Design of urban tunnels in soft ground using TBM

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ABSTRACT
This paper presents a state of the art of urban tunnel design using a tunnel boring machine with earth pressure balance focusing on the major issues concerning this subject. Analytical and numerical methods used to compute surface settlements, face-support pressures and lining loads are summarized. Parametric studies were conducted to define the influence of soil properties on the lining loads design. Appropriate observations about numerical design were done.
Specific features regarding urban environment, ground conditions and TBM technology are summarized and appraised, such as overburden, behavior of structures due to tunnel induced subsidence, groundwater, soil properties and grouting stabilization.
Relevant conclusions regarding planning, design and construction of tunnelling are pointed out.

RESUMEN
Este artículo presenta un estado del arte del diseño de túneles en ambiente urbano, construidos con máquina tuneladora y sistema EPB, enfocándose en los problemas más importantes de este tema. Se resumen los métodos analíticos y numéricos usados para calcular los asentamientos superficiales, las presiones de frente y las cargas en el revestimiento. Se realizaron estudios paramétricos para definir la influencia de las propiedades del suelo en el diseño de las cargas sobre el revestimiento. Se realizan observaciones importantes sobre el diseño numérico.
Se resumen y analizan características específicas sobre el comportamiento urbano, las condiciones de suelo y la tecnología de la TBM, tales como sobrecarga, comportamiento de estructuras debido a los asentamientos provocados por el túnel, piezometrías, propiedades de los suelos y estabilización de la lecha.
Finalmente, se señalan conclusiones relevantes sobre la planeación, el diseño y la construcción de túneles.

INTRODUCTION

Ground behaviour is one of the most important features in urban tunnelling, since stability and deformations may cause severe issues. When selecting the tunnelling system for soft soil conditions, the use of tunnel boring machines (TBM) with earth pressure balance (EPB) system, becomes a major advantage to overcome the main concerns regarding ground behaviour.

There are special aspects to be considered when using a TBM system: a) shallow overburden due to cost and functionality, b) structures on ground surface, c) foreign objects in ground such as drainpipes, foundations, anchors, etc., d) alignment and constraints regarding material transport and TBM access. Appropriate planning and geotechnical exploration may help to prevail over these difficulties.

Design of TBM tunnels must take into account the previous aspects; the main problems regarding design are summed up in the following points:

1) Measures to control ground behavior
2) Loads on the liner and structural design
3) Prediction of ground displacements

The existing methods to assess these three focuses will be discussed in the next sections.

In addition, geology, changing conditions in ground due to lenses, boulders, etc., and presence of groundwater must be considered for tunnel design in order to select the best construction method and reduce risk scenarios.

Soft soils represent a particularly difficult condition for tunnel construction. In Mexico City, as in many other cities, soil is characterized by high water contents and high plasticity, since the city is located in the central part of the basin where a lacustrine environment prevails. Low shear strength, high compressibility, consolidation and creep, just to mention some issues, pose a major problem regarding this kind of soils and importance must be attached to them.

Although there is a lack of major regulations regarding soft soil tunneling, a large amount of research has been conducted and provides a better insight into this problem.

This paper presents a review of the main features of soft soil tunneling with TBM-EPB.

1 GROUND BEHAVIOUR

When using a TBM with EPB, measures to control ground behaviour refers to tunnel heading stability and evaluation of face pressures.

Equilibrium condition is reached when the ground in the working chamber of the EPBS, achieves the maximum density for the acting pressure and the volume of the extracted material from the screw conveyor equalizes the theoretical one removed by the cutter head (Maidl et al., 1996). It is believed that excavation stability is controlled if the face support pressure is between the active and the at-rest ground pressure.

There are several methods to compute the required face pressures, from basic analytical methods to complex 3D finite element models. All of them must take into account face static stability, excavation stability both around the shield and the lining and hydrological conditions preservation (Russo, 2003).

