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Design and Performance of a Test Embankment on Very Soft Mexico Clay Deposit Improved by Vacuum and Surcharge Preloading with PVD's

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Abstract. A test embankment with a vacuum assisted surcharge loading and prefabricated vertical drains (PVD's) was constructed for the new International Airport of Mexico City. The main purpose of the trial test was to verify the capability of a drain to drain vacuum application method combined with a preloading fill to be successfully applied under extremely difficult soil conditions such as those existing in the construction area of the new International Airport of Mexico City. StarDrains were adopted to accelerate soft soil consolidation. To this aim an intensive monitoring program was carried out to control the effects induced in the subsoil by the consolidation process. Monitoring comprised all the period of construction and also a period of 60 days after vacuum switch off. Vacuum application lasted 6 months. During this time a vacuum pressure of approximately 60 kPa was delivered on average at drains caps (being 78 kPa the local average atmospheric pressure) and a final settlement of approximately 2 m was reached at the center of the test fill. Numerical analyses were carried out through Illicon code. Theoretical predictions based on the adopted model were very satisfactory as for the settlements observed in the center of the test fill at both ground surface and at depth. Measured lateral displacements in the outer area were consistent with, and even more satisfactory than, previous experiences and data reported in the literature. Differential settlements in the treated area were extensively monitored and critically analyzed. Pore water pressure variations measured by Casagrande and vibrating wires piezometers resulted in acceptable agreement among them, but theoretical predictions were of poorer quality compared to those of settlements. Possible reasons of the observed discrepancies were identified and discussed.

Keywords. Trial field embankment, vacuum preloading, drain, Mexico Clay, consolidation settlement.

1. Introduction

Vacuum consolidation technique with prefabricated vertical drains (PVD's) adopted to improve soft soil properties represents a valid alternative to the traditional method of direct preloading with drains since it avoids, or at least reduces, the need to get and manage large amounts of embankment materials and to carry them from the borrow pit

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to the construction site. Moreover, stability problems associated to preloading embankments of significant height and large lateral displacements in the neighboring area brought about by the poor characteristics of the bearing soil are avoided or minimized. According to Khan (2010) [1], additional advantages of the vacuum assisted preloading method consist in the enhanced cost effectiveness resulting from the more rapid application and more rapid removal of the vacuum compared to the placement and removal of the traditional surcharges.

A test embankment with vacuum assisted surcharge loading and PVD's was constructed for the New International Airport of Mexico City. The test site is located in the area of Texcoco Lake, resting over a deep formation of Mexico City clays which are extremely plastic materials exhibiting large and variable natural void ratios and water contents, with very poor geotechnical properties (Shelley [2]). The area is subjected to land subsidence due to a huge deep pumping program in the lower aquifers located below 60 m depth started at industrial scale in 1940. The ground water level is rather shallow.

A vacuum system based on a membraneless "drain to drain" method combined with PVD's having an inner star-shaped plastic cores (StarDrains®) was adopted (Ghionna et al. [3]; Lopez Acosta et al. [4]). In this system the vacuum is individually applied to each drain through specifically designed connection caps. The advantage with respect to the membrane system is that an air leak in the membrane doesn't affect the entire PVD system. The membraneless system has been extensively applied throughout the world (Chai et al. [5]; Khan [1]; Seah [6]; Chai and Carter [7], Tang and Shang [8]; Indraratna et al. [9], Dijkstra [10]).

2. Basic Principles of Vacuum Consolidation and Design Requirements

2.1. General principles

According to the experience gained in previous case applications, vacuum application reduces water pressures inside the drains in a relatively short time, bringing them from the initial hydrostatic profile (u_o) to (u_o-p_{vac}) where p_{vac} is the pressure decay induced by vacuum suction inside the drains; namely $p_{vac}= |u_{vac}|$ being u_{vac} (<0) the negative pressure applied by the vacuum system inside the drains. Both u_o and u_{vac} are referred to the atmospheric pressure (pa) at the considered site.

