20 m downstream of plug.
Long term disintegration of concrete	>25 MPa compressive strength. Concrete mix to be designed to best possible standards for resistance to acid attack, sulphate attack, and alkali-silica reactivity.

Table 1. Summary of recommended design criteria for permanent mine closure plugs.

MASS CONCRETE PLUG DESIGN

Mass concrete or monolithic plugs are so named because they are placed in one continuous pour and do not contain any steel reinforcement. They should be designed to resist failure from five possible failure modes, namely:
- Hydraulic jacking of rock surrounding the plug;
- Shear failure through the concrete, along the rock/concrete contact or through rock mass alone;
- Deep beam flexure failure;
- Excessive seepage around the plug and possible backwards erosion; and,
- Long-term chemical/physical breakdown of concrete, grout, or surrounding rock.

Suggested design criteria for each of the failure modes is summarised in Table 1.

Punching shear design

Punching shear failure should be assessed within the concrete, at the rock/concrete contact, and through the rock mass. The allowable shear stress for unreinforced concrete is given by (ACI, 1972):

$$f'_s = 2\sqrt{f'_c}$$

where, f'_s = concrete shear strength (psi)
f'_c = concrete compressive strength (psi)

For concrete with 25 MPa compressive strength, the allowable shear stress in the concrete is 839 kPa.

The shear strength of the rock mass can be estimated using the modified Hoek-Brown failure criterion, knowing the rock mass quality and the expected normal stress (Hoek et al., 1992).

Alternatively, design shear strengths for various rock qualities are provided in Table 2.

Rock Condition CSIR Rock Mass Rating	Design Shear Strength (kPa)	Allowable Hydraulic Gradient*
Very Good Rock Massive, hard, widely jointed 81<RMR<100	1500	15-30
Good Rock Hard to moderately hard, moderately jointed 61<RMR<80	900	10-14
Fair Rock Moderate to weak, moderately jointed 41<RMR<60	600	7-9
Poor Rock Weak, closely jointed or sheared 21<RMR<40	300	5-6
Poor Rock Very weak, possibly erodible RMR<20	150	3-4

*Allowable gradients can be higher if formation grouting is performed.
Table 2. Recommended design shear strengths and hydraulic gradients for tunnel plugs.

If the allowable shear stress in the concrete is lower than that of the rock, punching shear failure is controlled by the allowable shear stress of the concrete.

Plug length for static loading
For static equilibrium, the driving force equals the resisting force as shown below.

$$\frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{2(h+w)LU}{\rho gHwh} = 1$$

where
h = tunnel height (m)
w = tunnel width (m)
U = allowable shear strength rock mass, U should be substituted for fs if fs is lower
H = head of fluid on plug (m)
ρ = density of fluid (kg/m³)
g = gravitational constant (9.81 m/s²)

Example:
Consider a plug in a 4 m x 5m wide tunnel with a design head of 250 m of water. What plug length is required to resist punching shear failure?

U = 1500 kPa, fs = 839 kPa H = 4 m, w = 5 m,

Since $fs<U$, shear failure is controlled by the strength of the concrete. Solving for L,

$$L = \frac{(1000)(9.81)(250)(5)(4)}{(839,000 \text{ Pa})(2)(5+4)} = 3.25\ m$$

Applying a factor of safety of 3, the plug should be 9.75 m long.

Plug length for dynamic loading
The plug design should also consider loading from transient conditions such as earthquakes which can give rise to water hammer or material such as tailings or mud flowing up against the plug. Because these conditions are transient, a lower factor of safety can be accepted. It should also be noted that concrete and rock strength under dynamic loading is normally higher than for steady state loading.

Water hammer

If a plug is installed in a long tunnel, and an earthquake occurs, it may give rise to differential movement of the water and the surrounding rock. Since the water is restricted from flowing at the bulkhead, it is analogous to suddenly stopping the flow. Such a stoppage may cause a shock wave to propagate through the length of the tunnel and this shock wave could give rise to a much greater pressure on the plug. This phenomena is known as water hammer.

