Analysis, design, construction and behaviour of underground structures

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ABSTRACT
This paper reviews the accepted practice and discusses some lessons learned regarding the design, analysis, construction and observation of tunnels and deep shafts in soils. Some general reflections are presented based on specific experiences related to actual tunnels and shafts built in Mexico Basin.

RESUMEN
Se revisan las prácticas actuales y se discuten algunas lecciones aprendidas en materia de análisis, diseño, construcción y observación del comportamiento de túneles y lumbreras en suelos. Se hacen reflexiones de carácter general sobre el tema, basadas en algunas experiencias específicas relacionadas con obras construidas o en proceso de construcción en la cuenca de México.

1 INTRODUCTION

For the infrastructure development of large metropolis it is increasingly necessary to build large underground structures, particularly for transportation or drainage systems. These works are reaching increasing dimensions and depths, which leads to a challenging evolution of analyses and design methods, concurrently with innovative constructive procedures.

Lessons learned from the current practice for design, analysis, construction and behavior of tunnels and shafts in soft soils are discussed in this article, stressing the very particular geotechnical conditions encountered in the Valley of Mexico, and the more common constructive procedures currently used.

2 PART 1 TUNNELS

2.1 General concepts

The well known and now classic state of the art review on tunnels by R. B. Peck (Peck, 1969; Peck *et al.* 1972) is still mostly valid. The Mexican experience on this topic, especially for the Valley of Mexico, has also been the subject of useful technical publications (Farjeat, 1988; Moreno, 1991). Nevertheless, knowledge and analysis techniques keep evolving to face new challenges and to take into account new constructive procedures. Some tunnels are built at great depth. In occasions, they are located next to slopes with some instability potential, or in soils that can show significant piezometric changes during construction and operation stages, including the case of soils affected by regional consolidation process. The availability of powerful numerical models has opened the possibility to perform more refined analyses, taking into account these special conditions, complex geometries and soil heterogeneity.

To perform tunnel analysis and design in problematic soft soils, it is now deemed necessary to pay attention to the following limit states:

a) Failure limit states

The front face excavation failure, critical in soil tunnels constructed with the open front technique (Alberro, 1987; Tamez *et al.*, 1997; Soubra, 2000; Rangel *et al.*, 2007) is now generally controlled using closed front tunnel boring machines, that assure the front stability using air pressure or pressurized slurry or by means of balanced earth pressure. The Earth Pressure Balance (EPB) machines are now used in most tunnels in construction all over the Valley of Mexico not only for geotechnical reasons, but also because they allow overcoming the ecological problem of bentonitic slurry disposal.

The verification of the possible structural failure by compression, bending or bulging of the primary lining, generally built with precast segments or *dowels*, and also that of the cast in place final lining, both under the hydraulic and soil loading, is also an essential tunnel analysis and design subject. It has also been suggested to pay more attention to possible damage to the lining during the construction stage (C.B.M. Blom, 2002).

b) Service limit states

Both the design method and the constructive procedure used to excavate a tunnel in soils must ensure that the induced surface movements and those of the surrounding buildings will not exceed acceptable values (Reséndiz and Romo, 1981). Primary lining deformations must remain small enough to allow casting in place of the secondary and final lining, usually made using sliding casting molds.

Some specific subjects related to the needed analyses to review these service limit states are discussed below.

2.2 Loads transferred by the soil to the tunnel linings

To assess the soil loading on a circular section tunnel lining, many approaches are available going from the simplest ones to those that require the use of advanced numerical models.

a) Initial soil stress state

If it can be admitted that a tunnel excavation can be done without any change of the soil stress state (as by an act of magic), the lining will be subjected to the stress state corresponding to a vertical total pressure $\gamma_m z$, and to an horizontal total pressure equal to a fraction of the vertical one. Some analytical solutions are available to calculate the resulting circumferential forces acting on the lining ring as well as the bending moment, the shear forces and the radial displacements, in all the significant sections of the lining under this type of loading (Einstein & Schwartz, 1979). This simple analytical solution shows the importance of the hydraulic conditions of the soil (Rangel, 2010). In effect, the pressure transmitted by the water is isotropic and, as a result, the lining ring works essentially in compression while the effective stresses can induce shear and bending moments in the support system. These simple solutions constitute a valuable reference to evaluate by comparison the effect of other additional factors considered in more complex analysis methods. Similar analyses to the precedent, with the same advantages and limitations, can be performed in numerical form using directly structural software.

