Blow-out failures part 2: purely cohesive soils

In part 2 of this article, Dr Benoit Jones of Inbye Engineering looks at blow-outs in tunnels in purely cohesive soils – what the critical mechanisms are and how they can be predicted.

Introduction

As mentioned in the previous issue, a blow-out is a general term, which refers to any sudden release of face pressure. There are several ways in which a blow-out can occur in a purely cohesive soil:

1. Passive failure
2. Softening and erosion
3. Hydraulic fracturing

Passive failure is like a stability failure in reverse: a large cone of soil is forced upwards or outwards, failing along large shear surfaces. The main difference to stability failure is that the weight of the soil is a favourable rather than an unfavourable load, and the volume of soil involved is usually larger. Therefore, passive failure requires high face pressures.

Blow-outs can also be caused by support fluid finding a path to the surface, for example up a poorly-backfilled borehole, or by creating a path through erosion or hydraulic fracturing. Blow-outs can also occur away from the face – anywhere that an excessive pressure is applied. For instance, due to excessive grouting pressures, or excessive internal tunnel pressures finding a way through the lining.

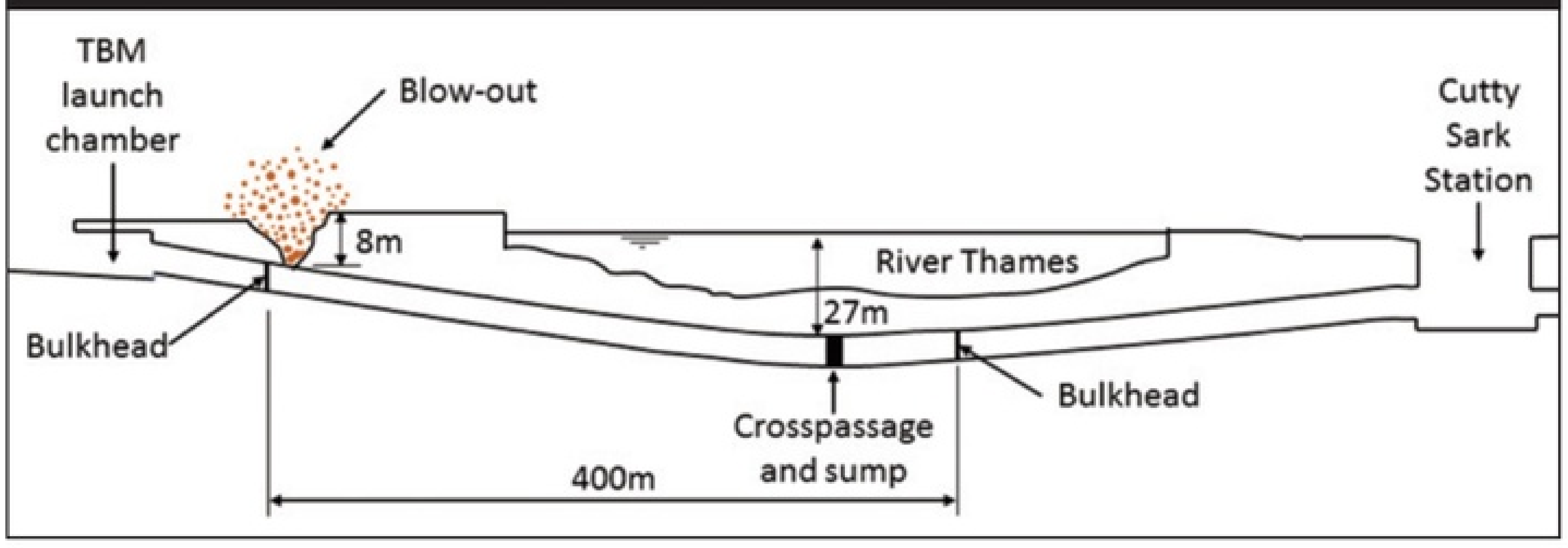
Looking back at the Docklands Light Railway tunnel blow-out described in the previous issue, compressed air was needed for construction of a crosspassage between the two running tunnels near the deepest point under the River Thames, where the water pressure was about 2.7 bar at axis level (270 kPa). One bulkhead was 60m beyond the crosspassage and the other one was much further inbye towards the portal, where the cover was only 8m, as shown in Figure 1. The blow-out occurred near the inbye bulkhead through the tunnel lining as the bulkheads were being tested up to 3 bar and pressure had reached only 2.1 bar (210 kPa). Eight of the tunnel's 1.2m long precast concrete rings were blown apart by the blast (Jones, 1998), and a crater 22m wide and 7m deep was created by the blast. The full overburden pressure at the crown at this location was less than 160 kPa. It is not known whether this blow-out began as a hydraulic fracture and the large crater was due to the huge volume of air that was released, or whether it was a passive failure.

