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Centrifuge modelling of the behavior of a tunnel in soft soil subjected to pore water pressure drawdown

Modélisation par centrifugation du comportement d'un tunnel en sol tendre soumis à une réduction des pressions interstitielles

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ABSTRACT: Field measurements of earth pressure acting on tunnel lining built in saturated clays in London, Chicago and Detroit revealed that the vertical pressure increases with time. In large cities such as Mexico City, Bogotá and Bangkok, soft clays layers are subjected to a consolidation process induced by the intense pumping of water from deep layers where, as the pore water pressure drawdown, the value of the earth pressure acting around the tunnel lining can suffer important variations that have to be considered in its structural design. In this paper, the long-term behavior of a tunnel lining in soft clay subjected to pore pressure drawdown is evaluated, through physical modeling in a geotechnical centrifuge. The evolution of the settlements developed at the surface of the model (regional subsidence) and the earth pressure developed at the tunnel lining were measured during pore pressure drawdown. It was possible to satisfactorily model the effect of pore pressure drawdown, a regional subsidence of up to 70 cm in a time of 4 years (17.5 cm/year), was simulated. The results show that this drawdown generated a significant increase on the earth pressure acting around the tunnel, mainly in the upper part.

RÉSUMÉ : Les mesures sur le terrain de la poussée des terres agissant sur le revêtement du tunnel construit en argile saturée à Londres, Chicago et Detroit ont révélé que la pression verticale augmente avec le temps. Dans les grandes villes comme Mexico, Bogotá et Bangkok, les couches d'argile tendre sont soumises à un processus de consolidation induit par le pompage intense de l'eau des couches profondes où, comme le rabattement de la pression interstitielle, la valeur de la poussée des terres agissant autour du tunnel le revêtement peut subir des variations importantes qui doivent être prises en compte dans sa conception structurelle. Dans cet article, le comportement à long terme d'un revêtement de tunnel en argile tendre soumis à unrabattement de la pression interstitielle est évalué, par modélisation physique dans une centrifugeuse géotechnique. L'évolution des tassements développés à la surface du modèle (affaissement régional) et la poussée des terres développée au niveau du revêtement du tunnel ont été mesurées lors de l'abaissement de la pression interstitielle. Il a été possible de modéliser de manière satisfaisante l'effet du rabattement de la pression interstitielle, un affaissement régional allant jusqu'à 70 cm en 4 ans (17,5 cm / an), a été simulé. Les résultats montrent que ce rabattement a généré une augmentation significative de la poussée des terres agissant autour du tunnel, principalement dans la partie supérieure.

KEYWORDS: Centrifuge modeling, tunnel, lining load, long term analysis, consolidation.

1 INTRODUCTION

In recent years, with the rapid growth of various cities, there has been a notorious increase in the use of tunnels built on soft soils, mainly densely populated urban areas, to serve a variety of purposes, such as transportation (road, rail and subway), and as part of sewage collection systems.

Earth pressure measurements acting on lining of tunnels built in saturated clays in London, Chicago and Detroit show that the vertical total earth pressure increases with time, reaching, in some cases, higher values than those initially obtained at a depth corresponding to the axis of the tunnel (Peck, 1969; Tchebotarioff, 1979). Ward and Thomas (1965) reported the long-term structural behavior of a segmented tunnel lining in London Clay subjected to a local seepage and presents the data of the developed stresses acting on the tunnel lining against time, after construction, in a period of about six years. Széchy (1971) referred to long-term measurements of earth pressures acting on a tunnel shield in Chicago for 10 years, showing the increase in earth pressure with time. More recently, Barratt et al. (1994) reports results of the loads developed in an instrumented ring in the north tunnel of the London Underground Jubilee Line below Regents Park (20 m deep) and in the Oxford Trunk Outfall sewer tunnel (15.5 m deep). At Regent Park, the results indicate that the vertical load acting on the lining reached the equivalent of 60% of the initial pressure after nine and a half years, and the horizontal load reached about 40% of the initial pressure. At Oxford, the loads on the support, although somewhat irregular, reached between 35 and 50% of the initial pressure, after seven and a half years of measurements. Japan Tunnel Association (1998) reported that shield tunnels for subway constructed in soft clay are experiencing long-term deformations.