b) Local stress release and ground unloading by tunnel excavation

A more realistic analysis of the stress conditions around a tunnel requires taking into account the construction procedure or at least, its more relevant aspects, including the local stress release occurring as the soil moves towards the excavation and the ground unloading associated to the removal of soil. Using a 2D approach, Alberro (1983) evaluated the local stress release around a tunnel in which the lining is installed immediately after the excavation, in elastic and viscoelastic media. He also presented a 3D displacement and stress analysis of the conditions prevailing when a circular tunnel is excavated in a semi-infinite elastoplastic medium, (1988). An analysis based on the plasticity theory made by Caquot and Kerisel (1966) shows that, for a sand in limit equilibrium, the mean soil pressure in the key of the tunnel can be significantly lower than the vertical total pressure $\gamma_m z$ in a proportion that depends on the relation between the depth of the tunnel crown and the tunnel radius, *D/r,* taking values from 0.85 for *D/r* =0.5 *to* values as low as 0.16 for *D/r* =4. These values are similar to those obtained experimentally by Terzaghi (0.78 and 0.20). Allowing the soil internal strength to develop, it is thus feasible to reduce significantly the pressure on the lining. Nevertheless, this condition is not attainable for very soft soils, since in this type of material the initial stress state tends to establish itself again after a while.

The unloading of the medium due to removal of soil produces a general upward tunnel movement ("bubble" effect) together with a change in the stress and displacements state in the surrounding soil. This phenomenon, quite significant in soft soils such as Mexico valley clays, can be assessed by means of elasticity theory analytical solutions (Mindlin problem), or by using numerical models.

c) Loading due to changes in piezometric conditions

Following the initial stress state changes around a tunnel in soft soil, dissipation of the induced pore pressure will lead to soil consolidation and to changes in the pressures on the lining. This effect has been estimated for Mexico City clay using numerical methods but the results have not always been convincing and should be calibrated by comparison with actual measurements.

The changes in the piezometric existing conditions around a tunnel due to pumping in the area also lead to changes in the loading acting on the lining and, in soft soils, to consolidation of the ground close to the tunnel (Farjeat, 1988; Tamez, 1997). These changes can generate important loading on the primary lining during the period in which this is the only lining of the tunnel, and on the long term, also over the secondary lining.

d) Additional loading associated to external sources

In special situations, the influence on the tunnel of other external sources, such as: seismic loading, weight of foundations in the surroundings, presence of water channels or levees in the close vicinity, etc. cannot be ignored.

2.3 Specific topics regarding primary lining analysis and design

To date, the primary lining is generally made with pre-casted segments forming rings assembled *in situ i*nside the rear portion of the tunneling machine as the excavation progresses. These rings are flexible due to the dowels joints.

In most soils and particularly in old clays, it is customary to take into account that the primary lining deformation leads to a raise in the horizontal pressure at the mid height of the tunnel and that the radial pressures reach a near isotropic condition, which is very favorable for the secondary lining working conditions (Lombardi, 2010). Obviously, this can only be expected in reasonably isotropic and homogeneous soils, because any stratification presenting contrasting rigidities leads to anisotropic conditions.

The dowels structural design is generally far from straightforward. The dowels work in highly hyperstatic conditions since the deformation of each ring depends on the interaction with contiguous rings with their own joints system. It has been tried taking this complex situation into account with a double articulated ring model (Maidl and Comulada, 2010), as shown in Fig. 1, in which the inner ring represents the contribution of two half rings of contiguous dowels. The results obtained with these models have been shown to be sensitive to the stiffness parameters used for the coupling springs.

Loading to be applied to this model can be determined using Winkler springs for representing the soil, but it is considered more realistic to perform a continuum mechanics analysis in order to represent explicitly the soil-ring interaction by means of numerical techniques like the Finite Element Method or the Finite Differences Method.