Softening and erosion

Softening and erosion appears to be most common for pressurised water conveyance tunnels during operation where the head of water is greater than the distance to the top of the clay or the surface. A leak out of the lining can, over time, gradually soften and erode the clay, eventually creating a path out of the clay to the surface or into a more permeable soil, resulting in flooding and/or excessive loss of water from the tunnel. This happened to a raw water tunnel near the village of Datchet, near London Heathrow Airport, in 2006, where over 30 years water eroded a path through the London Clay and then suddenly burst out, spouting 5m into the air and producing flood waters 1m deep (BBC, 2006). Fourteen properties were flooded, most only in their gardens, before the tunnel could be isolated. This was an unbolted concrete wedgeblock tunnel commissioned in 1976 (Pawsey & Humphrey, 1976), so it would have relied on the ground pressure from the clay, and the impermeability of the clay itself, to remain watertight (Wood, 2008). The tunnel was at approximately 30m depth below the ground surface, but the reservoir level was approximately 18m higher than this (Pawsey & Humphrey, 1976). Therefore, even though the overburden pressure at the tunnel depth, about 600 kPa, was significantly higher than the internal water pressure of about 480 kPa, a blow-out still occurred.

Predicting the length of time that softening and erosion will take to compromise a tunnel is difficult. However, it is considered good practice nowadays to design a tunnel lining to be watertight for water conveyance in soft ground, and so this risk hopefully only applies to older tunnels.

Figure 1: Long section of the DLR Extension South Drive (Jones, 1998).



Hydraulic fracturing

Hydraulic fracturing is a localised effect that can happen in the short term, or may occur after some softening and erosion has reduced the effective cover. This is where the clay is fractured by a high localised fluid pressure that exceeds the tensile or shear strength of the clay (Marchi et al., 2014), creating a path for the escape of support fluid. This will usually occur at the crown of the tunnel. In a tunnel with compressed air, the loss of air pressure could be followed by flooding of the tunnel and/or collapse of the face. In a slurry TBM, a sudden loss of slurry can be followed by over-excavation due to failure of the face caused by loss of support pressure. Also, release of the slurry at the surface or into an overlying lake or river could have environmental consequences.

For hydraulic fracturing to occur, a rule of thumb is to assume the fluid pressure has to exceed the full overburden pressure of soil and water (Holzhäuser, 2003), and generally accepted practice is to limit support fluid pressures to this value (e.g. Guglielmetti et al., 2008). This is usually significantly lower than the pressure needed for passive failure to occur.

Bezuijen & Brassinga (2006) show from field data and centrifuge tests that bentonite slurry blow-outs caused by hydraulic fracturing can occur at much lower face pressures than those needed for passive failure predicted by finite element or kinematic analysis methods. This is because these methods do not take account of the fact that slurry is a fluid. Bezuijen & Brassinga found this limit to be approximately the pore pressure plus 2 to 3 times the effective stress for their case.

Marchi et al. (2014) show from a large number of experimental tests and case histories of fracture grouting that fracture initiation in clay may be caused by either tension or shear. The most important factor is the confining pressure, otherwise known as the minor principal stress. As fluid pressure increases, the radial stress in the surrounding clay increases but the circumferential stress decreases. Failure occurs either when the circumferential stress reaches the tensile strength or when the difference between the radial and circumferential stresses causes shear failure. Other factors, such as the water content of the clay, the liquidity index, the stiffness, rate of pressure or injection contribute, but the lower bound to all the measured fracture pressures was the initial confining pressure, so it seems the fracture pressure cannot be lower than the minor principal total stress.

The pressure at tensile fracture is given by (Mitchell & Soga, 2005):

$$P_f = 2\sigma_0 - u_0 + \sigma_t'$$

where P_f is the fracture pressure, σ_0 is the initial confining pressure (the minor principal total stress), u_0 is the initial pore pressure and σ_t' is the tensile strength.

