

IMPACT OF ZERO BOND TO ROCK ON SHOTCRETE LINING DESIGN

Erik Stefan Bernard¹ and John C. Braybrooke²

¹ *Institute for Sustainable Industries and Liveable Cities (ISILC), Victoria University, Werribee, Australia*

² *Douglas Partners, Geotechnical Consultants, Sydney, Australia*

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ABSTRACT

Shotcrete linings usually achieve good bond to the underlying rock substrate when first applied during construction. This bond usually remains sound through many years of service. However, in some cases the bond may be inadequate or diminish with the passage of time causing the lining to detach from the substrate. To prevent detachment, and possible collapse in these circumstances, a shotcrete lining needs to be secured in place using a secondary means of attachment which, in the case of linings on hard rock, usually comprises rock bolts with external cover plates or spider plates on the ends of bolts. The potential causes of bond loss are many both prior to spraying and in service. Testing of bond strength frequently leads to apparent ‘zero bond’ results that may reveal an actual absence of bond or may be an outcome of the coring and testing process. If bond is relied upon in design it is important to consider the significance of zero bond results rather than simply dismiss them as artefacts of the testing process. This paper considers how partial or total bond loss can be considered in shotcrete lining design.

1 INTRODUCTION

Shotcrete linings on hard rock tend to be relatively thin. This is because their function is usually limited to supporting potentially unstable ground between adjacent rock bolts and possibly acting as a canopy for diversion of groundwater to drains. Since rock bolts act as primary support to the ground, the structural loads imposed on the shotcrete lining are relatively small. Nevertheless, the lining is often required to support its own weight in addition to a small surcharge associated with locally unstable ground and excess groundwater pressure, while remaining serviceable throughout the design life of the tunnel (usually 100 years). Shotcrete is also frequently used for patch repairs and local coverage of features such as drains.

The capacity of a lining to support the relatively minor loads imposed on it can be enhanced if bond to the ground is maintained. Based on the method of shotcrete lining analysis described by Barrett and McCreath (1995), the governing mode of load resistance for a lining that is bonded to competent hard rock is likely to be shear in most cases. This is because flexural failure is highly unlikely when the lining is bonded to and acts in composite action with the substrate. To ensure a shear-based failure mode, satisfactory bond to the ground must be confirmed. This is normally done by testing the in situ bond strength between the lining and substrate at selected points throughout the works. However, experience has indicated that the measured in situ bond strength often falls below the required minimum, partly due to the inherently high variability in results, but also due to an apparent absence of bond in some locations evidenced by the high incidence of ‘zero bond’ results.

Zero bond results may indicate an actual absence of bond (e.g., Figure 1) or may be due to the coring and testing process. Simply dismissing zero bond results as an artefact of the testing process is an unconservative approach that would never be tolerated for other structural parameters such as compressive strength. One therefore needs to consider the causes of zero bond more thoroughly and investigate the effect that an actual absence of bond has on the structural resistance of a shotcrete lining.

2 THE SIGNIFICANCE OF ZERO BOND RESULTS

The use of pattern bolting and shotcrete has long been a common form of ground support in tunnels with authors such as Barton et al (1974) and Bieniawski (1974) summarising extensive case histories relating the degree of support required to various rock mass classification systems. These papers high-light that very thin (50mm +/-) layers of shotcrete are widely used in better rock mass conditions. The introduction of fibre reinforced shotcrete as a replacement for mesh reinforced shotcrete was accompanied by a change toward a more deterministic method of lining design and thickness estimation. Such an approach to the design of lining thickness was described by Barrett & McCreath (1995) who stated:

“If adhesion is maintained, then the failure of the shotcrete will be controlled by direct shear. If adhesion is lost...then, and only then, does the flexural failure mechanism become kinematically possible”.



Figure 1: Examples of delamination zones evident in core holes a) inside shotcrete, and b) at the shotcrete/substrate boundary, leading to zero bond strength results.

For the case of a shear-based resistance mode, the zone immediately around a loaded area is taken to support the load through direct tension between shotcrete and ground at the boundary around the unstable rock (Figure 2). Thus, shear failure of the lining will only occur if adequate bond in direct tension is present. If bond is inadequate the lining will suffer adhesion loss followed by failure in bending.

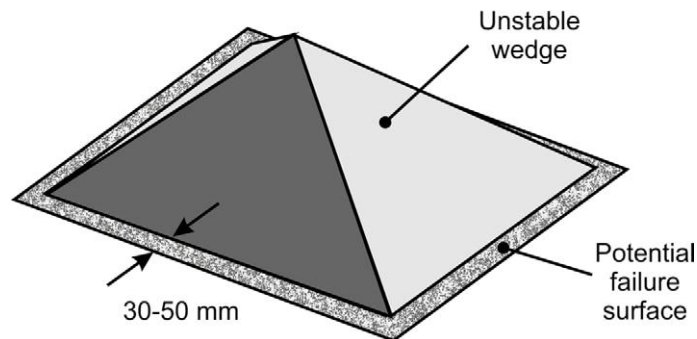


Figure 2: Bond failure zone around falling wedge, according to Barrett and McCreath (1995) based on work by Fernandez-Delgado et al (1975) and Holmgren (1987).