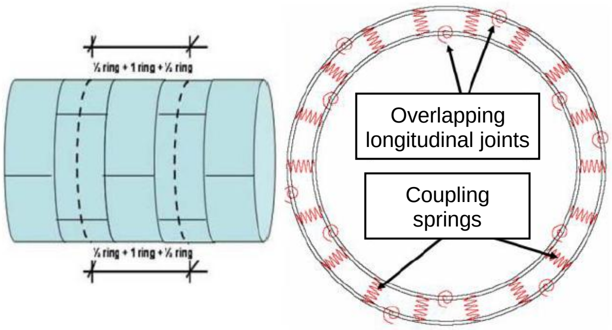


Figure 1. Double ring model (Maidl and Comulada, 2010).

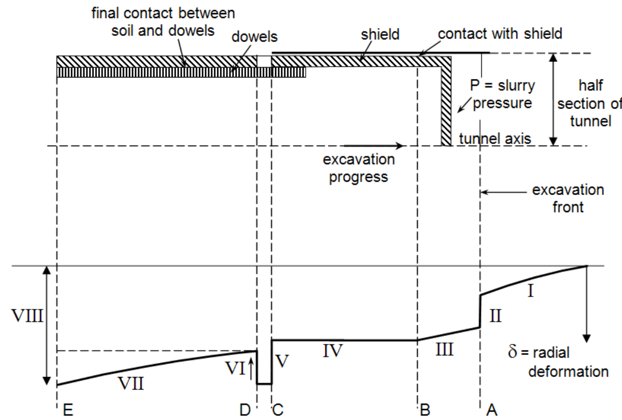
In this type of modeling it is customary to accept that, due to the joints, the dowel ring stiffness is equal to a fraction α of the stiffness *EI* of a continuous ring of the same width. The accepted α value is around 0.2 to 0.3. Nevertheless, there is evidence that the value of this parameter is not independent of the pressure conditions around the tunnel and that it tends to diminish sharply when the tunnel is submitted to a significant stress deviator. The stiffness reduction considered must be congruent with the double ring model behavior. Thus it is necessary to perform an iterative process between the continuous model and the structural one of Fig. 1. This process is cumbersome and convergence problems may arise, particularly in heterogeneous materials.

In the Finite Element modeling, the details of the constructive procedure must be represented as accurately as possible, including: the front shield pressure, the tunnel contraction that occurs in the annular space between the tunnel boring machine body and the dowels (Fig. 2), and the mortar pressure applied to fill this same space.

To achieve an adequate representation of the constructive process it is in principle necessary to carry out 3D analyses. Nevertheless, 2D analyses are commonly performed for the sake of simplicity and to reduce the computing time required. The merits as well as the shortcomings of these two scopes have been evaluated by Kastner *et al.* (2010).

Adequate representation of contraction is very important because the estimated stress state around the first lining of dowels depends on it. In bi-dimensional models it is possible to use diverse techniques to take into account the contraction effects: the progressive softening (Leca and Clough, 1992), the confinement-convergence method (Panet, 1995; Lombardi, 2010) and the reduction of the excavation diameter by an empirical factor. This last option is already implemented in leading commercial software (Plaxis, 2010).

The experience in the Valley of Mexico shows that a primary lining consisting of dowels may present an inadequate behavior under certain situations. During the excavation of a tunnel in a zone of very soft lacustrine clays, large deformations of the dowel primary lining (Fig. 3) were observed. An increase of the horizontal diameter of the tunnel of about 20 cm was measured.



Possible effects:

I. Convergence ahead of the excavation front
II. Overexcavation
III. Deformation until contact is made with the shield
IV. Additional convergence when the shield has a conical shape
V. Step at the edge of shield
VI. Possible effect of the mortar grouting
VII. Possible deformation of mortar with time
VIII. Convergence "initial deformation"

Figure 2. Initial deformation around a tunnel boring machine (Lombardi, 2010).

