Evaluation of Slurry TBM Design Support Pressures using East Side Access Queens Bored Tunnels Data

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Abstract

In practice, there are a number of approaches that can be used to determine the required support pressures to maintain stability while tunneling in soft ground. These approaches include both conventional rule of thumb (e.g. lateral earth pressure assumptions) and limit-equilibrium methods (e.g. wedge stability model and reduction in face support due to slurry infiltration). The differences in required pressures from these various methods can often be substantial, and leads to confusion and uncertainty in which support pressure should be used. The extensive TBM instrumentation and ground deformation monitoring during the East Side Access Queens bored tunnels project provides the opportunity to analyze the TBM support pressures achieved and their comparison to the various estimates for required support as well as the observed ground deformation. The results show that when the face pressure is equal to or greater than rule of thumb and wedge stability (safety factor = 1.5) required minimum support pressures, near zero deformation was maintained. The minor deformation that was observed is related to the estimated annulus shield gap pressure being less than the vertical overburden.

INTRODUCTION

In soft ground tunneling, ground deformation is controlled by support pressures at the face, radial shield and liner annular gap. Design estimates for these support pressures can be calculated using a number of methods including both conventional 'rule of thumb' and more complex limit-equilibrium models. These methods consist of their own assumptions, limitations and safety factors which can lead to substantially different recommended support pressures to achieve stability. This can often lead to confusion and uncertainty in which support pressure should be used.

The East Side Access Queens bored tunnels project involved the construction of four near surface, closely spaced metro transit tunnels beneath the rail yards and mainline railroad tracks in Sunnyside yards in Queens, New York (see Figure 1). The tunnels were constructed by the joint venture of Granite Construction Northeast, Inc., Traylor Bros. Inc., and Frontier-Kemper Constructors, Inc. in 2011 and 2012. The tunnels were excavated using two 6.9 m (22.5 ft) diameter Herrenknecht slurry shield TBMs primarily through variable glacial till soils and outwash deposits. The project was considered to be very successful in that the majority of ground deformation was below 10 mm, enabling the rail tracks to remain in service throughout the entire duration of the project.

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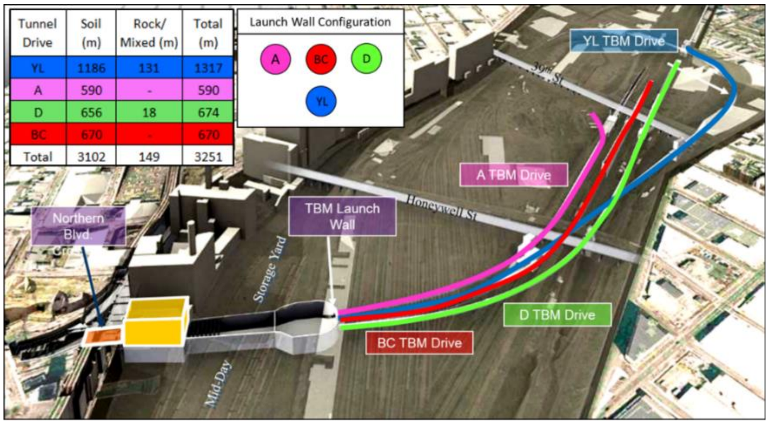

Figure 1: Queens ESA launch wall configuration and tunnel drives.

Ground deformation control was achieved by careful attention to the TBM support pressures, namely at the face, radial shield and liner annular gap. The conventional empirical approaches for estimating settlement that will occur due to tunneling are based on past experience and evaluation of measurement data from completed projects (Mair et. al. 1996), and highly dependent on the projected volume loss that will occur, in addition to the tunnel depth and ground conditions. However, the volume loss that occurs is dependent on the type and quality of the tunnel drive and construction, i.e. the ability to achieve stability at the cutterhead face and radial gaps around the TBM shield and segment liner.

This paper begins with a brief background of the project and to the common methods for determining the required face and radial support pressures. A detailed analysis of the machine data, namely the support pressures, is then presented and compared to conventional rule of thumb (e.g. lateral earth pressure assumptions) and limit-equilibrium methods (e.g. wedge stability model and the reduction in face support due to support fluid infiltration). The paper then circles back to deformation to see if there is a relationship between observed settlement and support pressures.

