CURTA ANÁLISE DOS PROBLEMAS E DO CÁLCULO DA ESTABILIDADE E DOS ASSENTAMENTOS DE TÚNEIS POUCO PROFUNDOS

A SHORT SURVEY ON CONSTRUCTION PROBLEMS AND MODELLING OF STABILITY AND SETTLEMENTS OF SHALLOW TUNNELS

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RESUMO

A construção de túneis pouco profundos tem hoje grande desenvolvimento sobretudo nos meios urbanos e suburbanos. Estes túneis, especialmente os que são escavados em solos ou rochas brandas, põem importantes problemas de estabilidade que são aqui apresentados em conjunto com as técnicas necessárias à sua resolução. Também se apresenta a modelação numérica da estabilidade na frente de escavação e dos assentamentos à superfície provocados pela construção de um túnel.

ABSTRACT

The construction of shallow tunnels has today a large development, mainly in urban and sub-urban areas. The great flows of traffic into and out of the large towns can be dealt with only by tunnelling. This paper presents a short survey of the construction methods and numerical modelling for shallow tunnels. Since tunnels in soft ground are those that put bigger problems, they are treated with some more extension. Construction techniques and excavation methods, together with stability of the face, ground movements and monitoring are dealt with.

1. INTRODUCTION

Soft ground or weak rock put the most important problems. The stability of the face of the tunnel is critical. In soft clay and water bearing sands excavation can be done only after ground treatment in the open face type of construction.

When using tunnel boring machines the stability requires earth pressure balance machines (EPBM) or slurry shield machines to ensure the stability of the face.

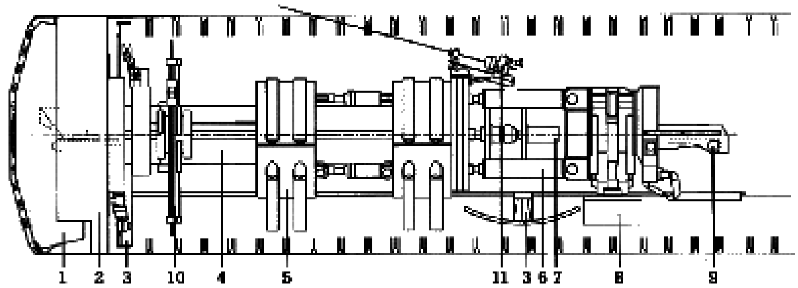
The problem of estimate the state of stress and deformation near the face is fully three-dimensional in all excavation steps. However, at present, most of the modelling in use is two dimensional with corrections to take account of the 3D situations and excavation and construction stages.

2. METHODS OF EXCAVATION AND CONSTRUCTION TECNIQUES

For tunnel in rock and for open face construction the excavation is done by the drilling and burst technique usually with partial face excavation by heading and bench or by side drifts methods (Kovári, 1998).

Temporary support is done by rock bolts together with thin sprayed reinforced concrete lining, when needed.

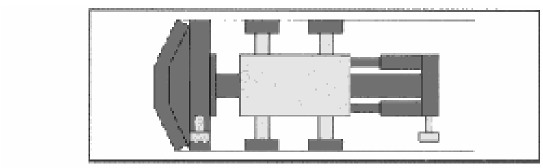
The weak rocks and soils are excavated with the usual excavation machines: The face may be excavated by parts of by full face excavation.
In the first case safety against collapse is greater but the working space may be small and the progress in the excavation can not reach large values. The reverse happens with full face excavation.
Using tunnel boring machines there is no access to the face. For the excavation of rocks the head of the TBM has a large number of cutting disks (Fig.1). Other tools such as scrapers can be added on the head of the machine to disaggregate the hard soils or soft rocks.



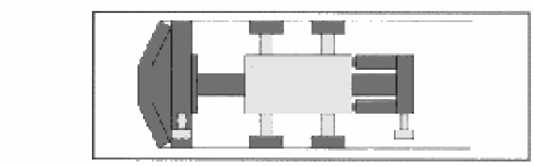
1.Cutter head. 2. Cutter head shield, hydraulically adjustable. 3. Support installation system and transport system. 4. Inner Kelly. 5. Outer Kally, two-piece, with grippers and adjusting cylinders. 6. Thrust cylinder.7. Cutter head drive. 8. Rear support. 9. Belt conveyor. 10. Roof bolting drill. 11. Probe drill.

