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Optimum pressure for rockmass grouting between TBM segmental linings of a twin tunnel

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ABSTRACT: Grouting pressure in the rock mass between newly installed TBM segmental linings during tunnel construction is typically limited at the point of injection to 100 kPa or less above groundwater pressure. However, because the grout pressure dissipates away from the point of injection, the grout pressure out in the rock mass may not be sufficient to overcome the hydrostatic back pressure, and the viscous resistance and cohesion of the grout, which means the grout does not adequately move out into the rock mass, displace the groundwater, and seal the rock mass. Barton and Quadros (2019) have suggested that the pressure can drop by 50 percent only 1 meter from the grout hole. This reduction is due to various factors such as applied pressure, viscosity and cohesion of the grout (Gustafson, Claesson & Fransson. 2013). This paper looks at some of the factors to consider when establishing limits on grouting pressure.

1 INTRODUCTION

Twin tunnels for highways and transit typically have cross passages between the tunnels. These cross passages are typically constructed after both the running tunnels have been excavated and lined, if lining is required. Prior to cutting through the lining and excavating the rock between the tunnels, it is common to drill through the lining and into the rock between the tunnels, and then grout that rock to control potential groundwater inflows.

In order to successfully control groundwater, the grout must be injected at a high enough pressure to displace the groundwater, not just at the point of injection, but also some distance out into the rock mass. Because the pressure dissipates with distance into the rock mass, the pressure at the point of injection must be considerably higher than the hydrostatic groundwater pressure, or the grout will not displace the water. The problem is that injection pressures are often limited to avoid damaging the lining. If the pressure limitations are too conservative, then it is quite likely that the grouting program will not successfully control groundwater. Uncontrolled inflows can cause all sorts of problems, including severe ground settlement and damage to structures.

2 GROUND CONDITIONS

The porosity and permeability characteristics of the ground are important. The most common situation for rock tunnels is for the groundwater to occur in a network of interconnected, open fractures. In most cases, the matrix of the rock between the fractures is tight or nearly tight, but in some cases the matrix can have considerable porosity and permeability itself. The goal of the grouting must be to create a thick, low-permeability halo around the excavation by filling the fractures (and matrix if necessary) with grout. The grouted halo should be a few meters thick because the grouted zone is not a membrane, but a zone of reduced-permeability rock.

The groundwater pressure around the grout zone is also important. The grout injection pressure must first overcome the hydrostatic back pressure before any grout can be injected. Furthermore, as the hydrostatic pressure increases, the grouted halo around the excavation needs to be thicker to effectively control the inflows, because it is the hydraulic gradient through the grouted zone that drives the inflows to the excavation.

3 CEMENT GROUT AS A BINGHAM FLUID

Cement grout is a slurry, not a liquid; it is a Bingham fluid, not a Newtonian fluid. As a Bingham fluid, it has both a viscosity and a cohesion, whereas a Newtonian fluid, like water or oil, has only viscosity (Figure 1). In both cases, the viscosity controls the rate at which the fluid can flow through a particular pipe or fracture. However, for grout, if the flow rate drops too low and the shear stress falls below the cohesion, the grout seizes: it stops flowing and starts behaving more like a soil. Once it seizes, continued pressure only serves to consolidate the grout through pressure filtration (which is desirable). All of this occurs well before the grout begins to take its initial set.