Analytical methods are divided into global limit equilibrium methods (LEM) and limit analyses stress methods (LASM). LEM consider soil as a rigid body and assume a stress distribution along the failure surface found through an iterative procedure, while LASM conduct stress analyses to provide upper and lower bound solutions. There exist several limit equilibrium methods, e.g. the method of Jancsecz and Steiner (1994) where the vertical pressure resulting from a soil silo and acting on soil wedge is computed according to Terzaghi's theory or the method of Anognostou and Kovari (1996) based on the sliding mechanism proposed by Horn, where the analysis is performed in drained conditions and a distinction between the stabilizing water pressure and effective pressure in chamber of EPB is presented.

A series of numerical model have been developed to evaluate face pressures using different failure surfaces such as linear, spiral logarithmic, circular and cylindrical.

2D models that may be conducted on transverse or longitudinal sections, allow for yield zone and strain development analyses, however, given the restriction of the model itself, it is not possible to evaluate the face stability.

3D numerical models are an integral tool to assess face-stability conditions. External forces such as traction or even forcing displacements, are used to model shield operation, while advancement of the shield machine is modelled by applying external forces such as jacking forces behind the shield machine and slurry pressures at the cutting face. Hence, the 3D model can be as detailed as desired, always considering important factors as the mesh size, boundary constraints, physical parameters and construction process.

Laboratory research on this topic has been carried out. Russo (2003) reported 1:10 scale tests and concluded that tunnel radio and pressures in the ground to be excavated define the driving process of the TBM, also that the passive state develops in the ground when the volume of the extracted material is less than the theoretical volume and the active state occurs when extracted material is more than the theoretical.

Consequently, only the control of face-pressures is not enough to guarantee excavation stability, but presence of pressure fluctuations and extraction regime must also be considered.

1.1 Pore pressures

Build-up of the excess pore pressure due to TBM advance is another effect that has to be taken into account when there are lenses of permeable materials embedded into soft ground. Injection of bentonite slurry or foam into the working chamber, cause infiltration and generation of excess pore pressure.

Since stability is affected by the reduction of the shear strength and a higher support pressure is required, it is necessary to ponder the effect of the generated excess pore pressure; to do so, a wedge stability analysis can be conducted using a stationary groundwater flow model.

The amount of water displaced depends on the slurry injection rate and instrumentation can be used to measure excess pore pressures at the time TBM stops to calculate the dissipation over time and so, the impact on the support pressures (Broere, 2003).

2 LOADS ON LINING

The use of segmental concrete lining has the following advantages: a) ensured longitudinal thrust resistance to the TBM during excavation, b) guaranteed support for the TBM back-up equipment, c) continuous support of the shield excavation in order to prevent surface settlements, d) prevention of water flow into the tunnel by installing a lining which is immediately impermeable, just to mention some examples.

The International Tunnelling Association establishes the next design loads to be evaluated when designing a shield tunnel lining (2000):

1) Geo-static loads, to evaluate load effects on lining segments and ground
2) Thrust jacking loads, to evaluate load effects distributed on segments by distribution-pads
3) Trailer and other service loads, including main bearing loads, divided by the number of wheels
4) Secondary grouting loads, extending regular grout pressure
5) Dead load, storage and assembly loads, bending moment influence

The loads resulting from vehicles moving in the tunnel or from the material passing on therein are not usually considered, unless the tunnel is close to the surface, or in loose soil of very low bearing capacity. Exceptions to this are water pressure tunnel.

It has been noticed that effective stresses on the lining are a function of the construction process; also it was found on instrumented sites (Craig and Muir Wood, 1978) that there is a 50-70% increment of the equivalent overburden stress, during the first few months.

Compression of the soil at the lateral walls will increase the stress and increase the ration of horizontal and vertical stresses, making compressive failure the most feasible mode of failure; on the other hand, large deformation effects will increase the magnitude of bending moments and although it might be reduced due to lining flexibility, it is important to consider the inertia of the lining ring for an appropriate design.

In the following sections, special characteristics to be considered for lining design are described.

