Tunnel Face Stability for Conventional Underground Excavation

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ABSTRACT
Maintaining stability of tunnel face is an essence to ensure safety during underground construction at any depth, through any ground condition and excavated conventionally or using TBM. This is especially the case in urban environments, where tunnels are typically shallow and often driven in increasingly difficult ground conditions. While a progressive collapse of the tunnel face may lead into a chimney caving mechanism, the influence of a tunnel collapse or extensive deformations in urban environment can be catastrophic, and even limited soil deformations may damage buildings. Keeping the tunnel face stable becomes more challenging when the ground is soft, non-cohesive and permeable with groundwater seepage. TBMs with advanced technologies maintain face stability by using earth or slurry pressure on the excavation face; but not all tunnels can be excavated using TBM due to site limitations. In conventional excavation methods, the tunnel face should be stabilized by using face dowels, face shotcrete, jet grouting, staggered excavation sequences and other techniques, so called “tools in the toolbox” in the tunneling industry.

This paper discusses some case histories of tunnel collapses due to face instability followed by a literature review on exiting stability analysis approaches for tunnel face. Different approaches and tools that are common and innovative and used to increase the safety factor of face stability in conventional mined tunnels are discussed. After demonstrating the importance of monitoring the face deformation, some guidelines are recommended to control this deformation to ensure safety and minimize impacts on exiting urban infrastructure.

1 INTRODUCTION
With the continuous increase in underground urban development, tunnels need to be built in increasingly difficult ground conditions. In most cases, the ground by itself is not stable and face stability is achieved by earth or slurry pressure on the excavation face, in bored tunnels, and by using tools such as face dowels, face shotcrete, jet grouting, and staggered excavation sequences, in conventionally mined tunnels.

Maintaining the stability of tunnel face becomes more challenging when the ground is soft, non-cohesive and permeable with groundwater seepage. Typically, ground can be classified into three main categories as depicted in Figure 1 based on their core-face stability.

In general, we would categorize tunnel failures into three main failure modes: (1) crown failure, (2) core failure, and (3) crown-core failure. Figure 2 schematically depicts these failure modes for an SEM tunnel. The principal stress directions affect the stability of the tunnel face. Core failure is more prevalent when the maximum principal stress, $\sigma_1$, direction is parallel to the tunnel drive and crown failure is more prevalent when the tunnel drive is perpendicular to $\sigma_1$ direction. Figure 3 illustrates diagrammatic core failure mechanism.

![Figure 1. Ground Behavior Categories](image1)

(a) Stable  (b) Stable in short term  (c) Unstable

![Figure 1. Ground Behavior Categories](image2)

(1) Crown failure  (2) Core failure  (3) Crown-core failure

![Figure 2. Tunnel Failure Categories (Health and Safety Executive, 1996)](image3)
This paper discusses some case histories of tunnel collapses due to face instability followed by a literature review on exiting stability analysis approaches for tunnel face in Sections 2 and 3, respectively. Section 4 briefly discusses common conventional sequential and full-face excavation methods as well as tools that are used to increase the safety factor of face stability in conventional mined tunnels. Section 5 presents a guideline for controlling deformation and face stabilization in full-face conventional excavation.

2 CASE HISTORIES OF TUNNEL COLLAPSES

This section briefly discusses some case histories of tunnel collapses due to face instability found in literatures. Table 1 summarizes the case histories and potential causes of their face collapses.

Table 1. Case Histories of Tunnel Collapses

<table>
<thead>
<tr>
<th>Case History</th>
<th>Cause of Face Failure</th>
</tr>
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<tbody>
<tr>
<td>Trans-Caucasian Railway</td>
<td>Local coaly &amp; clayey shale layers weakened by oblique slip surfaces &amp; fissures damped with water</td>
</tr>
<tr>
<td>National Highway A16</td>
<td>Geology</td>
</tr>
<tr>
<td>Grauholz Project</td>
<td>Thin layer of overburden, partially cohesionless soil layers, &amp; changing the face support from bentonite to air</td>
</tr>
<tr>
<td>Seoul Metro</td>
<td>Weathered granite at the tunnel face &amp; high groundwater pressure</td>
</tr>
<tr>
<td>Vereina Tunnel</td>
<td>High ratio of UCS/σ, was high &amp; principle stress direction</td>
</tr>
<tr>
<td>Laerdal Tunnel</td>
<td>Principle stress direction</td>
</tr>
<tr>
<td>Tala Hydroelectric</td>
<td>Poor control of groundwater at crown</td>
</tr>
</tbody>
</table>