Figure 1 - Tunnel boring machine

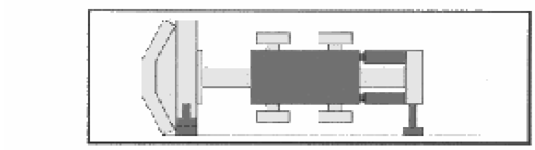
To push the head into the rock the machine is gripped to the tunnel walls. The operating cycle is shown in Fig. 2.



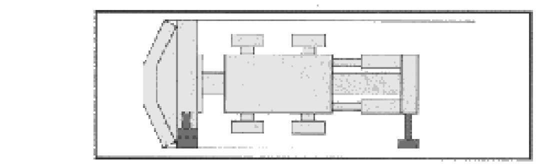
1. The machine is griped in the tunnel:Boring can start.



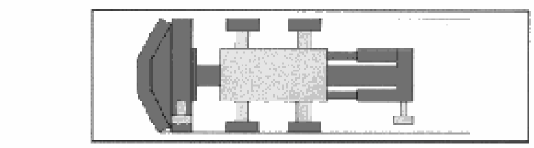
2. The cuttter head is at the end of the stroke: Stop boring.



3. The front and rear supports are extended and the grippers are retracted; the outer kelly slides smoothly forwards.



4. The machine is now aligned using the rear support.



5. The grippers are extended and the supports retracted; the machine is now ready for a new boring cycle.

Figure 2 - Operating cycle

For soft ground earth pressure balance tunnelling machines (EPBM) are used. The excavation soil and water go from the cutter head to a pressurized chamber through earth pressure balanced doors. The excavation materials are removed from that chamber by means of a screw conveyor (Fig.3) up to a belt conveyor.

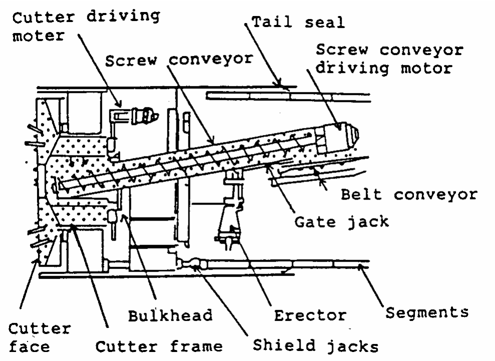


Figure 3 - Principle of the earth pressure balance machine (Fujita, 1989)

The pressure in the chamber can be controlled in order to ensure that the earth pressure balance is maintained at the face of the excavation.
For water bearing sandy soils slurry shield machines Fig. 4 are used. In these machines the face is supported by pressurized bentonite.

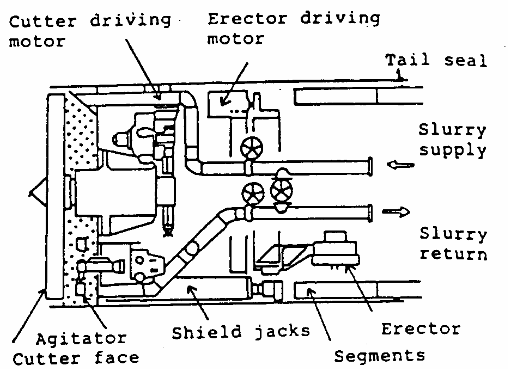


Figure 4 - Principle of slurry shield machine

This expansive clay mixes with the sandy soil and makes a cake around the tunnel. Most of the failures occur with this kind of soils for high water levels in the ground ahead of the tunnel face. There are already EPBMs able to deal with earth and water pressures up to 8 bar.
For hard rock troubles arise when there are faults and or badly fractured rock masses ahead of the tunnel face. Kovári *et al*. (1993), refer subsidence in faulting zones and collapse and falling of rock pieces ahead of the cutting head of the tunnel machine which have caused stand stills and low performance in the excavation procedure.
On the other hand troubles with tunnelling through water bearing sands are referred by Ian Clarke, ed. (1990) concerning the abandon of a tunnel machine due to the rush of water in the Thanet Sand Bed (London) with a static pression of 3 bar. The machine has been recuperated two years later freezing the zone, making a shaft and relaunching the boring with an EPBM.
The control of the position of the tunnel during the excavation is done by laser beams in the case of TBMs and by underground surveying in the case of open face excavations. Since the weight of a tunnel boring machines is smaller than the weight of soil corresponding to the same volume of tunnel, the head of the machine tends to go up and therefore the direction of its axis

has to be corrected frequently. On the other hand due to the relaxing of the rock or soil, the diameter of the cross section of the tunnel tends to be reduced. Therefore, there is always an over excavation, both in boring and open face tunnels.

