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Numerical analysis of vacuum consolidation of virgin Lake Texcoco clays, Mexico City

Analyse numérique de la consolidation sous vide d'argiles vierges du lac Texcoco, à Mexico

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ABSTRACT: As a part of the development of the New International Airport of Mexico City (NAICM), vacuum consolidation has been proposed around Passenger Terminal Building (PTB). A compensated foundation system has been developed for PTB and Gate-house while ground improvement using vacuum consolidation has been considered for the rest of the surrounding areas. The vacuum consolidation is intended to limit long-term differential settlements between the PTB and Apron along with pre-stressing the ground to carry repeated heavy aircraft loading.

As Lake Texcoco clays have never been treated using vacuum preloading technique in past, trials were proposed to establish the effectiveness of this technique. Two separate trials were undertaken with and without membrane such that contractor could have the flexibility to choose either method for the main works. A series of 2D Finite Elements (FE) analyses were carried to back-analyse the monitored behaviour of both trials in term of vertical and lateral displacements, and pore pressure measurements. The paper provides detailed information on the two trials undertaken. The results from the FE back-analyses and their comparison with the measurements are presented. The performance of both trials has been discussed and conclusions have been made on the effectiveness of methods of vacuum consolidation for Lake Texcoco clays.

RÉSUMÉ: Dans le cadre du développement du nouvel aéroport international de Mexico (NAICM), une consolidation sous vide a été proposée autour du terminal passagers (PTB). Un système de fondation compensée a été mis au point pour le PTB et le Gate-House, tandis que l'amélioration de sol par consolidation sous vide a été envis-agée pour les zones environnantes. La consolidation sous vide a pour but de limiter le tassement différentiel à long terme entre la PTB et Apron, ainsi que de précontraindre le sol pour supporter le chargement répété d'avions lourds.

Les argiles du lac Texcoco n'ayant auparavant jamais été traitées de cette manière, des essais ont été proposés pour en établir l'efficacité. Deux essais ont été menés respectivement avec et sans membrane, de manière à of-fir un choix à l'entrepreneur. Une série d'analyses éléments finis 2D (FE) a été réalisée afin de prendre du recul sur les mesures extraites des deux essais, en termes de pression interstitielle et de déplacements verticaux et la-téraux. Le document fournit des informations détaillées sur les essais réalisés, ainsi que sur les résultats des analyses rétrospectives FE et leur comparaison avec les mesures. La performance des deux pistes a été dis-cutée et des conclusions ont été tirées sur l'efficacité des méthodes de consolidation sous vide pour les argiles du lac Texcoco.

Keywords: Vacuum preloading, soft soil, Mexico clays, consolidation, PVDs.

1 INTRODUCTION

1.1 The NAICM project site

The Mexico City Basin was open until about 700,000 years ago when volcanic activity caused the creation of the Chichinutzin Range, which acted to close off the Basin. Several large lakes were formed subsequently by periods of glaciation and persistent rains within the last 100,000 years. Lake Texcoco is one of the largest lakes that formed during this period.

Although the soil of the downtown area of the Lake Texcoco has been studied extensively in the past (Santoyo et al., 2007, Diaz-Rodriguez, 2003, Auvinet, 2002), the region on the east of Nezahualcoyotl dam (Figure 1) built by Aztec remained untouched until 1960.

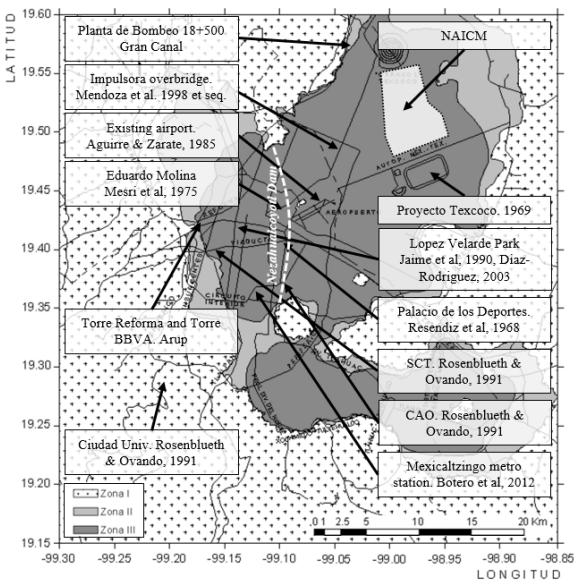

Figure 1. Lake Texcoco Region of Mexico City Basin (O'Riordan et al., 2018, 2017).