2.1 Trans-Caucasian Railway Project, Russia

Dilizhan Tunnel experienced two face collapse incidents while the tunnel was being excavated using TBM through coaly and clayey shales. In June 1983, while driving the tunnel, a rock-fall from the face and the roof took place with about 270 m$^3$ of the rock fall-out. In July 1983, while finishing the work with a seemingly stable condition at the face, the front part of the decks had a sudden fall. Two miners were completely covered with earth. The collapse was caused by the existence of local coaly and clayey shale layers weakened mostly by oblique slip surfaces and fissures damped with water (Vlasov et al. 2001).

2.2 National Highway A16 Project, Switzerland

Mont Russelin Tunnel located in Delémont, Switzerland, had an incident of face collapse with water inrush in 1989 while being excavated using TBM through limestone and tectonised marlstone with overlapping planes and faulted zones formed of alternating sub-horizontal layers. The progress of the drive was halted after about ten meters because of subsidence caused by incoming water with the molasses. Furthermore, the TBM cut into a major karst causing major problems. Moreover, an unstable or collapsing cutting face and a trapped shield occurred when traversing the marls of the anticline. The overbreak in front of the head reached 5 to 6 meters and sometimes risky human intervention was necessary in front of the head of the TBM (Kovari and Descoeudres, 2001).

2.3 Grauholz Project, Bern, Switzerland

A face collapse with settlement at the surfaced occurred in 1990 during construction of the Tunnel. The tunnel was excavated using a mix-shield TBM. The geology includes difficult, constantly changing subsurface condition with varying glacial moraine deposits, ice-marginal deposits of silt, sand, and gravel. Various problems such as collapse of the face and settlements of the surface in shape of craters led to a low average advance rate. The face collapse occurred when the bentonite slurry was pumped out, such that the upper part of the face was supported by overpressure. The reason for the collapse was combination of three factors: the thin layer of overburden, partially cohesionless soil in layers, and changing the face support from bentonite to air. A further collapse occurred, blocking the cutterhead during an attempt to return to hydraulic support after changing the cutters (Isaksson, 2002; Herrenknecht, 1992).

2.4 Seoul Metro – Phase 2, Korea

In January 1993, during construction of Seoul Metro tunnel near Yongdungop in soft rock using NATM (New Austrian Tunneling Method) by drill-and-blast, a face/daylight collapse occurred. Figure 4 presents a photo of the collapse at the surface.
The tunnel collapsed starting from the left side of the crown after removing spoil. Approximately 900m² of loose material flowed into the tunnel and water inflow of up to 300 liters/minutes was recorded. Possible cause of failure was weathered granite at the tunnel face and high groundwater pressure. Remedial measures consisted of backfilling the crater with soil followed by cement grouting and chemical grouting. Lessons learnt was insufficient ground investigation, unexpected groundwater inflow, no tunnel face stability analysis, and no consideration of blasting effects close to weathered zone with shallow cover (Lee and Cho 2008).

2.5 Vereina Tunnel of Central Station, Switzerland

The tunnel located in Klosters, Switzerland, had an incident of face collapse with roof caving in 1994 while being excavated using TBM through crystalline rock with sections of severely sheared and altered rock; and another section of stable crystalline rock with a tendency for brittle fracture. There were cave-ins on more than one occasion both above and in front of the cutterhead. Indeed the TBM became jammed at certain spots. Therefore, the TBM drive was extremely time-consuming (Kovari and Descoeudres, 2001).

Since the crown seems to be generally stable, this is face stability issue where the ratio of UCS/σ; was high and due to principle stress direction. This is a problem when tunneling in glacial moraine parallel to principal stress direction. It is believed they may have cause excessive wear on machine when tunneling parallel to principal stress. It should be mentioned that there are terminal or end moraines and lateral moraines. Lateral moraines are in general parallel to principal stress whereas end moraines are perpendicular to principal stress.

2.6 Laerdal Tunnel, Norway

In June 1999, Laerdal Tunnel in Sogn and Fjordane had an incident of face collapse with rock burst while being excavated conventionally by drill-and-blast through banded and veined Precambrian gneisses with gabbroitic composition and massive syenitic or monzonitic augen gneisses. The tunnel suffered rock burst over much of its route. The scheduled breakthrough had been delayed because of an 18 m long, up to 10 m deep tunnel collapse at the face. A fall of rock 10 km from the Aurland portal was grouted in place and remined. Approximately 1200 m³ of collapsed materials filled the excavated area. Due to adverse rock conditions there are high stresses, which have necessitated the application of heavy support with rockbolts and fiber reinforced shotcrete (Seidenfuss, 2006).