The pressure at shear fracture is given by (Soga et al., 2005):

$$P_f = \sigma_0 + nc_u$$

where n is a constant and c_u is the undrained shear strength. For a clay with a positive liquidity index, n = 1, but for a clay with a negative liquidity index (i.e. the water content is below the plastic limit) n = 1.5 to 2.

Therefore, either the minor principal total stress or the lower of the tensile or shear fracture pressures should be set as the maximum face pressure when compressed air or slurry is used.

Passive failure

Since hydraulic fracturing is the critical mechanism when support fluid is used, passive failure in clay is only likely to be the limiting case for earth pressure balance TBMs. Most studies of passive failure in clay, involving numerical models or kinematic limit state analysis, have assumed a uniform face pressure (Mollon et al., 2013), whereas in an EPB machine we would expect a face pressure that increases with depth. Despite this shortcoming, a design chart by Mollon et al. (2013) based on their asymmetric 'M2' velocity field method is shown in Figure 2, with values given in Table 1 to aid interpolation for design purposes. The values of N_γ and N_c from the design chart should be used in the following equation:

$$\sigma_b = \gamma D N_\gamma - c_u N_c + \sigma_s$$

where σ_b is the critical face pressure for a passive failure blow-out, N_γ is a stability number taking account of soil weight and N_c is a stability number taking account of soil cohesion.

This equation is derived from the more general stability equation, which is:

$$\sigma_c = \gamma D N_\gamma - cN_c + \sigma_s N_s$$

where N_γ, N_c and N_s are stability numbers for the effect of soil weight, cohesion and other effects, respectively. For undrained constant volume behaviour, N_s = 1 and $c = c_u$. In order to make the equation equal to the undrained stability equation traditionally used for clays, all that is needed is to substitute $N_\gamma = C/D + 0.5$ and $N_c = N$. But in Mollon et al.'s 'M2' velocity field method, $N_\gamma \neq C/D + 0.5$, because the velocity field is asymmetric (the maximum velocity is set 0.4D above the centre of the face), so the more general form must be used.

Figure 2: Design chart for critical passive failure blow-out (from Mollon et al., 2013).