The effect of water hammer is modelled as a piston sliding in a cylinder filled with a fluid at rest. The additional pressure (P_H) required to set the fluid in motion to velocity (v) can be estimated by (Westergaard, 1931):

$$P_H = c(v)\rho$$

where:

c = acoustical velocity of water (1437 m/s)

v = ground velocity (m/s)

Normally, the design earthquake movement for a site is quoted in terms of an acceleration.

The relationship between v_{max} and a_{max} is given approximately by Seed and Idriss (1983) as:

$$\frac{V_{max}}{a_{max}} = 55 \, cm/sec/g \, (for \, rock)$$

Example: A plug is to be built in a long tunnel where the maximum credible earthquake has an acceleration of 0.4 g. What is the additional pressure on the bulkhead that may occur during the earthquake?

The ground velocity v_{max} = 55 cm/sec/g (0.4 g) = 22 cm/s

Therefore,

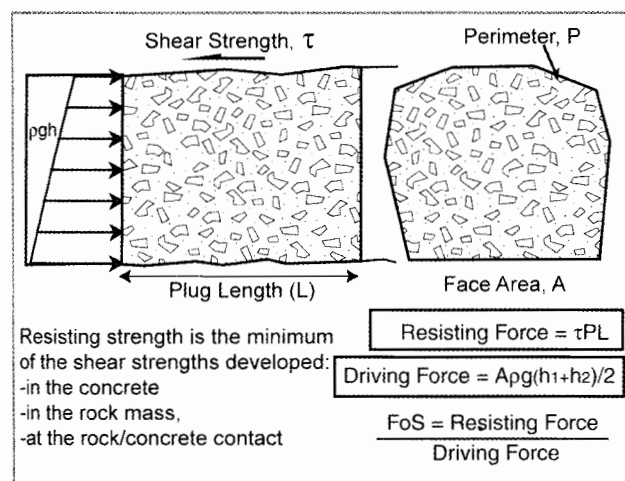$$P_H = (1437 \, m/s)(0.22 \, m/s)\left(1,0000 \, \frac{kg}{m^3}\right) = 316 \, kPa$$

Figure 3. Punching shear analysis.

Plain concrete deep beam bending design

If the design for punching shear strength results in a plug length that is less than the maximum dimension of the tunnel, the design should be checked for resistance to deep beam flexure. The following design process is based on the American Concrete Institute's "Building Code Requirements for Reinforced Concrete" (ACI 318-95). Because of the inherent difficulties in placing the concrete on the invert and crown of the tunnel, the beam should be conservatively assumed to be a one-way beam spanning between the walls. The one-way design assumption in effect adds an additional potential factor of safety of two, depending on the degree to which good bonding between the rock and concrete can be achieved through good construction practice and contact grouting (Abel, 1998).

The length of the unreinforced, concrete bulkhead must be sufficient to keep the tensile bending stress in the downstream face below the ACI allowable concrete tensile stress (f_t). The ACI code directs that a strength reduction factor of 0.65 be used in the design. The design tensile bending stress in the concrete should not exceed the allowable tensile strength of the concrete.

The required plug length (L) is given by:

$$L = \sqrt{\frac{6M_u}{b\,(f_t)}} \qquad f_t = 5\sqrt{f_c} \; (f_c \text{ in psi})$$

where,

Mu = Factored design bending moment;

B = unit height of beam; and

f_t = allowable tensile strength of concrete.

Hydraulic jacking

Hydraulic jacking is resisted when the minimum effective principal stress at the upstream face of the plug or any other part of the pressurised workings, exceeds the hydraulic head on the plug. The minimum principal stress (σ_3) can be conservatively estimated as the load due to the depth of rock cover above the tunnel.

A common rule of thumb for estimating the depth of cover requirements for pressure tunnels is that the depth of cover should be one-half the static head within the tunnel.

