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Geotechnical Analysis and Design of Tunnel Shafts in Difficult Soft Soil Conditions at Mexico City

Carlos Omar VARGAS[a,1], Jorge Bricio GUILLÉN[a], Raúl BERNAL[a] and Oscar LUNA[a]
[a] *Comisión Federal de Electricidad, Gerencia de Estudios de Ingeniería Civil, Ciudad de México.*

Abstract. This article presents the stability assessment of a tunnel shaft designed with the flotation method during construction stage (short term behavior) by analytical and numerical methods, projected in the lacustrine soils of Mexico City. The ultimate and serviceability limit states were reviewed in accordance with the requirements included in the Manual of Civil Works published by the CFE. For short-term behavior, the following conditions were evaluated: panel stability, ring trench stability, core stability, general stability and stability against uplift forces by pore water pressure and buoyancy, in addition, the horizontal displacements in the slurry wall are presented. Likewise, the geotechnical characterization, definition of the geotechnical models and the general characteristics of the tunnel shaft are described.

Keywords. Tunnel shaft, difficult soft soil conditions, flotation method.

1. Introduction

The tunnel shafts are underground structures that are intended to serve as access for the construction of a tunnel, in addition to providing ventilation and maintenance in the operation stage. The methods of construction, for this type of structures, depend on the nature of the soil that hosts it, but also on the advances and technical resources that are available [1]. In the case of the construction of tunnel shafts in soft soils, the constructive method of major application is the “Flotation Method”, which was developed by the engineers Jorge Cravioto and Abel Villarreal in 1969, this procedure has had a great acceptance in general practice because it reduces the possibility of failure of the walls by extrusion and considerably reduces the probability of deep seated failure by shear and by uplift forces [2].

The geotechnical design of these structures requires the evaluation of the stability of the different stages of construction in order to guarantee the good behavior of the structure in short and long term. Stability analysis are associated with the construction stages that can generate any soil failure, these analysis are commonly known as: 1) stability of the slurry wall trenches, 2) excavation of the perimeter trench or precut core, 3) stability of the shaft core, 4) general stability or deep-seated failure stability against

[1] Corresponding Author, Carlos Omar Vargas; E-mail: comarvm19@gmail.com.

uplift forces caused by pore pressure and 5) flotations stability. Another key element for the short term evaluation is the estimation of horizontal displacements during the excavation of the slurry wall trenches, in order to have an annular space (between interior panels of the screen) enough for the immersion of the buoyancy thrust. Generally, these analyses are carried out by analytical methods, having satisfying results for their evaluation; however, the use of numerical modelling in the design of these structures allows evaluating conditions that can hardly be determined with analytical methods. However, the stability evaluation by both methods is still recommended.

The objective of this paper is to show the stability review, as well as the analysis and design criteria for a tunnel shaft designed with the flotation method in its construction stage by means of analytical and numerical methods, projected in difficult soft soil conditions at Mexico City.

2. Geotechnical characterization

The case under study is located in the lacustrine zone of Mexico City, particularly in the Texcoco's Ex-Lake; this geotechnical zone is made up by highly compressible soft clays, so it is necessary to carry out a rigorous campaign of geotechnical exploration in order to have the site geotechnical characterization.

2.1. Geotechnical exploration and laboratory tests

For geotechnical site investigation, the following types of tests were carried out: a standard penetration test borehole with continuous altered sampling reaching a depth of 90.4 m, a piezocone test (CPTu) deepening up to 75 m, a vane shear test borehole (VST) with 21 vane shear tests, up to a depth of 32.2 m.

The pore water pressure conditions were measured through piezocone tests. Figure 1a shows the dynamic and static pore water pressures at the study site.

The laboratory tests took in consideration the performance of triaxial UU and CU tests and one-dimensional consolidation tests. In addition, triaxial CU tests were carried out with a load-unload-reload cycle in order to obtain and calibrate the parameters for the constitutive model of hardening soil.

2.2. Stratigraphic interpretation

Based on the results of the field work and laboratory tests, the stratigraphic interpretation of the site define the next formations or geotechnical units conventionally known in the Valley of Mexico, having in general: Surface crust (*Costra superficial,* CS), superior clayey formation (*Formación arcillosa superior*, FAS) constituted by clays of high plasticity and very soft consistency, hard layer (*Capa dura, CD*), lower clayey formation (*Formación arcillosa inferior*, FAI), superior stratified series (*Serie estratificada superior*, SES) composed by intercalations of clays, silts and sands, deep clayey formation (*Formación arcillosa profunda*, FAP), lower stratified series (*Serie estratificada inferior*, SEI) composed by a sequence of silts and sands of dense to very dense density and clays of very firm to hard consistency.