The significance of adhesion is illustrated by the following example in which lining capacity is assessed according to the method by Barrett and McCreath (1995). For a lining of thickness t and orthogonal bolt spacing s , with shotcrete shear strength τ , the direct shear resistance V around a square failure zone (Figure 2) is found as

$$V = 4\tau st \tag{1}$$

The punching resistance V_p around surface-mounted bolt cover-plates or bolts with a positive means of attachment to the shotcrete such as mesh or handle-bar plates, is found as

$$V_p = 4\tau t(p + t) \tag{2}$$

where p is the plate perimeter length. In contrast, the flexural load resistance will depend on the moment resistance m of the shotcrete, which can be expressed as

$$m = f_i t^2 / 6 \tag{3}$$

for a flexural strength of f_i (taken at either first crack or post-crack). If the load is taken to act as a point load at the centre of a square yield-line failure zone, the load resistance P_{PL} will be

$$P_{PL} = 16m \tag{4}$$

but if the load is taken to act as a Uniformly Distributed Load (UDL), the load resistance P_{UDL} will be

$$P_{UDL} = 24m/s^2 \tag{5}$$

where the failure zone is taken to comprise a square with negative yield lines at the supports and positive yield lines across the diagonals. Importantly, the adhesive load resistance P_a along the perimeter of the unstable wedge is found as

$$P_a = 4sw_a f_b \tag{6}$$

where w_a is the width of the adhesion zone and f_b is the bond strength. It is presently assumed that the rock fails as a pyramid-shaped wedge with a 60° side inclination and density equal to 24 kN/m³.

If we take $\tau = 2.0$ MPa, $f_t = 3.0$ MPa, $f_b = 0.5$ MPa, cover plate side length to be 200 mm, and assume the adhesive width w_a around the unstable wedge to be equal to the thickness of the lining, we obtain the series of plots of Factor of Safety (FOS, based on unadjusted resistance/load) against bolt spacing s for the failure modes represented by Equations 1 to 6 shown in Figure 3. The direct shear resistance (solid black line) is clearly greater than the load to cause adhesive failure (red line) around the periphery of the unstable wedge. Punching around the cover plates offers a lower FOS but does not have an adhesive failure limit when the plates are on the outside of the shotcrete lining or are physically attached to the shotcrete. In contrast, the flexural modes of failure offer a lower degree of load resistance. For the case shown, adhesion to the substrate is maintained across the entire range of bolt spans so the lining can be taken to support the wedge in shear. However, there may be some problem with punching around the bolts for spacings greater than 2.2 m. Similarly, if the peripheral bond strength is diminished in some way then the likelihood of flexural failure is increased. A similar set of curves can be generated for other wedge shapes and failure zones.

The shear-based failure mode around unstable blocks and wedges observed by Fernandez-Delgado et al (1975) and Holmgren (1987) (which subsequently formed the basis of the design method by Barrett and McCreath (1995)) requires that a satisfactory bond exists at the boundary to support the load. The ground load associated with an unstable wedge or block is supported by the bond at the shotcrete/rock interface around the perimeter of the loaded area (Figure 2). Holmgren (1987) approximated the highly loaded width of this zone to be about 30-50 mm while others have taken the critical width to be larger than this (Sjölander et al, 2021). If de-bonded areas exist within this highly loaded boundary zone, then only part of the perimeter is available to support the wedge or block. There is no way of identifying in advance whether the boundary of an unstable wedge will coincide with a de-bonded portion of a lining or not. It is therefore important that the *probable incidence* of de-bonded zones be identified, and that the impact of zero-bond areas be accounted for in estimating the average bond strength that can be used to calculate the carrying capacity of the lining.

As an example, assume that 100 bond tests are conducted in a length of tunnel lining using a laboratory-based test such as AS1012.24 [6] or the EFNARC Specification (1996). Assume 50% of the extracted cores produce a zero result (due either to an initial absence of bond or ‘breakage’ during drilling), but the remaining 50% of tests that resulted in an intact core indicate an average bond strength of 500 kPa. If the ‘zero’ results were, in fact, the result of poor drilling that led to breakage of the cores, then the 500 kPa estimate of bond strength could be valid. However, if the drilling was conducted in areas where some of the shotcrete had de-bonded from the substrate prior to drilling, the average bond strength may be 250 kPa rather than 500 kPa. It is therefore necessary to record all incidences of ‘zero’ results whenever *in situ* pull-tests are conducted or cores are extracted for testing in the laboratory and attempt to identify the cause of each zero result.

Table 1: Incidence of zero results in published coring programs, and recent Australian projects.

Sources	Method	Age	Surface Prep.*	No. of Cores	Zero results (%)
Malmgren et al (2005)	SS137342	2 yr	Grinding	45	44
	SS137342	2 yr	Hydro-scaling	25	12
NIOSH (2010)	Custom	7 d		185	5
Holter (2015)	Custom	14 d	Grinding	80	0
Sandstone	SS137342	17 yr	Grinding	85	53
	EN1542	17 yr	Grinding	24	50
	AS1012.21	4 yr	Grinding	127	53
	SS137342	4 yr	Grinding	28	78

* Surface preparation before testing

As shown in Table 1, the frequency of zero results can be very high and, as a consequence, will affect the average bond strength if the zero values are attributable to actual de-bonding of the lining. The data in Table 1 also suggests that the incidence of zero bond results increases with the age of the lining possibly indicating that a time-dependent process such as drying shrinkage or thermo-hygral cycling could be responsible for the apparent age-dependent bond loss.