From then on, notwithstanding possible limited fluctuations or disturbing effects caused by other factors (such as unstable water flow of the air-water mixture along the drain, presence of thin draining layers in the surrounding soil, etc.), a relatively constant water pressure equal to (u_o-p_{vac}) is maintained at the lateral boundary of the drains all along the whole consolidation process. Initially (i.e. at time $t=0^+$), pore water pressures in the neighboring soil remain unaffected by vacuum application and equal to the hydrostatic value (u_o). It turns out that an initial positive hydraulic head difference Δh_{vac} (being $\Delta h_{vac} = p_{vac}/\gamma_w$) takes place between the surrounding soil and the drain boundary originating a water movement directed from the soil to the drain. This hydraulic process is conceptually similar to that occurring in soil around the drains in the traditional preloading technique. The main difference resides in the fact that in the latter technique the initial hydraulic head difference Δh_q between soil and drain boundary originates from the excess pore water pressures Δu_q induced in the subsoil by the surcharge load (q) applied on ground surface while the water pressure inside the drains remains unchanged

and equal to u_o, i.e. $\Delta h_q = \Delta u_q/\gamma_w$. Note that in the previous equation, u_q is referred to the atmospheric pressure, too.

The similarity between the consolidation processes developing in the two afore mentioned preloading techniques has been assumed as theoretical basis in several calculation methods adopted in practice (i.e. Illicon code as per Funk & Mesri [11]), by simply posing $\Delta u_q = p_{vac}$. However, this similarity has not a general validity. The main reason resides in the fact that vacuum suction induces in the subsoil an isotropic compressive normal stress field with a corresponding effective stress path in q-p' space that moves (Lopez Acosta et al. [4]) progressively away from the shear strength envelope. This significantly enhances the stability of the embankment (Indraratna [12]). Moreover, the contractive strains in the horizontal direction tend to reduce the lateral deformations induced by the surcharge load on the external soil. Finally, the vacuum suction extends up to a limited space below the drain tip while the traditional surcharge loads can attain longer depths depending on dimensions of the loaded area. Over time, with the consolidation process going on, pore water pressures in all points located around the drains tend to decrease; the decreasing rates depend on time and distance from the drains boundary. The process comes at its end ($t=t_{100}$) when the pore water pressures in the whole neighboring space around the drains equalize the water pressure (u_o-p_{vac}) inside them. According to Indraratna [12] the use of vacuum pressure increases the rate of excess pore pressure dissipation as a direct result of an increased gradient towards the drain.

An important issue of the vacuum assisted technique is the time at which the vacuum pumps must be switched off. Such time is dependent on the design criteria concerning both the stability and the settlements of the permanent infrastructures/structures built on the treated area under service loads. Such problems are strictly controlled by the average consolidation degree (U_f) of the soil at the end of vacuum application and the average overconsolidation ratio (OCR_f) after vacuum removal. The former one has a positive impact on the shear strength of soil due to the increase in effective stresses caused by consolidation while the latter limits the settlements under service loads. Since the suction applied by the vacuum pumps cannot exceed the atmospheric pressure (realistic values of pumps efficiency span in the range from 80% to 90%), in most cases it is deemed appropriate to adopt a combined version of the technique in which a limited surcharge load is added to the vacuum preloading. This is done by installing the vacuum system on top of a limited fill height and placing additional layers of surcharge materials on it. The risks of shear failure of the preloading fill are minimized by the isotropic stress field induced in the subsoil by the vacuum suction. Despite the relevant international experiences, several issues need to be further clarified to enhance design reliability. They can be briefly resumed as follows (Indraratna & Rujikiatkamjorn [13]; Indraratna et al. [9], Chai et al. [14], Ong & Chai [15], Kahn [1]):

- actual magnitude and distribution of vacuum pressure along the drain length;
- permeability and compressibility of the soil in the smear zone around the drains;
- long term decay of drains discharge capacity;
- reduced efficiency of vacuum system caused by thin layers of permeable soils;
- prediction of lateral movements near the boundary of the treated area;
- nonlinear soil behavior and hydraulic properties.

2.2. Field and Laboratory Investigations

An extensive description of soil investigation is included in Ghionna et al. [3]. Site investigations included: 3 drilling boreholes with continuous coring (BH) for high quality soil sampling; 3 boreholes with SPT tests in sandy layers; 7 CPTU with dissipation tests; 22 Field Vane tests carried out in 4 boreholes.

Particular attention was devoted to the one-dimensional compressibility properties of cohesive layers at short and long term: the laboratory test program included 21 stress controlled oedometer tests and 11 long duration "creep" tests. A preliminary analysis concerning sampling quality conducted through the oedometer method proposed by Lunne et al. [16] evidenced a satisfactory quality class of samples. Unconsolidated undrained triaxial tests (TX-UU) were also conducted to have supplementary data on the undrained shear strength (c_u) of fine-grained materials.