BACKGROUND

The four tunnels totaling 3,251 m in length (refer to table in Figure 1 for individual tunnel lengths) were constructed by the joint venture of Granite Construction Northeast, Inc., Traylor Bros. Inc., and Frontier-Kemper Constructors, Inc. in 2011 and 2012. Two 6.9 m (22.5 ft) diameter Herrenknecht slurry shield TBMs were used. The cross-section at the launch wall (Figure 1) illustrates the four tunnel configuration. At the launch wall, excavation of tunnel YL began at a depth of 22.9 m below the existing ground surface. Tunnel A began 11.9 m deep and tunnels D and BC 11.7 m deep. Tunnel YL was driven first, followed by tunnels A, D and BC. The project is described in detail in Robinson & Wehrli (2013a,b). For this paper, only results from tunnel BC are presented.

Geology

The ground conditions encountered during tunneling primarily consisted of glacial till soils and outwash deposits, consisting mostly of sand with silts/clays and gravel. In addition, large boulders (up to 2 ft) and cobbles were frequently encountered in the glacial till stratum. The majority of tunnels A, D and BC were excavated in the glacial till soil. The first 130 m of tunnel YL was excavated in fractured gneiss bedrock while the rest of the excavated alignment encountered glacial till and Gardner's clay soils. The longitudinal geological cross section for tunnel BC is presented in Figure 2.

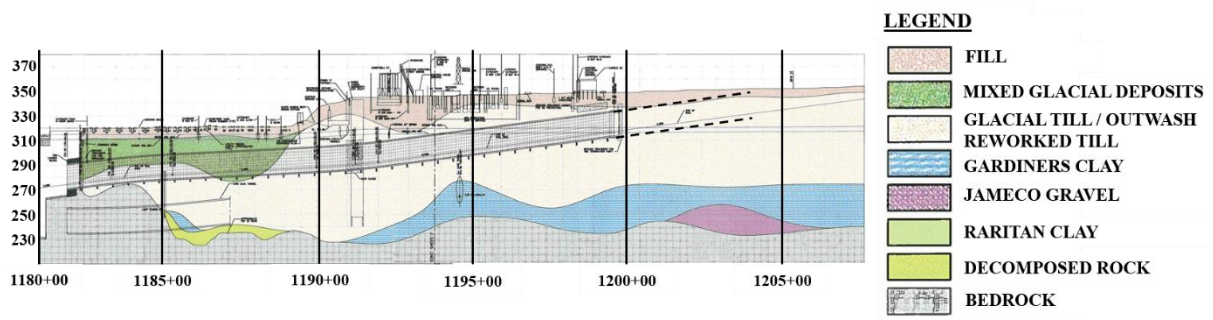

Figure 2: Geological cross section for tunnel BC.

Ground Deformation

This paper focuses on the ground deformation measurements taken on the mainline tracks between the Honeywell St. and 39th St. bridges (refer to Figure 1). Ground deformation monitoring was performed using an automated motorized total station (AMTS) with a measurement frequency of 10/day. The AMTS survey started well before the start of tunneling and continued until well after tunneling was completed. The majority of the rail monitoring points had settlements less than 10 mm (0.4 in).

Support Pressures

In order to limit surface settlement, support of the face and annulus around the shield and liner gaps is utilized to counteract the soil and pore water pressures. The support comprises of three primary components: (1) slurry at the cutterhead for face support, (2) slurry around the TBM shield for radial shield gap support and (3) grout around the liner annulus for liner annular gap support. Figure 4 illustrates the primary components of a typical slurry shield TBM and the applied pressures.

