Design of tunnel lining in consolidating soft soils

Conception du revêtement des tunnels dans des sols mous en processus de consolidation

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ABSTRACT: In this paper, a detailed description of the methodology employed for analysis and design of the final lining of a tunnel that will be part of Mexico City drainage system is presented. The tunnel crosses soft clayey soils of the lake zone of Mexico Valley. These clays are submitted to an on-going subsidence process associated to intense pumping of water from the aquifer in the urban area. The obtained results allow a better understanding of the soil-tunnel interaction as the medium is submitted to the effects of a double process of consolidation: firstly due to changes in effective stresses generated by the tunnel excavation and, secondly, due to the long term piezometric drawdown in the soil. It is shown that Finite Element Method (MEF) is a powerful tool for the analysis and design of tunnels in these difficult conditions. FEM allows considering different constitutive models, and representing soil consolidation due to tunnel excavation, piezometric drawdown and interaction between the tunnel lining and the surrounding soil.

RÉSUMÉ : Cette communication décrit la méthode suivie pour la conception du revêtement définitif d'un tunnel qui fera partie du système d'assainissement de la ville de Mexico. Le tunnel traverse les argiles molles de la zone lacustre de la vallée de Mexico. Ces sols sont soumis à un processus de consolidation dû au pompage d'eau intensif de l'aquifère de la zone urbaine. Les résultats obtenus permettent de mieux comprendre l'interaction entre sol et tunnel sous l'effet d'un double processus de consolidation associé aux changements de contraintes effectives causés par le creusement du tunnel puis aux abattements piézométriques dus au pompage dans le sol à long terme. On montre que la méthode des éléments finis (MEF) est un outil puissant pour la conception des tunnels dans ces conditions difficiles. La MEF permet d'utiliser divers modèles de comportement du sol et de considérer la consolidation due au creusement, les abattements de pression ainsi que l'interaction entre le tunnel et le sol environnant.

KEYWORDS: tunnel lining, soft soils, piezometric drawdown, Finite Element Method.

1 INTRODUCTION

In the lake zone of Mexico City valley, tunnels liners are subjected to a double consolidation process. A first process is due to an effective stress change generated by the excavation itself and primary liner installation (Kirsch 1898, Morgan 1961, Wood 1975, Curtis 1976, Alberro 1983, Bobet 2001, Auvinet and Rodríguez-Rebolledo 2010, Zaldívar *et al*. 2012), whereas a second process is due to the piezometric drawdown originated by dewatering of the deep aquifer (Alberro and Hernández 1989, Farjeat and Delgado 1988, Equihua 2000, Flores 2010). It is known that the first process only affects the primary liner and that the excess of pore pressure dissipates sometime after tunnel excavation (Gutiérrez and Schmitter, 2010). The second process acts in a permanent fashion (long term) on both liners over the serviceability period of the tunnel.

Hence, in order to design the secondary liner, long term mechanical properties of the liners and of the soils have to be considered.

This paper presents a description of the methodology employed for geotechnical design of the permanent liner of a 62 km long tunnel that will be part of the drainage system of Mexico City ("Túnel Emisor Oriente" TEO), in a 2.8 km stretch that crosses clayey soils of the lake zone of Mexico City valley. The inner diameter of the tunnel is 7m. The primary liner is a 35cm thick ring of prefabricated segments and the secondary liner is a 35cm thick cast-in-place concrete continuous ring.

2 THE GEOTECHNICAL MODEL

2.1 *Soil profile*

The stratigraphic sequence in the area of interest mainly consists of the following layers:

Layer CS. Superficial crust, 0 to 3m thick, average specific weight of 14kN/m^3 and mean water content of 33%.

Layer B. Highly plastic soft clays and silts with microfossils, 22 to 24m thick, average specific weight of 11.3kN/m^3 and mean water content of 293% (e_0 =6.6)

Layer C. Silts interbedded with sandy silts, 1.5 to 2.5m thick, average specific weight of 15kN/m^3 and mean water content of 56%.

Layer D. Highly plastic soft clays and silts, 9.5 to 13.5m thick, average specific weight of 12kN/m^3 and mean water content of 165% (e_0 =3.9).

Layer E. Stiff silts interbedded with sandy silts (hard layer), 5 to 7.5m thick, average specific weight of 16kN/m^3 and mean water content of 40%.