It is worth mentioning that, for the evaluation of short-term stability, the numerical models only consider the FAI-SES contact.

3. Geotechnical models

Different geotechnical models were built assigning geo-mechanical properties to the different units, according to their mechanical behavior and type of analysis.

For stability analysis, the Mohr-Coulomb model was used for all soils, while for the short-term behavior analysis, the Hardening Soil model were used for cohesive soft soils. Table 1 and Table 2 show the parameters associated with these models.

The values of cohesion (c) and friction angle (ϕ) were obtained from the results of the triaxial compression tests UU and CU. Elastic modulus in undrained condition (Eu), were obtained as a function of the undrained shear strength (Su), which was obtained from the lower envelope that was determined with field vane (VST) (Figure 1b) and piezocone tests of the site and of the surrounding surveys, looking for the most unlikely condition. The compressibility properties of the soils were determined from one-dimensional consolidation tests (e_0, Cc, Cr), while the parameters of shear strength in terms of effective stresses were determined from triaxial CU tests.

The mechanical parameters for the hard layer (CD) were obtained from the results of the phicometer and pressuremeter tests in resistant strata of the lake zone; these parameters were also correlated with the piezocone data, in places where a hard layer was penetrated during the test.

Table 1. Geotechnical model for stability analysis and short-term displacements.

ID	Unit	Depth (m) From	Depth (m) To	γ (kN/m^3)	S_u (kPa)	E_u (MPa)	ν_u --
UG-1	CS	0	1.2	13	22	2.8616	0.4
UG-2	FAS1	1.2	4	11.6	8	1.0696	0.49
	FAS2	4	32	11.6	$f(z)^1=8+z$	$f(S_u)^2= 1.0696+0.128*z$	0.49
UG-3	CD	32	34	16	-	32	0.38
UG-4	FAI	34	49	11.9	$f(z)^1=-160+6z$	$f(S_u)^2= -20.4344+0.768*z$	0.49

γ, Saturated unit weight, S_u, Undrained shear resistance, E_u, Undrained elastic modulus y ν_u, Undrained Poisson ratio
ν_u , Undrained Poisson ratio, c´, Effective Cohesion, ϕ ´, Effective friction angle, E_{50}, Reference stiffness modulus, E_{oed}, Tangent oedometer modulus y E_{ur}, Reference Young's modulus for unloading and reloading.

Table 2. Geotechnical model for stability analysis and short-term displacements.

ID	Unit	Depth (m) From	Depth (m) To	ν'_u -	c´ (kPa)	ϕ´ (°)	E_{50} (kPa)	E_{oed} (kPa)	E_{ur} (kPa)	ν_{ur} -
UG-1	CS	0	1.2	-	5	28	-	-	-	-
UG-2	FAS1	1.2	4	-	2	35	930	400	6700	0.2
	FAS2	4	32	0.33	2	35				
UG-3	CD	32	34	-	10	38	20.4	-	-	-
UG-4	FAI	34	49	0.33	2	35	930	400	6700	0.2

γ, Saturated unit weight, S_u, Undrained shear resistance, E_u, Undrained elastic modulus y ν_u, Undrained Poisson ratio
ν_u , Undrained Poisson ratio, c´, Effective Cohesion, ϕ ´, Effective friction angle, E_{50}, Reference stiffness modulus, E_{oed}, Tangent oedometer modulus y E_{ur}, Reference Young's modulus for unloading and reloading.

4. Shaft tunnel proposal and constructive procedure

Figure 2 shows the geometry of the tunnel shaft together with its elements and the stratigraphic interpretation of the site. The general constructive procedure considered for the analysis was as next shown: a) Construction of the guide wall and exterior slurry wall, b) Excavation of an interior perimetral trench to isolate the shaft core of the excavation, c) Excavation of the shaft core while simultaneously replacing it with slurry, d) Floating tank placement and construction by stages of the final structure of the tunnel shaft, e) Filling of the annular space (between the structure and the slurry wall) with grout and f) Placement of interior structures (fill, half-round, towers) and construction of the cap slab. Figure 3 shows the construction stages of a tunnel shafts with "Flotation Method".