2.7 Tala Hydroelectic Project, Bhutan

In 1999, Tala Hydroelectic Project had a face collapse in their tunnel while excavating through moist crushed rock by drill-and-blast. Because of geological problems, the commissioning of the plant had been delayed and the costs escalated. The project required 22.25 km x 50 m² headrace tunnel, 2 x 992 m long pressure shafts, a 2.2 km x 60 m² tailrace tunnel and an underground powerhouse. Around 80% of the strata were classified as poor or very poor, which caused considerable tunneling delays. Blockages in inclined and vertical shafts delayed completion. The excavation at the 6 km mark had to deal with a 15 m long face collapse when it intersected and aquifer 122 m from the portal. Geologists described the conditions as the worst for tunneling that had been met anywhere in the world yet. The excavation of the tunnel took about 20 months, which in normal condition would have taken about two months (Seidenfuss, 2006).

3 FACE STABILITY ANALYSIS APPROACHES

A number of authors has described external failure mechanisms of the tunnel face and derived formulas to calculate a suitable support pressure by analytical or empirical means. Historically, the first models were upper and lower bound plasticity solutions for an associated cohesive material. Later, solutions for non-associated Mohr-Coulomb material and limit-equilibrium models were published. The different models will be described briefly in this section (Broere 2001).

Broms and Bennermark (1967) derived one of the first models by deriving a relationship describing the stability of unsupported vertical openings in an undrained cohesive material. With this relationship they define the stability ratio N to be equal to the difference between total overburden stress and support pressure divided by the undrained shear strength. They found that an opening in such conditions would become unstable for N > 6. While Broms’ and Bennermark’s stability number is for soft ground tunnels, Hendron and Fernandez (1983) used ratio of α/UCS for rock tunnels.

Davis et al. (1980) investigate the stability of an idealized partially unlined tunnel heading in a Tresca material and introduce the distance P between the face and the point where a stiff support is provided. They use the vertical opening as presented by Broms & Bennermark as one of three limit cases for their stability analysis. They further derive upper and lower bound plasticity solutions for the stability of a plane strain unlined cavity and a plain strain ‘long wall mining’ problem, obtained by taking P goes to infinity.

Atkinson and Potts (1977) derived the minimal support pressure for an unlined cavity in a dry cohesionless material. They differentiate between two limit cases. The first case is a tunnel in a weightless medium with a surface load, the second a tunnel in a medium with γ > 0 but without surface loading. For this second case, they derive two lower bound solutions. For cases where c > 0 and φ > 0, which include all practically relevant cases, it can be shown however that, in these solutions, the overburden is independent lower bound solution.

Leca and Dormieux (1990) propose a series of conical bodies. Combined with different stress states, similar to those proposed by Davis et al. (1980), they derive lower and upper bound limits for both the minimal and maximal support pressure of a lined tunnel in a dry Mohr-Coulomb material. They present two sets of graphs for the
dimensionless load factors, one for minimal and one for maximal support pressures. Using these graphs the resulting support pressures can be calculated. Their [unsafe] upper bound solution for the minimal support pressure is in good correspondence with the results from laboratory and centrifuge tests on tunnels in sand by Atkinson et al. (1975) and centrifuge tests by Chambon & Corté (1994), which all fall between the bounds set by Leca’s upper bound solution and Atkinson’s lower bound solution. The accompanying lower bound solution, however, shows a depth dependence of the support pressure not observed in laboratory tests.

The minimal support pressures needed for a semi-circular and spherical limit equilibrium mechanism, which roughly resembles the mechanism used by Broms & Bennermark have been calculated by Krause (1987) in a limit-equilibrium analysis using the shear stresses on the sliding planes. Of the three mechanisms proposed, the quarter circle will always yield the highest minimal support pressure. As Krause already indicates this may not always be a realistic representation of the actual failure body. In many cases, the half-spherical body will be a better representation.