The area came under intense geotechnical scrutiny in the 1960s culminating in the Proyecto Texcoco volume that was produced as a memorial to Nabor Carillo, an important Mexican geotechnical engineer at UNAM.

The same area has now been used for the construction of one of the largest airports in the world: the Nuevo Aeropuerto Internacional de la Ciudad de México (NAICM).

At the time of writing this paper, as a result of political intervention, the project has been suspended. The foundation for all major structure including passenger terminal building (PTB) was complete but vacuum preloading in the Apron area could not be implemented.

1.2 Site Geology

Similar to the downtown area, the eastern zone of Lake Texcoco bed is characterized by deep Lacustrine deposits of compressible clays, with interbedded sands and volcanic glass to a depth of about 200m. The softest clays from the Formación Arcillosa Superior (FAS) unit in the upper 25 to 30m of the sequence are characterized by very high plasticity and very high void ratios due to the ubiquitous presence of diatoms: the skeletal remains of organisms that thrived in the nutrient-rich lakebed.

FAS is followed by 2m to 5m thick dense sands/volcanic glass knows as Capa Dura (CD), which overlays lower clay units known as Formación Arcillosa Inferior (FAI), followed by alternate layers of sand and lower clay deposits. The groundwater is highly saline and the site was formerly a brine extraction works, comprising over 400 wells between 30m and 60m deep between 1945 and 1995. The deep abstraction still continues for drinking water purposes causing ongoing regional subsidence in the range of 15cm to 20cm/year.

As a part of the NAICM development, a comprehensive ground investigation was undertaken to obtain detailed information on site history, ground conditions, physical and mechanical properties of different geological units and groundwater conditions (O'Riordan et al., 2018, 2017). The FAS unit has been the focus of investigation being the founding stratum for all structures proposed for NAICM.

2 PROBLEM DESCRIPTION

2.1 PTB – Apron Interface

The ongoing subsidence in the Mexico City area makes foundation design a challenging task as it requires a balance of settlement and emergence control (O'Riordan et al., 2018). The PTB and the Gate-house foundations comprising of a shallow pile-enhanced raft was designed on the principles of a compensated foundation. The basic idea behind a compensated foundation was to avoid any significant changes in the ground stresses (keeping OCR intact), to accommodate a wide range of ground conditions anticipated across the site, and more importantly to follow the subsiding ground.

The commercial Apron, on the other hand, is expected to support the repeated heavy concentrated loading from aircrafts and therefore could not be designed using compensated foundation principles. As a result, ground improvement using surcharge in combination with vacuum consolidation was proposed. In order to limit the impacts of expected large settlement on the PTB and Gate-house foundation, in particular, the ground improvement zone was restricted to about 20m away from PTB in between Gate-houses and 16m away from the Gate-house foundation as shown in Figure 2.

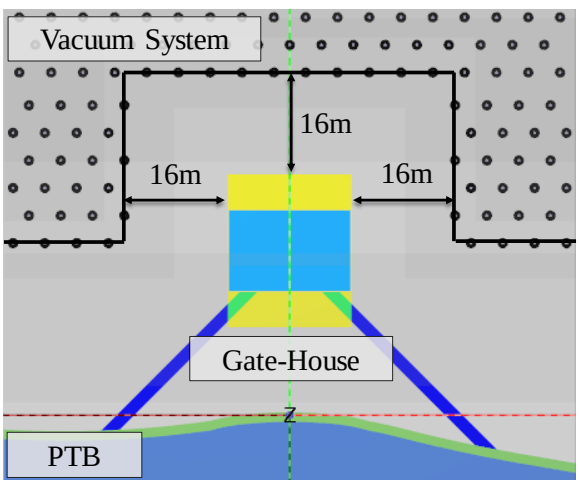

Figure 2. Gate-house relative position to PTB raft.