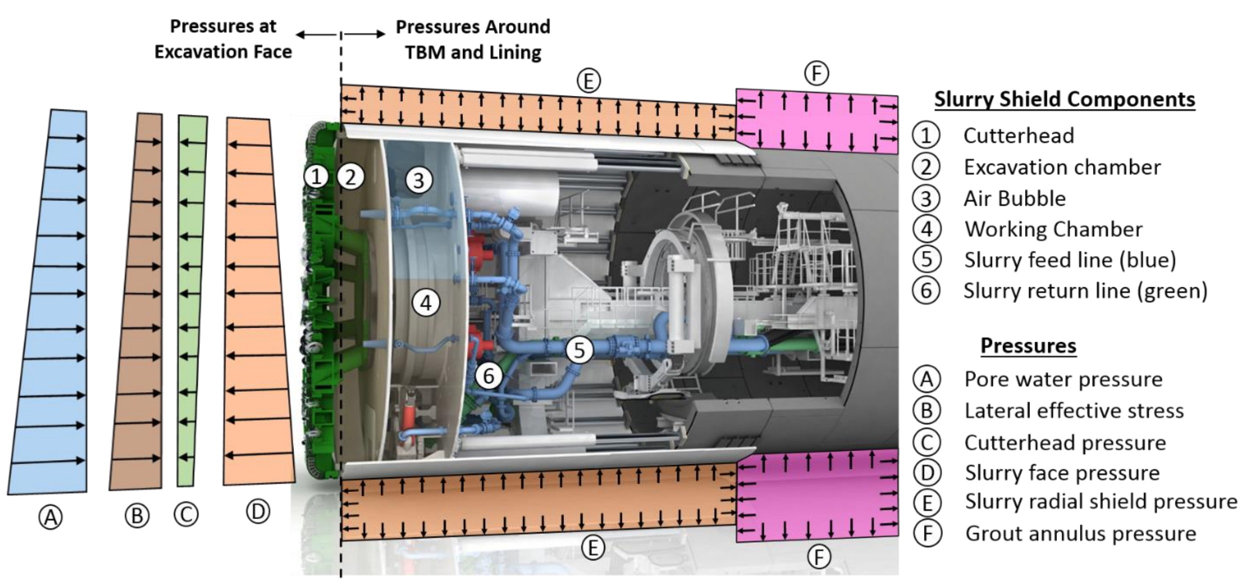

Figure 3: Slurry shield components and active pressures

In practice, there are a number of approaches used to estimate the required face support. Perhaps the most widely used approach is where the required face support s_a equals the active lateral earth pressure (lateral effective stress under active conditions), pore water pressure, or a combination of the two, plus a nominal safety margin. An often quoted rule of thumb for the support pressure at the crown is (COB 1996):

$$s_a = K_a \sigma_v' + p + 20 \text{ kPa} \tag{1}$$

In actuality, the face support employed varies from one project to the next and is generally based on previous experience of the contractor. For example, Table 1 lists the support pressure used in several Japanese slurry TBM tunneling projects. It is evident that the parameters used for estimating the necessary support pressure can be substantially different between projects.

Table 1: Support pressure used in several Japanese slurry TBM tunneling projects (Kanayasu et. al. 1995)

D [m]	Soil type	Support pressure used
6.63	gravel	water pressure + 10-20kPa
7.04	cohesive soil	earth pressure at rest
6.84	soft cohesive soil, diluvial sandy soil	active earth pressure + water pressure + fluctuating pressure (~ 20kPa)
7.45	sandy soil, cohesive soil, gravel	water pressure + 30kPa
10	sandy soil, cohesive soil, gravel	water pressure + 40-80kPa
7.45	sandy soil	loose earth pressure + water pressure + fluctuating pressure
10.58	sandy soil, cohesive soil	active earth pressure + water pressure + fluctuating pressure (20kPa)
7.25	sandy soil, gravel, soft cohesive soil	water pressure + 30kPa

Another approach for determining the required face support is to use limit-equilibrium methods to determine the minimum slurry pressure required for stability of three-dimensional failure bodies. The wedge stability model, first introduced by Horn (1961), consists of a prismatic soil wedge loaded by a soil prism (Figure 5). The forces acting on the wedge include the weight of the soil prism (G_s) and the weight of the wedge (G_w), counteracted by the shear resistance on the side walls of the wedge (T) and the slip plane (K), and the required support force (s_{min}).