An often-encountered limit-equilibrium model is the wedge model, which assumes a sliding wedge loaded by a soil. A number of slightly different implementations have been described in the literature. Murayama (1966) calculated the minimal support pressure using a two-dimensional log-spiral shaped sliding plane (Krause 1987). A three-dimensional model as sketched in Figure 5 was already been outlined by Horn (1961), using a triangular wedge with a straight instead of log-spiral shaped front. His article offers no practical elaboration and an implementation of this model is first published by Jancsecz & Steiner (1994). They incorporate the influence of soil arching above the crown and present their results in the form of a three-dimensional earth pressure coefficient for different values of overburden and angle of internal friction $\phi$. These results are valid for a homogeneous soil only. Anagnostou and Kovári (1994 and 1996) show the eroding influence of slurry infiltration on face stability using a similar model. Mohkam and Wong (1989) described a limit equilibrium model using a roughly log-spiral shaped wedge, which has to be obtained from a variational analysis over the unknown position of the failure plane.

Figure 5. Wedge and Silo Model (Broere 2001)

A problem common to all global stability models mentioned above is that they are only suited for isotropic, homogeneous soil conditions, nor do they consider the state of stress. While practical experience shows that the stability of the face is often a problem in heterogeneous soils (Mair and Taylor 1997; Broere and Polen 1996), more so than in homogeneous soils, it cannot be quantified straightforwardly by any of those models. In such conditions, one may attempt to establish upper and lower limits to the support pressure by simplifying the geometry of the problem or using averaged ground properties. There are no clear methods to make such simplifications or obtain such averages and a certain amount of engineering judgement is necessary. To deal with this problem Belter et al. (1999) introduced a two-dimensional wedge model that can include layered soils above crown, but not in front of the face. It remains unclear from the brief description, however, whether this model includes arching effects of the soil above the tunnel or to what extent these heterogeneities influence the required support pressure. The effect of layered soils above the face can easily be included in Terzaghi’s arching formulae (1943) and thereby in the wedge models described by Jancsecz & Steiner or Anagnostou & Kováří. The model sketched by Katzenbach is a two-dimensional simplification of such a model.

A more precise assessment of the safety margins available under field conditions is hindered by the practical problems as well as the financial consequences of a full-scale tunnel face collapse, and most efforts in this direction have been made using laboratory models. Establishing a relation between (partial) safety factors and an acceptable risk level for the entire tunneling project is also complicated by the fact that most stability models have no explicit dependence on time, excavation speed or excavation distance.

Apart from the analytical models mentioned above, a number of numerical models have also been described in literature. The lion’s share are finite element calculations tailored to a certain project and are hardly adaptable for general cases. An exception is the model reported by Buhan et al. (1999). They describe the framework for a three-dimensional finite element model of the face of an EPB machine, where the drag forces of seepage flow
towards the face have been included. This model is applied to an 8m diameter tunnel, and the influence of varying overburdens on stability has been studied. They also find that the stability safety factor depends solely on the ratio of horizontal to vertical permeability and not on their individual magnitudes.

4 CONVENTIONAL EXCAVATION METHODS

This section briefly discusses two common conventional excavation methods: SEM and full-face followed by a comparison of core-face stabilization design for a tunnel, which is currently under construction in Slovakia. Face stabilization toolboxes are briefly explained at the end of this section.

4.1 Sequential Excavation Method (SEM)

SEM, also commonly referred to as NATM, is a concept that is based on understanding of the behavior of the ground as it reacts to the creation of an underground opening. In its classic form, the SEM/NATM attempts to mobilize the self-supporting capability of the ground to an optimum. Initially formulated for application in rock tunneling in the early 1960’s, NATM has found application in soft ground in urban tunneling in the late 60’s and has since enjoyed a broad international utilization in both rural and urban setting. Depending on size of the opening and quality of the ground, a tunnel cross-section is subdivided into multiple drifts. Refer to AFTES (1978) and Chapter 9 of “Technical Manual for Design and Construction of Road Tunnels – Civil Elements” by Federal Highway Administration of the US Department of Transportation (Publication # FHWA-NHI-10-034; December 2009) for history, background and guideline of SEM/NATM.

4.2 Full-face Excavation Method

Full-face excavation method, also known as ADECO-RS, is based on the analysis and control of the deformation response of the tunnel during excavation. Full scale testing, laboratory testing and numerical analysis carry out the analysis. The scope of the analysis is to achieve a theoretical evaluation of the deformation response by classifying the deformation response of the tunnel core-face in three main categories as detailed in Figure 1. The control part of the method consist in adopting stabilization measures of the core-face of the tunnel, which could be pre-confinement actions and/or confinement actions. Ultimately, the experimental onsite measurements of the deformation response are compared with the theoretical evaluation of the deformation response determined during the analysis to arrive to the final design calibration.