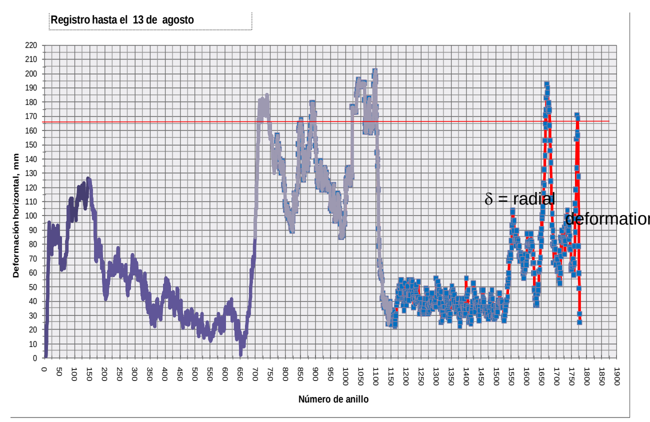


Figure 3. Horizontal deformation measurements in dowel rings (Meza and Auvinet, 2010).

These unusual deformations have been attributed to a local loss of confinement of the tunnel associated to vertical cracks in the soil and/or to general ground horizontal movements detected in the levees of a nearby channel, located close a rock hill. Nevertheless, the more likely source of the problem is probably the fact that the tunnel crosses the pressure bulb generated by those levees recently re-leveled and, as a consequence, was excavated in a soil under consolidation process. The deformation evolution confirms this last hypothesis. It was necessary to temporally reinforce the primary lining with steel frames in this span, but the problem was solved as soon as the definitive lining was installed.

2.4 Secondary lining analysis and design specific topics

It was already mentioned that the secondary lining is expected to be subjected mainly to compression stresses as a consequence of the primary lining deformation and the developing of a near isotropic soil pressure. The secondary lining could thus theoretically be designed practically without reinforcement. This assertion is only valid if the soil stress conditions, and particularly the piezometric conditions, do not change during the life of the tunnel. When water pressures may diminish significantly due to pumping, very important non symmetrical pressures can develop on the secondary lining. This severe condition can be easily simulated analytically or, in a more realistic way, by means of numerical techniques such as the Finite Element Method. The results of such an analysis are shown in Fig. 4, where the long term behavior of a shallow tunnel was evaluated in presence of regional consolidation induced by pumping, in conditions typical of those prevailing in Mexico City.

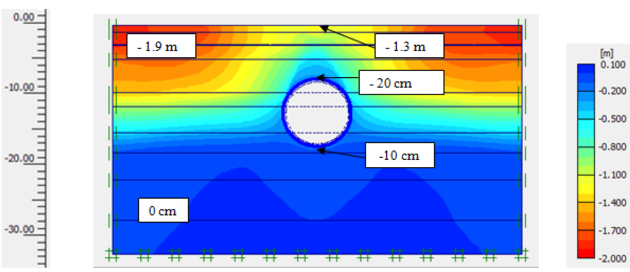


Figure 4. Numerical modeling of regional consolidation effect on a shallow tunnel. Vertical displacements (Regal, 2009).

The results show that the piezometric level drawdown condition is indeed critical, and more so when the tunnel rests on some resistant layer. This condition then generally rules the lining design. From the structural point of view, it is difficult to define if under this type of loading both the primary and the secondary lining will work together or that the loads will be taken entirely by the secondary lining. Preliminary calculations made with simplified models seem to support the second hypothesis. This problem is currently being evaluated by means of physical modeling. In the case shown in Fig.4, it can also be seen that a bump will be progressively formed on the surface directly above the tunnel. The actual pressure drawdown is difficult to forecast since it depends on the future pumping conditions. A design option can be to consider that the piezometric level drawdown will only be a fraction of the total drawdown during the useful life of the tunnel, but this implies that in the occurrence of complete piezometric level drawdown, expensive tunnel reinforcement works would be required, a non desirable heritage to the future operators.

3 PART 2: SHAFTS

3.1 General concepts

Shafts are vertical wells of large diameter used to get the tunnel boring machines down to the excavation level, but also to perform other constructive operations, and to give access to operators for tunnel maintenance works.