However, concerns still remained on the behaviour of these very soft structured clays as larger than expected deformation zone could severely impact the PTB and Gate-house foundation. Also, since the in-situ stresses were expected to change significantly under the preloaded area, a large difference in the stress state between the PTB and Apron was envisaged resulting in the potential long term differential settlement issues. As the PTB and Apron are required to function within a very small range of differential settlements during the design life, developing a broad understanding of the behaviour of the Lake Texcoco clays under vacuum consolidation became vital.

Consequently, two fully instrument large scale vacuum consolidation trials were carried out. A significant amount of information on the stress-deformation behaviour and effectiveness of vacuum consolidation application was obtained from these trials.

The impacts of ground improvement on the adjacent foundations during construction and in the long term was studied using a comprehensive 3D Finite Element (FE) analysis. As a critical step in carrying out this exhaustive 3D analysis, the back-analysis of the trials was carried out using 2D FE to ensure that adopted soil constitutive model is capable of simulating the observed behaviour with reasonable accuracy. With a fully calibrated soil model, high-quality forward predictions were made using 3D FE.

This paper focuses only on the back analysis of the two trials. The details of the 2D FE analysis and comparisons of the key monitoring results with the predictions are presented.

3 GROUND IMPROVEMENT USING VACUUM PRELOADING

3.1 Vacuum consolidation trials

In practice, there are two main methods to undertake vacuum consolidation, namely the air-tight

sheet method (membrane system) and the vacuum-drain method, also known as a membrane-less system (Chai and Carter, 2011), Figure 3.

Both methods were employed to assess the performance of each system such that the contractor could have the flexibility to choose either method for the main works.

The vacuum consolidation trials were carried out in accordance with the provided specifications requiring measurement of lateral and vertical displacements, porewater pressure (pwp) developments in the ground and development of a potential vertical crack next to the trial during and after the trial.

Another aspect of interest was to establish the magnitude of suction that can be achieved and maintained by the two different systems. Ground investigation before and after the vacuum consolidation was carried out to assess the impacts on ground properties.

3.1.1 Membrane system trial

The system requires placing an air-tight sheet on the ground surface and sealing the periphery of the sheet by embedding it into the ground.

The trial comprised of a rectangular area of 50x70m. Prefabricated Vertical Drains (PVDs) were installed in a triangular grid with a centre to centre distance of 1.2m. PVDs were installed up to 28m below ground level (bgl). The preload embankment consisted of a 2m thick Tezontle (locally sourced pumice rock). The impervious membrane was preceded by a geotextile, and installed in the middle of two sand protective layers. The membrane has been sealed and anchored into a network of peripheral trenches at a variable depth of 0.5 m to 1m in a mud prepared by mixing the natural ground and bentonite.

The vacuum pressure was applied using 2 pumps. The vacuum system was in operation for about 180days, maintaining an average vacuum pressure of about 69kPa for the first month, and approximately 62kPa thereafter.

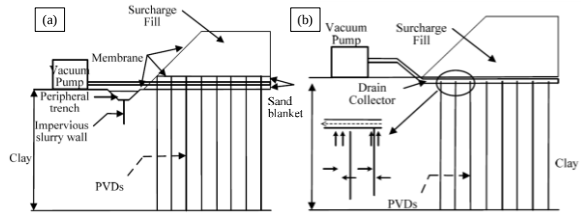

Figure 3. Vacuum-assisted preloading system: a) membrane system; (b) membrane-less system (Indraratna 2009).

3.1.2 Membrane-less system trial

This system uses a surface or subsurface clayey layer as an air-sealing layer and involves the application of the vacuum pressure directly to PVDs with specially made caps. This kind of PVD is usually referred to as a capped PVD or simply CPVD (Chai et al., 2008). Each PVD is connected to a geosynthetic pipe placed on the ground surface, which is connected to a vacuum pump. Unlike the membrane system where an air leak can affect the entire PVD system, in this membrane-less system, each drain acts independently (Indraratna 2009).

A total of 10 vacuum pumps were installed to apply suction to the horizontal pipes connected to PVDs. A vacuum pressure variable between 70kPa to 53kPa was maintained for about 180days.

4 FINITE ELEMENT ANALYSIS

4.1 Back-analysis of vacuum trials

A 2D Plaxis FE model of vacuum consolidation trials was developed to back-analyse the monitored performance. The calibration was done by comparing predicted ground movements and pwp development with monitoring records.