Layer F. Highly plastic soft clays and silts interbedded with volcanic ashes, average specific weight of 13.2kN/m^3 and mean water content of 115% (e_0 =2.9).

2.2 *Piezometric conditions*

The initial pore pressure distribution was obtained from piezometric stations installed in the area. A typical pore pressure profile is presented on Figure 1. A significant drawdown with respect to the hydrostatic condition is observed from a depth of 7m down, reaching up to 400 kPa at a depth of 56m. The phreatic level depth (NAF) varies from 3 to 5m.

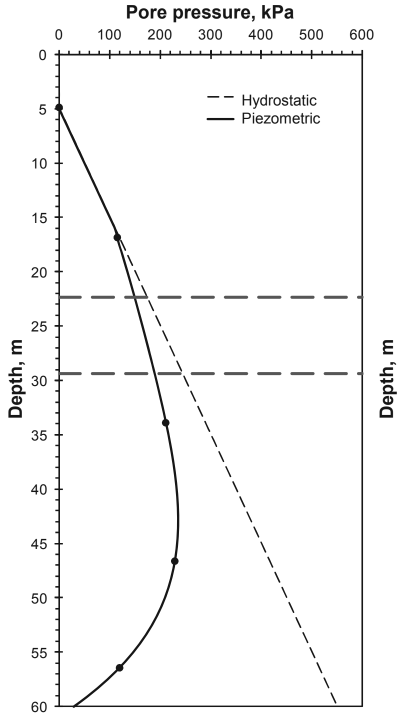

Figure 1. Pore pressure profile

An indirect and approximate way to forecast future pore pressure profile consists of employing a numerical model to evaluate the magnitude of the piezometric drawdown required to induce the regional subsidence that is expected to take place at the end of the design period (50 years) in the area. Such regional subsidence can be estimated from surveys performed on surface references and referred to a deep benchmark. The estimated regional subsidence after 50 years varies from 3.8 to 5.8 m.

3 NUMERICAL MODELLING

3.1 Software

A numerical model was developed using the Plaxis 2D software (www.plaxis.nl). Since a long term assessment is required, the analysis was conducted in terms of effective stresses, taking into account drained parameters and non-drained initial conditions (Plaxis bv 2008, Schweiger 2005, Rodríguez-Rebolledo 2011).

3.2 General characteristics of the model

Figure 2 shows the finite element mesh and Table 1 presents the soil properties of the different layers used in the numerical model. Mohr-Coulomb and Soft-Soil type models were employed for hard and soft soils, respectively.

The modelling is performed in several stages related to the constructive procedure of the tunnel:

Stage 1. Tunnel excavation and installation of primary liner. The construction condition right after installation of the primary liner is modelled. In order to take into account the effect of the joints between segments, a reduction factor of stiffness of the ring parameter (α), estimated by means of an interactive procedure, is introduced. The procedure consists of varying the magnitude of such parameter so that convergence is obtained between the geotechnical and the structural models. A value of $\alpha = 0.2$ was obtained.

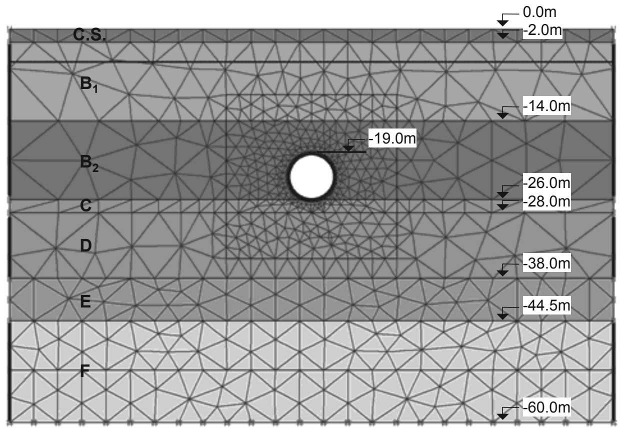

Figure 2. Finite element mesh

Table 1. Soil properties for the numerical modelling

Layer	Model	κ*	λ*	*OCR*	ν'	ν_{ur}	K_o=K_o^{nc}	k_x=k_y m/day
CS	MC	---	---	---	0.30	---	1.00	$1x10^{-2}$
B_1	SS	0.043	0.260	1.0	---	0.15	0.43	$1x10^{-5}$
B_2	SS	0.035	0.250	1.0	---	0.15	0.43	$1x10^{-5}$
C	MC	---	---	---	0.33	---	0.50	$1x10^{-1}$
D	SS	0.026	0.227	1.0	---	0.15	0.43	$1x10^{-5}$
E	MC	---	---	---	0.33	---	0.50	$1x10^{-1}$
F	SS	0.023	0.201	1.0	---	0.15	0.43	$1x10^{-4}$