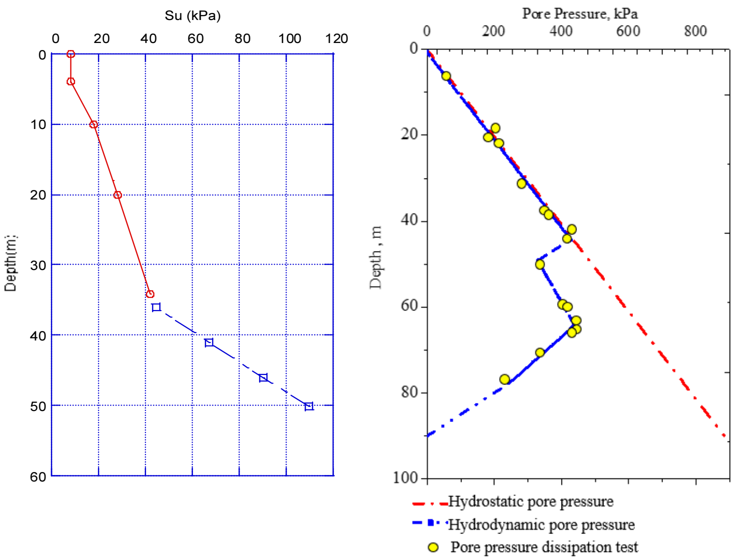

Figure 1. a) Pore water pressures at the study site and b) Undrained shear resistance envelopes (*Su*).

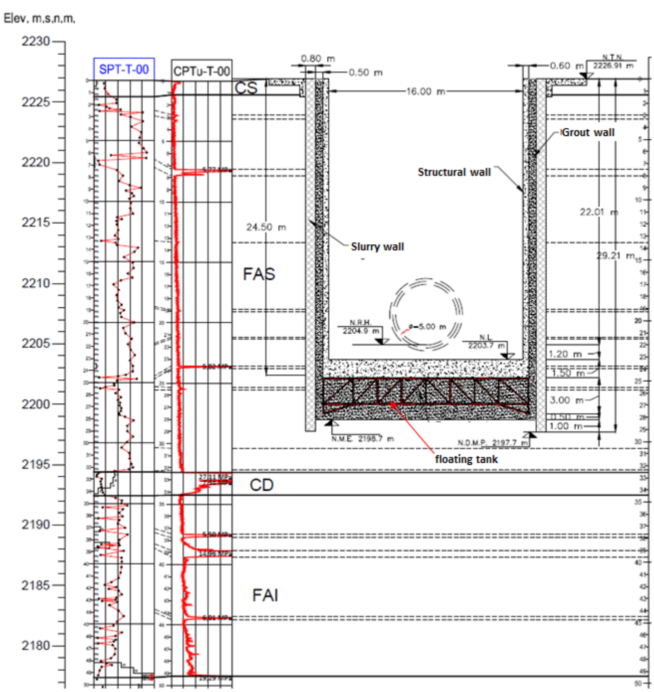

Figure 2. Stratigraphy of the site under study and geometry and tunnel shaft elements in study case.

5. Analytical methods for stability assessment

The stability analysis was carried out during the different stages of construction of the tunnel shaft with limit equilibrium methods and numerical modeling (FEM). Stability of the slurry wall trenches was evaluated by analytical and numerical methods (3D), annular trench, shaft core and stability against uplift forces were computed by analytical methods, while general stability and buoyancy were computed by analytical and numerical methods (2D).