This kind of structure can be built using diverse techniques, among which the most relevant ones in Mexico City are the traditional direct excavation for firm soils (Moreno, 1991); the flotation method (Auvinet *et al*., 2010), the precast ring lining method (Zemva, 2010), and the slurry walls method (Moreno, 1991), the last three for soft soils. The slurry wall technique had presented constructive problems in the past, such as clay extrusion in joints, but is now again successfully used thanks to improving precision and control during construction. All these different techniques can be combined in several ways (Contreras y Alanis, 2010).

To verify the stability of this deep structures, attention must be given to reviewing several relevant limit states among which are the following (Auvinet, 2006, 2010; Tamez *et al*., 2007).

a) Failure limit state

The aspects to be considered are mainly: the guide wall stability, the stability of the trench excavated for construction of the perimetral walls, the stability of the excavated walls against general shear failure or local extrusion, and the stability of the excavation bottom against shear failure or uplift pressures. Besides, in the case of excavations made by traditional procedures, the possibility of flooding, and also the occurrence of piping of the surrounding ground must be reviewed. In all cases it is necessary to verify the flotation condition of the finished shaft. The shaft seismic behavior must also be evaluated especially in soft clays where amplification phenomena can be important (Pérez-Rocha and Avilés, 2010; Rangel *et al*., 2010; Zemva, 2010).

b) Service limit state

The shaft must be constructed without inducing non desirable movements or cracking that could affect surrounding buildings and public services. Protruding of the shaft structure from the surrounding ground in presence of regional subsidence can also have undesirable effects.

3.2 Particular topics regarding shaft safety when built by the flotation method

The flotation technique has become one of the most popular shaft construction methods in the Valley of Mexico lacustrine zone. It is performed as follows: First a circular concrete guide wall is built in order to guide the following excavations. Next, an annular trench, stabilized with bentonite slurry is excavated by sections. Then the soil central core inside the wall is excavated stabilizing the excavation also with bentonite slurry. Then a cylindrical floating tank is placed inside the excavation and the concrete shaft is built over this tank. As the construction progresses the slurry is partially displaced to allow both tank and structure to descend by stages until the design depth is reached. At this point, the annular space between soil and shaft structure is filled with mortar. The Fig. 5 shows a finished shaft sketch.

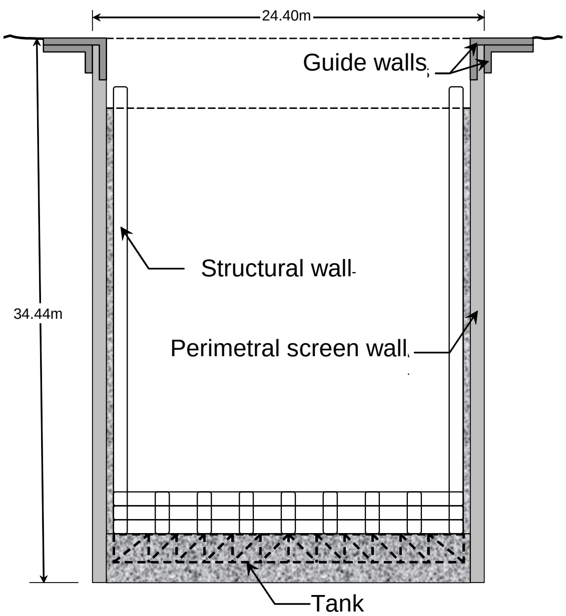


Figure 5. Shaft constructed by the flotation method with perimetral wall.

To increase the safety of the excavation it is now customary to build a perimetral screen wall with a self setting mixture of slurry and cement before the central soil core excavation is made. This screen increases the safety against wall failure, allows preventing the soil cracking and can be useful to control the piezometric conditions at the bottom of the excavation.

The main shaft stability analyses to perform are the following:

a) Guide wall stability

The guide wall stability depends on the soil characteristics near the surface. Attention must be given to the possible existence in the site of undesired fills, organic soils and cracking. It has been useful to realize a soil detailed exploration within the first meters with a portable dynamic penetrometer. When necessary, the superficial soil layers can be replaced by competent materials such as geosynthetic-reinforced gravel or soil-cement.