3. METHODS FOR SUPPORT OF THE EXCAVATION AND GROUTING PROCEDURES

In the case of tunnelling in rock, the excavation is supported by bolting and sprayed concrete for the case of the open face construction technique. The final lining may be done by a thicker shell of sprayed reinforced concrete or a ring of pre-cast reinforced concrete segments.
For soft ground tunnels and open face excavation, there is need of ground treatment before excavation. This is done by jet grouting "umbrella arches" just outside of the roof of the tunnel (Fig. 5). Horizontal micropiles or fibber glass pipes may be used also to make the "umbrella".

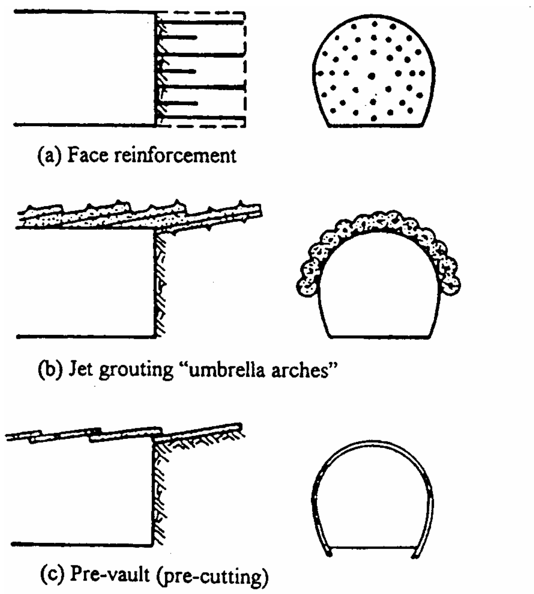


Figure 5 - Ground treatment and pre-lining techniques (after Schlosser and Guilloux, 1995)

The short term support of the excavation may be done by steel ribs. Long term lining is made of sprayed concrete or a ring of pre-cast reinforced concrete segments.
For the case of bored tunnelling in soft ground the shield of the machine supports the earth and water pressure within a very short term. The final lining, usually made of a ring of precast reinforced concrete segments is collocated as soon as excavation progresses. The gap between the tunnel ground surface and the lining after the passing of the shield is filled with grout.
When there are buildings above the excavation pre-stabilization of the ground above the roof elevation of the tunnel may be needed. That is done by grouting at low pressure or by jet grouting.

4. GROUND MOVEMENTS AT THE SURFACE

The relieve of pressure in the soil or rock and the loss of soil and rock volume ahead of the excavation face is the most important factor for the ground movements both at the surface and in depth.

The maximum settlement at the surface above the tunnel roof can be estimated on the base of the volume loss. The shape of the settlement curve normal to the tunnel axis seems to be that of a Gaussian distribution, already proposed by Peck (1969) and after by many other authors.
The volume lost depends on the type of ground and also on the type of excavation technique including the skill of working team. The minimum volume loss is obtained in bored tunnelling. However, large ground losses and surface settlements may happen when the stability of tunnel face is not well assured.
A good estimation of the volume loss can only be made on the base of previous case histories with similar conditions of ground and construction techniques. A tail void between the shield and the lining in the back of a boring machine, the deformation of the lining and consolidation in clayey soils, also cause ground movements. The immediate grouting of the gap between the lining and the tunnel wall is the way to reduce the ground movements due to tail void.