The Norwegian criterion is suitable for sloping topography. Precedent practice using the Norwegian cover criterion (Bergh-Christiansen and Dannevig, 1971) suggests a FOS between 1.1 and 1.3 is appropriate.

The minimum rock cover C_{RM} is given by:

$$C_{RM} = \frac{h_s \gamma_w F}{\gamma_R \cos\beta}$$

where,

C_{RM} = minimum rock cover measured to the nearest point on the ground surface;

h_s = static design water head;

γ_w = unit weight of water;

γ_r = unit weight of rock;

β = slope angle of topography.

Figure 4 illustrates the use of the Norwegian Tunnel Criterion for selecting the location of a tunnel plug. The criterion assumes the topography is constant in the direction normal to the plane of the section (plane stress conditions).

Allowable hydraulic gradient

In many cases, the length of the plug is governed by the allowable hydraulic gradient for the rock conditions that will prevent piping and downstream erosion of the tunnel walls or by a maximum acceptable seepage. The hydraulic gradient is defi-

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$$C_{RM} = \frac{\gamma_w h_s F}{\gamma_r \cos \beta}$$

where,

C_{RM} = Minimum rock cover measured from the nearest point on the ground surface
h_S = Static design water head
γ_w = unit weight of water
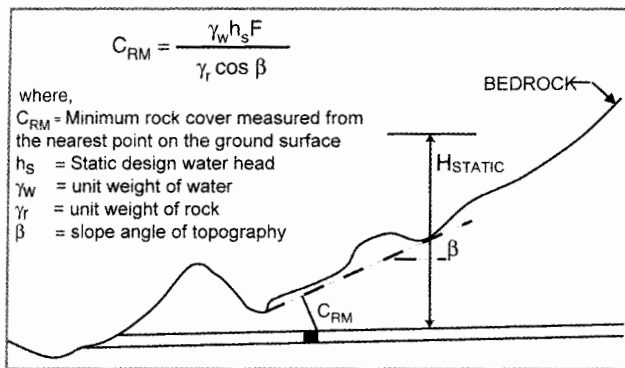γ_r = unit weight of rock
β = slope angle of topography

Figure 4. Norwegian cover criteria for unlined pressure tunnels.

ned as the design head of water divided by the length of the plug. In accordance with Darcy's Law, the seepage through the surrounding rock mass will increase proportionally with the gradient (although this is rarely the case in practice).

Seepage should never occur through a plug or along the rock concrete contact but seepage will always occur to some extent through the surrounding rock mass. The extent of seepage is a function of the hydraulic gradient across the plug, the hydraulic conductivity of the rock mass, and the degree to which formation grouting or upstream liners are used. An acceptable leakage criterion for permanent mine closure plugs is not well established. In South Africa, acceptable long term leakage through the rock mass immediately surrounding a plug ranges from 0.25 to 1 litre/second (Garrett et al., 1961). Coogan and Kintzer (1987) indicate that 2.5 litres/second was unacceptable for a hydroelectric diversion tunnel. Grouting work subsequently reduced the flow to 0.8 litres per second and this was considered acceptable. Researchers in South Africa have shown that for ungrouted plugs an allowable gradient of 38 is appropriate in competent rock (Garrett et al., 1961). Chekan (1985) suggested a maximum gradient of 56 for ungrouted plugs and estimated that grouting of the rock mass can reduce the thickness of the plug eightfold.

Allowable seepage

It is generally assumed that an acceptable leakage criterion for ARD plugs should be more restrictive than for other types of plugs. However, there are practical limits to how much the seepage can be controlled even with extensive formation grouting. The most important factor controlling the seepage is the natural permeability of the rock mass. Where the hydraulic conductivity of the rock mass is less than 10^{-5} cm/s, plugs can easily achieve seepage rates of less than 0.5 litres/second (measured 20 m downstream of plug). In rock masses of higher hydraulic conductivity, curtain grouting and formation grouting can be effective, but to a certain extent will simply alter the groundwater seepage, such that it enters the tunnel further downstream from the plug. There should be absolutely no seepage from the rock/concrete contact. Seepage for 1-2 metres downstream of the plug should be limited to occasional drips.