4.2 Numerical model

As a part of the site-wide NAICM design, the HSS model was used extensively in a range of FE analysis for excavation design and foundation

settlement assessment. The model was calibrated against a range of laboratory and in-situ test data to develop input parameters prior to use. The model was further validated by comparing the ground response recorded during a large trial excavation (O'Riordan et al., 2018, 2017). The input parameters developed are summarised in Table 1.

Table 1. Plaxis ground model input parameters

HSSmall model properties										
Soil Unit	Layer Thick.	Unit Weight γ_t	Coeff of permeability k_v	E_{oed}^{ref}	E_{ur}^{ref}	m	c^{ref}	Friction Angle	γ_{70}	G_0^{ref}
		(kN/m^3)	(m/s)	(kN/m^2)	(kN/m^2)		(kN/m^2)	(deg)		(kN/m^2)
1 Crust	1.5	14	1.00E-07	10,000	10,000	-	10	30	-	-
2 FAS	28.5	12	4.00E-09	300	6,000	0.8	0.1	41	1.00E-03	1.25E+04
3 CD	1.5	17	5.90E-05	1,250	6,250	0.8	0.1	38	1.00E-04	2.60E+04
4 FAI	11.5	13.1	2.00E-09	375	3,000	1	0.1	36	1.00E-03	1.25E+04
5 SES	11	14	1.00E-06	1,000	5,000	1	0.1	35	1.00E-03	2.05E+04
6 FAP	13	13.5	4.00E-09	800	5,000	1	0.1	35	1.00E-03	2.00E+04
7 SEI	23	16	1.00E-06	1,850	10,000	1	0.1	35	1.00E-03	4.00E+04
8 FAP 2	15	13.5	4.00E-09	900	5,450	1	0.1	35	1.00E-03	2.25E+04
9 SEI 2	15	16	1.00E-06	2,000	10,000	1	0.1	35	1.00E-03	4.10E+04
All soil units:	$k_v = k_h$		$\upsilon = 0.2$		OCR (1930) = 1					
	$E_{50}^{ref} = E_{oed}^{ref}$		$p^{ref} = 100$		Dilatancy = 0					

4.3 Model settings and parameters

The FE model developed to reproduce the geometrical configuration of the trial is shown in Figure 4. The model was simplified by considering symmetry of the problem to reduce the computational time.

PVDs were modelled using special drain elements that have the ability to simulate the back pressure during suction application. The same elements can also perform as normal drains as vacuum pressure is switched off.

The soil permeability in the PVD area was adjusted iteratively in order to consider the equivalency between the axisymmetric drain conditions and plane strain analysis. The smearing effects due to PVD installation were added to obtain the final equivalent permeability for the PVD zone (Indraratna 2009).

A fully coupled flow-deformation analysis was considered during vacuum consolidation. Appropriate groundwater head flow boundaries were applied to simulate the actual pwp development in different soil layers.

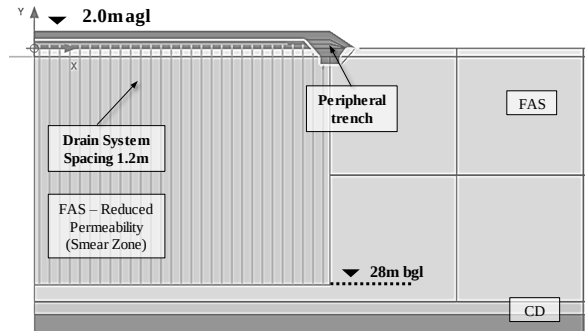

Figure 4. 2D Plaxis view of FE model of vacuum trials

4.4 Stage construction

The stage construction adopted in the FE model closely followed trial sequence. The key stages considered are as follows: Stress initialisation followed by construction of a 1m thick tezontle platform within 30 days period; installation of PVD layers and construction of 2nd layer of 1m thick tezontle within 30 days period; followed by about 180days of vacuum consolidation.

The vacuum pressure applied to PVDs attempted to reproduce the suction monitored at the vacuum pumps, Figure 5.