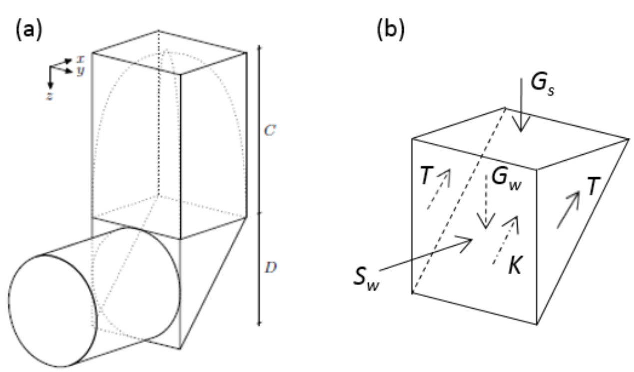

Figure 5: (a) Wedge and silo model; (b) forces acting on the wedge.

According to the Janssen/Terzaghi silo theory, the vertical effective stress acting on the top of the sliding wedge is reduced by the shear stresses acting on the sliding surface of the soil prism, often referred to as 'soil arching.' The wedge stability model has been further developed to incorporate different soil layers above the TBM (Jancsecz & Steiner, 1994) and at the tunnel face (Broere, 1998). See Broere (2001) for a comprehensive overview of the wedge stability method. The limit-equilibrium model defines the minimum required support pressure or force at failure (FS = 1). It is common practice to reduce the shear strength of the soil by a safety factor to provide a more conservative estimate of the slurry face pressure. In this study, the friction angle φ' was reduced by a factor of 1.5.

When tunneling in coarse-grained, more permeable soils, consideration must be given to the reduction in effective slurry face support as a result of slurry infiltration. Neglecting the variation in penetration distance *e* over the tunnel face, Anagnostou & Kovari (1994) derived the following expression for the reduced effective slurry support:

$$\frac{p_{SL}}{p_{SL(O)}} = 1 - \frac{e}{2Dtan\theta} \text{ if } e < Dtan\theta \quad (2)$$

$$\frac{p_{SL}}{p_{SL(O)}} = \frac{Dtan\theta}{2e} \text{ if } e > Dtan\theta \quad (3)$$

where p_{SL} is the reduced support force, $p_{SL(o)}$ is the support force at $e = 0$, D is the cutterhead diameter and θ is the angle of slip between the horizontal and the wedge plane.

In the East Side Access Queens bored tunnels project, the CQ031 contractor determined the required face support pressure based on the Leca/Dormieux method using a safety factor of 2.0, and estimated settlements were derived from the convergence-confinement method. This analysis determined a baseline for the minimum pressure required to maintain face stability and a maximum allowable pressure to safeguard against blowout. Both settlement and face pressure were optimized together, but keeping desired minimum settlement criteria a priority. Overall, face pressure was increased as necessary without reaching the blow out maximum pressure. Using varied sections along the combination of the tunnel alignments, a series of face pressures and settlements were established and then interpolated to provide the proper face pressure with respect to tunnel chainage. The face pressure criteria was then finalized to values achievable by the TBM.

In addition to the face stability, support against collapse and significant deformation of the radial shield and liner annular gaps must also be achieved to limit ground deformation. To prevent deformation of the cavity at the crown, the slurry and grout support pressures should match the vertical total stress at the crown. The vertical stress will not be geostatic, however, as soil arching does occur. On the other hand, in the case of the slurry support in the radial shield gap, the pressure should not exceed the vertical total stress to prevent a blow-out situation. The radial shield slurry pressure is hydraulically linked to the slurry face pressure, which requires a unique balance between face and radial shield support pressure to achieve stability against both collapse and blow-out.

RESULTS

Stability of the face was assessed using two methods for determining the minimum support pressure (1) 'rule of thumb' as provided in Equation 1 (s_a) and (2) wedge stability analysis (s_w) using a factor of safety = 1.5. Stability at the crown around the shield and liner was assessed using both the geostatic vertical surcharge (σ_v) and the reduced vertical surcharge according to the Janssen/Terzaghi silo theory ($\sigma_{v(silo)}$) at the crown.