This should be considered an achievable target for most rock masses. Environmental regulators may impose stricter design criteria.

Finite difference groundwater modelling programs such as MODFLOW can be used to predict the amount of seepage if there is a good estimate of the hydraulic conductivity around the plug. MODFLOW is a three-dimensional finite difference modelling program developed and used extensively by the US Geological Survey. MODFLOW simulates steady and non-steady flow in an irregularly shaped flow system in which aquifer layers can be confined, unconfined, or a combination of confined and unconfined. Flow from external stresses, such as flow from reservoirs in the mine, areal recharge, evapotranspiration, and flow to drains (tunnels), can be simulated. Hydraulic conductivities or transmissivities for any layer may differ spatially and be anisotropic (restricted to having the principal direction aligned with the grid axes and the anisotropy ratio between horizontal co-ordinate directions is fixed in any one layer), and the storage coefficient may be heterogeneous.

Models can be used to quickly assess the relative effects of design changes such as:

- the effect of plug length on the predicted seepage;
- the effect of hydraulic conductivity on the predicted seepage; and,
- the effect of using an upstream liner or other measures to reduce seepage.

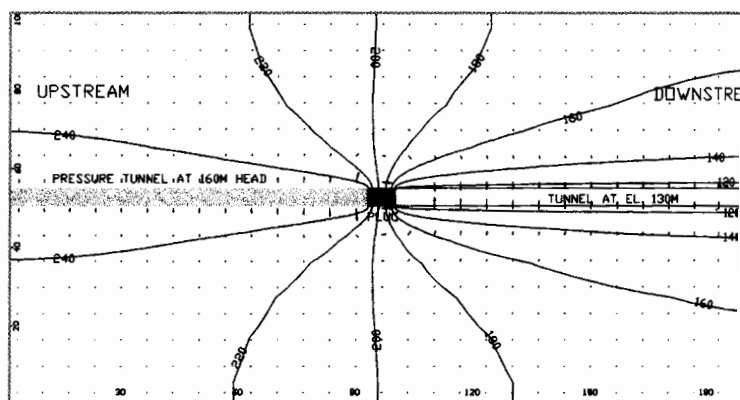Figure 5 shows the results of a MODFLOW analysis indicating flowpaths and equipotential lines.

Figure 5. View of modelled seepage paths and equipotentials around tunnel plug.

Chemical stability of concrete, grout and rock

If the plug is a permanent mine closure plug, the plug design should also assess the long term chemical stability of the concrete, grout, and rock formation. Concrete structures are normally designed for a 50 or 100 year service life so the concrete will require special admixtures to extend its service life. Very often the mine water impounded behind plugs can be acidic and high in sulphate. Both of these conditions are detrimental to concrete. A high fly ash (30-50% of cementitious) concrete is recommended for permanent plugs. A Type II portland

cement in combination with a Type F fly ash will provide low shrinkage and good sulphate resistance.

Cementitious grout is also susceptible to leaching. There are many reported cases where grout curtains beneath dams require re-grouting after the original grout had leached out. Slag based microfine grouts and silica fume additives can provide better resistance to aggressive groundwater than normal portland cement grout.

Where a plug is located in soluble rock or where there are joints with soluble infilling, great care must be taken to thoroughly grout these joints. Small leaks in soluble material will rapidly turn into large ones in these formations.

TUNNEL PLUG CASE HISTORIES

Examples of plug failures

Most tunnel plugs do not fail catastrophically, although two that have are presented below. A more typical type of failure is like that of the Chandler Tunnel described below where through poor construction or design, or unrecognized geological conditions, the plug performance does not meet expectations.