2.1 Inertia of the lining ring

To identify the acting stresses and strains, especially those that involve the joints, flexural inertia is of main interest. Its magnitude depends on the number of segments in the ring and also on the type of subsoil that interacts where the tunnel is founded. Because the various rings are continuously rotated it is not practical to simulate the presence of the joints in determined positions, for example, with reduced thickness or special interfaces (Guglielmetti et al., 2007). The procedure suggested by the Japanese Tunnelling Association is described below: 1) the ring considered singularly is characterized by zones with both high and low flexural inertia, that is, the joints and the segments, respectively, 2) a sequence of rings is such that a joint in one ring corresponds to a segment in the previous and subsequent rings, 3) such a configuration allows the excess moment that cannot be sustained by the joints in the adjacent segments to be transferred to the previous and subsequent rings.

To correctly numerical modeling: 1) correction of the elastic modulus of the ring, according to a factor ξ, 2) calculation of the stress characteristics and 3) modification of the value of the bending moment, increasing and decreasing the value for the segment and joint, respectively, by the same ξ factor (the normal force remains the same).

2.2 Thrust jacking loads

Confirmation can be made for the action of the jacks, because a joint can be considered as an external surface of the concrete which is just bearing a load, acting on a determined area.

In the numerical analysis this phase is the one which allows mainly high normal forces with low bending moments, the only one compatible with the specific type of structures to be exerted on the segments.

2.3 Longitudinal behaviour

The lining is loaded in the longitudinal direction due to the stage construction and the jack forces acting on the tube, hence bending moments and shear forces develop.

In order to analyze the longitudinal behaviour of a tunnel, Bogaards and Bakker (1999) proposed a staged application of forces on the tunnel ring. External forces are applied on the new segmental concrete ring and removed from the previous ring and so on, until the tunnel is finished (Figure 1). Therefore, to derive the stress state at a certain stage of construction, the previous stages of construction have to be summed up.

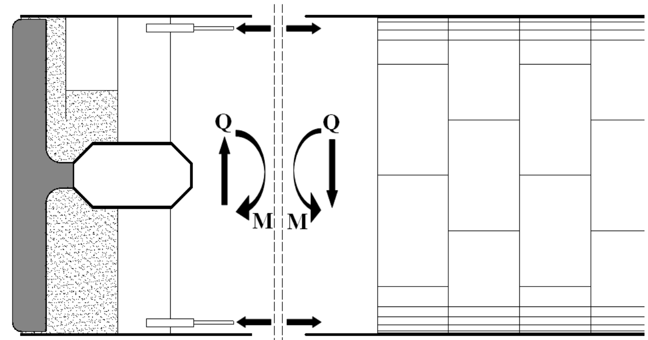

Figure 1. Staged loading to simulate longitudinal behavior

In order to obtain better results from this analysis, instrumentation can be used to monitor TBM forces at small intervals to gather a large amount of data. Besides, more attention should be paid to the subgrade reaction of the hardening grout material in the zone behind the TBM, since the application of this load in each step is a delicate matter. To account for the flexible behaviour of the tunnel rings due to joints between segments, it is necessary to use a reduction factor to the axial strain.

Solution of this model shows a constant bending moment when the TBM is far away from the measurement point and results from the model agreed with real measurements.

3 GROUND DISPLACEMENTS

During tunnel construction, unavoidable deformations will appear in the surrounding soil. Seeking to reduce the impact of such ground displacements, it becomes necessary to evaluate them before tunnel construction. There are three basic components of tunnel deformations (Guglielmetti et al., 2007): 1) immediate settlements which are presented right after the pass of TBM, and are a function of the tunnel face stability, the velocity of advance, the time necessary of installation and the time necessary to fill the tail-void. This settlement along the tunnel axis begins at a small distance ahead of the tunnel face and stops when the grout injection of the tail void has hardened enough to counteract any further radial displacement, 2) long-term settlements, due to a) the primary consolidation and b) secondary consolidation (a creep phenomenon which is mostly controlled by the rate at which the skeleton of compressible soils can yield and compress).