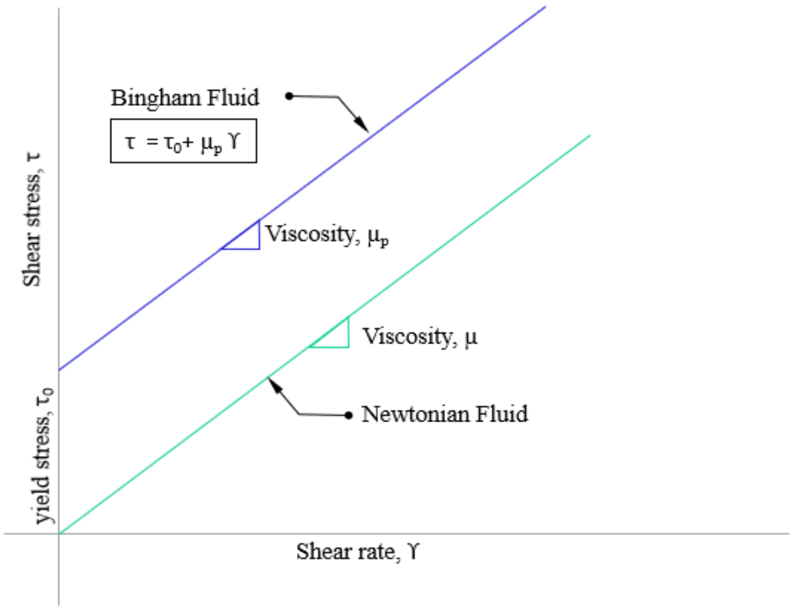

Figure 1. Bingham Fluid

4 PRESSURE ATTENUATION

Grout pressure attenuates with distance from the point of injection from three processes. The first is head loss due to viscous resistance to flow as the grout is forced through the fractures. The second is radial dissipation as the grout spreads out through the fractures from the grout hole. The third is from the grout seizing and consolidating due to cohesion. The pressure also increases or decreases as the grout is pushed lower or higher in the rock mass from the point of injection.

Head loss and radial dissipation can be modeled together using the radial form of the cubic law (derived for this paper based on Young, et al, 2011). This is a simple model based on a grout hole intercepting a single, extensive fracture at a high angle. In this model (Equation 1), Δh is the head loss along the fracture between the borehole wall at radius r_1 and some more distant point in the fracture at radial distance r_2; Q is the grout pumping rate, a is the aperture of the fracture, μ is the viscosity of the grout, and γ is the unit weight of the grout.

$$\Delta h = \frac{-Q}{2\pi a^3} \frac{12\mu}{\gamma} \ln\left(\frac{r_2}{r_1}\right) \quad (1)$$

For example, a typical 1 to 1 mix (by mass) of Portland cement grout might have a viscosity of 5.6 mPa·s and a unit weight of 14.3 kPa/m. If the pumping rate is 30 L/min = 0.5 L/s, and the fracture has an aperture of 0.4 mm, and the borehole has a diameter of 64 mm (r_r = 32 mm), then the head loss to a point 2.0 m away at the same level would be 24 m. Converting to pressure, where z is the elevation difference between the grout hole and the point at r_2 (z = 0 in this example):

$$\Delta p = \gamma(\Delta h - z) = 345 \text{ kPa} \quad (2)$$

If the tunnel were 30 m below the water table, then the groundwater would create a hydrostatic backpressure of about 300 kPa. Therefore, to push the grout 2 m horizontally through that fracture, the injection pressure would have to be 645 kPa, or more than twice hydrostatic pressure.

Real fracture networks are complex, unique, and difficult to model, but the simple model shown above does illustrate the nature of the problem. If a grout hole crosses more than one fracture of similar aperture, then Δh would be cumulative, but if one is much finer than the other, then the grout will probably seize in the finer one and stop flowing. The flow is typically laminar, so the roughness of the apertures is not relevant, except that, at the scale of these fractures, a little roughness can change the aperture considerably.

Figure 2 shows the excess pressure (Δp) required to push the above-described grout a certain distance through single fractures of various apertures. Obviously, finer fractures require more excess pressure. What is more interesting is that there is a pressure threshold needed to push the grout more than about 0.5 m along the fracture. For example, for a 0.2mm fracture, the excess pressure required to move grout more than 0.5m is estimated to be around 11 bar (1,100 kPa).