Tapian Pit Drainage Tunnel, Marcopper Mine

The Marcopper Mine is an open pit copper mine located on Marinduque Island, Philippines. Mining of the Tapian Pit was completed in 1991, and the pit was being backfilled with tailings from the nearby San Antonio Pit. When the Tapian Pit was being mined, a 2.5 m x 3.0 m tunnel was driven from the Makulapnit River valley to intersect the bottom of the pit to facilitate drainage. Before backfilling the pit, a 6 m long concrete tunnel plug was constructed approximately 100 m from the portal. At this location, the tunnel was 30 m below surface at El. 170 m. Backfilling of the Tapian Pit began in 1991 and by 1996, there were 21 million tonnes stored to El. 320 m.

In March, 1996, there was a sudden uncontrolled release of tailings from the tunnel. The tailings were fully liquefied and flowed full in the tunnel for several days. The tailings flowed intermittently for several months after while emergency grouting was carried out to try to stop the flow. Approximately 4 million tonnes of tailings were released. The tailings flowed into the Makulapnit and Boac Rivers and were carried 20 km down to the ocean. The mine was immediately shut down and at this date, the owners are continuing to deal with the environmental costs of this unfortunate incident.

Although the original plug site could not be accessed for safety reasons, the cause of the failure is believed to be hydraulic jacking of the rock surrounding the plug. The confining pressure at the plug location (equivalent to approximately 80 m of water) was too low to resist the hydraulic head on the plug. The rock surrounding the plug is categorized as poor to fair. The water likely jacked open joints in the rock mass surrounding the plug. Once opened, there would have been very high gradients locally, gradually eroding a direct path

around the plug. Once a direct path was developed, water would have flowed through the channel at a velocity perhaps exceeding 100 km/hr. This in turn would have led to rapid erosion and widening of the channel around the plug. This unfortunate occurrence illustrates the importance of carrying out a thorough site investigation and assessing all possible failure modes in the design.

Merrispruit Mine, South Africa

During shaft sinking at the Merrispruit Mine in South Africa, uncontrollable water inflows were intersected at a depth of approximately 1500 m. A parallel plug was installed some distance up the shaft to prevent flooding of crosscuts and another shaft. The plug was 18 m long and there were four 250 mm diameter pipes cast into the concrete with valves to control the flow. The pipe sections were approximately 10 m long and had flanged joints approximately halfway through the plug. The plug concrete was poured as the water was rising in the shaft and it was only about six days old when the water reached the plug. Soon afterwards, the plug failed by punching shear failure. The 9 m long top half of the plug was suddenly pushed about 100 feet up to the next sublevel. About 90 men working the shaft lost their lives.

It was determined by an inquiry that as the concrete cured, it rose the temperature of the concrete sufficiently to affect the gasket in the flanged joint, which was made from a bitumen-like material. The gasket failed, which in turn caused hydraulic fracturing of the weak concrete. The plug was then half the design length and was not sufficiently long to resist the water pressure, which was about 15 MPa.

#1 Chandler Tunnel, Colorado

Abel (1998) reports that initial efforts to seal the #1 Chandler Tunnel at the Summitville Mine in Colorado were unsuccessful because geological conditions were not adequately accounted for in the design. The plug was located a distance of 100 metres from the portal at a depth of 29 metres. The maximum head developed on the plug was 52 metres. The tunnel was approximately 2.4m wide by 2.4m high. The plug was constructed to be 2m long using dry mix shotcrete with steel reinforcement on the upstream and downstream ends of the plug. The plug had a 150 mm diameter stainless steel pipe within it to carry the water during construction.

Subsequent triaxial compressive testing of a part of the rock surrounding the plug indicated a friction angle of only 7° and cohesion of 57 psi. The plug leaked at a rate of 10 gpm, which was considered excessive. As a consequence, formation grouting was carried out in the rock a distance of 6m downstream of the existing plug and a 6m extension was added to the plug. After completion of the extension, the seepage was reduced to zero.