κ* = slope of the swelling line
λ* = slope of the normal compression line
OCR = over consolidation ratio
ν' = Poisson's ratio
ν_{ur} = unload-reload Poisson's ratio
K_0 = coefficient of earth pressure at rest
K_0^{nc} = lateral earth pressure at rest for normally consolidated states
k_x = k_y = permeability coefficients for *x* and *y* directions
MC = Mohr-Coulomb model
SS = Soft-Soil model

Stage 2. Consolidation of the medium due to the excess of pore pressure generated by the excavation of the tunnel and installation of the primary liner. At this stage it is assumed that the excess of pore pressure generated by the excavation and installation of the primary liner is dissipated before the secondary liner is built (Gutiérrez and Schmitter, 2010) and therefore that it will only affect the primary liner.

Stage 3. Construction of the secondary liner. The construction of the definitive liner is modelled at this stage. Interface elements between primary and secondary liners are used to simulate the discontinuity between both liners.

Stage 4. Consolidation of the medium due to excess of pore pressure generated by construction of the secondary liner and piezometric drawdown. The behaviour of the tunnel is predicted for the next 50 years. In order to do so, a reduction of the elastic modulus of the concrete due to plastic flow has to be applied (F_R, Table 2). The regional subsidence obtained from the numerical model for a total drawdown of the current piezometric conditions was equal to 6.2m. This value is close to the maximum value estimated from field measurements for a period of time equal to 50 years (3.8-5.8m, section 2.2).

The primary liner was modelled using volume elements and the secondary liner using plate type elements. Two interfaces were also included: a primary liner-soil interface and, as mentioned, a secondary liner-primary liner interface.

3.3 Results and discussion

Figure 3 shows the excess pore pressure around the tunnel originated by construction itself (stage 1). Underneath the tunnel floor, it can be observed that an excess of positive pore pressure is generated due to unloading, whereas close to the lateral sides of the tunnel excess of negative pore pressure develops due to loading. This means that the unloading associated to the removal of the weight of the excavated soil produces an upward general movement of the tunnel, Figure 4 ("bubble" effect, Auvinet and Rodríguez-Rebolledo, 2010).

Table 2. Liners properties

Stage	Liner	Type of element	f'_c MPa	α	F_R	E MPa	e m	EA MN/m	EI MNm^2/m
1 y 2	Primary	Cluster	35	0.2	1.00	5,206	0.4	----	----
3 y 4	Primary	Cluster	35	0.2	0.57	2,968	0.4	----	----
	Secondary	Plate	50	1.0	0.57	17,152	0.4	6,003	61.3

α = liner reduction factor of stiffness
f'_c = concrete axial unconfined strength
F_R = reduction modulus due to plastic flow
E = modulus of elasticity
e = liner thickness
I = modulus of inertia