An extreme case occurs in soft clays subjected to the consolidation process induced by the pumping of water in the deep layers in the urbans area where, as the pore water pressure drawdown, the top of the tunnel lining is subjected to load increments, increasing the difference between the vertical and horizontal earth pressure and inducing additional bending moments and compression forces (Tamez et al. 1997; Rodríguez et al. 2013). This phenomenon is also known as regional subsidence and it is characteristic of cities such as Shanghai, Bangkok, Tokyo, Kuala Lumpur, Jakarta, Singapore, Bogota, and Mexico City (Rodríguez-Rebolledo et al. 2015; Rodríguez-Rincón, et al. 2020). Tamez et al. (1997) reports that, in the lacustrine deposits of Mexico City, this phenomenon was originated in the second half of the last century and continues to progress gradually, according to piezometric and superficial settlements measurements, recommending considering their effect on the design of tunnels built in the lacustrine deposits of Mexico City to provide adequate rigidity to the lining that to guarantee an appropriate long-term tunnel operation. Kusakabe and Ariizumi (2005) identified five tunnels in Tokyo metropolitan area who exhibit typical long-term deformation. These five tunnels were constructed in a layer of soft clay in the period of 1960s to early 70s, experienced lowering of the ground water level due to pumping, deformed in a form of elliptical shape, with the appearance of cracks, and deformed gradually with time for some 10 to 20 years.

Geotechnical centrifuge is an advanced physical modelling technique widely used by researchers worldwide as a reliable tool for simulating and studying geotechnical problems. This technique enables the stress conditions in the full-scale construction to be reconstructed using greatly reduced-scale construction models and provides an attractive modelling approach in geotechnical engineering (Huang et al. 2014). It also provides physical data for investigating mechanisms of deformation and failure and for validating analytical and numerical models. Due to its reliability, time and cost effectiveness, centrifuge modelling has often been the preferred experimental method for addressing complex geotechnical problems (Ng, 2014).

In this paper, the long-term behavior of a tunnel lining in soft clay subjected to pore pressure drawdown is evaluated, through physical modeling in a geotechnical centrifuge. The evolution of the settlements developed at the surface of the model (regional subsidence) and the earth pressure developed at the tunnel lining were measured during pore pressure drawdown.

2 CENTRIFUGE MODELING

2.1 *Equipment*

The test was undertaken in the 40 g-ton geotechnical centrifuge at Universidad de los Andes, Geotechnical Models Laboratory, Bogotá, Colombia. The equipment consists of a beam geotechnical centrifuge with an effective radius of 1.70 m, a maximum acceleration of 200 g and a maximum loading mass of 400 kg, as presented in Figure 1.

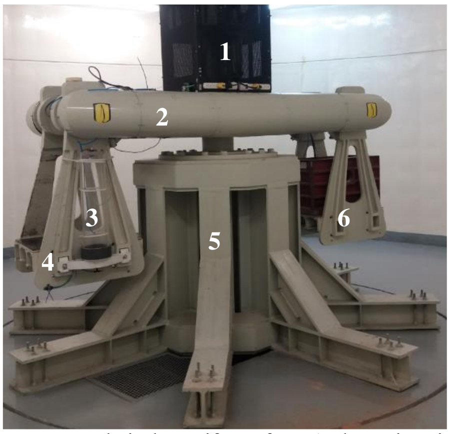

Figure 1. Beam geotechnical centrifuge of Los Andes University: 1- data-acquisition system; 2- beam; 3- tank for water control; 4- modelling boxes platform; 5- motor axis; 6- counterweight platform.