Chemical analyses were also conducted on ground water in order to evaluate possible negative effects on drain filters durability.

2.3. Soil Profile and Groundwater Level

Field investigation evidenced a relatively uniform soil profile consisting of the following units (ref. to Ghionna et al. [3]):

- Formacion Argilosa Superior (FAS): highly plastic and extremely compressive soft clay of lacustrine origin with water content ranging from 150% to 300% and void ratios ranging from 4 to 8; the unit extends approximately from 0 to 30 m from ground level (GL);
- Capa Dura (CD): very dense to hard slightly cemented sandy silt layer from 30 to 31.5 m from GL; the number of SPT blows is usually very high;
- Formacion Argilosa Inferior (FAI): soft clay of lacustrine origin with occasionally interbedded lenses of fine sands and volcanic ashes of centimetric thickness, extending from 31.5 m to 45 m below GL;
- Formacion Argilosa Profunda (FAP): soft clay with characteristics similar to FAI extending at depths below 45 m from GL; this unit is similar to FAI and FAS but it exhibits lower void ratios and natural water contents due to water pumping in the deeper aquifers.
- The ground water table (GWT) was found at very shallow depths, approximately 1 m from GL.

2.4. Soil Properties

Some of the key soil properties obtained by laboratory and field investigation are shown in Figure 1. The initial in situ vertical permeability coefficients ($k_{v,0}$) was estimated by combining the coefficient of consolidation (c_v) evaluated from CPTU and the coefficient of volume compressibility (m_v) derived from oedometer tests results. In the adopted approach, in situ permeability was isotropic (i.e. k_h/k_v= 1). The decrease in permeability during consolidation was computed from oedometer tests, by a best fitting procedure of k_v vs. e, between e_0 and e_p (being e_0 the in situ void ratio and e_p the void ratio at end of primary consolidation) according to Taylor [17] relationship written in semi-logarithmic version:

$$\log (k_v / k_{v,0}) = (e - e_0) / C_k. \quad (1)$$

In this expression $k_{v,0}$ is the permeability at the in-situ void ratio (e_0), whereas C_k is a permeability change index. Tavenas et al. [18] proposed to link C_k to the in-situ void ratio through the following empirical equation: $C_k = 0.5 \bullet e_0$. For these soils, more appropriate values $C_k/e_0 = 0.35$ for FAS formation and $C_k/e_0 = 0.45$ for FAI formation have been adopted in the numerical analyses.

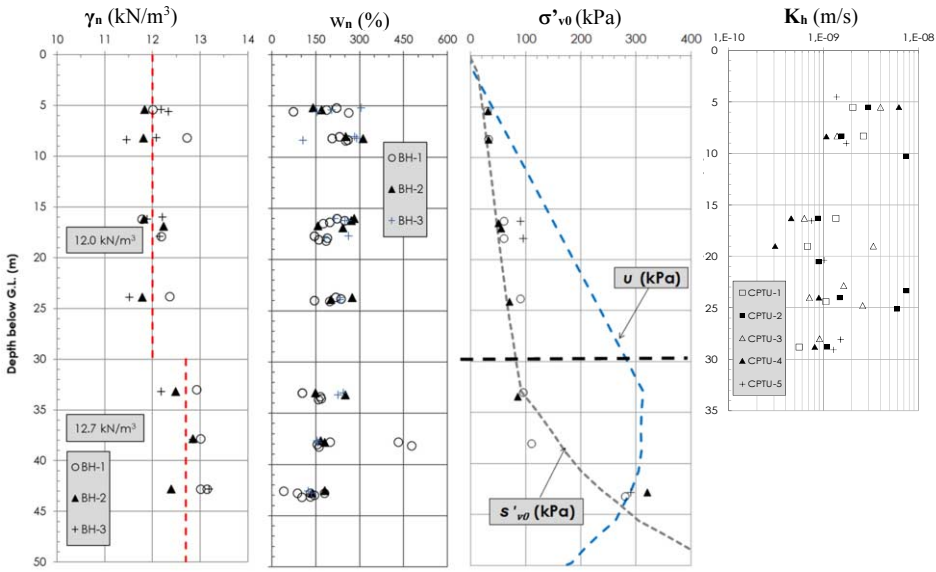

Figure 1. Index properties for FAS and FAI layers: unit weight, water content (Legend ○=BH1, ▲= BH2, +=BH3) – Effective overburden stress - Horizontal permeability from CPTUs.