Full-face excavation approach can be described by the following assumptions: (a) The ground is considered as a construction material; (b) the importance of the ground deformational response during excavation; (c) the advantage of driving the tunnel always full face in every stress-strain situation; (d) the importance of the invert in difficult ground conditions (to “close the ring”); and (e) the relevance and value of stress-strain monitoring during face advance.

The complexity of the deformation response it is not simply limited to the tunnel cavity but it extend beyond it, involving the volume of ground ahead of the face. This region, which is called “advance core”, is affected by two types of deformation response: “extrusion” and “pre-convergence” (see Figure 6). The extrusion manifests itself on the surface of the face along the longitudinal axis of the tunnel while the pre-convergence of the cavity can be described as the convergence of the theoretical profile of the tunnel ahead of the face. These primary components are functions of the relationship between the strength and deformation properties of the advance core and its original stress state.

The full-face excavation approach has been applied for the first time in the Tasso Tunnel (Italy, 1988), then San Vitale (Italy, 1990) and Vasto Tunnel (Italy, 1991). Starting from its very beginning, this method has aroused considerable interest and rapidly established itself as an advantageous alternative. See Lunardi (2000, 2015a and 2015b) for more information about ADECO-RS.

4.3 Core-face Stabilization Design

The Visnove tunnel is a double lane tunnel of the D1 Highway Lietavská Lucka – Visnove – Dubna Skala, about 7,450 m long located in the Zilina district, Slovakia. The tunnel is currently under construction. Figure 7 effectively compares core-face stabilization design using SEM and Full-face excavation method.

4.4 Face Stabilization Toolboxes

Staggered excavation, which is a principal of sequential excavation method, is known to be an effective measure in reducing construction impacts of conventional excavation on existing structures because it provides early ring closure. On the other hand, face dowels can be effectively used to stabilize the face while excavation can be done full-face which means even faster closure of a stiffer ring. Nowadays, face dowels are being used in sequential excavation method as well; although the ring will be closed slower than full-face approach. Figure 8 depicts isometric of staggered excavation method versus full-face with face dowels.
To design the fiberglass face dowel, temporary support pressure on the face can be determined as (Peila 1994):

$$\sigma_c = \min\left(\frac{4N_bA\sigma_b}{\pi D^2}, \frac{4N_bS_l\tau_b}{\pi D^2}\right)$$ [1]

Where $N_b$ is number of dowels, $A$ is cross section of the GFRP dowels, $S_l$ is lateral surface of the dowels, $\tau_b$ is shear stress on the lateral surface of the dowels, and $\sigma_b$ is yielding stress of the dowels material.

Spiling or pipe-canopy became an essential part of conventional excavation regardless of full-face or sequential method. The spiling increases face stability by reducing the weight of ground silo on the top of the wedge (See Figure 5 for definition of Silo and Wedge).

Other tools such as face shotcrete and jet grouting are also being used in conventional excavation. Face shotcrete is common in both Sequential and Full-face excavation method to prevent from localized face failure at each round of excavation. When ground is granular and cohesion-less, jet grouting are utilized to improve the ground and provide stability during tunnel excavation.

5 GUIDELINE FOR CONTROLLING DEFORMATION

Figure 1 classified ground into three different categories based on their behavior during underground construction with some photos of tunnels under construction. Figure 9 below presents the ground-reaction curve for each of these categories. Before starting the excavation, numerical and/or closed-form analyses are performed and the tunnel alignment will be divided into reaches and each reach will be categorized based on the ground condition.

During excavation, the extrusion and convergence are measured with proper instrumentations such as sliding micrometer. The ground, which was categorized during design stage, then, will be verified or re-categorized, based on results of measured deformations. Figure 10 presents an outline/example of such monitoring.
Based on ground category, defined during the design stage and verified by measuring deformation, the pre-support and initial support are specified. Figure 11 effectively summarizes a guideline for controlling deformation in conventional excavation using full-face method. The ground types correspond to Figure 1 definition.

6 SUMMARY

This paper discussed some case histories of tunnel collapses due to face instability. A literature review on exiting stability analysis approaches for tunnel face was presented. Common conventional sequential and full-face excavation methods were discussed together with “tools in the toolbox” that are used to increase the safety factor of face stability in conventional mined tunnels. A guideline was given for controlling deformation and face stabilization in full-face conventional excavation.

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