Face Stability

Figure 6 presents the face pressure results (reported at the springline) for tunnel BC. Data for the first 200 ft of the alignment is generally unreliable and has been omitted from the results. The active slurry pressure (p_{SL}) was measured using pressure sensors in the excavation chamber, and assumed that the pressure of the slurry support in front of the cutterhead is the same as the pressure in the excavation chamber, a reasonable approximation (Bezuijen and Talmon, 2014). The reduction in effective face support resulting from slurry infiltration was also considered, although the penetration depth was found to be minimal and only resulted in a maximum pressure reduction of 5%. The total geostatic vertical stress (σ_v) at the springline is also presented to assess the potential for blow-out.

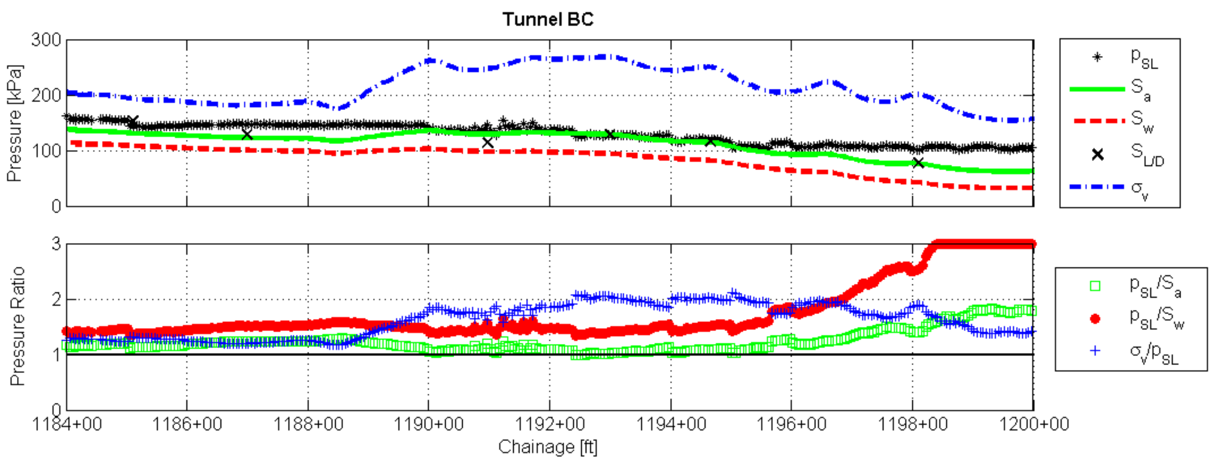

Figure 6: Face support and minimum requirement for stability at springline (top plot); pressure ratios (bottom plot) - tunnel BC

For tunnel BC, the face support p_{SL} was approximately 150 kPa (1.5 bar) at the beginning of the drive and gradually reduced to 95 kPa by the end of the drive. p_{SL} was found to be slightly greater than the rule of thumb (s_a) for the first 600 ft shown (average p_{SL}/s_a = 1.2) and then essentially equal to s_a for the next 600 ft of the drive. For the final 800 ft of the drive, p_{SL} was significantly greater than s_a as p_{SL}/s_a gradually approached above 2.0 by the end of the drive. p_{SL} was 1.3-3.0 times greater than s_w for the first 1400 ft shown, and over 3.0 times greater for the last 600 ft of the drive. p_{SL} always remained considerably below σ_v to prevent blowout. It can also be observed that p_{SL} was equal to or greater than the required design support pressures from the Leca/Dormieux method ($s_{L/D}$).

One important observation from Figure 6 is that the two methods for estimating face support produce substantially different results. Using the wedge model with a safety factor of 1.5 yields a considerably lower minimum support pressure (15-55%) than the rule of thumb approach, which includes a safety margin of 20 kPa. Note, without the 20 kPa safety margin, several sections of the alignment would demonstrate that p_{sl} is less than s_a. For the wedge model, if one reduced φ' such that p_{sl}/s_w = 1, the safety factor would increase to 2.5. Lastly, it is fairly well understood that arching exists above the crown when C/D > 2-3, however, full overburden is assumed in the 'rule of thumb' method.