This case history demonstrates the need for a thorough site investigation to assess the geotechnical and hydrogeologi-

cal properties of the proposed plug site. Such studies can often reduce the need for extensive formation grouting which is often the most expensive part of the tunnel sealing project.

Successful tunnel plug examples

Successful mine sealing and flooding is achieved if the flow downstream of the plug is within accepted limits when the design head on the plug is achieved. Although the plugging will immediately reduce the drainage from a portal, the water quality will likely decrease initially as exposed workings are flooded. Depending on the rate of seepage into and out of the flooded workings, the water quality may take several years to improve.

Keno Hill Plug, Yukon

The Galkeno Mine, part of the Keno Hill silver camp in the Canada's Yukon Territory operated from the late 1950's to the mid 1960's. The Galkeno 900 adit was driven to test the down dip extension of the upper Galkeno workings. Acidic drainage from the mine flows out of the Galkeno 900 adit and the company was requested to seal the tunnel to reduce the flow and flood approximately 240 vertical m of the mine to reduce further ARD generation. The original flow from the adit was approximately 7 l/s and contained 35-40 ppm zinc.

The plug was located approximately 365m inside the portal at a depth below surface of approximately 200m. The design head on the plug was 244 m, which is the distance to the next highest adit. The tunnel is 2.4 m wide and 2.4m high. The plug was constructed in 1994 and consisted of a 10 m long radial grout curtain at the upstream end of the plug, consolidation grouting of the rock mass along the length of the plug, a 7.5 m long parallel plug, and contact grouting. Curtain grouting and consolidation grouting was carried out prior to placing the concrete. The plug was located in closely jointed rock with jointing striking sub-parallel to the axis of the adit. More geologically favourable sites were not available. It was anticipated that seepage around the plug would be high.

Despite a considerable amount of grouting work, there was still significant seepage through the rock mass when the plug went into service. Nonetheless, the mine workings are known to have flooded since there has been an increase in flow from the upper portal. Since construction, there has been a gradual decrease in the concentration of zinc to approximately half of its original level. The flow from the portal is approximately 3-4 l/s. It is estimated that a half of this flow can be attributed to seepage around the plug though the jointed rock mass.

Tapian Pit Drainage Tunnel Plug (New Plug)

After the failure of the original Tapian Pit Drainage Tunnel Plug (described above), a new plug was constructed following a very detailed site investigation. A site was selected based on the original mapping of the tunnel as well as geotechnical boreholes drilled from surface. The plug site was also selected where the depth of cover at all points upstream of the

plug was adequate to resist hydraulic jacking. The confining stress was confirmed by hydraulic jacking tests carried through boreholes drilled from surface to a distance no closer than 6 m from the tunnel.

The water in the tunnel was flowing at a rate of 50 litres/s prior to construction of the new plug. A cofferdam was constructed to collect this water into a 300 mm cement lined ductile iron pipe. A 3m thick safety bulkhead was quickly constructed upstream of the true plug location to protect workers in case tailings in the pit and the tunnel were remobilized. This safety bulkhead became the upstream form for the actual plug. The hydraulic conductivity of the rock mass was profiled using a double packer assembly. The testing indicated that the hydraulic conductivity was highest closest to the tunnel wall and that it approached the background hydraulic conductivity about 5 m from the tunnel. Radial consolidation grouting was carried out to a depth of 5 m radially from the tunnel prior to placing the concrete. A slag based microfine cement grout was used for the injection grouting to penetrate the very fine jointing in the rock. A total of 8 tonnes of microfine grout was injected.

A 15 m long radial grout curtain was installed at the upstream end of the plug to form a seepage cutoff and to effectively increase the length of the seepage path around the plug. The grouting was carried out using a split spacing and descending stage sequence to gradually reduce the hydraulic conductivity of the rock mass to 1 Lugeon (10^{-5} cm/s). A total of 24 tonnes of microfine grout was injected to create the grout curtain.

The design head on the plug was 201 m. An allowable gradient of 10 was selected for the rock mass, resulting in a 20 m long plug.