The internal dimensions of the model box developed for this experiment are: 15 cm deep, 55 cm wide and 58 cm high (Figure 2). The main parts of the box were made of aluminum with a front wall of resistant acrylic and a thickness of 64 mm. Small holes were drilled on the right side of the aluminum box to place the pore pressure transducers on the ground at different depths. At the left side bottom of the box, a hole was added to connect the model with the external water tank to maintain the water level, i.e., to maintain the initial piezometric conditions. On the right side of the box, another hole was added to enable the water to drain during the test, i.e., to induce in the model a pore pressure drawdown.

Figure 2. Model box.

2.2 *Model configuration and instrumentation*

Figure 3 shows the configuration of the centrifuge model. The stratigraphy was divided into two layers: the upper layer made up of sand and the lower layer made up of a reconstituted soft clay, in which the tunnel is situated. The tunnel is supported by a thin sand layer that acts as a drain at the bottom of the model. The water table was 2 cm deep.

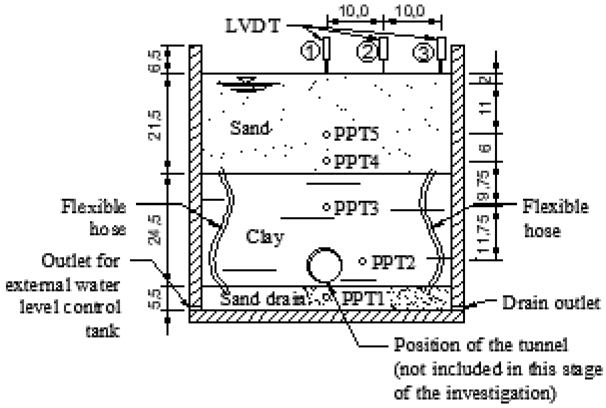

Figure 3. Configuration of centrifuge model.

The test was performed for a gravitational field of 80g. Consequently, the dimensions of the model must be scaled up by a factor of 80 to obtain the prototype dimensions, while the stress and strain are scaled by a factor of 1. Thus, the thickness of the sand layer and soft clay layer at the prototype was 17.2 m and 19.6 m respectively, while the depth of the water table was 2.0 m.

The instrumentation of the model box was selected according to the variables to be monitored during the tests. For the measurements of settlements on the soil surface, three LVDTs were used; to measure the pore pressure in the soil mass during the drawdown, five piezometers (PPT) were installed. Figure 3 presents the distribution of these instruments in the model.

A 12.0 cm long hollow aluminum cylinder, with an outer diameter of 7.5 cm and wall thickness of 5 mm was used to simulate a prototype tunnel with concrete linings of 6.0 m in outer diameter and wall thickness of 50 cm. The thickness of the hollow cylinder was selected to maintain the model-prototype similarity in terms of flexural rigidity.

The instrumentation of the tunnel lining is shown in Figure 4. Eight earth pressure transducers (EPTs) with an equal spacing of

45 degrees were installed in the aluminum lining to obtain the earth pressure distribution around the tunnel.

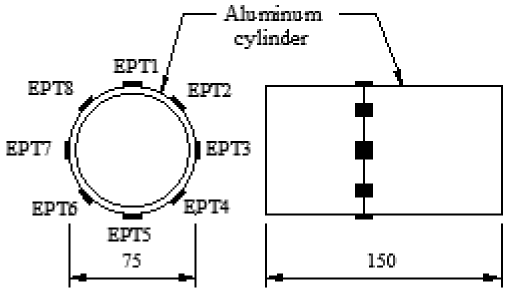

Figure 4. Arrangement of instrumentation in the tunnel lining.

2.3 *Model preparation and testing program*

The clay stratum used for the test consisted of a laboratory reconstituted soft soil prepared by mixing a powder conformed by 50% kaolin clay (Caumin P055) and 50% diatomaceous soil (50K-50DS) from Bogotá with a water content of approximately 1.5 times the liquid limit of the powder (102% moisture content). First, the slurry was manually homogenized; then, an industrial mixer stirred the sample for 30 min with special care to avoid the accumulation of lumps at the bottom of the container. The result was a slurry of liquid consistency with good uniformity and without bubbles. Table 1 summarizes the main properties obtained for the 50K-50DS mixture.