3 SIZE EFFECT FOR BONDED LININGS

Shotcrete, rock, and the boundary between them are all quasi-brittle materials that are affected by the presence of flaws (whatever their scale). Fracture science has shown that as the size of a loaded member increases, it is more likely that points of weakness will exist within the zone subject to tensile stress with the result that the apparent mean strength will fall (Neville, 1996; Bažant and Planas, 1998; Pan et al, 2022). The high incidence of zero bond results obtained in field testing indicates that bond strength is a parameter that exhibits both considerable variability and frequent points of weakness (Table 1). Unfortunately, all the bond tests presently available for shotcrete linings were conducted using specimens of about the same size (64-75 mm diameter). It is therefore difficult to identify a size effect using these results. However, Shu et al (2024) conducted laboratory tests on samples of young concrete cast against old concrete to examine the relation between various factors and size effect. They found that for direct tension specimens of between 70 and 200 mm cross-sectional width there was an approximate 45 percent fall in mean tensile strength between the smallest and largest specimens for relatively smooth interfaces. Based on a Weibull model of statistical size effect, this would extrapolate to a 48 percent fall in tensile strength for members with a 1000 mm cross-sectional length relative to a 100 mm core test. If a Bažant model of energy-based size effect is used, then the fall in mean tensile strength will be about 53 percent for a 1000 mm wide tension zone compared to a 100 mm core test (2024).

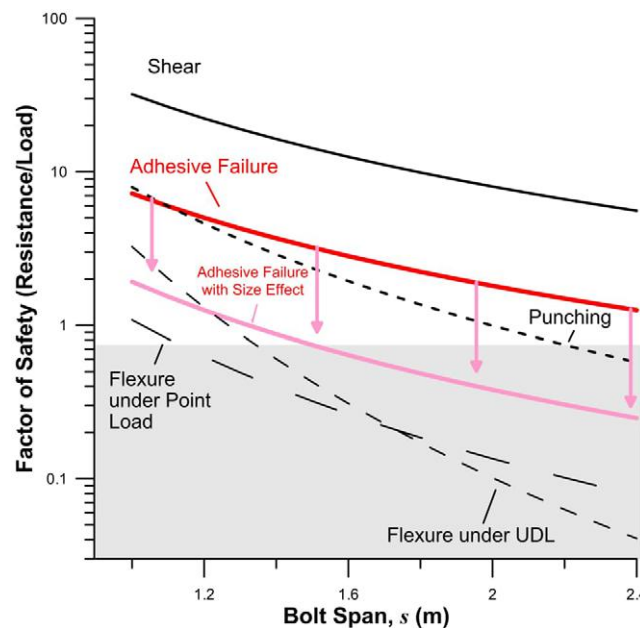


Figure 3: Factor of Safety on failure by various modes for a 75 mm thick shotcrete lining of between 1.0 and 2.4 m bolt span, calculated using method by Barrett and McCreath (1995).

As described later in this paper, there are numerous potential causes of zero or near-zero bond strength at the shotcrete/substrate boundary so their occurrence cannot be dismissed. Local flaws will diminish the effective bond strength of a large area of shotcrete loaded in tension. When local flaws affecting strength are present it is appropriate that size effect be accounted for by factoring down the mean bond strength obtained in tests as the size of the loaded boundary between shotcrete and substrate increases. For engineering assessments of bond in shotcrete linings we are generally not concerned with differences between statistical size effect, energy-based size effect, and other phenomena, thus a relatively simple size effect model is most appropriate. For a limited data set of bond specimens, the size-corrected tensile bond strength f_b for a wedge falling through a shotcrete lining (as per the Holmgren tests (1987) and Barrett and McCreath design method (1995)) can be approximately related to the apparent bond strength f_{test} obtained in testing by the simple expression $f_b = \alpha f_{test}$ in which α is a size correction factor of the form:

$$\alpha = \left(\frac{l}{l_0}\right)^{-\frac{1}{n}} \quad (7)$$

where l is the length of the side of an unstable wedge, l_0 is a characteristic reference length (e.g. the diameter of a tensile test core), and n is about 3 (see Bažant and Planas, 1998) depending on the characteristics of the process under consideration (for example, the heterogeneity of the material, the rate of strain, and duration of loading). Clearly, the proportion of ‘zero bond’ results obtained in bond tests on specimens of differing size will strongly influence the magnitude of n but insufficient information is presently available to determine the exact magnitude of n . Nevertheless, as an example, if $n = 3$ one can estimate that a bonded area of 1000 mm side length exhibits an average tensile strength f_b about 0.42 times the mean apparent tensile strength f_{test} obtained for a 75 mm diameter core extracted from the same area. Thus, using this bond strength adjusted for size effect the load to cause adhesive failure around an unstable wedge will be diminished, as shown by the pink line in Figure 3. In this case, when the bolt spacing exceeds 1.5 m, the lining will experience adhesion loss and flexural failure will occur in the lining.

The form of the size effect relationship for shotcrete bonded to a rock substrate should be examined in greater detail when more comprehensive information about the incidence of zero bond results and the effect of size becomes available.

4 THE SIGNIFICANCE OF ADHESIVE FAILURE

The widely used method of lining design by Barrett and McCreath (1995) is based on falling block tests conducted by Holmgren (1987) and Fernandez-Delgado et al (1975). These tests examined the behaviour of shotcrete linings in response to the intrusion of falling blocks into a tunnel space. Some of their tests extended to examination of post-peak behaviour in which the bond strength in the zone around the periphery of a falling block was exceeded, a process now known as ‘adhesive failure’. All such tests resulted in a rapid fall in load resistance as the shotcrete lining around the falling block proceeded to peel off the substrate. Following bond failure, the laboratory linings resisted load purely in flexure as they spanned between the edge of the falling block and the nearest point of support (such as a rock bolt, see Figure 4a). Since the flexural resistance of shotcrete linings is usually relatively low compared to shear resistance, the onset of adhesive failure leads to strongly deflection-softening behaviour and may result in the falling block breaking through the lining.