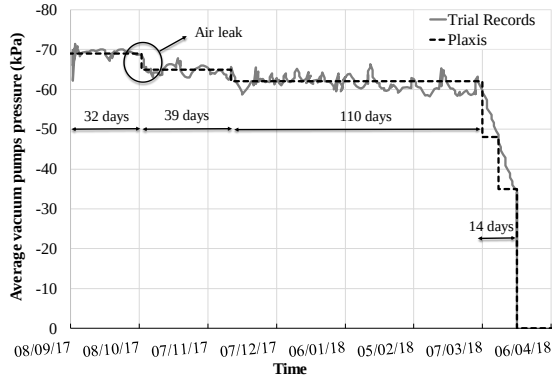

Figure 5. Average vacuum pressure records from monitoring of trials and applied vacuum pressure for numerical analyses.

The vacuum pumps switch off was modelled in two steps to consider a gradual reduction in the applied vacuum pressure. In the last stage, a nor-

mal consolidation was performed for 90days during which PVDs acted as simple drains aiding the recovery of the pwp.

4.5 Results

The results from numerical analyses were found to be generally consistent and as expected based on the published literature (Chai et al., 2005; Gouw et al., 2012). The pwp increased during construction of the embankment while a drop in pwp to below hydrostatic was predicted as suction was applied. As expected, horizontal outward displacements were predicted during the embankment construction which reverses to large inward displacements at the edge of the embankment with the continual application of the suction pressure. The ground settlements continue to increase through all stages and evolved as a typical settlement trough.

5 NUMERICAL MODEL VALIDATION

5.1 Porewater Pressure

The pwp were measured through the trial embankments by Standpipe Piezometers and Electric Piezometers installed at different depths, i.e. 2, 8, 12, 20 and 27m bgl. As shown in Figure 6, FE predictions were found to be in close agreement with the measurement at various depths for both in the centre and outside of embankment.

The recovery in pwp post vacuum switch off was also modelled. The curves at 27m and 20m bgl showed a similar trend to monitoring data, while the shallower curves showed a drop in pwp after initial recovery. This may be due to the continual normal consolidation.

Figure 7 shows a comparison of the measured pwp profile with depth against the FE predictions. A reasonable fit can be seen between the predictions and measurement of the pwp evolution with depth. Figure 7 also indicates that applied suction tends to make lower than expected impact at shallower depths.

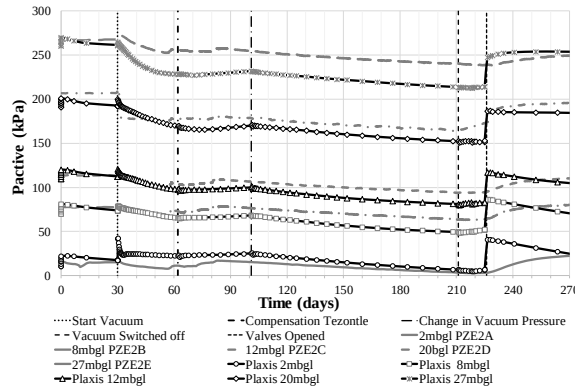

Figure 6. Pwp development comparison between numerical back-analyses and monitoring records from vacuum trials.

As a result, shallower strata does not seem to get benefited from the ground improvement and in fact, could become softer than in-situ state due to remoulding.

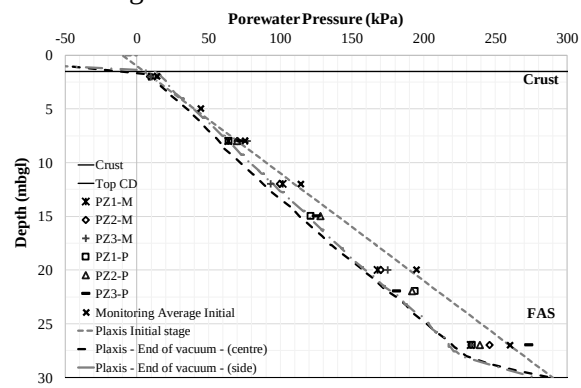

Figure 7. Comparison of pwp development with depth within FAS between numerical back-analyses and piezometer readings.

5.2 Vertical displacements

Figure 8 shows the comparison between the measured settlements with time at different depths with the numerical predictions at the centre of the embankment. A comparison of predicted and measured expansion post vacuum pump switch off is also presented. As shown, the FE predictions were found to be in a good agreement with the measured settlements and expansion profiles.