Table 1. Summary of properties for 50K-50DS (unit weight and compressibility parameters at consolidation pressure of 200 kPa).

Parameters	Value	Unit
γ	13.9	kN/m³
w_L	102	%
w_P	50	%
I_P	52	%
Gs	2.49	-
Cc	0.707	-
Cs	0.110	-
λ	0.307	-
κ	0.047	-
c_v	36.79	m²/year

For the model assembly, the effective stress profile with depth was generated using the one-dimensional compression technique, which was performed under normal gravity (1g). This procedure is equivalent to the standard consolidation test, where the load is applied in stages until the required compression stress is reached. According to Madabhushi (2014), it is much easier to consolidate the clay outside the centrifuge in a 1-D consolidometer than to directly consolidate the clay layer in the centrifuge. This procedure was used by Thaher and Jessberger (1991), Horikoshi and Randolph (1996), Leung et al. (2004), Tran et al. (2012), Lam et al. (2013) and Rodríguez-Rincón et al. (2020).

The 50K–50DS slurry was prepared as described but in larger quantities and using a larger industrial mixer. At the base of the model box, a 5 cm thick layer of coarse sand was placed to enable bottom drainage of water during the consolidation process. A geotextile was placed on top of the sand layer and a filter paper on top of the geotextile. Subsequently, the slurry was gently poured until it reached a thickness of 40 cm and, at the same time, the tunnel was accommodated at the bottom of the clay layer. The condition of free vertical movement of the tunnel relative to the side wall was achieve experimentally, although its vertical displacement is restricted by the drain sand layer.

The consolidation process was performed in two stages. For the first stage, the model was subjected to one-dimensional compression by manually and gradually (double load increased) applying vertical loads from 1.33 kPa to 6.25 kPa. For the second stage, the vertical stress was generated by a pneumatic consolidometer; the starting value was 12.5 kPa, and gradually doubled to 200 kPa. During the consolidation stages, the water level was maintained on soil surface using a feeding tank, that allow to keep the soil completely saturated.

When the soft soil layer was consolidated up to 200 kPa, the clay surface was smoothed to reach the required thickness of 24.5 cm (19.6 m in the prototype). A filter paper was placed on top of the clay and subsequently a geotextile. Finally, a 21.5 cm thick layer (17.2 m in the prototype) of coarse sand (D_{50} = 0.70 mm, Cu = 2.35, Cc = 0.92, $\gamma_{nat\ max}$ = 15.3 kN/m³, $\gamma_{nat\ min}$ = 14.8 kN/m³, $\gamma_{sat\ min}$ = 19.1 kN/m³, e_{max} = 0.94, emin = 0.56) was placed to enable top drainage of water during the pore pressure drawdown consolidation stage at centrifuge (Figure 5). To enable the upper sand layer to drain during the centrifuge tests, two flexible hoses were added to connect the top sand layer to the bottom drain (Figure 3). To minimize the adhesion between the internal walls of the box and the soft soil, the walls surfaces were lubricated and coated with two layers of plastic PVC films, and there was also lubrication between them.

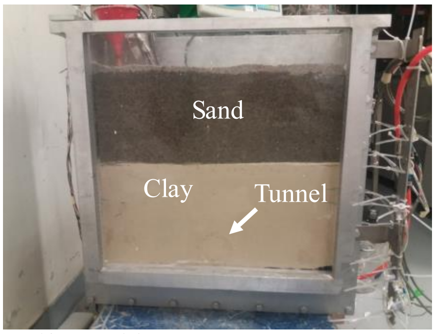

Figure 5. Centrifuge model.