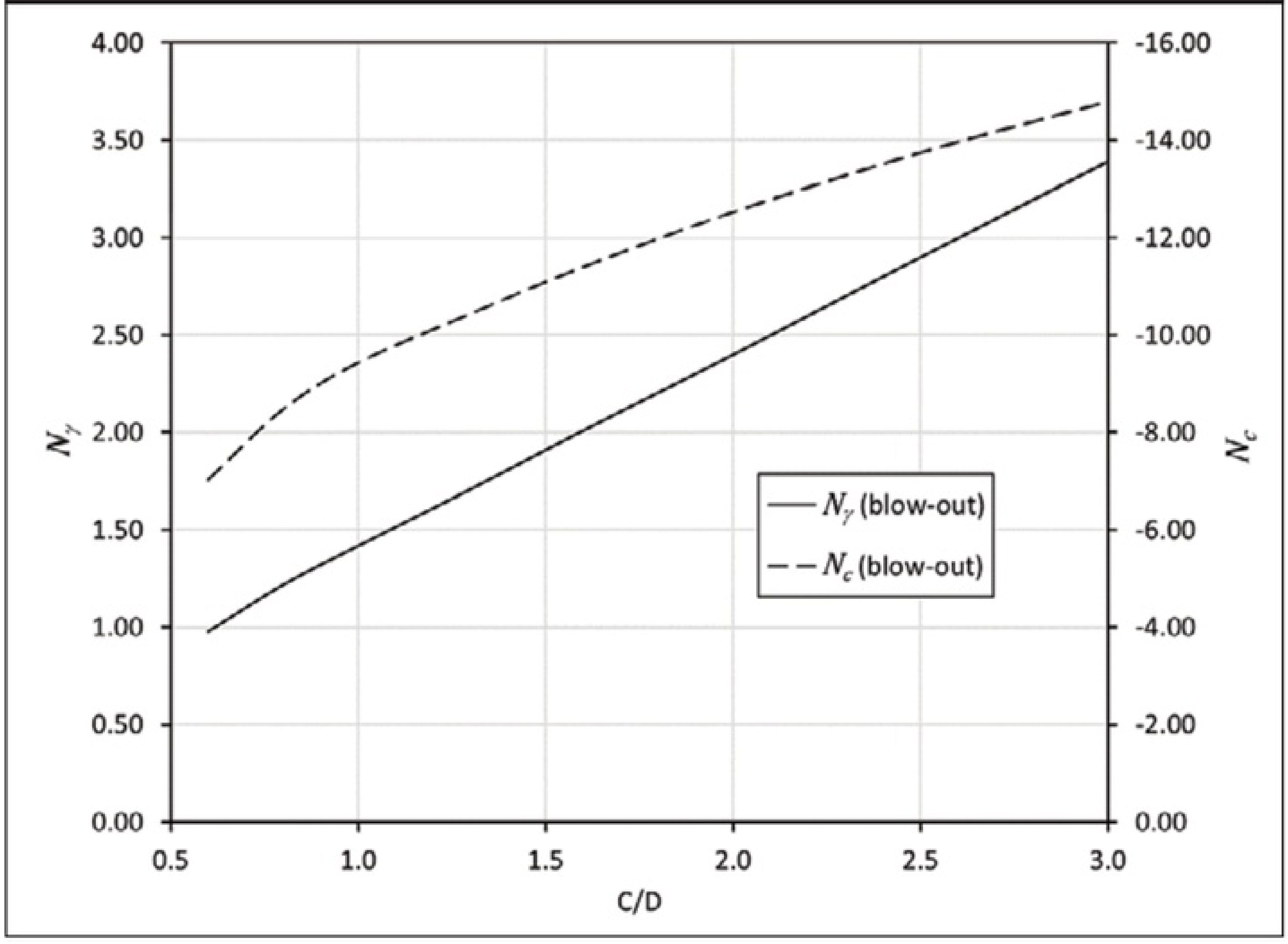


Table 1: Values used to produce Figure 2 for critical passive failure blow-out (from Mollon et al., 2013).

C/D	N_γ (passive failure blow-out)	N_c (passive failure blow-out)
0.6	0.98	-7.02
0.8	1.22	-8.47
1.0	1.42	-9.43
1.3	1.71	-10.44
1.6	2.01	-11.40
2.0	2.40	-12.53
2.5	2.90	-13.75
3.0	3.39	-14.80

Unfortunately, no centrifuge modelling of passive failure in clay has been published, but it is corroborated to some degree by Mollon et al.'s (2013) numerical modelling of a heading using FLAC3D, for which a comparison is shown in Figure 3 for $D = 10$ m, $\gamma = 18$ kN/m³ and $c_u = 20$ kPa.

Figure 3: Comparison of critical passive failure pressures using the design chart based on 'M2' velocity field kinematic analysis by Mollon et al. (2013) and a 3D finite difference model in FLAC3D, also by Mollon et al. (2013), for $D = 10$ m, $\gamma = 18$ kN/m³ and $c_u = 20$ kPa.