5. NUMERICAL MODELLING

5.1 Simulating the face support

Limit equilibrium solution:
- upper and lower bound 2 D solutions for clays, Davis *et al* (1980)
- upper and lower 2 D bound solutions for sands, Leca and Dormineux (1990)
- 3 D solution based on silo-theory, (Anagnostou and Kovári, (1994)

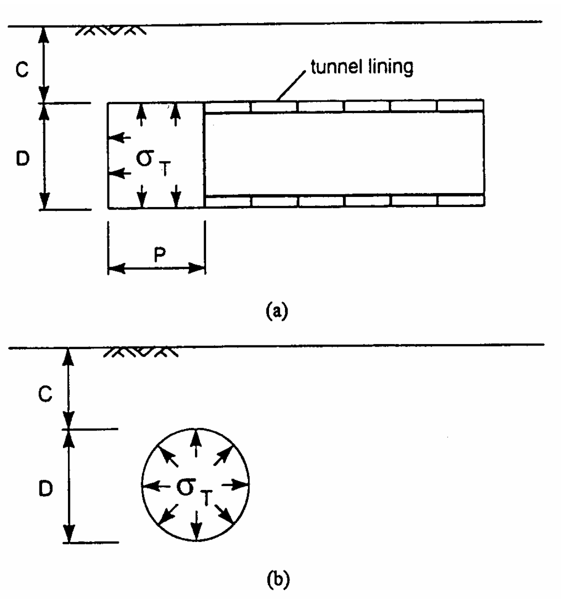


Figure 6 - (a) Tunnel heading in soft ground; (b) two - dimensional idealization of a tunnel heading

5.2 Evaluation of support pressures

For clays the stability ratio N is defined (Mair and Taylor, 1996) as

$$N = \frac{\sigma_s - \gamma\ z - \sigma_T}{q_u}$$

where

γ = unit weight of the soil

z = depth of tunnel axis (C+ D/2)

σ_s = surface surcharge (if any)

σ_T = tunnel support pressure (if any)

q_u = undrained shear strength at tunnels axis

Mair and Taylor present a family of curves for N_c (N critical) as functions of C/D and $\gamma D/ q_u$ (Fig. 7) based on upper and lower bound calculations (Davis et al). For C/D about 5 the upper bound solutions give N about 6. This is the value for the stability ratio at collapse considered by Broms and Bennermark (1967) and Peck (1969).

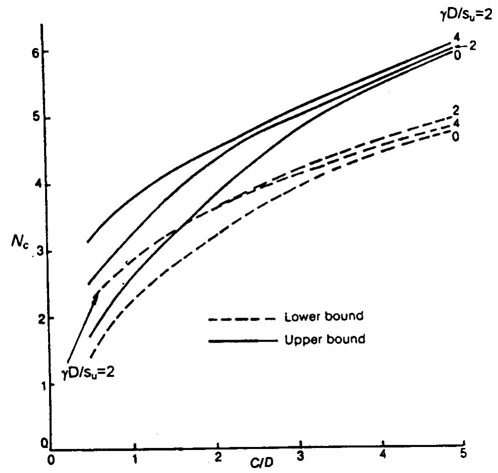


Figure 7 - Upper and lower bound critical stability ratios for plane strain circular tunnel (Davis *et al.*, 1980)

For a sandy soil (c' = 0, ϕ = 35°) Mair and Taylor (1996) give results for the dimensionless face pressure $\sigma_T / \gamma D$ calculated from upper bound and lower bound solutions in 2 D (Atkinson and Potts, 1977) and in 3 D (Leca and Dormineux, 1990), and also from limit equilibrium (Anagnostou and Kovári). They found that lower bound solutions give significantly higher pressures (σ_T / γ_D about 0.3) than the upper bound solutions (σ_T / γ_D about 0.08 to 0.15). Also they found that the face pressure would be independent of the ratio C/D (ratio of the depth to the diameter of the tunnel).

Limit equilibrium solution give a value of $\sigma_T/\gamma D$ about 0.15 similar to the upper bound 2D solution.

Attkin and Potts also gave results from centrifuge tests on lined tunnels in dry sand showing $\sigma_T/n\gamma D$ to be between 0.06 and 0.13, where ng is the centrifuge acceleration.

For the case of slurry and EPB tunnelling in water bearing sands Anagnostou and Kovári (1994) studied the stabilising force to be exerted by the slurry on the working face of the tunnel. They found that the force depends on the infiltration of the slurry into the ground which is a function of the grain size of the sand and the yield strength of the slurry.