In order to verify the guide wall working conditions in the presence of excavation equipment and heavy cranes, the Finite Element method has been used. It is considered that the guide wall is an axisymmetric body subject to local loading. The technique proposed by Wilson (1965) to model axisymmetric geometric problems with non axisymmetrical loading conditions has been used (Dilosquet, 2003; Auvinet & Rodriguez, 2004). The local loading is approximated by using the Fourier series in order to continue taking advantage of the axisymmetrical conditions. Nevertheless, recently the 3D Finite Element Method is used.

b) Soil core stability

The soil core is a cylinder confined with slurry. The stability of this mass under its own weight can be verified by a simple evaluation of the shear stresses that develop inside the core. The Tridimensional Finite Element Method has been used for this purpose (Fig. 6). For deep excavations in the Valley of Mexico clays, the safety factor is close to one. In reality the core failure is not really critical since, even in the case of failure, the excavation can proceed.

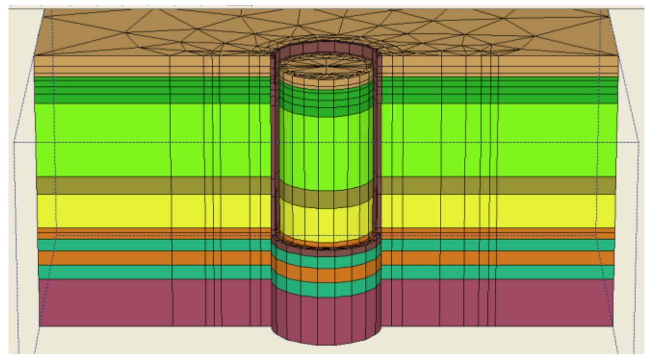


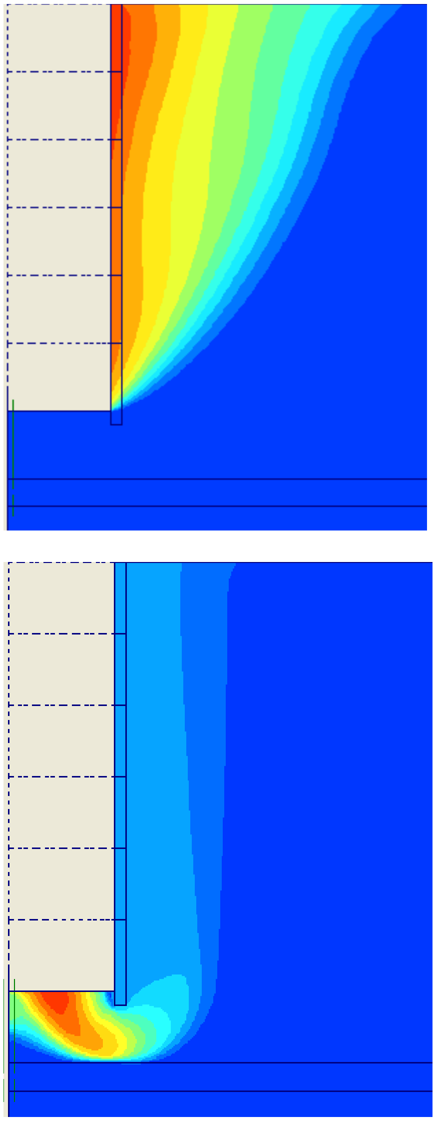
Figure 6. Central core 3D modeling

c) Soil cracking, slurry loss

During the excavation of the perimetral annular trench, some incidents have occurred, like slurry loss or soil cracking under the slurry pressure. In the Valley of Mexico clays, cracking can take place when the slurry level is only one meter above the piezometric level. As it was said, the construction of a perimetral screen wall with a self setting mixture of slurry and cement or mortar can be useful to prevent the occurrence of soil cracking.