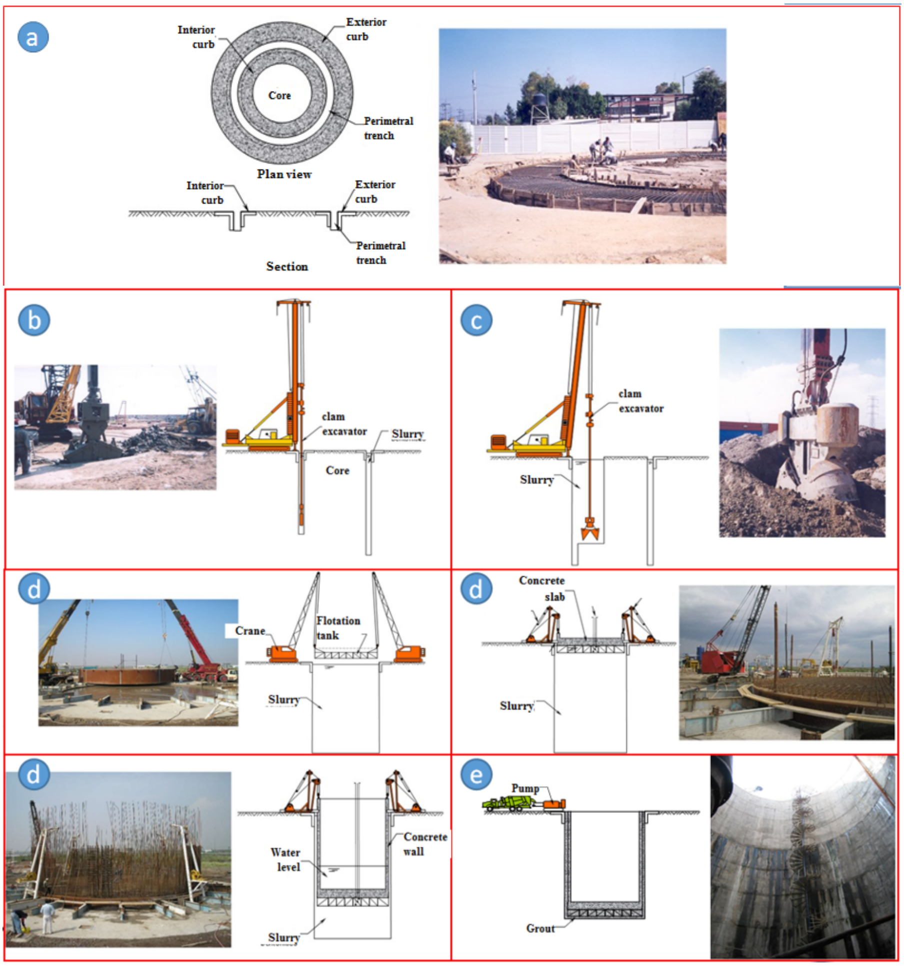

Figure 3. Construction stages of a tunnel shafts with "Flotation Method" (Modified from [1]).

To guarantee a safety condition of the tunnel shaft, it is considered acceptable to resort to the concept of safety factor (SF). It was revised that, for the different load combinations and possible failure mechanisms, the minimum values of the safety factors are satisfied. The minimum safety factors and conditions for tunnel shaft with the flotation method are: Trench stability for the slurry wall (SF $\geq$ 1.5), core stability (SF $\geq$ 1.1), stability against deep seated failure (SF $\geq$ 1.5), stability against uplift forces caused by pore water pressure (SF ≥ 1.5) and stability against buoyancy (SF ≥ 1.1). Figure 4 presents the different failure mechanisms and analysis schemes used in analytical methods.

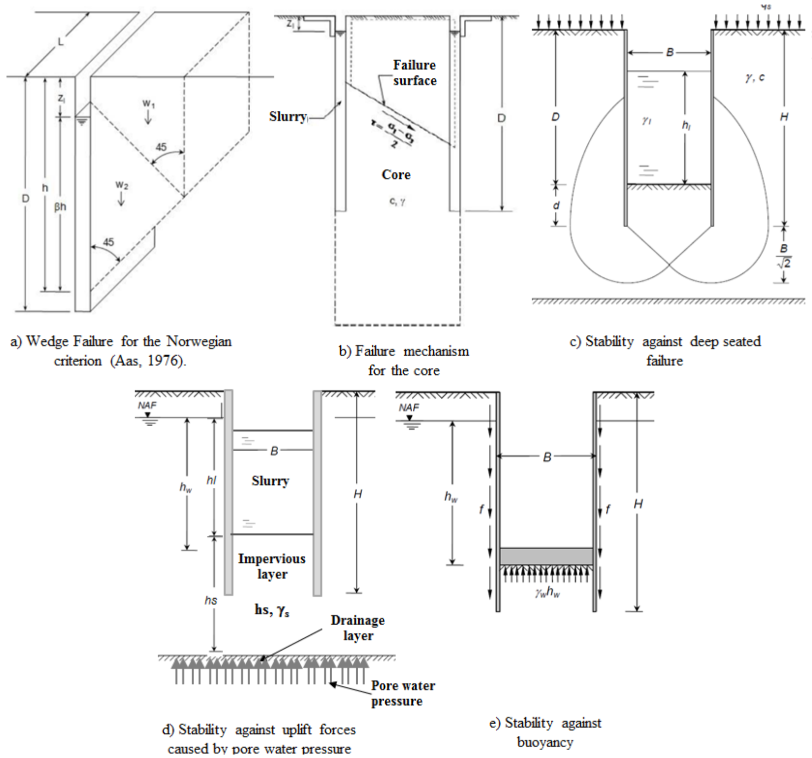

Figure 4. Failure mechanisms and analysis schemes (Modified from [1]).