For the case of a 10 m diameter tunnel Anagnostou and Kovári give the factor of safety as function of the excess slurry pressure (Δp), the concentration of bentonite associated with the yield strength of the slurry τ_f and grain size d_{10} of the sandy soil (Fig. 8).

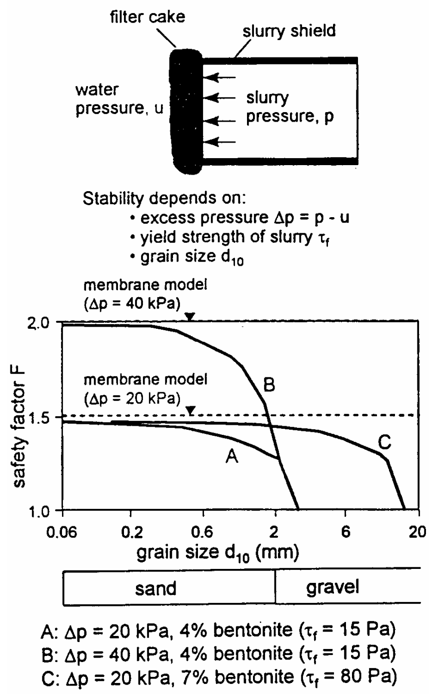


Figure 8 - Safety factor against face instability for a slurry shield (after Anatgnostou and Kovari, 1996)

It can be seen that for coarse soil, with d_{10} over 2 mm, the increase in slurry pressure does not increase safety. The slurry will infiltrate deeper and there will be fluid loss. Only for fine grained soil, the increase in the infiltration pressure will increase the factor of safety.
For coarser soils only the increase of bentonite content in the slurry will increase the safety factor.

5.3 Evaluation of surface displacements.

As already told in chapter 4, the settlements at the surface depend essentially on the volume loss during excavation. The settlements start before the tunnel working face reaches the observation site and reaches its maximum S_{max} some distance after the tunnel face passes the observation site.
As already told, the transverse settlement curve has the shape of a gaussian distribution (Fig. 9).

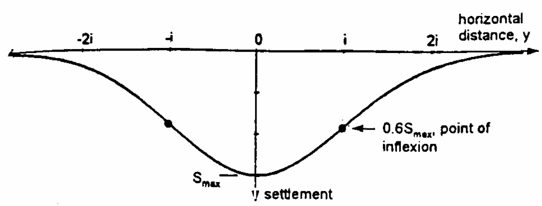


Figure 9 - Gaussian curve used to describe the transverse settlement trough

$$S_y = S_{max} \exp(-y^2/2b^2) \qquad (1)$$

where
S_y = settlement;
S_{max} = maximum settlement;

y = horizontal distance from the tunnel centre-line;
i = horizontal distance from the tunnel centre-line to the inflexion point.

Integrating (1) the volume V_s of the surface settlement trough (per unit length of the tunnel) is obtained

$$V_s = \sqrt{2\pi}.i.S_{máx.} \tag{2}$$

Usually V_s is expressed in terms of a percentage fraction of the excavated area of the tunnell.
Many authors, starting with Peck (1869), proposed a linear relationships between r and the depth H of the tunnel axis:

$$i = KH \tag{3}$$

An average value of K = 0.5 has been found for tunnel in clays (Mair and Taylor, 1996)

Loganathan and Poulos (1998) also proposed a closed-form analytical solution to predict surface settlements:

$$S_{z=0} = 4\varepsilon_0(1-\nu)R^2 \frac{H}{H^2+y^2} \exp\left(-\frac{1.38y^2}{(H+R)^2}\right) \tag{1'}$$

where
ε_0 is the ground loss (ratio), ν is the Poisson´s ratio of the soil above the tunnel, R is the tunnel radius, H the tunnel depth and **y** is the lateral distance from the tunnel centre-line.
On the basis of centrifuge testing Loganathan, Poulos and Stewart (1999) claim that (1´) gives better results than (1).

5.4 Finite element modelling in 2D and 3D

The construction of tunnels and other underground openings put the most complex problems of numerical analysis in Geomechanics. Three dimensional analysis seem to be the most suitable numerical method to tackle this kind of problems, since the state of stress and strain near the working face of the tunnel is fully three dimensional. However, most of the analysis for design is still two dimensional. That is due to the complexity of the problem.
To be complete the numerical simulation would take account of all stages of construction:
- Excavation either by parts or full face.
- Temporary support and its rigidity.
- Final lining properties.
- Changing of the state of stress and strain with time for squeezing rocks and clays.