3. Test Embankment and Drains

A schematic plan view of the test embankment is shown in Figure 2. It has a rectangular shape of 56 x76 m at the base and 50 x 70m on top. It consists of a 2 m high lightweight gravelly-sized material of volcanic origin ("Tezontle"), compacted to a dry unit weight (γ_t) of approximately 12.2 kN/m^3, laid in three layers. A fourth layer of the same material but with finer grain size ("Tezontle fino"), is interposed on top of the second layer to protect the vacuum lines. A final vertical pressure of about 24.4 kPa was applied on the ground surface by the surcharge fill.

PVD drains consist of cylindrical vertical pipes of plastic material having a star shaped internal core wrapped by a protective geotextile filter ("Star Drains").

The main characteristics of PVD StarDrains are summarized in Table 1. The first 3 m of the pipes from the top (namely 1.5 m below the ground water level) were sealed by an impermeable plastic tape to reduce possible air inflow from ground surface. Chai et al. [5] demonstrated the possibility of using the surface soil as a sealing layer; the efficiency of the method, however, is strictly dependent on the permeability of the surface layer and the capability of the upper part of the drains to prevent or minimize air

entry. Using this method (Chai et al. [14]), an almost linear trend of vacuum pressure with depth in the upper portion of the drains is considered appropriate.

Drains bottom was approximately 2 to 2.5 m above the Capa Dura . The installation was carried out from the top of the second layer of Tezontle, namely 0.5 m above the ground level (GL), that in the test fill area is at 2228 m a.m.s.l.. At this elevation the average value of the atmospheric pressure is approximately 78 kPa. However, a lower value (namely $q_w \approx$ 75 m^3/year) was adopted in the numerical analyses, to take into account possible long-term effects caused by buckling, kinking, clogging, etc. (Miura et al.[19], Aboshi et al. [1]; Lee and Karunaratne [21]).

Table 1. Characteristics of PVD StarDrains.

Outer drain diameter	30 mm
Mandrel outer diameter (for installation)	60 mm (cylindrical)
Sealed section length of drain from G.L.	3.0 m
Depth of drains from initial G.L.	28 m
Drains spacing (triangular array)	1.2 m
Total number of drains	3045
Drains discharge capacity	>1000 m^3/year (at 300 kPa confinement) 75 m^3/year (adopted for numerical analyses)

3.1. Vacuum System

The vacuum system consisted of 6 vacuum pumps each one feeding 10 lines of flexible plastic pipes laid horizontally on the surface of the first Tezontle layer.

An average drains number varying from 50 to 51 drains was connected to each line. An air-water separator tank was used at each pump location in order to draw apart and detect the amount of water extracted from the drains. The vacuum pressure recorded at pump manifolds was on average approximately 70 kPa. Taking into account that the atmospheric pressure at the trial fill site is about 78 kPa, the efficiency of the pumps was about 87%, with no single pump working below 80%. The head loss between the pumps and the ends of the vacuum lines on top of their farthermost drains was estimated to be approximately 30%, being about 15% on top of drains located at mid points of the vacuum lines.

4. Instrumentation

In addition to 246 topographic benchmarks, installed both on top of the test fill and all around the boundaries of the embankment, geotechnical instrumentation was installed as shown in Figure 2. Settlement plates were installed at the base of the first Tezontle layer and magnetic multilevel settlement gauges (extensometers) extend from approximately 2 m to 47 m depth. Couples of piezometers (5 couples per “isla”) at 8 m, 15 m, 22 m, 27 m and 37.5 m, were formed by a Casagrande piezometer and an electric vibrating wire piezometer.

No significant variations of ground water level were observed in the whole testing period in deep monitoring wells.

Precision leveling of benchmarks and measurements of the other instruments was carried out daily.