d) Excavation stability

The excavation general stability after removing the soil core can be verified using limit analyses methods as those proposed by Nash & Jones (1963) and Alberro & Auvinet (1984). Nevertheless it is now more common to use numerical methods. Using, for instance, the Finite Element Method, it is possible to take into account geometrical, stratigraphical and mechanical details with good accuracy. The safety factor is determined by the mechanical parameters reduction method. The mechanism to consider usually is the wall general failure, even when it is known that the soft layers extrusion mechanisms can also be critical. When the excavation is surrounded by a mortar wall, the most critical mechanism is the bottom failure (Fig. 7).



a)

b)

Figure 7. Potential shear failure mechanisms a) without perimetral mortar wall b) with mortar wall

Influence of undrained shear strength of clay c_u, and of the slurry level on the safety factor for excavations with or without perimetral mortar is shown in Fig. 8. The slurry level is obviously a very critical parameter. The importance to carry on a strict control of the slurry level has been demonstrated through formal reliability analyses (Arias, 1997, Orduño, 2010). The safety factor diminishes with the shaft diameter, because the favorable 3D effects tend to disappear (Fig. 9).

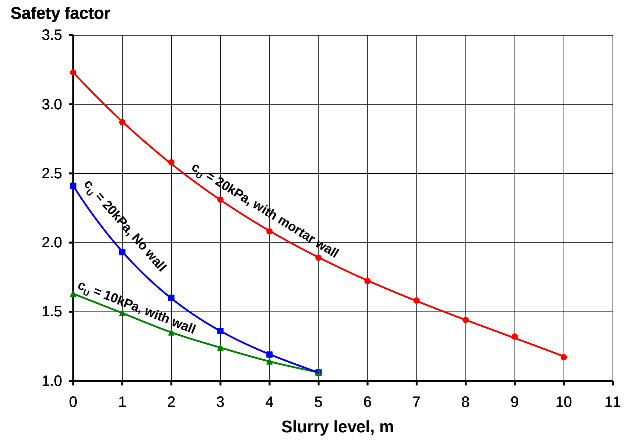


Figure 8. Influence of undrained clay shear strength and slurry level on the safety factor.

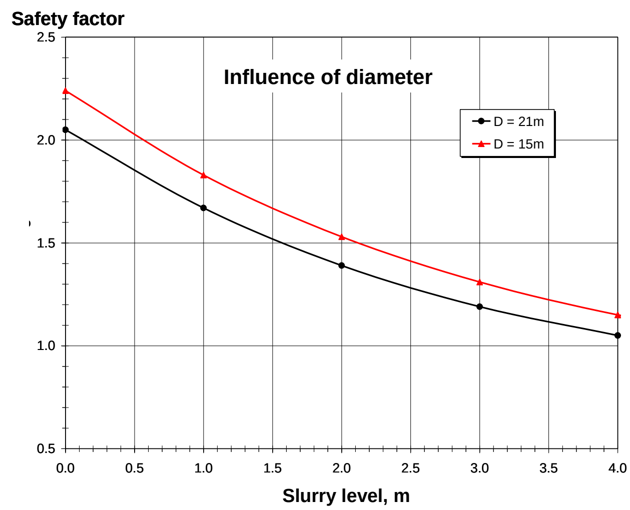


Figure 9. Influence of shaft diameter and slurry level on the safety factor.

e) Bottom failure by uplift pressures

When the bottom of the shaft excavation is located slightly above a relatively permeable layer, the piezometric conditions inside such a layer have great importance for stability. The pressures can be in hydrostatic conditions, presenting a slight drawdown due to pumping or, on the contrary, presenting artesianism. This last condition is obviously the most critical. The safety factor is obtained comparing the uplift pressure to the soil weight in the bottom of the excavation plus the slurry weight. The bottom failure by uplift pressure can also occur when the slurry level drops for any reason. A failure of this kind was registered in Mexico City when a contractor let the slurry level get down one meter before the flotation tank was installed, in order to avoid slurry overtopping. In this case, it is considered that the slurry weight may have induced an overpressure inside the pervious layer. The transitory flow condition that develops when the slurry level decreases leads to the formation of unfavorable seepage forces towards the excavation. This situation can be easily modeled numerically performing a transitory flow analysis using the Finite Element Method (Fig. 10).