After the model preparation, the box was weighed to determine the counterweight. The box was placed on the platform of the centrifuge arm and connected to the external water level control tank (Figure 6) through the hole at the bottom of left side of the box to guarantee the initial hydrostatic conditions at the first stage of the test. The hole at the bottom of the right side of the box was connected to an electromagnetic valve, which was automatically activated, and enabled water to drain during the second stage of the test. On the left side of the platform, a horizontal tank was installed to receive the water drained from the model. Finally, the LVDTs were installed at the top of the box and the PPTs were saturated.

The test was performed for a gravitational field of 80g, and consisted of two stages, defined as illustrated in Figure 7. In stage 1, the initial pore pressure and effective stresses conditions were developed and stabilized, i.e., hydrostatic and geostatic conditions. In stage 2, the pore water pressure drawdown was simulated.

For the design of the model, in addition to considering the capacities of the centrifuge, it is necessary to consider the permitted maximum daily continuous operating time. Due to cost and security reasons, according to the university regulations, the maximum continuous running time for the large centrifuge is defined in nine hours. Thus, the flight time intervals of the test for the stage 1 and 2 was approximately 40 min (0.67 h) and 500 min (8.33 h) respectively. The flight time for stage 2, where the acceleration was kept on 80 g, simulate a long-term behavior of about 6 years in the prototype.

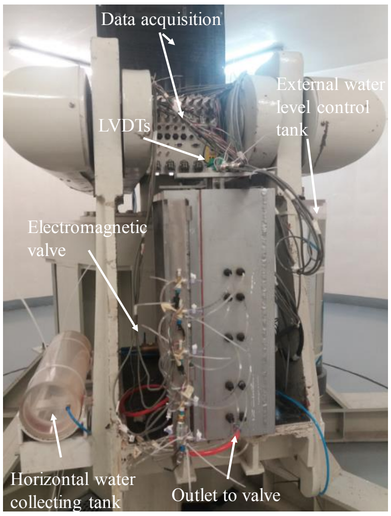

Figure 6. Model box placed on the platform of the centrifuge arm

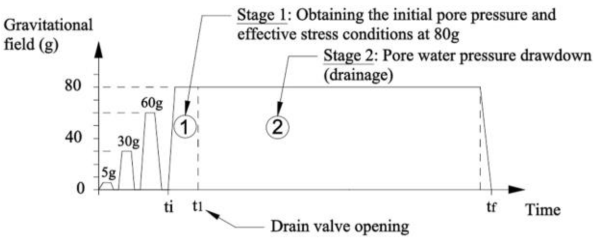

Figure 7. Testing stages

3 RESULTS AND DISCUSSIONS

For practical purposes and for a better understanding of the problem, all the values presented herein were transformed from the centrifuge model to the prototype scale.

3.1 *Pore pressure profile*

Figure 8 shows the pore pressure profile obtained during the test. The PPT 1 failed to work during the test and therefore, the result at this depth was not recorded successfully. However, the trend of piezometric drawdown is clear enough. For Stage 1 (0.00 year), the initial hydrostatic condition was satisfactorily reproduced. For Stage 2, pore pressure decreases with time immediately after the onset of the drainage and the 80% of a total pore pressure drawdown was reached in 5.84 years.