Therefore an elasto-visco-plastic analysis would bee needed. However, most of the analyses are elasto-plastic only. Even for these there is no agreement in what concerns the best constitutive law to be adopted. The Mohr-Coulomb plasticity law with an associated flow rule is the most common.
For deep tunnels, homogeneous isotropic ground and plane strain conditions, starting with a hydrostatic state of stress σ_0, and assuming a Mohr-Coulomb constitutive law a closed form solution can be derived (Gioda, 1983). In this solution a "characteristic " or ground reaction curve is obtained, giving the internal pressure ratio σ_r/σ_0 as a function of the radial displacement δ_r.
Starting from this concept Panet and Guenot (1982) proposed the simple expression

$$\sigma_r = (1-\lambda)\ \sigma \tag{4}$$

where σ_0 is the initial isotropic ground stress prior tunnelling and λ is an unloading parameter. The stress removed from the soil before lining installation is $\lambda\sigma_0$ and the pressure applied to the lining is $(1-\lambda)\sigma_0$ (Fig.10).

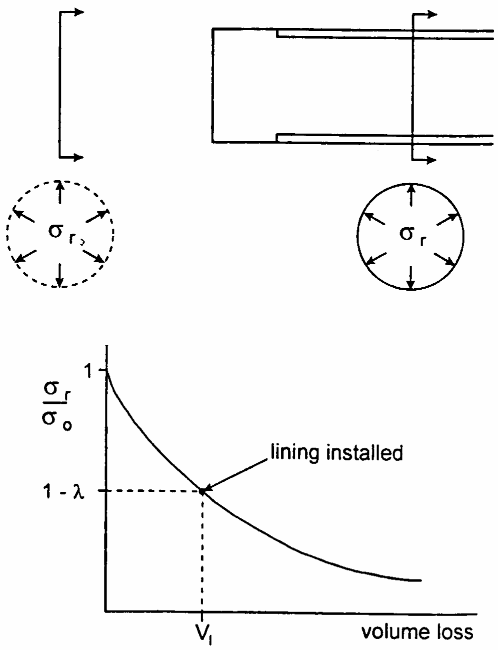


Figure 10 - Application to 2D FE analyses of the principle of convergence-confinement method (Planet and Guenot, 1982)

As the stress is removed from the tunnel boundary radial displacement δ_r occurs, and the corresponding volume loss V_l can be calculated:

$$V_l = 2\pi R.\ \delta_r \tag{5}$$

where R is the radius of the tunnel.
The volume loss V_l will be a practical value obtained from experimental results and can take account of 3D effects. δ_r is the radial displacement.
The volume loss V_l should be in accordance with field measurements (convergence measurements) and the corresponding lining stress $(1-\lambda)\ \sigma_0$ should be realistic.
A better approach seems to be the assumption that the initial state of stress is

$$\sigma_1 = \sigma_v = \gamma z \quad \text{and} \quad \sigma_2 = \sigma_3 = K_0\sigma_1 \tag{6}$$

where σ_1, σ_2, σ_3 are the principal stresses at depth z, and γ is the unit weight of the ground.
For this condition, although simple, there is no closed form solution and we must resort to a numerical solution, usually by finite elements.
The transverse surface displacements curve can be evaluated and it is found that the shape of this curve very much depends of the values chosen for K_0. For heavy consolidated clays it is usual to take K_0 significantly greater than 1. However, K_0 must be reduced on account of unloading due to 3D effects.
Using a 2D analysis and K_0=0.5 Addenbrooke *et al.* (1997), found a better agreement to field data for the transverse curve in London clay (Fig. 11).