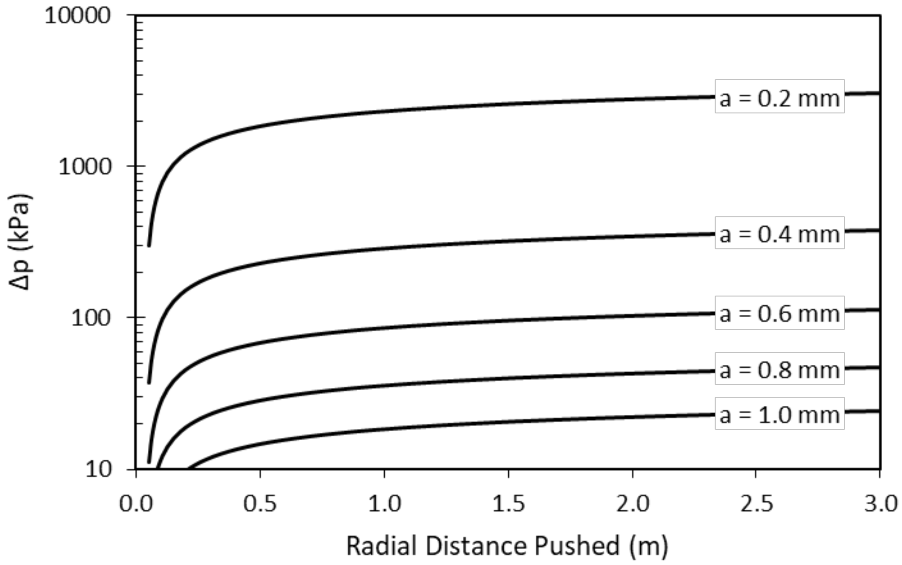

Figure 2. Excess Pressure Needed to Push Grout out into the Formation Assuming Tunnel was 30m below Water Table)

4.1 Example Projects – Waterview Auckland TBM Tunnel

Grouting in the Waterview Connection was described previously in published papers by McCarrison (2017) and Okada, Giauque & Bhargava (2017). What follows is an interpretation of what was published.

The Waterview Connection consisted of twin segmentally lined tunnels with 16 cross passages. The rock was weak siltstone and sandstone with closely spaced joints and generally low permeability. The cross passages were grouted as shown in Figure 3, prior to breaking through the lining.

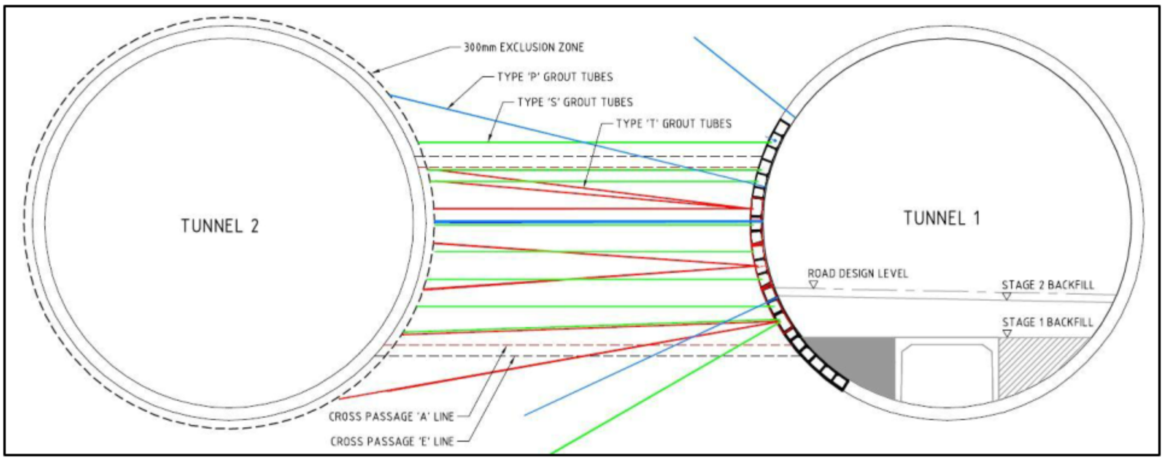

Figure 3. Typical Cross Passage Grout Holes Arrangement (Figure Extracted from the Waterview Connection Project Cross Passage Tunnels: Safety in Design and Construction (Okada, Giauque & Bhargava. 2017))