The concrete mix for the plug was designed to have the following properties: low heat of hydration, low shrinkage, low settlement, good pumpability, low segregation during placement, good resistance to sulphate attack and alkali-silica reactivity. The mix design is provided in Table 3.

Material	Units	Quantity
Portland Cement, ASTM C150 Type II	kg/m³	220
Fly Ash, ASTM C618 Type F	kg/m³	143
Coarse Aggregate	kg/m³	670
Fine Aggregate	kg/m³	1070
Free Water Added		0.42 w:c
Water Reducing Retarder	l/m³	1.5 – 1.8
Superplasticizer	l/m³	1-3
Air Entraining Agent	l/m³	1-3

Table 3. Tapian Pit Drainage Tunnel Plug concrete mix design.

To protect the plug concrete from aggressive groundwater, the upstream form was covered with 3 layers of a composite bentonite geotextile.

Four thermocouples were embedded in the concrete to monitor the temperature rise and subsequent cooling in order to guide when contact grouting should begin. Contact grouting was carried out when the temperature of the concrete core cooled to within 5° of the rock temperature. Contact grouting was carried out by drilling diamond drill holes through the contact from the downstream end of the plug. Holes were grouted to refusal at a pressure of 3 Mpa.

As part of the plug design, a finite difference seepage model was carried out to estimate the amount of seepage through the rock mass that could be expected. This work predicted seepage of 11 l/s into the tunnel downstream of the plug with 100 m of head. The measured seepage was 6 l/s when the head was at 100 m.

The new Marcopper plug also includes a graded soil filter downstream of the concrete section. The purpose of the filter is to act as a second plug to retain tailings in the tunnel even if the concrete deteriorates after hundreds of years. The sand meets standard geotechnical filter criteria for the tailings. A schematic cross section of the new Tapian Pit Drainage Tunnel Plug is shown in Figure 6.
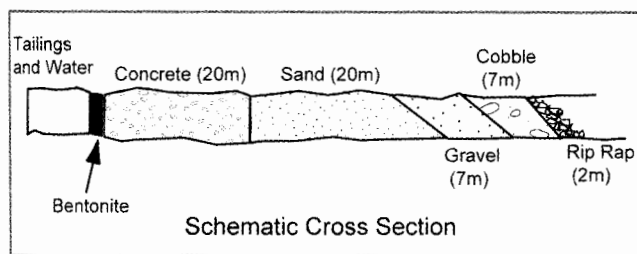

Figure 6. New Tapian Pit drainage tunnel plug.

CONCLUSIONS

Flooding of an underground mine located in mountainous terrain involves sealing all major sources of leakage including drill holes, major geologic structures, and adits. Long term sealing of mine adits can be effectively achieved using unreinforced concrete plugs. The allowable hydraulic gradient, the allowable seepage, and the shear strength of the concrete and surrounding rock mass govern the length of the plug. Plugs must also be located sufficiently far in a mine such that the confining stress exceeds the design head on the plug to prevent hydraulic jacking. To resist degradation of the concrete in contact with ARD, it is recommended that a Type F fly ash in combination with Type II portland cement is used in the concrete mix.

A thorough geotechnical and hydrogeological assessment is recommended for any permanent plug. These studies will ensure the plug is located in the best possible location to ensure the effectiveness of the seal and to reduce construction costs.

The best design in the world can be useless unless there are strict quality assurance controls during construction. It is strongly recommended that the design engineer be on-site during construction to modify the design as conditions dictate and to ensure that the final as-built plug conforms to the design objectives. Plugs should be monitored occasionally for a period of several years after the design head is achieved.

The guidelines presented above are considered appropriate for tunnel plugs requiring a minimum 100 year service life. Where a longer term is required, the designer must consider measures above and beyond the standard requirements presented. These measures may include, installation of impermeable coatings upstream of the plug to reduce seepage, a longer plug length, and/or a clay core "earth plug" analogous to an earthdam.

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