These differences highlight the confusion and uncertainty often expressed in the industry today over which method should be used to determine the minimum support pressure. While the 'rule of thumb' approach provides a conservative support pressure for face stability, this support pressure could be too high such that it overcomes the reduced vertical surcharge from arching that a blow-out can occur. Conversely, while it's understood that arching exists, the mechanics of arching are not fully understood causing it to often be neglected from the calculations. In practice, a great value is placed in the experience of the TBM operator, especially in slurry TBMs. The ability to fine tune the face pressure based on the readouts from the TBM sensors while interpreting the data from the ground deformation monitoring is key to a successful operation. To this day, contractors and TBM operators continue to debate which approach is appropriate.

Annulus Stability

Figure 7 presents the gap pressure results for tunnel BC acting at the crown. The slurry pressure supporting the radial shield gap (p_{SL}) was interpolated from the pressure measurements in the excavation chamber. It is assumed that the slurry pressure in the radial shield gap follows the same gradient as the slurry in the excavation chamber. The grout pressure supporting the liner annulus gap at the crown (p_G) was interpolated from the measured grout injection pressure. The potential for blow-out due to the slurry pressure was also assessed.

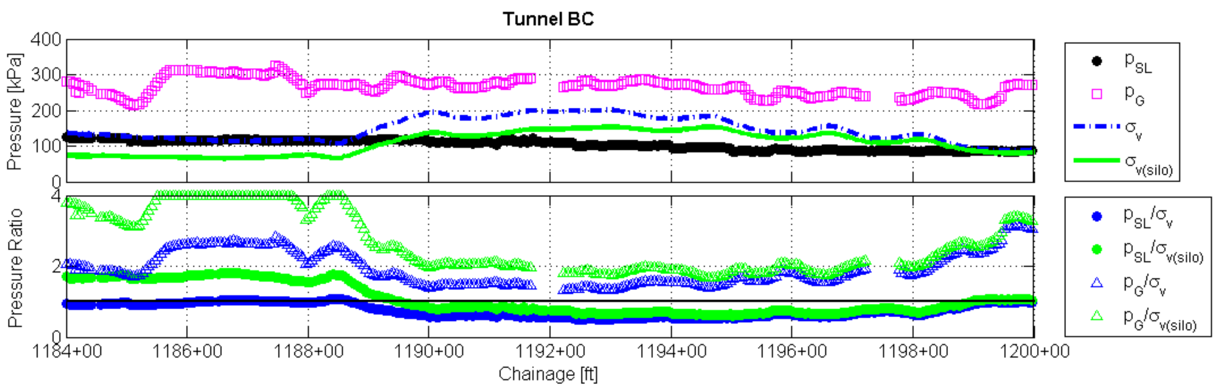

Figure 7: Slurry and grout pressures compared with vertical surcharge at crown (top plot); pressure ratios (bottom plot) – Tunnel BC

With regards to stability of the radial shield gap, p_{SL} was found to be greater than $\sigma_{v(silo)}$ and generally equal to σ_v for the first 500 ft of the alignment shown. Between chainage 1192+00 and 1201+00, however, p_{SL} was less than σ_v and $\sigma_{v(silo)}$ (average p_{SL}/σ_v FS = 0.53; $p_{SL}/\sigma_{v(silo)}$ FS = 0.71). This section of the alignment is beneath the mainline tracks and AMTS monitoring zone and provides a stong case for the slight settlement observed. Convergence of the soil around the radial shield gap would be limited, however, to the difference in cutting diameter to tail shield diameter (50 mm for the TBM used on this project).

Concerning the stability of the liner annulus gap, p_G consistently exceeded σ_v and $\sigma_{v(silo)}$, indicating that stability at the crown of the liner annulus was well-achieved. Since the grout was mixed with an accelerant for rapid curing, blow-out due the high grout pressures was not a concern and did not occur.

DISCUSSION

As demonstrated in the results, face and liner annulus support pressures were generally well-achieved and corroborate the near-zero ground deformation observed throughout the project. Albeit small, there was some ground settlement that warrants discussion. For the analysis, 10 AMTS monitoring points placed directly over the tunnel BC alignment over a span of 475 ft were compared with the resulting pressure ratios. Figure 8 presents the immediate settlement due to tunnel BC *only*. For this set of monitoring points, ground settlement ranged from 0-8 mm, with a larger settlement occurring between 1191+00 and 1192+00.