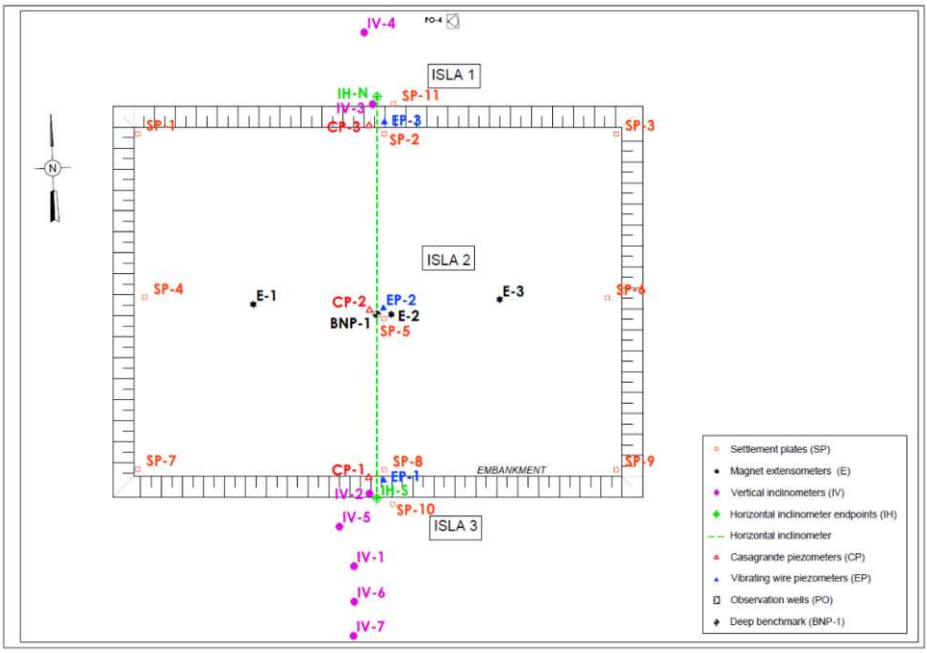

Figure 2. Monitoring instrumentation.

5. Construction History

The loading sequence involved the following phases:

1. 2016/07/15: placing on GL a 0.5 m thick Tezontle blanket (working platform),
2. from 2016/08/15 to 2016/10/01: placing a 0.3 m thick protective layer of fine Tezontle; the placement was in sequence above three different zones,
3. from 2016/08/17 to 2016/10/10: installation of Star-drains,
4. 2016/10/10: positioning vacuum lines and connecting drain caps,
5. from 2016/10/10 to 2016/10/14: vacuum activation and performance checks,
6. from 2016/10/20 to 2016/10/31: completion of Tezontle fill (total 1.2 m),
7. 2017/04/12: end of vacuum application.

The overall duration of vacuum application was 180 days. Monitoring data continued to be collected for a period of 60 days after vacuum switch-off.

6. Analysis of Monitoring Data

6.1. Surface and Deep Settlements of Test Fill

The time trend of surface settlements measured by settlement plates on the test fill is shown in Figure 3. A final settlement of 1.97 m at the center of the embankment was reached in six months. Approximately 96 mm were monitored at the center before the application of the vacuum and the remaining in the subsequent period. The higher settlement rate corresponds to the initial period in which the effect of the vacuum was

enhanced by the placement in two steps of the last 1.2 m high Tezontle layer. The settlements stopped very quickly after the vacuum pumps were switched off; a negligible additional settlement (approximately 18 mm at the center of the fill), was observed in the final monitoring period. Based on results of a previous field test conducted in an adjacent area with a traditional surcharge fill without vacuum, the performance of the new test fill with vacuum was very satisfactory, with much shorter time to reach the same settlement. It is worth mentioning that at the end of the trial the central part of the embankment was partially submerged (approximately 1m below the GWL); the capability to handle with possible fill submergence was of concern for the selection of the numerical code adopted.