$$E = \alpha F_R \left(4{,}400\sqrt{f'_c}\right)\left[MPa\right]$$

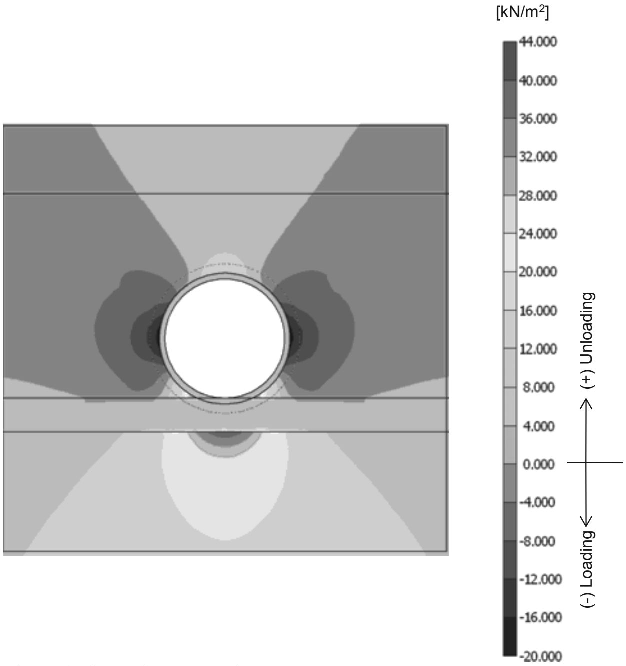

Figure 3. Stage 1, excess of pore pressure

Once the excess of pore pressure is dissipated (stage 2), effective stresses increase in the soil located below the tunnel. The clayey soil in this zone becomes a pre-consolidated material and therefore is less compressible than the soil around it. Because of that, once the definitive liner is installed (stage 3) and the excess of pore pressure generated by the water pressure drawdown is allowed to dissipate (stage 4), the rate of subsidence of the soil underneath the tunnel decreases (Figure 5). Therefore, the tunnel experiences an apparent emersion with respect to the surrounding soil.

Such emersion causes the soil around the tunnel to hang from the primary liner, generating negative skin friction over its upper part and inducing development of limit stress conditions in some areas (Figure 6). The forces that try to make the tunnel move downward induce, in turn, significant upward reaction forces and some plastification in the hard layer (support layer).

The analysis results show that the final liner is subjected to a very unfavourable loading condition from a structural point of view (Figure 7). While the upper part of the tunnel (point A) is loaded in vertical direction, the lateral sides (point B) experience confinement loss. This decrease of the horizontal stress can be estimated in a simple way by applying Terzaghi's effective stresses principle, that is:

$$\Delta\sigma_x = \Delta u\left(1 - K_0\right) \tag{1}$$

where: $\Delta\sigma_x$, is the total horizontal stress increment; Δu, is the pore pressure increment and K_0, is the coefficient of earth pressure at rest.

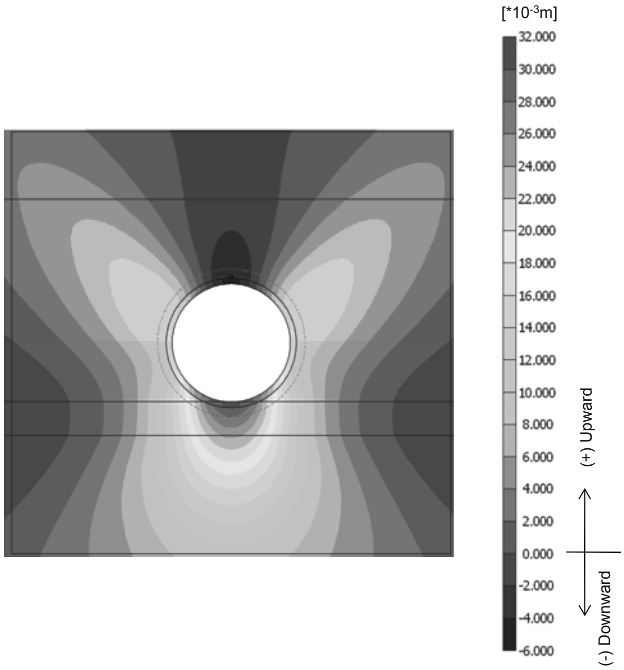

Figure 4. Stage 1, vertical displacements

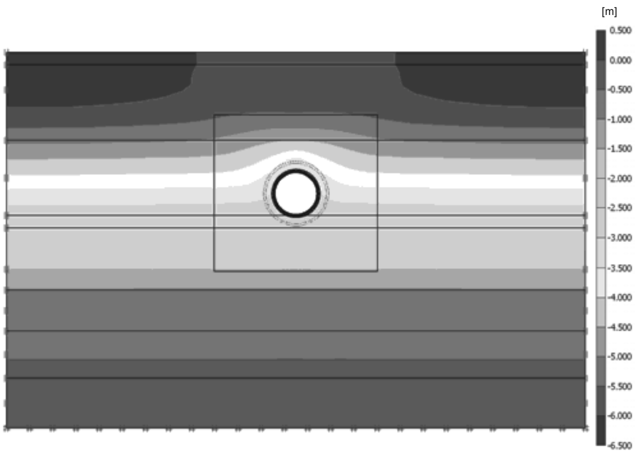

Figure 5. Stage 4, vertical displacements

The total stress increment at point A has to be zero as a result of the drawdown of pore pressure ($\Delta\sigma_y$ = 0). Variations of the total stresses with respect to the amount of piezometric drawdown at points A and B are displayed on Figure 7. This figure also presents, for comparative purposes, the results obtained from finite element modelling.