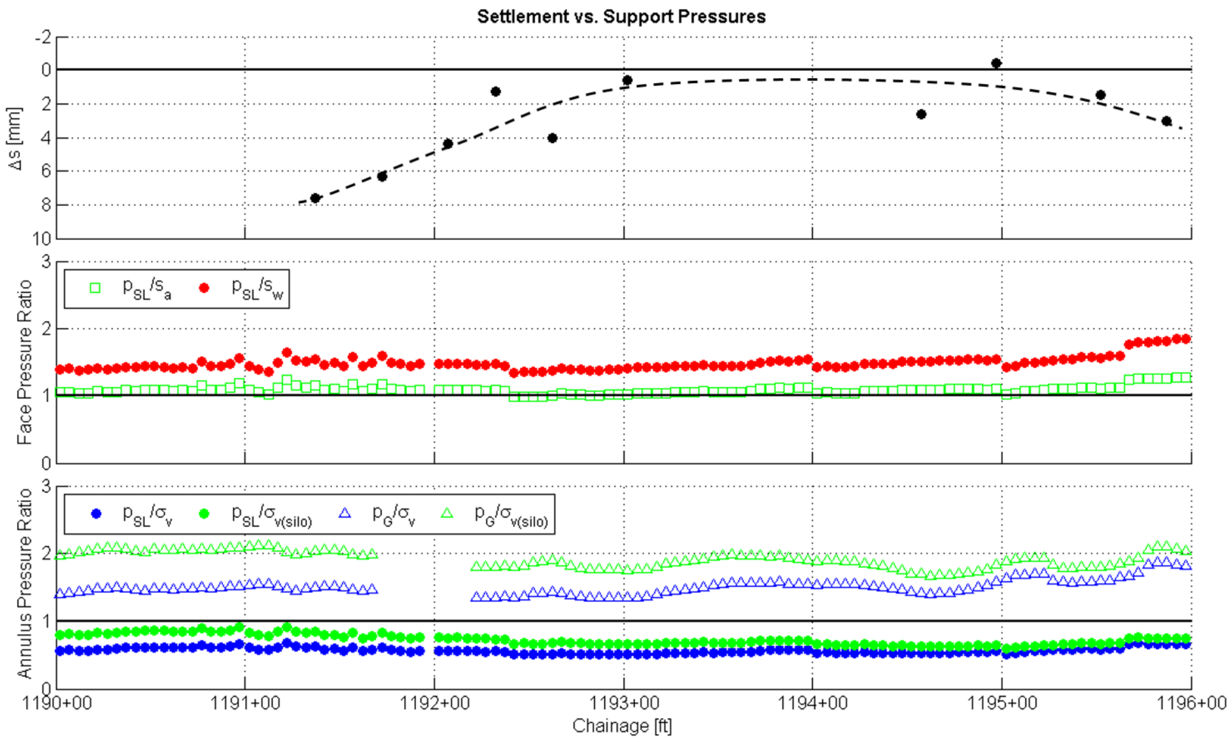

Figure 8: Observed settlement (top plot) compared to face stability (middle) and crown stability (bottom plot).

As reiterated in Figure 8, the face pressure ratios were ≥ 1.0 for P_{SL}/s_a and ≥ 1.3 for p_{SL}/s_w. For the crown stability, grout pressure in the liner annulus was high enough to achieve a pressure ratio ≥ 1.5. The slurry pressure at the crown of the radial shield gap was significantly less than the total overburden stress. If arching was minimal, this may be the reason for the slight settlement observed throughout.

To further analyze, the change in deformation for one monitoring point in relation to the relative position of tunnel BC cutterhead is presented in Figure 9. The settlement data was zeroed at x = -300 ft to isolate the influence of tunnel BC only. The moving average of 20 readings is also presented. As evident, the moving average of the ground settlement remains at essentially zero until the cutterhead has passed the monitoring point. This supports the notion that P_{SL}/s_a ≥ 1.0 and p_{SL}/s_w > 1.4 was sufficient to prevent deformation. However, immediately after the cutterhead has passed, a mild settlement of 2mm occurs before gradually consolidating an additional 1.5 mm by the time the cutterhead is 100 ft pass the monitoring point.