After several observations, Peck (1969) established that the transverse settlement trough caused by a tunnel can be described by a Gaussian error function (a bell shape). Since the behaviour of soft soils can be considered as undrained, the volume of surface settlement is equal to the excavated soil volume in excess to the theoretical volume of the tunnel, usually known as volume loss.

Longitudinal settlement can be considered as an s-like curve and according to Attewell and Woodman (1982) it can be obtained through transverse settlement. Assuming that the point of inflection, i_x which represents the standard deviation in the Gaussian equation, is the same for both, transverse and longitudinal curves (i.e. $i_x = i_y$), the longitudinal settlement results a function of this value and the tunnel diameter.

In order to compute surface settlement there are analytical and numerical methods. Analytical methods assume a homogeneous soil and study the phenomenon of new stress field caused by tunnel construction. Numerical methods, mostly based on finite elements, allow for a more complex modeling of soil, with several strata, advanced constitutive laws and realistic boundary conditions.

3.1 Behaviour of buildings due to tunnel induced subsidence

Influence of tunnel construction on surrounding buildings and civil works has to be evaluated. However, given the complex tunnel-soil-structure interaction process, it is difficult to accurately assess the tunnel induced deformations.

A common practice consists of evaluation of settlements in greenfield conditions, ignoring the interaction between soil and structures. Although this practice results very rough, it is the first approximation that should be made. Risk must be evaluated considering that the structure follows the greenfield settlement and a better approximation can be made through numerical models based on finite elements or finite differences.

In such methods the structural continuity, the foundation, the building orientation, previous movements and the soil-structure interaction must be considered to assess risk.

As studied by Franzius (2003), soil-structure interaction can be studied in numerical models by using interface elements with simple constitutive model properties, such as the linear elastic or the Mohr-Coulomb model. The tunnel modeling should be realized step by step considering the construction loading conditions like position of back-up train, backfill, jacking forces and grouting conditions.

4 CASE HISTORY

A 10 m diameter tunnel for a metro system was built on very soft lays in Mexico City. Monitoring of its behaviour was carried out before, during and after TBM pass. Design was carried out, using 2D finite element models where transverse sections were analyzed to define loads on lining due to short-term processes as excavation and construction of a 1.1 m concrete bench, and long-term loads as consolidation and ground-water extraction. Service loads were ignored.

The geotechnical profile used for modeling is shown on Table 1, where c: cohesion in kPa, ϕ: friction angle in º, and γ: unit weight in kN/m^3.

Table 1. Geotechnical profile

Layer	Depth (m)	c' (kPa)	ϕ (º)	γ (kN/m^3)
Fill	2.5	4.9	30	15.7
Crust	5.0	10.7	30	15.9
Soft clay	7.0	55.9	36	15.0
Sand	0.5	0.0	35	16.0
Soft clay	8.0	65.7	36	14.9
Sand	0.5	0.0	35	16.0
Soft clay	1.5	105.9	36	15.7
Stiff silty sand	15.0	147.2	32	16.2

Pore pressure distribution is shown on Figure 2 for short-term and long-term conditions.

The geotechnical model was simplified for modelling purposes; however, field and laboratory tests were taken into account to fully understand the shear strength and deformability parameters. It was found that in order to model short and long-term conditions it was preferably to use drained parameters.

A reduction on pore pressure and an increase on shear strength were used to simulate consolidation and groundwater extraction for long-term design.

A series of revisions were carried out to investigate the effect of pore pressures reduction and the increase of K_0.

In Figure 3 it can be seen how the ratio between the maximum bending moment and its corresponding axial force, reduces as the relation K_0 (σ'_h/σ'_v) is increasing. As it can be observed, the slope is greater for smaller values of K_0 and it becomes smaller for higher values.