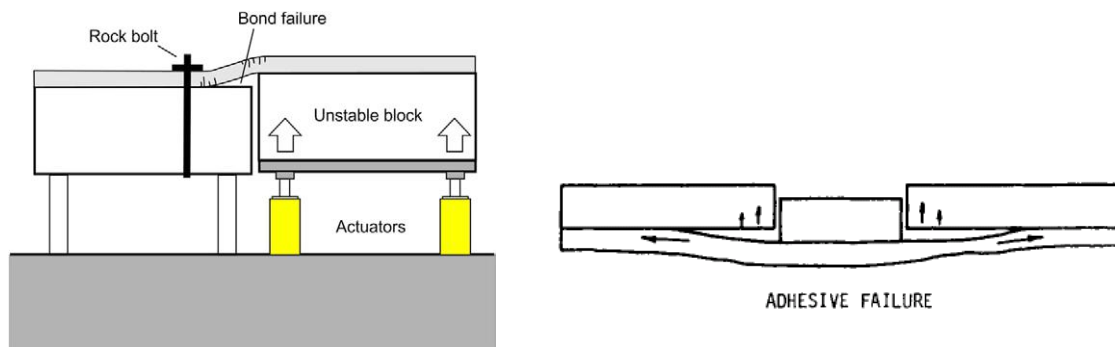


Figure 4: a) Adhesive failure in falling block test used by Holmgren (1987) to assess shotcrete lining capacity, and b) Fernandez-Delgado et al (1975) tests to assess shotcrete lining capacity.

Adhesive failure primarily involves separation of the lining and substrate by peeling action (Figure 4b). Peeling is the process by which tensile failure occurs at an adhesive surface that is eccentrically loaded. Eccentric loading is known to have a deleterious effect on the apparent tensile strength of concrete; that is why bond tests such as EN1542 and SS137243 have been developed to eliminate eccentric loading and achieve an ‘optimal’ bond strength estimate (see the Appendix for a more detailed discussion of available bond test methods and their limitations). However, ground loads are usually applied to shotcrete in an eccentric manner with the result that a peeling mode of failure is more likely than direct tension. Moreover, peeling is more likely to occur at an interface of regular geometry (typical of excavations made using a road header or TBM) than irregular geometry (such as blasted rock surfaces) because geometrically regular surfaces exhibit fewer physical obstacles to the propagation of a long splitting crack.

The significance of peeling is that the load resistance that can be sustained by a lining suffering peeling-based adhesive failure is much lower than can be expected if the load was supported in direct tension at the bonded interface. Although tests of peeling resistance between concrete surfaces are rare, peeling between concrete and bonded fibre board has been

examined by several researchers and found to result in an approximate 80-90% fall in apparent tensile bond strength compared to direct tension (Dai et al, 2007; Alam et al, 2012; and Zhang 2014). It is therefore reasonable to conclude that a lining subject to adhesive failure will support very little residual load at the substrate boundary and instead will primarily resist load through flexure between the falling wedge and points of anchorage (such as rock bolts with plates on the outside of the shotcrete, handle-bar plates, etc).

The bond strength at the shotcrete/substrate interface in regions beyond the 30-50 mm wide boundary around a falling wedge cannot be relied upon for residual support due to the low resistance of the peeling mode of failure that would be induced in these areas. Moreover, it is self-evident that a pre-existing de-bonded area adjacent to a falling wedge will degrade the load resistance of the lining because it will promote peeling action rather than resistance in direct tension. For these reasons, the magnitude of bond strength achieved at the periphery of a potentially unstable block must be maintained during the life of a lining. Zones of de-bonded lining will increase the likelihood that an unstable block will coincide with low-resistance zones thereby increasing the likelihood of a flexural mode of failure. Since it is not possible to predict in advance where zones of weak bond may exist, it is prudent to control this risk by either applying a size effect factor to the bond strength or designing the entire lining as if no bond exists anywhere at the shotcrete/substrate boundary.

5 POSSIBLE CAUSES OF POOR BOND

Bond between shotcrete and a substrate such as rock is achieved by chemical and physical means as soon as the shotcrete is sprayed onto the surface. Long-term bond is created by hydration of the cementitious components within shotcrete paste forming tensile linkages with the substrate material, and by physical interlock between the hardening cement and the rough substrate surface. Under ideal conditions, the high speed of impact between an impinging jet of shotcrete and the substrate combined with the relatively high strength typically achieved by a shotcrete paste produce a bond between lining and substrate of between 0.5-2.0 MPa in direct tension (Hahn and Holmgren, 1979). However, the actual bond strength achieved will vary with many factors, including the composition and surface roughness of the substrate, the strength of the shotcrete and substrate, cleanliness of the substrate, and quality of spraying. If bond is relied upon for effective operation of a lining or retention of local shotcrete features such as patch repairs or drains, it is important that the designer understands the factors that influence long term retention or loss of bond to the substrate.