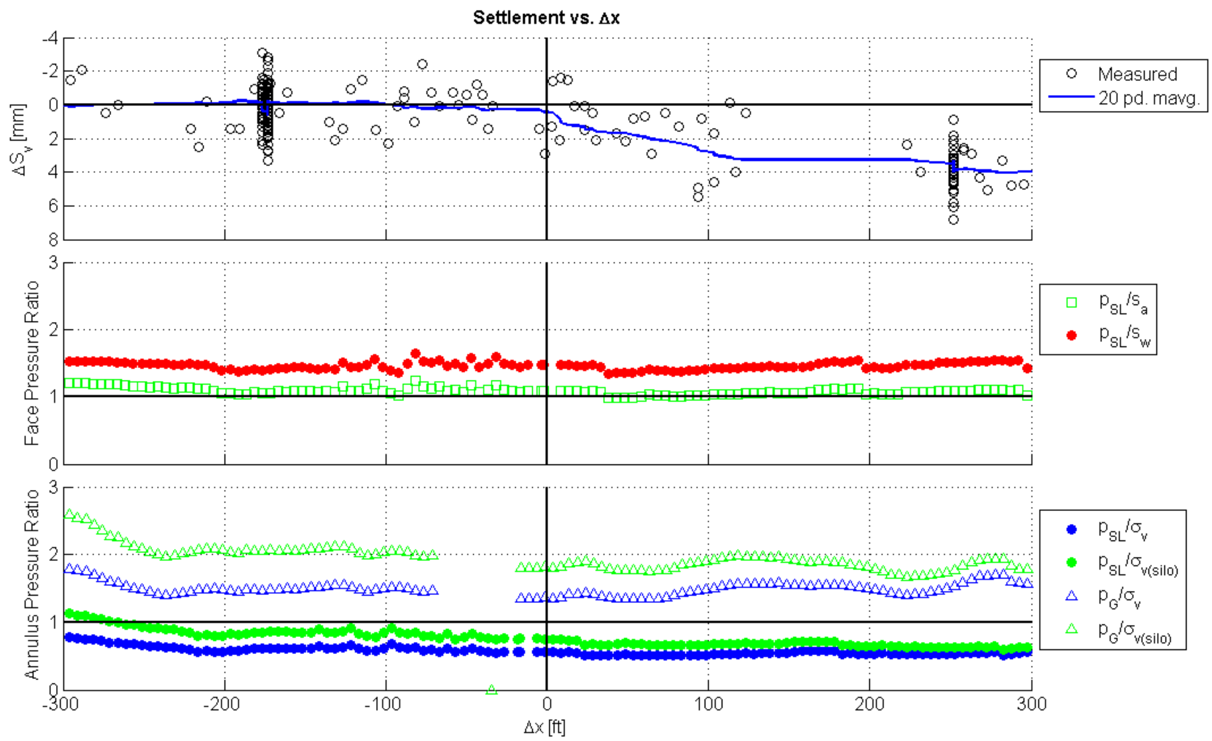

Figure 9: Observed settlement in relation to relative position of cutterhead (top) compared to face stability (middle) and crown stability (bottom).

The ground behavior indicates that an initial small deformation occurred at the crown of the radial shield gap, causing nearly 2 mm of immediate settlement followed but 1.5 mm of consolidation settlement. After the initial settlement, the rate of settlement was quickly reduced, most likely due to the high grout pressures in the liner annulus. It is plausible that if the grout support pressure was not high enough, additional settlement would have occurred.

CONCLUSION

Analysis of the different methods for determining required support pressures revealed that the methods can produce substantially different minimum required pressures to achieve stability against both deformation and blow-out. Ground deformations in this project were limited by well-controlled support pressures at the face and annulus that met or exceeded minimum required pressures estimated from ‘rule of thumb’ and limit equilibrium methods. The slight, but inconsequential settlement that did occur appears to have been a result of the comparatively low slurry pressure at the crown of the radial shield gap.

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