Measurement of horizontal stresses and pore pressures at the site, reported that K_0 fell in a range from 0.45 to 0.62 where the tunnel laid on very soft soils with low pore pressures, where the crown was at 10 to 13 m depth.

The relation M/A significantly impacts the segmental concrete lining design and a small value leads to a less amount of reinforcement steel. As the K0 value is expected to change as the consolidation takes place, it is important to properly assess the real value in situ through instrumentation as pressure cells, piezometers and pressuremeters.

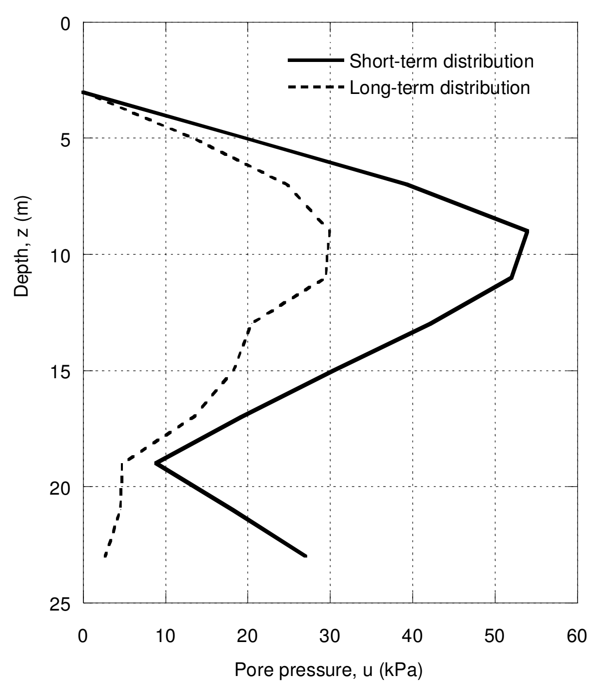

Figure 2. Pore pressure distribution

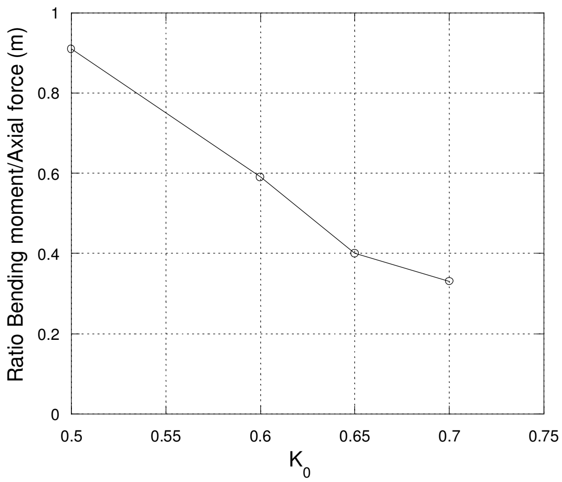

Figure 3. Relation between K_0 and ratio M/A

Figure 4 presents the deformed mesh of a transverse section of the modeled tunnel where long-term effects can be observed. The computer program Plaxis 2D based on finite element method was used to perform these analyses. The tunnel was modeled with a plate element and three different sets of material properties were defined to represent the TBM, the segmental concrete lining at short-term conditions and the segmental concrete lining at long-term conditions after the construction of the bench. Since this program works in two dimensions it is not possible to evaluate the face stability; however, considering the reduction on stiffness it is achievable to estimate the loads on the linings and with the transverse section of settlements, the total longitudinal settlement can be assessed.