5.1 DISRUPTION OF BOND DURING CONSTRUCTION

There are numerous factors that can potentially reduce or eliminate the bond between shotcrete and its substrate during construction. Based on the authors' many years of experience in lining construction and inspection, these detrimental factors can be summarised as:

Insufficient cleanliness of the substrate. Even though the rock is usually washed prior to spraying it is inevitable that there will be some areas which will remain contaminated with dust and other unconsolidated material that limit the development of effective bond.

Surface contamination with oils, other hydrocarbons, or set accelerator. This often occurs during spraying due to leaks from machinery and poor control of accelerator.

Variations in the *roughness of the substrate*. This depends on rock type and excavation techniques. Friable rocks such as shale or schist will exhibit very poor adhesion due to fragmentation within the rock.

Deleterious *mineral composition* of the substrate. In rock, the presence of clay, mica, talc, or chlorite minerals may lead to expansive reactions or poor adhesion, resulting in degraded bond strengths.

Poor spraying technique. The spraying nozzle should always be perpendicular to the substrate surface and at an optimum distance from the surface, normally between 1-2 metres. However, if in some areas the nozzle happens to be further away the shotcrete will impact the substrate at a lower velocity thus affecting adhesion. In other areas where the nozzle is too close the shotcrete will exhibit excessive rebound. Both these processes can diminish the bond achieved.

Seepage of groundwater from the rock. Groundwater seepage should be controlled and channelled away from the area of work prior to the application of shotcrete to the substrate. Excessive groundwater that is not channelled away may compromise adhesion in that area.

Excessive dosage of set accelerator in the shotcrete can have a substantial detrimental effect on bond. Excessive use of set accelerator can also produce layers of weak shotcrete within the lining (Figure 1) that act both as potential delamination surfaces and channels for water ingress through the completed lining. Excessively accelerated shotcrete that hardens before it impacts the substrate is more likely to bounce off and interfere with the motion of trailing airborne concrete particles further reducing the effectiveness of compaction.

Poor mixing of the supplied concrete that creates lenses of hydration stabiliser or paste-free material within the as-placed material can lead to sloughing and detachment of wet shotcrete.

Poor mix designs that lead to low cohesion within the shotcrete also diminish bond to the substrate; this can cause internal detachment of the wet lining from the substrate (which is hard to detect) or fall-outs.

Bond disruption may also occur due to ground movement in the vicinity of the newly sprayed shotcrete lining. Ground movement may be caused by relief of pre-existing stresses in the ground because of excavation of the tunnel space, or by break out of the ground or nearby temporary shotcrete structures as part of the excavation process. For example, the practice of spraying a primary shotcrete lining close to the face during double- or triple-heading excavation sequences and then breaking out parts of the temporary shotcrete to complete the excavation is highly likely to disrupt the bond between the ground and the adjacent primary lining. Similarly, a heading and bench excavation in a high horizontal stress environment will cause additional convergence of the sidewalls. In an excavation with a flat roof this can induce pressure within a thin shotcrete lining in the roof, causing it to shear or buckle. In these circumstances it is prudent to either remove the compromised shotcrete and apply a new lining to the rock or include a secondary means of attachment between the primary shotcrete lining and ground (such as handle-bar plates or spider plates on the ends of the rock bolts) to anticipate the bond disruption likely during the excavation sequence. Surface preparation techniques also influence the achievement and retention of bond in local repairs (Bissonette, et al, 2012; Reny, 2013).

Regardless of the cause, the outcome of bond disruption is areas of low or non-existent bond between the shotcrete lining and substrate that can affect the load resistance and serviceability of the lining. The size of de-bonded areas can range from a few square centimetres to several square metres.

5.2 BOND DISRUPTORS SPECIFIC TO ROAD HEADERS

There are several phenomena that occur during excavation of ground using road headers that warrant special attention.

Each of these phenomena are associated with the characteristic grooves that are left in the surface of the rock following excavation by road header (Figure 5).

Formation of Cracks in Grooves. The high compressive stress applied to the rock face at the tip of each excavation pick can generate cracks within the rock that remain after the pick has passed. These cracks can then act as initiation points for the growth of larger cracks if the face of the rock is subject to sustained tensile stress.

Rock Flour in Grooves. The high compressive stress acting at the tip of each excavation pick can also grind the constituents of the rock into a flour that remains as a loose surface coating to the more competent rock underneath. Recent investigations of mature shotcrete linings on sandstone excavated by road header have revealed grooves in the substrate filled with silica flour. This material requires careful cleaning to remove but this clearly does not occur in all cases.

Accumulation of Over-spray in Grooves. Even if the surface of the excavated rock is carefully cleaned prior to spraying, the spraying process itself can lead to over-spray associated with the shotcrete jet accumulating as loose debris in adjacent grooves (Figure 5). This over-spray consists of small loose particles that rapidly dry out and form an unconsolidated layer at the base of each groove that later acts as an initiation point for cracks.

5.3 IN-SERVICE DISRUPTION OF BOND

Even if good bond is initially achieved between shotcrete and substrate, in-service conditions can degrade the magnitude of bond strength or extend the size of existing de-bonded regions. Among the most important contributing factors to in-service bond disruption are:

Ground movement. On-going ground movement following the initial re-distribution of stress that accompanies excavation may occur in some cases due either to nearby excavations or changes in ground conditions such as time-dependent loss of strength of thin layers under stress (Oliveira and Diederichs, 2017) or creep of highly stressed ground.