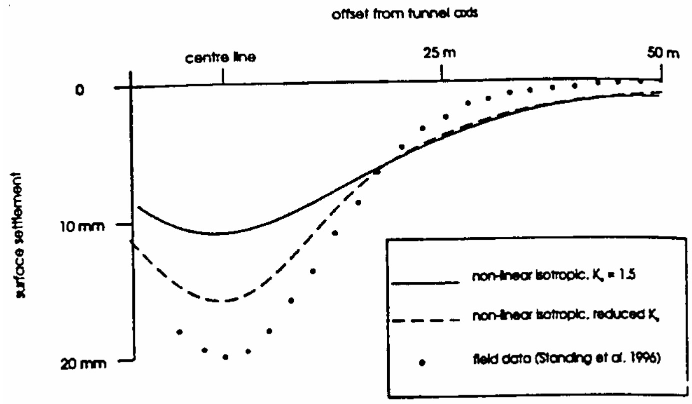


Figure 11 - Influence of K_0 on FE predictions of surface settlement (Addenbrooke, *et al.*, 1997)

It should be noted that the output of any 2D or 3D FE elasto-plastic analysis depends not only of the chosen initial state of stress, but also on the deformation and strength characteristics of the ground and on the constitutive law considered. Therefore the lack of agreement between measured and computed values for displacements and stresses derive not only from the assumed initial state of stress but also from the other input data not being representative of the actual characteristics of the ground.

Since there is always a range in the ground characteristics, several sets of calculations must be done obtaining a range of results also. Then, the measured values might be inside this range.

6. MONITORING OF TUNNELS

Displacements and stresses that result of any kind of calculations cannot be fully reliable, since any model is a great simplification of reality, the initial state of stress is not well known and the deformation and strength characteristics that enter in the computer program never represent well the actual ground behaviour.

Therefore, to get safety during the construction of a tunnel several kinds of measurement are needed.

The oldest and most important measurement is that of convergence. Usually three marks are fixed on a cross section of a tunnel: one at the roof and one at each side of the wall at a level near that of the centre. The distances between the marks in this triangle are measured and registered along the time. For a stable tunnel the difference in the readings of two consecutive time steps will rapidly decrease along the time. (Therefore, the frequency of the readings can decrease). If this is not the case the safety against collapse is not assured.

It should be noted that in any excavation in soft ground the factor of safety decreases with time, since pore pressures start to be negative just after the excavation and tend to zero or positive along the time. Negative pore pressures give an apparent cohesion to the soil, which disappears after some time.

Measurement of settlements at the surface, mainly when there are buildings, is another monitoring need. Transverse underground displacements by means of inclinometers are also a usual measurement. Also the measurement of the horizontal displacements ahead of the working face in soft ground should be done. In depth settlements above the roof are sometimes measured. Also extensometers are installed radially from inside the tunnel.

Further to these measurements, complementary horizontal and inclined borings are usually made, mainly in soft ground open-face tunnels. In this case the construction of the tunnel may be said to be performed by the "observational method", which requires a less detailed design.

In bored tunnels, borings ahead of the working face can also be done, stopping the machine and drilling horizontal holes through special openings made in the head of the machine. Sometimes also from inside the machine holes can be drilled to install bolts and anchors in the roof and walls of the tunnel.

7. CONCLUSIONS

In the open face tunnelling, where there is easy access to the tunnel face, the lining can be done with spread concrete and various ground treatment techniques can be used namely that of "umbrella jet grouting". In the case of bored tunnels the most usual lining is that of pre-cast reinforced concrete segments.

Face stability is fundamental to avoid failure. Kinematics upper bound and statically admissible lower bound solutions are available to estimate the tunnel support pressure at the tunnel head. Also limit equilibrium solutions exist to access the tunnel face stability pressurized by the slurry or EPB shields.

Ground movements depend mainly on the value of the ground volume loss during excavation. Surface and sub-surface short term troughs can be reasonably approximate as Gaussian curves. S_{max} and the widths of these curves can be predicted by F.E. analysis, but low values for the earth pressure at rest should be used (K_0 <0.5) to better fit the measured values, even in the case of pre-consolidated soils.
Realistic ground volume losses should be entered in the calculation of the maximum surface settlement. Small values of the volume loss are obtained with slurry and EPBM shield machines.

A good plan of monitoring is essential to the safety of the tunnel and of the buildings at surface. Movements both at surface and in depth must be measured and alarm limits for those movements must be stated.
Compensating grouting may be needed under the foundations of the buildings if high settlements are found during the tunnel excavation.
Complementary borings from inside and outside of the tunnel should be performed, mainly in the case of the "observational method" of construction.

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