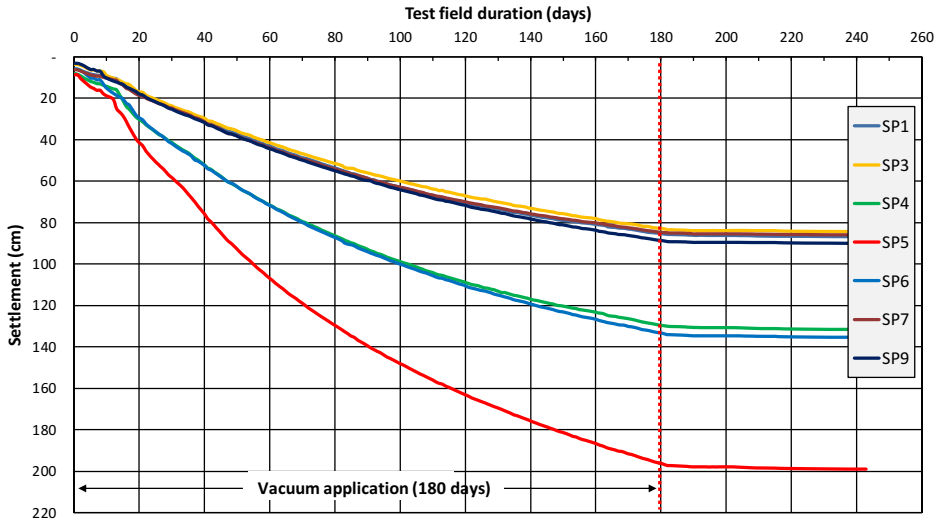

Figure 3. Settlement measured by plates.

It must be mentioned that some amount of settlements is due to the loading applied by the Tezontle layers on ground surface (24.4 kPa). From a general point of view, settlements induced by traditional preloading systems with drains can extend well below the base of the drains, depending on dimensions of the loaded area compared to the length of drains. When partially penetrating drains are used (as in our case) settlements at these depths are likely to take a longer time to develop compared to the corresponding ones in the vacuum treated zone. This will have a negative impact on the final consolidation degree at the end of vacuum application.

6.2. Differential Settlements in Points of The Treated Area.

Figure 4 summarizes data obtained from the horizontal inclinometer. Curves evidence isochronic settlement profiles that have almost symmetrical shapes and trends similar to that exhibited by traditional embankments. The high values of differential settlements observed between the center and the boundaries of the embankment are an effect of the inward movements of the lateral boundaries of the treated area caused by vacuum.

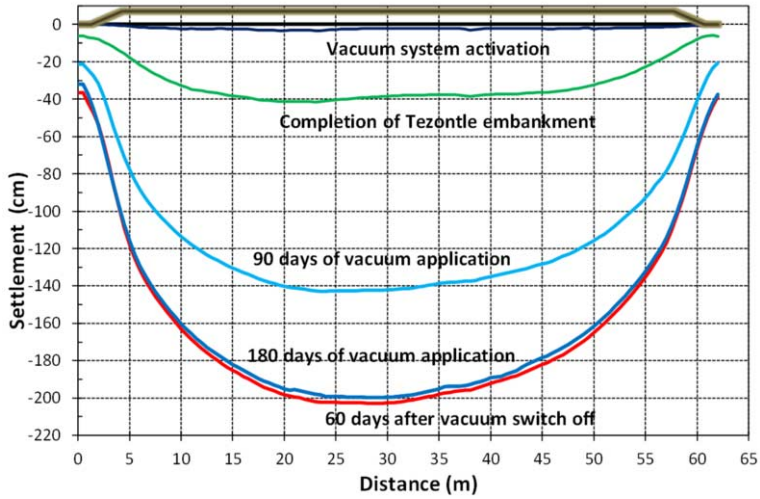

Figure 4. Time history of settlements measured by horizontal inclinometer.

Boundary settlements vary almost linearly with the corresponding ones detected at the center of fill. Along the transverse centerline a ratio varying from 0.63 to 0.69 between lateral and central settlements was observed; this range is in good agreement with that predicted by the elastic theory for traditional embankments. A similar range (from 0.64 to 0.69) was obtained for points along the longitudinal centerline while it was quite lower (0.42 to 0.48) for points located at the corners of the treated area. In any case even these ranges were in satisfactory agreement with those predicted by the elastic theory for traditional embankments.

Settlements in the order of several centimeters (7 to 10) have still been recorded at a distance of approximately 40 m from the edge of the trial embankment at the end of vacuum application.

6.3. Horizontal Displacements Outside the Treated Area

An exhaustive analysis of the lateral displacements measured by the vertical inclinometers installed in the subsoil at several distances from the lateral boundary of the treated area is very complex, depending on several factors. An important role is also played by possible uncertainties connected to the vacuum technique such as: actual intensity of vacuum pressure on top of drains, vacuum pressure profile inside the drains, influence of possible lenses of more permeable strata, etc.

For these reasons Khan[1] suggested that the most appropriate way to analyze the lateral displacements outside the treated area is to base the analyses on empirical approaches. Attempts in this direction are reported in Ong & Chai [15], Chai et al.[22], Mesri & Khan [23], Indraratna et al. [9].