Drying shrinkage. Shotcrete exhibits a high degree of inherent drying shrinkage that is often made worse by poor curing practices. Experience has shown that most shotcrete tunnel linings experience significant shrinkage cracking. Differential shrinkage between the surface and interior parts of a lining can induce curling at cracks and other boundaries that may lead to de-bonding (Sjölander and Ansell, 2017). This is a slow process that can take years to manifest itself.

Thermo-hygral Cycling. Seasonal variations in temperature and humidity can drive thermal or moisture gradients between a shotcrete lining and its substrate that generate shear stresses at the boundary between these elements. This type of

environmental cycling can exacerbate crack widths (Holter, 2015) and promote progressive de-bonding by a similar mechanism to that associated with drying shrinkage (Silfwerbrand, 1997).

Chemical and physical degradation of the shotcrete or substrate. Excavating a tunnel through ground introduces oxygen to a previously anaerobic environment. This will lead to chemical changes, including acidification of some minerals, that may subsequently attack the shotcrete lining and substrate rock. The shotcrete may also chemically interact with the ground because of the introduction of an alkaline material to a previously acidic environment. Both these processes can potentially reduce the magnitude of the bond strength achieved early in the life of a lining.

Crystal growth in the zone between lining and substrate. The drained nature of most tunnels, and the drying environment typical of many tunnels with forced ventilation, act to draw moisture out of the surrounding ground. Dissolved minerals such as mirabilite, epsomite, and halite can thereby be transported from the ground to the tunnel lining with this moisture. Upon evaporation, the expansive pressure associated with growth of crystals precipitating from solution can steadily destroy porous materials and prise a lining away from its substrate (Hunter, 2026).

Sustained Tension. Materials such as concrete exhibit a reduced tensile strength when loaded in sustained tension due primarily to the slow growth of cracks (Rüsch, 1960). If a significant tensile or shear stress is applied to a lining boundary for a sustained period of time the apparent bond strength will diminish.

Weak Planes in Substrate. The achievement of a strong bond to the substrate is of little value if the substrate itself includes de-bonded internal planes. This is often the case in layered rocks where weak strata (or bedding planes) may exist immediately inside an apparently competent surface. Thin layers of shale, mudstone, and other weak materials within an otherwise competent rock mass or schistose foliation can lead to low tensile strengths that render the bond between shotcrete and substrate irrelevant.

Any of these processes can take years to progressively de-bond a lining from its substrate, so the presence of a strong bond soon after construction does not guarantee that bond will persist throughout the life of a lining. The fact that so many potential degradation mechanisms exist supports the requirement for a means of effectively assessing bond at any point in the life of a shotcrete lining if bond is relied upon to support a load (including self-weight).

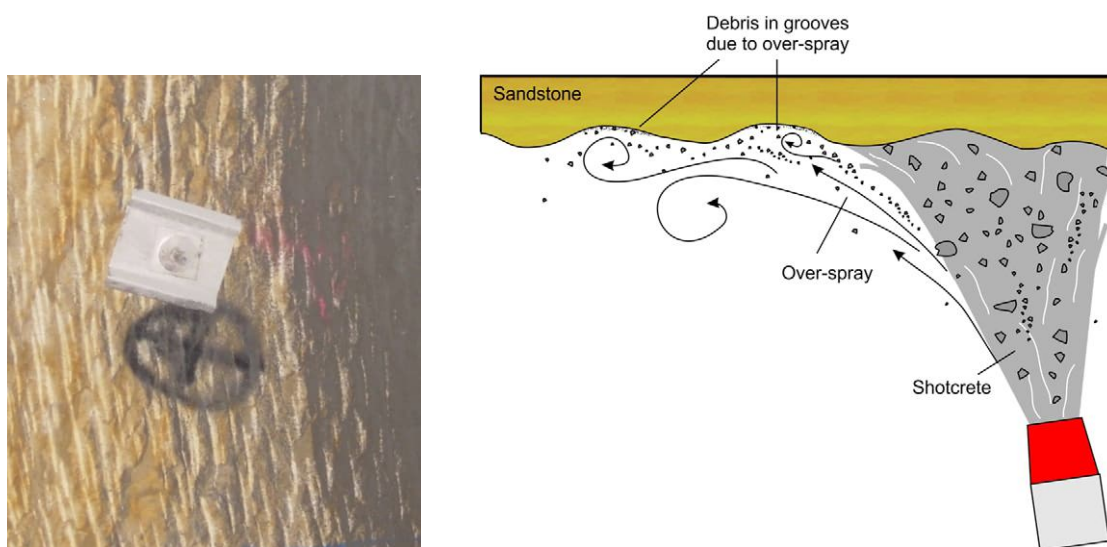


Figure 5: a) Accumulation of loose over-spray in pick grooves, and b) adjacent to an active spraying area.

6 THE INCIDENCE OF ZERO RESULTS

One of the realities of bond testing is the frequent occurrence of so-called ‘zero’ results. These ‘zero’ results may be caused by an actual absence of bond at the shotcrete/substrate boundary or may be caused by breakages induced by the drilling and testing process. The high frequency of ‘zero’ results is illustrated by the data in Table 1 which shows the zero rate in published studies and tests conducted in various tunnels.

When investigating bond strength every ‘zero’ result should be recorded and investigated to identify whether it indicates a true area of de-bonding or not. Hammer sounding prior to coring is useful in identifying whether de-bonded zones are likely to be present. If gaps are observed in the core hole either at the boundary with the substrate or within the lining

then the ‘zero’ result must be counted as valid (Figure 1). An endoscope may be useful for this purpose. Even if a gap is not visible at the substrate boundary, profuse dripping of water after coring may indicate that a narrow delamination plane exists.