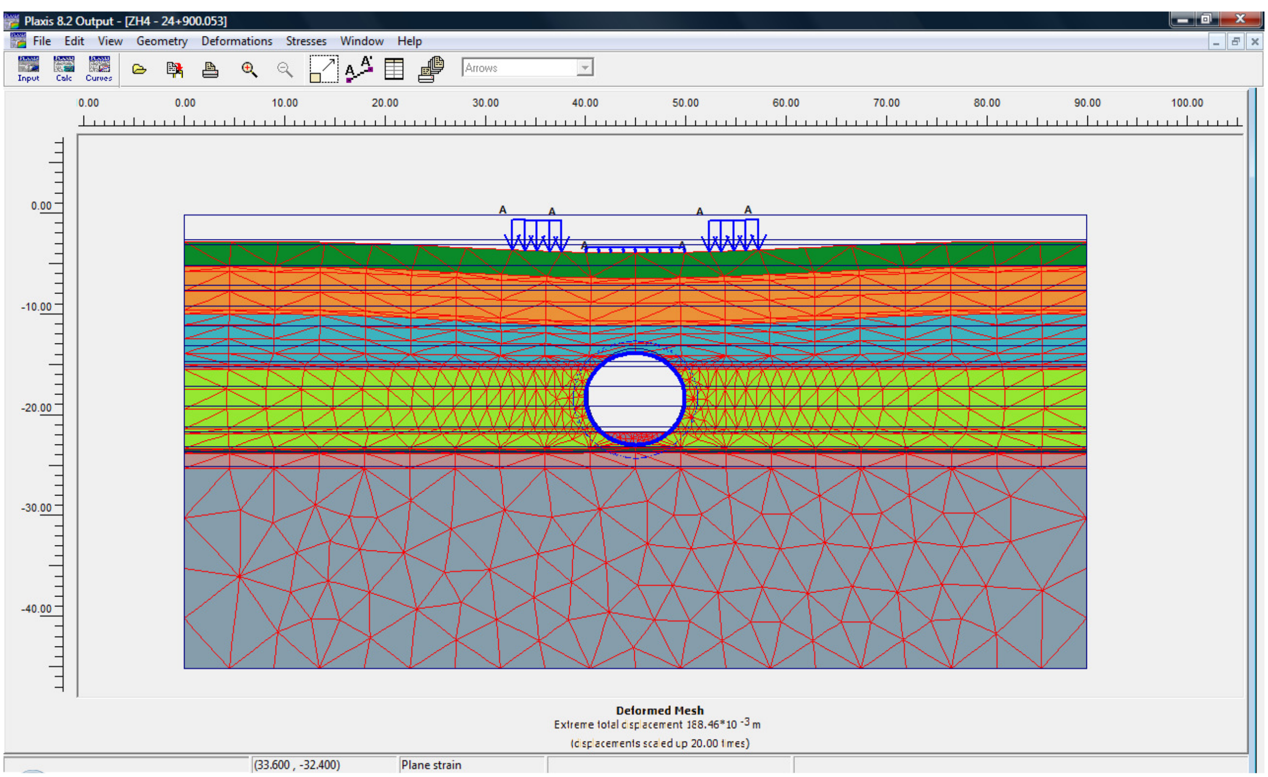

Figure 4. 2D finite element model of tunnel lining

The transverse model was calibrated using topographic measurements inside the tunnel and they compared fairly well.

Immediate settlements were also compared with those obtained from bench marks in the surface and results from the transverse model were slightly superior, which agrees with the assumptions made by Attewell and Woodman (1982) where the settlements obtained from transverse curves where bigger than the ones obtained at longitudinal profiles, however the difference is negligible.

Lining loads were also obtained for short-term and long-term conditions and structural software was used to predict the mechanical elements on the lining to do the design.

Consolidation loads were considered since this is the most relevant condition for long-term period; however, they represented a smaller percentage of the total load than the immediate loads applied during the first months.

Relevant conclusions regarding modeling are given in the following sections.