5.1. Trench stability (slurry wall)

For the analysis of the trench stability for the slurry wall, it is possible to use the criteria published by the Norwegian Geotechnical Institute proposed by Aas [3]. This method is based on the assumption of a three-dimensional failure mechanism (Figure 4a). The sliding wedge is formed by two prisms of equal with to the excavated trench that can slide both vertically and horizontally [3].

$$FS(h) = \frac{c}{h\left(\gamma - \beta^2\gamma_l + \dfrac{q_s}{D}\right)}\left(2 + 0.94\frac{h}{L}\right) \tag{1}$$

Applying the criteria of Aas [3] and using the variation of undrained shear resistance as a function of depth, the calculation of the SF was carried out varying the depth of the wedge of analysis, in figure 5a the values of SF are presented for the case under study. In these it is appreciated that the analysis of the critical wedge is the one formed at 3.0 m depth associated with a SF = 2, this value is greater than the minimum required for this condition.

5.2. Core shaft stability

After the construction of the slurry wall, a cut of the excavation area is made, through a perimeter trench, which releases the soil core of the excavation, so it is necessary to verify the stability of the central shaft core.

The stability can be verified by assuming that the shaft core is a soil specimen that is going to be tested in a triaxial compression test, in which the slip failure will occur when the shear stress developed in the specimen, would be greater than his undrained shear resistance.

For this condition, the central core of the shaft is subjected to a state of horizontal stresses, which is less than the horizontal geostatic stress before the excavation (K_0 being the lateral coefficient of the soil at rest). Figure 5b shows the plot of SF obtained against the depth of excavation of the trench. According to these results, it is observed that the core is stable in this failure condition, presenting SF values greater than 1.1.

5.3. Stability against deep seated failure

When using a retention system that guarantees the stability of the walls of the excavation, it is necessary to verify the stability against deep seated failure.

For a shaft excavated in saturated fine soil, the safety factor is defined as:

$$FS = \frac{c_u\left(N_c i_c s_c + \alpha J\right)}{\gamma D + q_s - h_l \gamma_l} \tag{2}$$

An aspect of great importance in this mechanism, is the height of the stabilizing slurry, since the pressure of this fluid in the excavation will help to avoid the horizontal displacement of the walls (general failure) or the lifting of the bottom of the excavation (deep seated failure), that is why during the excavation and casting process, it must be guaranteed that the slurry level is located at any time at natural surface level or at the height that guarantees an FS greater than the minimum required. To expose the effect of the slurry in this failure mechanism, in Figure 5c the safety factors are presented varying the height of the slurry by means of equation 2. According to the results of this figure, it is observed that to guarantee an FS equal to or greater than the minimum required, it is necessary to maintain a slurry height greater than or equal to 15 m. However, it is recommended to maintain the level of slurry at natural surface level to guarantee a good behavior.

Additionally, this plot allows reviewing the security in case of having a drop of the slurry level during the construction procedure.

5.4. Stability against uplift forces

When the excavation is carried out in an impermeable layer, which in turn rests on a permeable stratum, it must be considered that the water pressure in this stratum can raise the bottom of the excavation (Figure 4d).

For this condition, the security factor is calculated with the following expression:

$$FS = \frac{\gamma h_s + \gamma_l h_l + \alpha c_u J}{u} \tag{3}$$

For the case under study, the pore pressure in the permeable layer was 312 kPa, with this value a safety factor of 1.0 was obtained, which is lower than the minimum required for shafts designed by the flotation method, so the analysis was made considering the relief of pore pressure (u) in that stratum. Figure 5d shows the FS against the values of relieved pressure (u-Δu) required to avoid the deep failure caused by the uplift forces.

The plot (figure 5d) shows that in order to reach a FS equal or greater than the minimum required (FS ≥1.5), the pore water pressure should be of 215 kPa approximately, meaning that there should be a pore water pressure relief (Δu) of 97 kPa in order to reach a stable condition against this kind of failure. In order to increase the safety factor against deep failure due to uplift forces caused by pore water pressure, it is necessary to decrease the piezometric pressure in the permeable strata or layers with a pumping system installed in that strata.