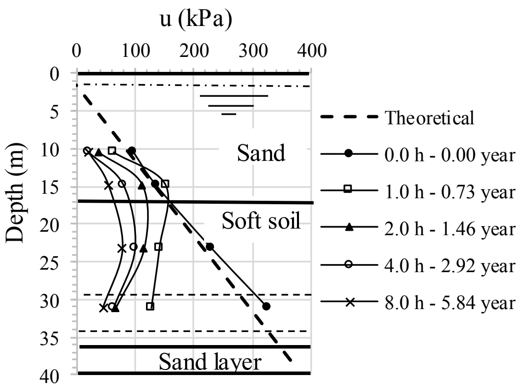

Figure 8. Pore pressure profile measure during Stage 2 of the centrifuge test

3.2 *Variation of lining earth pressure with time*

Figure 9 show the variation of earth pressure with time before the onset of the drainage process. The measured data clearly demonstrate that an increase in the earth pressure around the tunnel lining occur immediately after the drainage process. The main changes occurred within the first two hours (approximately 1.46 years on the prototype scale) after the start of the drainage process. The initial earth pressure measured by the cells placed in the upper and lower part of the tunnel lining (EP1 and EP5) presented the highest values, while the horizontal earth pressure (EP3) presented the lowest value. The greatest increase of earth pressure at the tunnel crown may be due to the restriction of the tunnel vertical displacement due to the presence of a hard sand layer above it. The measured vertical pressure at the tunnel crown (EP1) for stage 1 is close to the value of the vertical pressure theoretically predicted (measured vertical pressure equal to 475 kPa; theoretically value equal to 509 kPa) and the ratio between vertical (EP1) and horizontal (EP3) pressures was lower than expected (close to 0.3), probably due to the model preparation procedure.

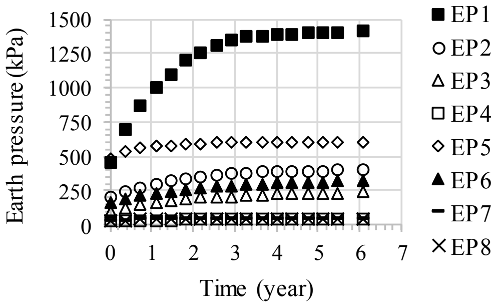

Figure 9. Development of normalized earth pressure with elapsed time

The earth pressure transducers EPT4, EPT7 and EPT8 showed extremely low values and therefore were not considered in the analyzes; however, taking advantage of the symmetry of the model, it was decided to replace these values with those obtained in EPT4, EPT3 and EPT2, respectively.

Figure 10 shows the variation of the normalized earth pressure (NEP) with time, calculated as the ratio between the total earth pressure measured on the lining tunnel (EPn) and the initial earth pressure (EP0) measured immediately before the onset of the drainage process. The measured data clearly shows that the earth pressure around the tunnel lining increase with time during the drainage process. On the tunnel crown (EP1), after 3 years of piezometric drawdown, the pressure increases almost 3 times, in comparison with the values before the onset. On the tunnel wall, for the same period, the horizontal earth pressure (EP3) increases almost 1.9 times, while on the bottom the pressure (EP5) increases only 1.2 times.

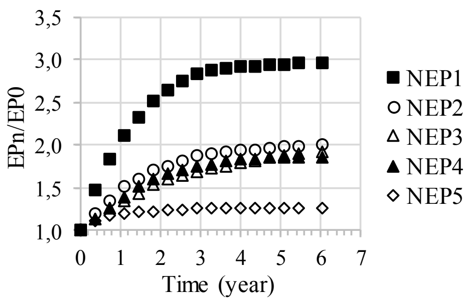

Figure 10. Development of normalized earth pressure with elapsed time

3.3 *Earth pressure distribution around the tunnel lining*

Figure 11 shows the circumferential distributions of the normal earth pressure (EP) acting on the tunnel lining based on the measurement at 1, 2 and 8 hours (approximately 0.73, 1.46 and 5.84 years in prototype scale) after the onset of the drainage process. It is confirmed that the variation of earth pressure around the tunnel is not uniform, presenting the highest value at the tunnel crown (EP1), following by the earth pressure on the tunnel bottom (EP5). The lowest values were observed on the horizontal earth pressure (EP3).

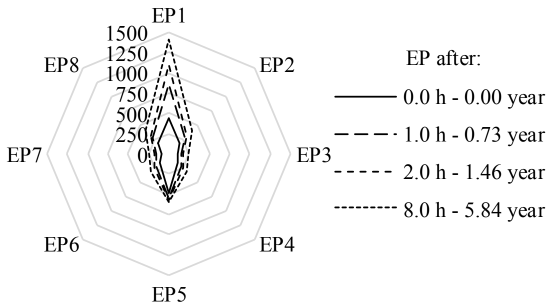

Figure 11. Distribution of normal earth pressure around the tunnel lining

Figure 12 shows the circumferential distributions of the normalized earth pressure (NEP) acting on the tunnel lining, based on the measurement at 1, 2 and 8 hours (approximately 0.73, 1.46 and 5.84 years in prototype scale) after the onset of the drainage process. It can be confirmed that the increase in the distribution of earth pressure around the tunnel lining due to piezometric drawdown is not uniform. A considerable increase in the normalized earth pressure at the tunnel crown (EP1) is observed, when compared to the obtained normalized earth pressure at other locations. The earth pressure at the tunnel bottom (EP5) presented the lowest increment.