It can be observed that the total stress at point A estimated with FEM increases as piezometric drawdown develops. This can be explained by the fact that the tunnel settles at a lower rate than the surrounding soil (apparent emersion). Hence, the soil above the tunnel's upper part pushes the liner downward, increasing the vertical stress in this area.

Regarding the horizontal stress (Point B), a significant difference can be observed between theoretical and FEM solutions for a zero drawdown. This can be explained by the fact that during the tunnel excavation stage the primary liner tends to push laterally the soil located in the side zones, generating an increment of the horizontal stresses. As the piezometric drawdown takes place, the FEM solution gets closer to the theoretical one. It is possible to conclude that the theoretical solution can be used with confidence for determining the decrement of the horizontal stresses (confinement loss) on the sides of the tunnel as a result of pore pressure reduction. This is not the case for the stress increment that develops on the tunnel upper part. Differences between both solutions can be explained by the fact that the FEM takes into account the interaction liner-soil that develops both during the construction and serviceability stages.

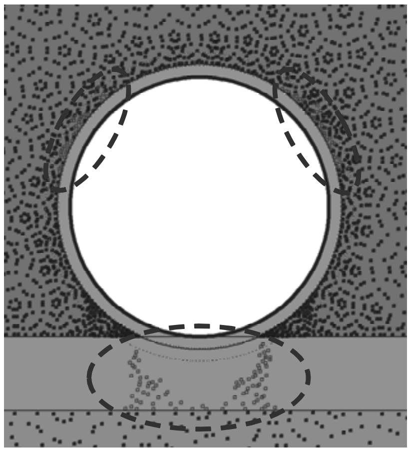

Figure 6. Stage 4, plastic stress point

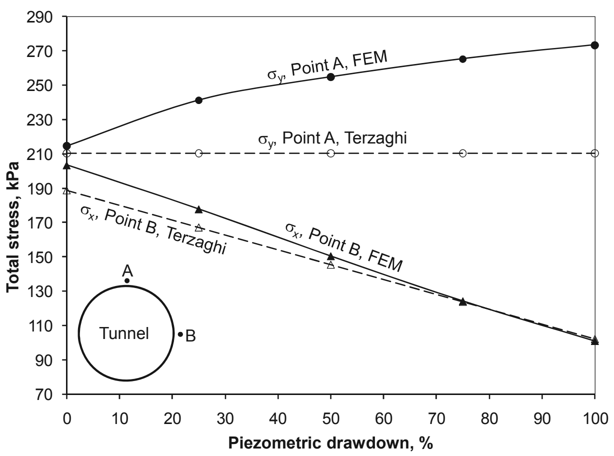

Figure 7. Variation of total stresses with piezometric drawdown

4 CONCLUSIONS

A detailed description of the methodology employed for the analysis and design of the definitive liner of a tunnel that will be part of the Mexico City drainage system has been presented. Part of this tunnel crosses through the clayey soils of the lake zone in Mexico City valley.

It has been shown that unloading associated to the removal of excavated soil produces a general upward movement of the tunnel (“bubble” effect). As the excess of pore pressure dissipates, a decrement of the effective stresses in the soil located underneath the tunnel is produced. The stress decrement originates that the clayey soil in such zone is transformed in a pre-consolidated material, which becomes less compressible that the surrounding zone. Because of this, as the definitive liner is placed and the excess of pore pressure generated by the piezometric drawdown dissipates, the soil below the tunnel settles down at a lower rate than the surrounding soil. Therefore, the tunnel experiences an apparent emersion with respect to the surrounding media.

It was observed as well that during the piezometric drawdown process, the secondary liner will be subjected to a very unfavourable loading condition from a structural point of view. Indeed, the upper part of the tunnel is submitted to a load increment generated by the apparent emersion of the tunnel, and at the same time, the sides of the tunnel experience a loss of confinement due to a reduction of water pressure. The unloading process developed on the sides of the tunnel can be estimated using Terzaghi’s effective stress principle.

It has been confirmed that the FEM is a powerful tool for analysing and designing tunnels in those difficult conditions, since it allows: 1) considering different constitutive laws for the materials involved; 2) simulating the phenomenon of consolidation of the medium, due to excavation and construction of the tunnel and piezometric drawdown and 3) considering the interaction between liners and soil.

5 ACKNOWLEDGEMENT

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