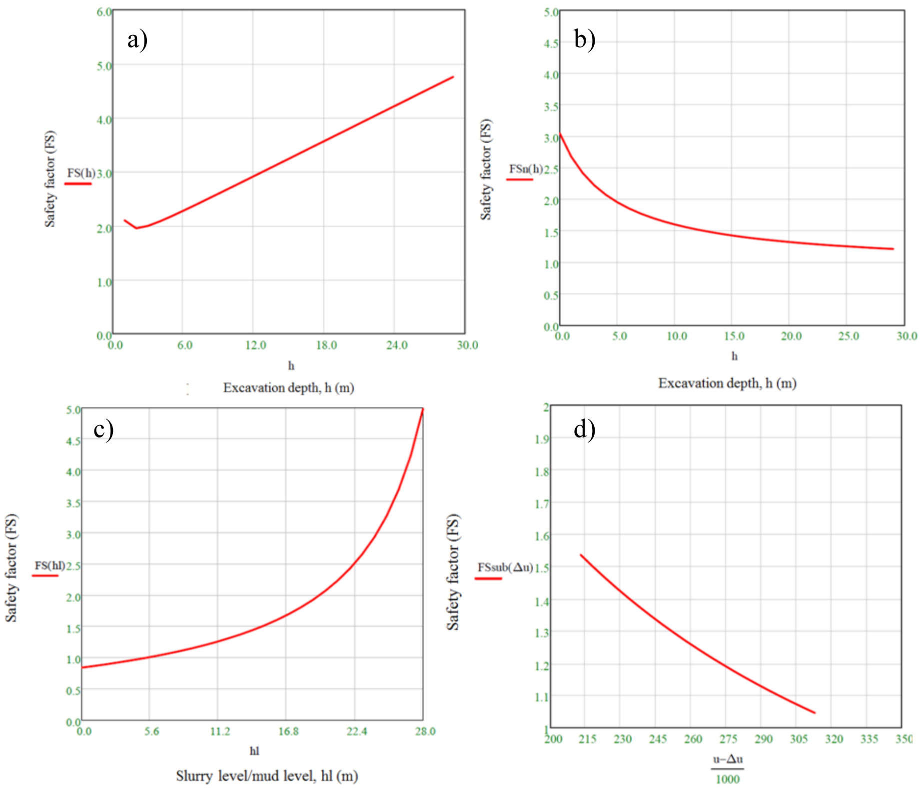

Figure 5. a) Plot of safety factor against the excavation depth of the slip wedge; b) safety factor of the core stability against the depth of the trench; c) Plot of safety factor against slurry height and d) Plot of safety factor against relieved pore water pressure.

5.5. Stability against buoyancy

The presence of a superficial groundwater level can generate the shaft buoyancy. This can occur after: building the bottom slab, removing the stabilizing fluid from the center of the shaft and/or interrupting the deep pumping used to avoid the failure caused by uplift pore water forces.

To verify the shaft stability against this type of failure, the safety factor is calculated as:

$$FS = \frac{\frac{W_e}{A_T} + fJ}{h_w \gamma_w} \quad (4)$$

For the study case, the safety factor is 1.43 and the compensation ratio was 1.21, meaning that the structure is lightly compensated. The bottom slab is usually used as an overweight to increase the safety factor against buoyancy.

6. Evaluation of stability and estimation of displacements with numerical methods

The geotechnical analyses with numerical methods were performed by the finite element method (FEM) with the 2D and 3D Plaxis software [4, 5]. The general stability or deep-seated failure was resolved by an axisymmetric model, while the stability of panels (slurry wall) was made by a three-dimensional analysis, this last since the geometric conditions of the problem are adequately represented with this type of analysis.

Likewise, it is worth mentioning that the thickness of the slurry wall in the analysis is 0.50 m since it corresponds to the effective thickness, discounting the possible verticality deviations between panels. The considerations of the model and the construction stages for the numerical simulation are presented below.

6.1. Model considerations for the stability analysis

The analysis was performed with an axisymmetric model with elements of 15 nodes and all the strata were considered until the contact of the lower stratified series (SEI).For the model boundary conditions, lateral boundaries were fixed for horizontal displacements, leaving them free for vertical displacements, while the bottom boundary was restrained in both ways; located at a distance that does not affect the final results. For the constitutive model, an elasto-plastic behavior was considered in accordance to the yielding criteria of Mohr-Coulomb.