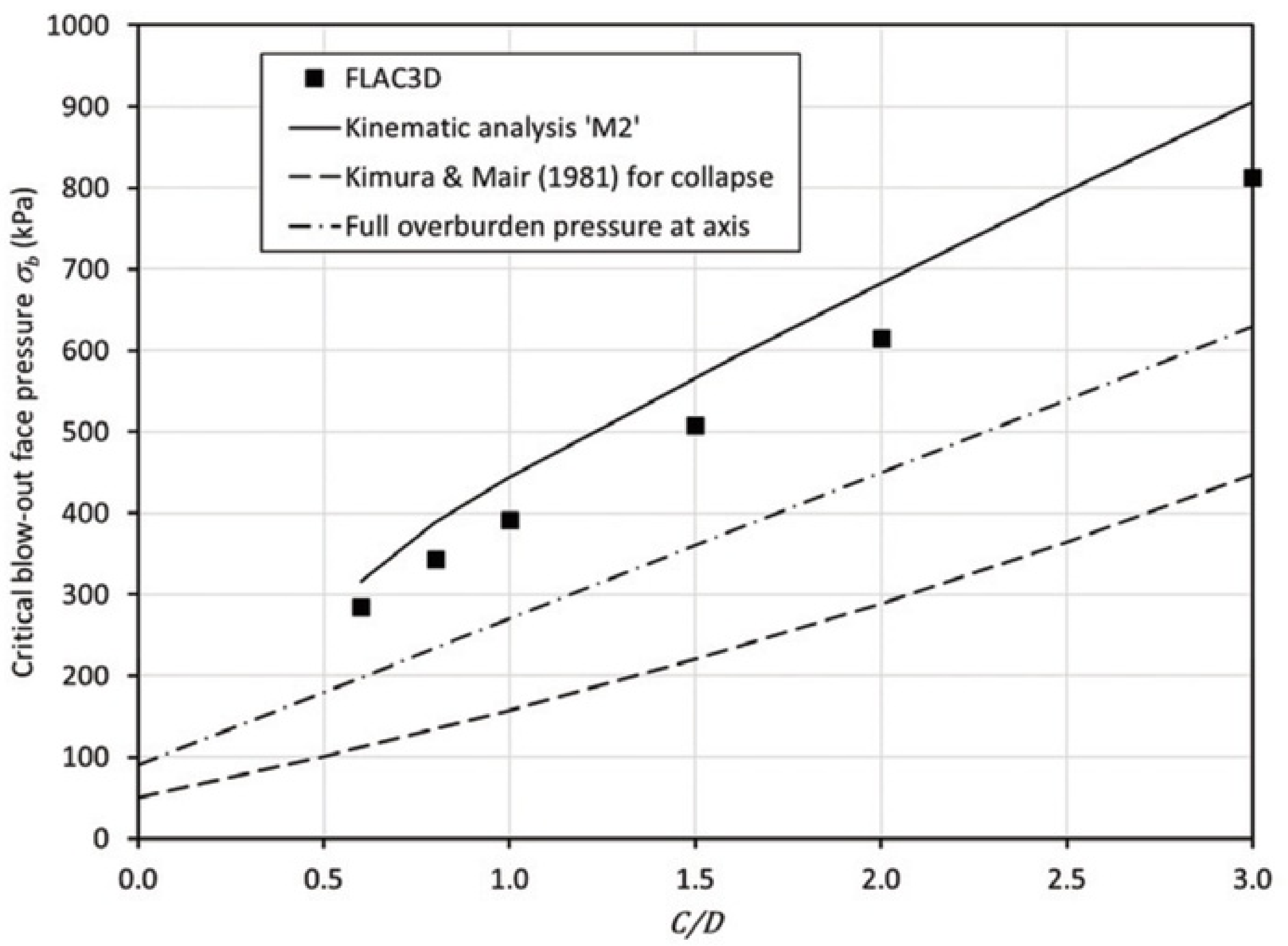


Figure 3 shows that the kinematic analysis overestimates the critical passive failure pressures compared to the numerical modelling results. This is unsurprising because it is an upper bound and defines when the ground must fail, and therefore will always be on the unsafe side. However, it must be close to the true collapse geometry and conditions, as it is not too far above the FLAC3D results.

Figure 3 also gives us a feel for the magnitude of face pressure required to cause passive failure. Even for this low value of undrained shear strength (20 kPa), the critical face pressure is about 1.5 times the full overburden pressure at axis level. At an undrained shear strength of 30 kPa the critical face pressure is double the full overburden pressure (Mollon et al., 2013). Therefore, when support fluid is used, hydraulic fracturing will almost always be more critical.

Also shown on Figure 3 is the minimum face pressure required to avoid collapse, from the design charts based on centrifuge testing produced by Kimura & Mair (1981), for the same geometry D = 10 m and P = 0 m, soil bulk unit weight γ = 18 kN/m³ and undrained shear strength c_u = 20 kPa. The aim of design would be to find a safe zone of face pressures between collapse and blow-out, allowing for factors of safety and for variability of the applied pressure. In very weak soils at low cover, a safe zone may be impossible to achieve.

In summary, to estimate the critical face pressure that would cause a passive failure blow-out, the design chart in Figure 2 could be used to provide an initial estimate, and if accuracy is of critical importance, or if soil layers or geometry are complex, a 3D numerical model could be used. Where a fluid such as slurry or compressed air is used for face support, a hydraulic fracturing blow-out is more likely, and face pressures should be limited to below the hydraulic fracturing pressure calculated at the crown of the tunnel.

Conclusions

There are many types of blow-outs, of which passive failure is just one. For all types of blow-outs, increasing cover will generally reduce the risk, but an understanding of the ground and the groundwater, and how they interact with the support method, will help predict blow-out scenarios.

Numerical and physical modelling appear to be fairly reliable methods of analysing collapse or passive failure of a heading. However, there has been much more focus to date on collapse and more research is needed to improve our knowledge of passive failure and blow-outs in general. Although relatively rare, the effects can be just as catastrophic as a collapse.

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