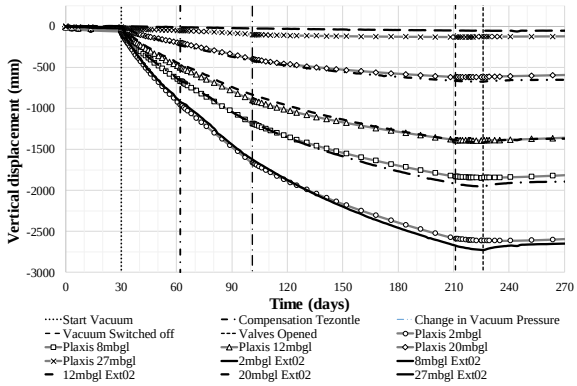

Figure 8. Comparison of deep vertical displacements development between numerical back-analyses and monitoring records.

5.3 Horizontal displacements

The horizontal displacements were measured using inclinometers installed 5, 10, 15 and 20m beyond the peripheral trench axis. Figure 9 presents the comparison at 5m away from the trench axis. The displacements comparison at the vacuum switch off is also presented. The numerical prediction followed the inclinometer 01 profile reasonably well for up to 5mbgl for different stages of embankment construction and vacuum application. Between 5mbgl and 20mbgl, FE analysis overpredicted the lateral movements by up to 210mm at 12mbgl. Although the reason for the difference is not clearly understood but it is believed to be a result of lower level of suction actually maintained in the outermost PVD.

After the vacuum switched off, the displacements envisaged during the consolidation are outward as confirmed by the monitoring data which showed outward lateral displacement of 17mm. The FE model, however, slightly overpredict the outward lateral displacement by approximately 40mm.

Considering the complexities around the application of vacuum in the field and replicating this in an FE model, the prediction on the horizontal displacements are still considered reasonable.

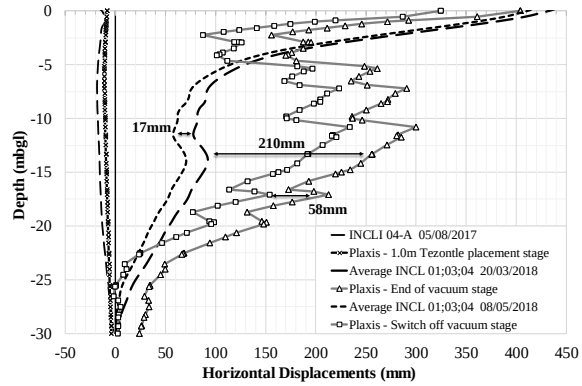

Figure 9. Comparison of horizontal displacements development between numerical back-analyses and monitoring records

5.4 Strength and stiffness increase

The estimation of the undrained shear strength s_u from the pre-vacuum CPTu tests showed a linear increase with depth which concurs with the design line for the PTB area. However, the post vacuum CPTu appears to suggest no appreciable strength gain was achieved. The measurements indicate a marginal upper bound increase of 2.9kPa to 9.4kPa along the depth. The Vane tests showed an increase of undrained shear strength by up to 3 times, which does not match with the CPTu tests. The results of other lab testing including UU tests were also inconclusive. It appears that further GI will be required to make any firm conclusions on the strength gain post vacuum consolidation.

6 CONCLUSIONS

The response of Lake Texcoco clays to ground improvement using vacuum consolidation was tested by means of two different vacuum consolidation trials. A detailed 2D Plaxis model was developed to back-analyse the performance of the trials. The main purpose of the analyses was to assess the performance of the HSS constitutive model and to form a basis for a detailed 3D FE.

Successful calibrations were carried out by comparing the predicted ground movements and pwp development with the measurements.

The HSS constitutive model adopted the Lake Texcoco clays appears to reproduce a reasonable fit to the overall behaviour of the vacuum preloading trials, both in terms of pwp development and displacements prediction. The successful 2D calibrations paved a way for a detailed 3D FE assessment of this complex problem. The evolution of pwp indicates that vacuum consolidation is unable to treat shallow strata successfully. This might be related to the significant initial softening under the embankment load resulting in visco-plastic behaviour.

The post vacuum GI although does indicate some minor improvement in strength but firm conclusions could not be made due to limitations with the available information.

7 ACKNOWLEDGEMENTS

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