For the analysis of stability and short-term behavior, the following stages were carried out, in which the state of stresses and deformations generated in the soil was calculated:

1) Definition of the initial state of stress considering the piezometric conditions of the subsoil before the construction of the shaft. 2) Excavation of panels stabilized with slurry with a unit weight γ = 14 kN/m^3, with an overload (q_s) of 15 kPa and construction guideline. 3) Excavation of panels with set slurry of unit weight γ = 14 kN/m^3, with overload (q_s) of 15 kPa and construction guideline. 4) Excavation of annular trench stabilized with bentonite slurry with a unit weight γ = 10.5 kN m^3. 5)

Complete excavation of stabilized core with bentonitic slurry of $\gamma = 10.5$ kN/m^3, with level of superficial slurry. 6) Construction of the tunnel shaft including bed, grout, bottom slab and walls. 7) Dissipation of excess pore pressure by constructive procedure. 8) Construction of the cap slab and half cane.

6.2. Failure mechanisms and safety factor with numerical simulation.

Figure 6a shows the contours of deformations with the phi-c strength reduction method; in this figure, it is observed that the failure mode is a general failure mechanism, which presents a safety factor of 2.0.

Figure 6b shows the failure mechanism caused by buoyancy, for this case, the safety factor value was greater than 3. Figure 7a show the 3D simulation of the panel construction, while in figure 7b its failure mechanism and safety factor associated whit this condition is presented. The safety factor in this figure is associated with the mechanism of failure of the overload, so that without overload the safety factor is greater than 3; it should be noted that for this simulation two panels were excavated, which represent the critical condition during construction process. Generally, this kind of analysis is usually carried out in an axisymmetric model, which is a very critical condition and does not represent the stage of the construction process, obtaining lower safety factors.

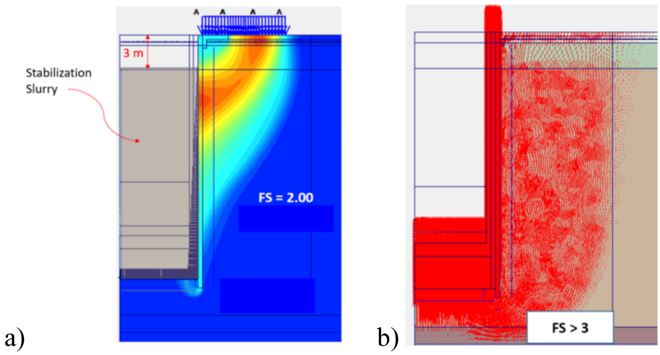

Figure 6. a) General failure mechanism and b) Failure mechanism with buoyancy condition.

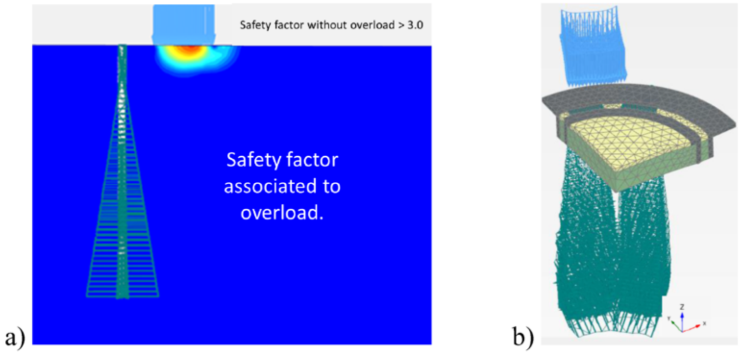

Figure 7. a) Panel analysis with 3D FEM and b) failure mechanism in the panel excavation.

6.3. Short-term displacement estimation

The horizontal displacements were estimated from the numerical simulation of the construction stages. These displacements were calculated by phase or stage and were added to determine the accumulated displacement on the wall. This was done in this way for practicality, since for the stage of wall excavation with slurry and overload, the best way to simulate the construction process is with a three-dimensional analysis, as in the construction procedure, this is carried out by panels (see figure 7a).

For the numerical simulation, the constitutive model Hardening-Soil (HS) was used in normally consolidated or slightly pre-consolidated clays (FAS and FAI), which allows a better estimation of the deformations since it considers the plastic deformations, beside the better simulation for the soil stiffness for loading, unloading and reloading. The parameters for this model were estimated from CU triaxial tests with loading and unloading, calibrating the results of these tests with the response of the model.