As previously identified, zones of delamination or de-bonding may be associated with sand lenses within the shotcrete, sloughing of wet shotcrete, layers of excessive set accelerator dosage, ground movement, layers of dirt and debris that were not cleaned off prior to spraying, or other contaminants such as spillage of construction polymers or oils. These phenomena can often be identified by close examination of the broken surfaces of the extracted core. Each of these factors may not result in an actual gap within the shotcrete or at the substrate boundary but can produce a zero or near-zero bond strength result that must be included in any estimation of average bond strength.

Problems during drilling of a core can lead to breakage and an apparent ‘zero’ result even for very strongly bonded shotcrete. A poorly anchored drill rig can either shake or shift position during drilling, or the drill rig may be bumped or stop functioning properly, leading to a change in drill inclination and increase in friction during operation. Cores can also be broken when the drill barrel is withdrawn upon completion of drilling. While evidence of poor drilling may sometimes be found in the form of a wavy or discontinuous core barrel surface, it is seldom possible to be certain about the actual cause of breakage.

7 SUMMARY

Experience with bond measurement from numerous hard rock tunnelling projects has demonstrated that bond between a shotcrete lining and its substrate can be disrupted by many factors both during construction and over the service life of a lining. This can result in a high proportion of ‘zero’ bond results when conducting tests in the field. Considerable difficulty usually occurs in distinguishing whether these ‘zero’ results are artefacts of the sampling process or whether they represent true areas of de-bonding.

The high incidence of zero bond results in field testing raises concerns about the effect local flaws have on the effective tensile bond strength of a shotcrete lining around a potential falling rock wedge. Since bond to the substrate affects the mode of failure that the lining will exhibit in response to the falling wedge (shear or flexure), and these two modes of failure typical exhibit substantially different peak load resistance, the existence of areas of zero bond between a lining and its substrate can influence the load resistance of a lining.

Given the significance of de-bonding to load bearing capacity, the most effective means of addressing the difficulties of assessing and accounting for the high variability and frequent occurrence of ‘zero’ bond strength is to either account for de-bonding by including a size effect factor on bond strength, or assume that bond is non-existent and design all thin linings on hard rock to resist load primarily through bending rather than shear. As described by Barrett and McCreath (1995), this requires points of attachment between the lining and rock to exist in the form of rock bolts with cover plates *over* the lining, or spider plates with adequate resistance to punching shear *within* the lining, so that the flexural resistance of the lining can be exploited.

CRedit authorship contribution statement

Erik Stefan Bernard: Conceptualisation, Formal analysis, Writing - original draft. **John C. Braybrooke:** Data curation, Investigation, Writing – review & editing.

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APPENDIX

BOND ASSESSMENT METHODS

Given the numerous possible causes of bond degradation outlined in this paper, and the significance of bond loss to lining behaviour, it is often necessary to quantify the degree of de-bonding that may arise or quantify the magnitude of bond strength remaining to support potentially unstable ground especially if shear resistance is relied upon in the lining. Methods of detecting and measuring bond strength have therefore been developed and are described below.

A1.1 HAMMER TESTING

Tapping the surface of a shotcrete lining to detect hollow internal zones is a partially effective means of identifying de-bonded zones but is prone to subjectivity and error. The method is sometimes known as the Norwegian Hammer method of sounding because it was first described in the Norwegian Concrete Association Publication No. 7 titled “Sprayed Concrete for Rock Support”. It involves tapping the surface of a shotcrete lining with a light hammer and listening for ‘drumminess’. Unfortunately, there are spurious causes of drumminess (such as the presence of drainage elements within the lining or delamination of the underlying substrate) and different operators vary in their opinion of what constitutes a drummy sound. Despite this, there is general agreement that a drummy sound is an indicator that de-bonding may be present.

A1.2 BOND STRENGTH MEASUREMENT

While hammer testing can identify areas of de-bonding, it cannot identify the magnitude of bond strength. To measure bond strength it is necessary to conduct a pull-test on a sample that includes the shotcrete/substrate boundary.

Bond strength is commonly assessed between concrete and overlying membranes such as floor coverings and shotcrete. Test methods for floor coverings (such as bonded flooring materials) are more widely used than for shotcrete so the tests and equipment used for shotcrete have often been adapted from test methods originally developed for floor coverings and repair media. The most common methods used for these applications include EN 1542, ASTM C1583, and ASTM D7234 which are all primarily used to assess overlays and repairs to a concrete surface. These tests are carried out *in situ* and numerous manufacturers have developed equipment which can be used to conduct this test. However, these methods are primarily intended to test geometrically smooth and regular surfaces, so the proprietary equipment available for this test method usually cannot accommodate the uneven surfaces typical of shotcrete linings. Similarly, ISO 4624 is an internationally recognised method used to measure the adhesive strength of paints and varnishes on various substrates using a metallic dolly with pivoting pin. For a concrete substrate, it recommends a sample diameter of 100 mm. In contrast to these *in situ* tests, AS1012.24 is an Australian standard that involves drilling a core through a concrete overlay and into the substrate to extract an intact core that is subsequently tested in tension in a laboratory. Several variants of this method exist in other countries.