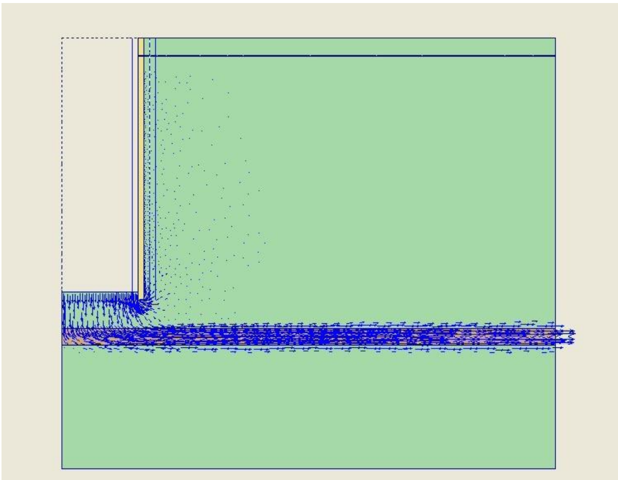


Figure 10. Transitory flow simulation using the Finite Element Method.

In order to avoid any bottom failure possibility due to uplift pressure, it may be advisable to install pumping wells. The efficiency of this technique can be evaluated also by numerical modeling (Fig. 11) and can be verified in the field with piezometers. To control the piezometric conditions, deep perimetral screens have also been built to seal the permeable layer.

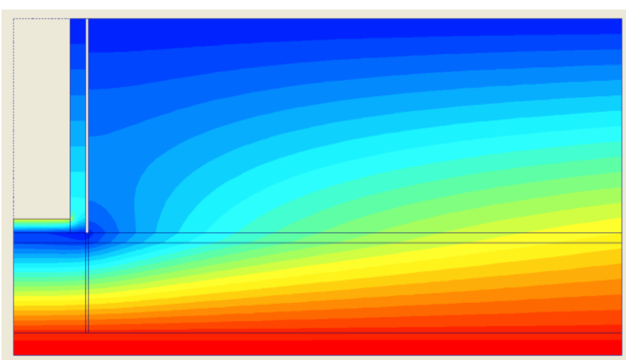


Figure 11. Modeling of pumping in the permeable layer to control uplift pressure.

f) Finished structure flotation

When the shaft construction is finished and the water ballast removed, the structure can float under certain conditions. This occurs when the Archimedes buoyancy force is bigger than the sum of the shaft weight plus the lateral friction between the soil and the shaft. This is more likely to occur for large diameter shafts because the lateral friction increases only with the diameter of the shaft while the Archimedes force grows with the square of the diameter. If necessary, the weight of the shaft must be increased or use of ballast considered to avoid buoyancy.

Service limit states also deserve great attention.

a) Induced movements on contiguous buildings during construction

In an urban environment, the neighbors are legitimely worried about possible damage to their property by shaft construction. The displacements around the shaft, calculated using for instance the Finite Element Method, are usually small. This condition has been verified by means of geotechnical instrumentation using inclinometers and survey points.

As a general advice, it is better to avoid the simultaneous construction of two adjacent shafts in order to avoid interferences and potential excavation instabilities when losing the problem symmetry.

b) Long term behavior

A very critical long term problem is apparent protruding of shafts built in soils subjected to regional consolidation process. In order to avoid damages to constructions in the neighborhood, it is convenient to locate the shafts as far as possible from these constructions.

4 CONCLUSIONS

The state of the art of underground works in soft soils presents a continual evolution. Sustained progresses are recorded in the constructive procedures, and simultaneously the available specialized software allows performing more and more sophisticated analyses. The yet cumbersome tridimensional analyses will soon evolve in a more friendly and accessible tool. At the same time it is necessary to improve knowledge of the physical phenomena involved in the construction of this kind of works. For this purpose it is necessary to invest in geotechnical instrumentation (Meza and Auvinet, 2010) considering every case as a unique opportunity to learn more about this type of problems. Also, in order to take into account the large remaining uncertainties in analyses and to introduce a higher degree of realism in design, basic analyses should be complemented with stochastic and reliability studies (Auvinet *et al*., 2001, 2001).

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