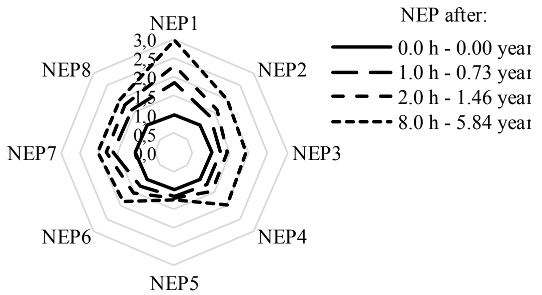

Figure 12. Distribution of normalized earth pressure around the tunnel lining

3.4 *Subsidence*

Figure 13 presents the progress of the surface settlement immediately after the onset of the drainage process. It can be observed that the soil surface showed a subsidence of approximately 73 cm after 6 years. The highest subsidence rate was obtained in the first 1.2 years of the drainage stage, after approximately 55% of the consolidation occurred (U = 55%).

In Figure 8 is observed that 80% of the total pore pressure drawdown was reached (U = 80%) in 5.84 years, but the results from the LVDT transducers (Figure 13) show that the settlement approximately stabilizes after 4 years, which indicate that the generated excess pore pressure has been almost completely dissipated (U ≈ 100%), showing that the pore pressure was not totally drawdown in the centrifuge model. The reason may be that the development of the water expelled to the horizontal tank generates a remaining hydraulic head, which prevents the development of the total pore pressure drawdown. Nonetheless, the magnitude of the achieved settlement was considered sufficient to simulate problems related to the regional subsidence in soft soils (≈17.5 cm/year). For the lacustrine zone of Mexico City, Auvinet et. al (2017) reported a mean subsidence value of 20 cm/year (from 0 to 40 cm/year).

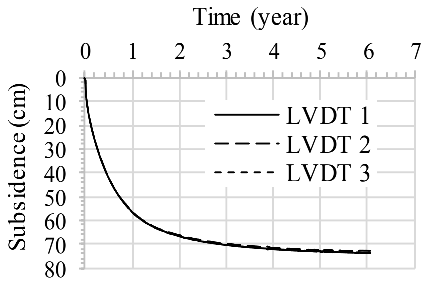

Figure 13. Surface settlement (subsidence) with elapsed time

4 CONCLUSIONS

A centrifuge model test was carried out to examine the long-term loads acting on a tunnel built in soft soil in consolidation process. The evolution of the settlements developed at the surface of the model (regional subsidence) and the earth pressure developed at the tunnel lining were measured during pore pressure drawdown. From the test results, the following conclusions can be obtained:

1- The pore pressure values obtained from the centrifuge piezometers show that the model satisfactorily reproduces the initial hydrostatic condition and the pore pressure drawdown profile for different time intervals. LVDT transducers confirm that settlements approximately stabilize after six hours, which indicate that the excess pore pressure generated has been almost completely dissipated. Thus, the methodology presented here enable to satisfactorily simulate the required piezometric conditions for this investigation.
2- It was possible to satisfactorily model the effect of pore pressure drawdown, a regional subsidence of up to 70 cm in a time of 4 years (17.5 cm/year), was simulated.
3- The results show that this drawdown generated a significant increase on the earth pressure acting around the tunnel, mainly in the upper part. This non-uniform distribution of vertical and horizontal earth pressures induces additional bending moments in the tunnel lining that must be considered in the long-term structural design.

5 ACKNOWLEDGEMENTS

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