Test procedures specific to the assessment of the tensile strength of shotcrete bonded to a substrate include the EFNARC European Specification for Sprayed Concrete, 1996 (Section 10.6). This specification covers the assessment of the tensile strength of shotcrete bonded to a substrate, but it lacks detail regarding how the procedure is actually implemented. Oddly, it includes both *in situ* testing and laboratory testing of an extracted sample but does not preference either. This procedure was followed by a second edition in 1999 that differed in several respects from the 1996 edition but introduced the recommendation for a minimum of six cores to be tested ‘from the same general area of the works’ to calculate one estimate of bond strength.

Swedish Standard SS137342 is the only actual standard in existence specifically for testing the bond between a layer of shotcrete and its substrate. It differs significantly from earlier guidelines and specifications in that it requires a double-core bit to drill an over-sized round groove around the primary core so that a chuck can be mounted concentrically over the sample to be pulled (Figure A1). This method is more detailed and better thought-out than the EFNARC specification for ensuring that eccentric loading is eliminated from the load train.



Figure A1: In situ testing of bond strength using a) drilled concentric core holes, b) mounting of a glued dolly to the end of a partially drilled core, and c) pull-testing, similar to the SS137243 test method.

The method by Seymour et al (2010) is not a standard but is a recommendation produced by NIOSH in North America. It uses a pull-pin drilled and glued into the end of a partially drilled core instead of the tension chuck required by SS137243 but is otherwise very similar to the Swedish method.

As part of research conducted into the behaviour of shotcrete sprayed over a waterproofing layer, Holter (2015) used a variation on the method by Seymour et al (2010) and ISO 4624 that incorporated an abrasive disk to grind a flat reference surface into the shotcrete face around the core. A concentric jack was then placed on the perpendicular surface to pull the core via a central pin and measure *in situ* bond strength (Figure A2). Expanding pins and epoxy grouting of the pin are both possible.

In general, the methods listed above can be divided into *in situ* tests, which are all based on pull-tests of a core drilled through the boundary between shotcrete and substrate, and laboratory-based tests in which an intact core sample comprising both the shotcrete lining and substrate is extracted and moved to a laboratory and tested in tension after preparation of the ends. Among the *in situ* tests, some use a dolly (or 'rondell') comprising a round metallic disk bonded to the end of the core to exert tension during testing, while others rely on an expanding pin or threaded metallic pin glued into a hole drilled through the centre of the core (usually before coring) to impart tension. The pin-based method avoids the problem of debonding due to a weak surface on the face the shotcrete, while the dolly-based method avoids potential breakage of the core due to splitting. The pin-based method is probably superior for longer cores (with a length/diameter greater than unity) while the dolly method is better for thin linings in which the length/diameter ratio is less than unity. When using a dolly it is necessary to grind away the weakest surface layers of shotcrete before starting the process of coring and gluing.

A1.3 CORE BREAKAGE

The laboratory-based tensile tests of bond strength suffer the problem that the core must be broken off at some point in the substrate so the sample can be extracted for transportation to the lab. This is almost always done by hammering a wedge into one side of the core slot, which introduces an eccentric load to the core resulting in flexural failure. This can lead to breakage at the shotcrete/substrate boundary for weakly bonded specimens, after which the sample cannot be tested. Thus, a positive bias is introduced into the statistical record because the weaker bond strength samples end up being discarded due to breakage and are thereafter excluded from further consideration.

Similarly, cores that break off during coring cannot be tested using either the *in situ* or laboratory-based methods. These samples are then discarded and seldom recorded as 'zero' results. It is usually quite difficult to determine whether a core that supposedly 'broke' during drilling was, in fact, broken off as a result of the drilling process, or was de-bonded from the substrate prior to drilling. This ambiguity may be partially clarified by tapping the surface before drilling to detect drumminess, but an absence of drumminess does not confirm the presence of bond because small areas of de-bonding often do not sound drummy. Even if a weak bond is present, the elimination of weak specimens by 'breakage' during drilling introduces a positive bias to the population of surviving samples.

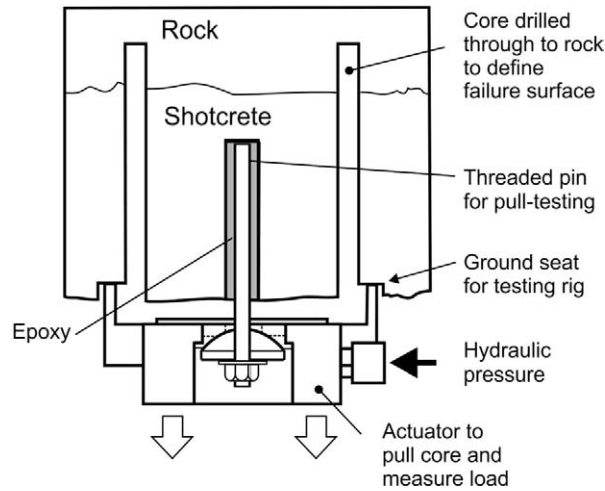


Figure A2: Test configuration similar to that described by Holter (2015) for pull-testing with drilling and surface grinding completed prior to installation of pull-pin in central hole.

The 1999 EFNARC recommendation (Section G9.5) for six samples to be tested in ‘the same general area of the works’ to calculate one estimate of bond strength does not provide any guidance on whether to include ‘zero’ results in the average bond strength estimate or not. The common practice in the field is to assume ‘zero’ results are caused by drilling problems, discard these samples, and keep drilling until the required number of non-zero results or intact samples are obtained. This clearly introduces a positive bias to the distribution of recorded results.