Most of the cross passages were successfully grouted and met the inflow criterion, possibly because the rock was already tight without grouting. Cross Passage 12, however, was different. It was in rock that was more fractured and under a major stream, with the hydrostatic head of about 37 m to the tunnel springline, which would have created a hydrostatic back pressure of about 3.7 bar in the grout holes.

Grout was successfully injected at pressures ranging from 5 bar up to 8.5 bar. At 8.5 bar, the injection pressure was more than twice the hydrostatic pressure. Strain gauges were applied to the lining to monitor deformation and nothing significant was observed. The designed capacity of the lining was not stated in the papers, but it was implied from the monitoring program that the pressures used significantly exceeded the design capacity with no ill effect. Why these high pressures were safe is likely to the attenuation of pressure as described above.

5 DISCUSSION

The goal of the grouting described in this paper is to consolidate the rock mass in preparation for excavating cross passages between already excavated and lined twin tunnels. Contact or tailskin grouting between the lining and the rock mass should already have been completed so, ideally, there should be no gap between the rock mass and the extrados of the lining. If, ideally, there is no gap, then the grout pressure should dissipate into the rock mass and only be weakly applied to the lining due to the elastic response of the rock mass. Also, since the grout is a Bingham fluid, it cannot hydrostatically transfer pressure like water. If the grout is flowing, it loses pressure with distance due to viscous resistance; if it stops flowing, it seizes and absorbs pressure like a consolidating soil. All of this means that there is a low probability that the injection pressure gets fully applied to the extrados of the lining.

However, in our experience, it is not uncommon for structural engineers to place severe limits on the amount of grouting pressure that can be applied to the rock mass. Sometimes the allowable pressures even account for the factor of safety in the lining design, so that the injection pressures barely overcome the backpressure from the groundwater. The consequence of these severe limits is that there is insufficient pressure at the point of injection to adequately grout the rock mass around the area to be excavated for the cross passage.

Equation 1 models the low-probability situation where there is an extensive gap between the segments and the rock mass, presumably due to ineffective tailskin or contact grouting. Equation 1 overestimates the pressure applied to the lining if the tailskin grouting is even a little bit effective. Even so, Equation 1 and Figure 2 show that the pressure dissipates markedly over a very short distance away from the grout hole.

A good way to manage the low-probability situation is to test and improve the tailskin or contact grouting in the vicinity of the cross passage, prior to grouting the rock mass. This can be done by drilling a pattern of short injection and bleed holes through the lining and then injecting grout from the lower holes to push the water, and then grout, up and out through the upper holes. Of course, if the holes don't make any water or take any grout, then it shows that the original grouting was fully effective in that area and that the low-probably situation does not occur. If the low-probability situation does not occur or was corrected, then there is no reason to place severe restrictions on the grout pressure. This testing and improvement need to be done in both tunnels on both ends of each cross passage prior to grouting the rock mass in between.

Bleeding down groundwater pressure so that lower grout pressures can be used is not a good idea in many situations, because it can lead to consolidation of the overburden soils and structural settlement, depending on the specific geotechnical conditions along the tunnel alignment.

6 LIMITATIONS

It is difficult to accurately predict the average aperture of a rock mass; therefore, the required excess grout pressure may be either overestimated or underestimated. On site testing is essential, and as discussed in Section 5, it is important to ensure that high-quality tailskin grouting has been carried out before applying higher excess pressure for pre-excavation grouting at cross passages.

7 REFERENCES

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