5 RELEVANT CONCLUSIONS REGARDING NUMERICAL ANALYSIS

Several researchers have conducted numerical analysis of soft soil tunnelling. Depending on the main focus (e.g. face stability, loads on linings or surface settlements), the models vary in input parameters, 2D or 3D configuration, mesh size, load applications and complexity. Some conclusions gathered from the consulted references show that:

1. Although, the tunnel excavation process is a 3D problem, 2D analysis provide a good insight into the issue and reduce computational needs and time. It has been noted that results from 2D analysis fairly compare with 3D results, as long as the stress redistribution is appropriately considered for each construction step.
2. It is important to consider the small strain behaviour of soil. Franzius (2003) found that It has been noted by many authors that when using small values of the K_0 (e.g. 0.5, either over the whole mesh or only as a local zone or reduced K_0) results obtained from the model show good agreement with field measurements in stiff clays. Nevertheless, analyses with higher values of K_0 (e.g. 1.5) settlement values are overpredicted when compared to field data. Although soil anisotropy can improve those settlements forecasting, the difference is slight.
3. In longitudinal direction, steady-state conditions are supposed to be achieved at a certain distance behind the tunnel face. This distance varies and it depends strongly on the K_0 value. As the K_0 value increases, the distance is reduced and as the K0 value diminishes, the distance raises.
4. For tunnel excavation, only 3D analyses are able to reliable provide an estimation of the excavation impacts and the needed pressure to stabilize the process.
5. 2D analyses might be used to obtain approximation of the strain behaviour at the face and the effect of the applied pressures, when an staged analysis is conducted on a longitudinal section.
6. As the complexity of 3D model increases, also the required time, machine cost and the uncertainty of input parameters. Hence, it is useful to conduct sensitivity analyses to evaluate the effect of changing input parameters.
7. Input parameters as the horizontal effective stress over the vertical effective stress (K_0) and pore pressure distribution are very significant when it comes to load lining computation, since a small change on those conditions can lead to very different amount of reinforcement steel on the lining. Hence, it is recommended to carry out Instrumentation campaigns in order to clearly define those parameters and conveniently identify any change during construction and operation.
8. The reduction factor that accounts for the influence of non-uniform strain distribution within the segments and takes into account flexibility due to the joints between segments must be calibrated by comparing the deformations obtained from the model to those measured on field. In the same way, deformations must be checked to properly simulate the external grout load used to compute face stability.

6 RELEVANT CONCLUSIONS REGARDING CONSTRUCTION

Construction process affects directly the loads induced on the tunnel and the soil. Any major modifications on the construction process can alter the deformations and cause severe consequences regarding the urban environment, hence it is important to take into account some aspects as:

1. Grout used for TBM-EPB tunnelling is often conditioned with additives for workability improvement, hence, its properties must be studied and considered when selecting those materials in order to avoid alterations on the surrounding soil.
2. Operation of the TBM is very important since the existence of pressure gradients in the working chamber can change the applied confinement at the face and the reported pressure on the bulkhead. Also, as it was mentioned before, appropriate control of the confinement pressure is not enough to guarantee face stability and external factors must be considered.
3. Estimations of the density of the excavated material might not be exact and thus, must be measured in situ to plan accordingly and avoid changes when computing face pressures.
4. Appropriate investigation of sites that can be affected by the tunnel construction, should be made. Special foundation cases, sewers, other tunnels, important building, etc., must be considered when defining the tunnel alignment to define solutions previous to the TBM pass. Instrumentation on such buildings and regular monitoring can help to prevent any damage.
5. Change in pore pressures can modify the tunnel design, hence it becomes important to avoid any process that might change the pore pressure distribution while constructing and during

operation to avoid structural damages or excessive deformations on the tunnel.

7 CONCLUSIONS AND FUTURE RESEARCH TOPICS

It has been seen that TBM-EPB tunnels have become very popular in urban areas due to its advantages during construction. As in any other process, it is necessary to take into account several factor to avoid damages to existing structures and allow for a proper operation of the new structure.

During the construction of a metro line in Mexico City face pressures, settlements and loads on linings were estimated and also measured for short-term conditions. The authors look forward to be able to use future data gathered from instrumentation to establish valid criteria for tunnel design in Mexico City, considering long-term effects, by calibrating the actual predictions and the proposed long-term parameters.

Also, it is of main interest to check the segmental concrete lining design to optimize future projects.

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