Figure 8a shows deformation curves with respect to depth for the different stages obtained with numerical modelling, while in figure 8b an example of the displacements obtained with the numerical model is shown (excavation trench). Figure 8a also shows the plot called "verticality", which includes displacements by deviation of the construction procedure of the wall, which was considered as a 0.5% of the depth of the wall. Figure 8 shows that the total cumulative horizontal displacements of the wall (screen) are in the order of 0.3 m. These displacements must be small or less than the clearance thickness or free space (50 cm), which indicates that there will not be any interference with the immersion of the shaft and the floating tank.

6.4. Analysis results.

The results of the geotechnical analysis show that the excavation of the tunnel shaft is stable for all the constructive stages that have been analyzed, having safety factors greater than those required. Table 3 shows the safety factors for each construction stage with the review of the stability by analytical and numerical methods with the finite element method. Regarding the short-term behavior, Figure 8 shows that the cumulative horizontal displacements of the wall (screen) adding the deviation by verticality are approximately 0.30 m, which are lower than the range of free space equal to 0.50 m.

Table 3. Safety factors with analytical and numerical methods.

Revision type	FS_{MA}	FS_{FEM}	$FS_{minimum}$
Trench stability	1.95	>3	1.5
Core stability	1.2	-	1.1
Stability against deep seated failure	2.07	2	1.5
Stability against uplift forces	1.05*->1.5	-	1.5
Stability against buoyancy	1.74	>3	1.1

7. Conclusions

For both analytical and numerical stability analysis, the determination of the variation of the undrained shear strength with depth allows to drop the tendencies of overestimating or underestimating the value of Su. In addition, this parameter is essential in order to evaluate the short-term behavior.

The geotechnical models were defined according to the type of analysis to be carried out. For the stability analysis, a Mohr Coulomb model was used, given that failures in this type of soil occur in undrained conditions. The Hardening Soil model was used to model de displacements, which allow representing the effects of loading, unloading and reloading behavior due to excavation process.

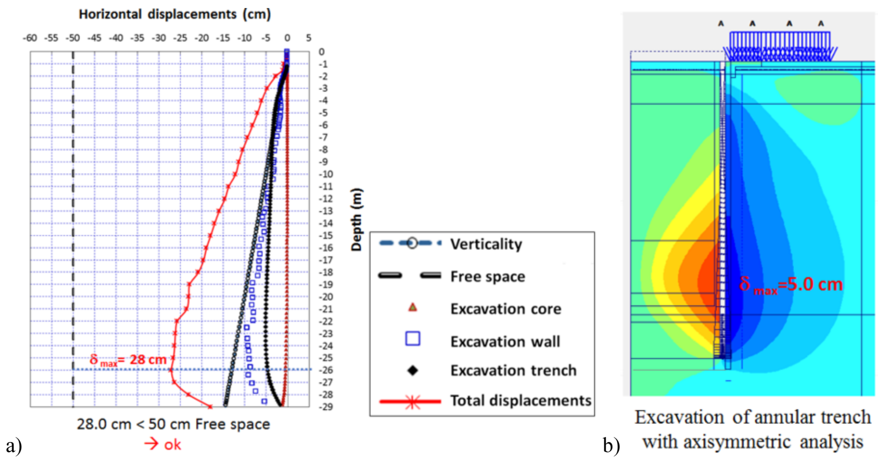

Figure 8. a) Cumulative displacements for the wall with the verticality deviation and b) displacements contour for excavation trench.

The displacement estimation was made by numerical modelling with axisymmetric and three-dimensional models, which allowed simulating the approximate conditions of the construction stages. The analytical methods continue being a great tool for the evaluation of the stability in this type of structures. In this paper it is observed that the failure mechanism and safety factors estimated with analytical methods provide adequate results for the design.

The use of numerical modeling allows determining relevant aspects such as plastified zones, identification of failure mechanisms such as the case of deep seated or general failure. In addition, they allow estimating the displacements according to the conditions of load in the ground, evaluating the stability of the constructive procedure in terms of displacements. This aspect is relevant for the design of the wall and feasibility of the construction procedure, since from these results it can be estimated that there will be no difficulties in the floating tank immersion due to these effects.

Likewise, an important aspect in this type of structures is the design of the geotechnical instrumentation, which will allow evaluating the real behavior of the structure during its construction, validating hypothesis and results of the analyses. Nevertheless, the analyses are the basis for defining the thresholds for the instruments within the auscultation plan.

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