Figure 5 depicts the different features of the lateral movements induced in the subsoil outside the treated area by a traditional surcharge load and by a vacuum pressure. Two opposite behaviors are observed. Under a traditional surcharge load soil tends to bulge in the outward direction while under vacuum pressure, displacements are in the inward direction. For simplicity it can be useful to describe in a synthetic way the approach followed by Chai et al. [24] to analyze the problem. The approach is based on

the consideration of an earth pressure coefficient defined as $K = p_{vac} / (p_{vac} + \sigma'_{vo})$ to be compared with the at rest horizontal earth pressure coefficient k_o. In the above expression σ'_{vo} is the effective vertical pressure and p_{vac} is the applied vacuum pressure. When $k \leq k_o$ there will be no inward lateral movements induced by vacuum and vice versa. Furthermore, since K decreases with depth, even the magnitude of the lateral displacements will decrease with depth.

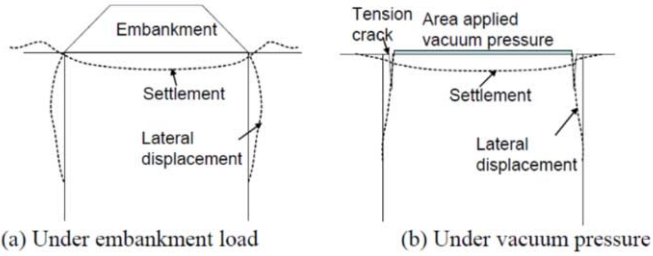

Figure 5. Ground deformations induced by embankment load and vacuum pressure (from Chai et al. [24]).

Chai et al. [24] found a relationship between the stress ratio k and the settlement ratio s_{vac}/s_l observed in comparative oedometer tests, being s_{vac} the settlement induced by an incremental vacuum pressure and s_l the settlement induced by a corresponding incremental surcharge load having the same intensity. The settlement ratio increased almost linearly with decreasing stress ratio evidencing that inward lateral movements induce a reduction in the vertical settlements caused by vacuum pressures with respect to those caused by traditional preloadings of the same intensity. This effect is significant only for high values of vacuum pressure and at low depths; in any case it should be taken into account when large values of inward displacements are observed. (Khan [1]).

Lateral displacements were measured through the vertical inclinometers installed at various distance from the north side boundary of the embankment. It can be concluded that:

- vacuum application induced significant horizontal displacements directed towards the embankment body; the displacements extended to significant depths reaching, in some cases, points near the bottom of the casings;
- the placement of the final layer of Tezontle caused lateral displacements in the outward direction with strong deformations on top of casing;
- after vacuum switch off, the inward lateral movements manifested a slight tendency to reduce or even revert;
- the maximum values measured at ground surface at the end of the test in the inclinometers installed nearest to the fill ranged between 170 and 270 mm, gradually reducing in magnitude moving away from the improved area;
- appreciable values (in the order of 50 mm) were still detected at a distance of 20 m from the border of the embankment.

Based on results gathered from nine case histories, Mesri & Khan [23] evidenced that the ground surface horizontal displacements (δ_s) at the boundary of the vacuum treated area are in an approximately linear relationship with the surface settlements (s_c) at the center of fill. Similar evidences, but with reference to data at depth, have been reported by Indraratna [25], who also ascribed this feature to a specific effect of the vacuum. The above conclusions have been confirmed in the present trial field.

6.4. Tension Cracks Close to The Outer Boundaries of The Fill.

Negative pressures applied by vacuum in soil below the test fill tend to develop tension cracks on ground surface (Seah [6]). Theoretical reasons explaining the formation of such cracks have been presented by Chai & Carter [7]. Tension cracks at a given depth in the subsoil near the boundary of the treated area tend to develop when the overall effective horizontal pressure $\sigma'_h = p_{vac} + \sigma'_{ha0}$ acting on a soil element delimiting the vertical plane of a potential crack is larger than the effective horizontal stress required to maintain a Ko stress state. In the above equation, σ'_{ha0} is the active pressure applied by the soil behind the vertical plane of the crack. Since σ'_h tends to decrease with depth no tension cracks are observed below a certain depth. In this trial field five local cracks were observed, four of them along the minor sides of the embankment (two for each side) and one along the major side. All cracks were oriented approximately parallel to the sides of the embankment, with lengths up to 420 mm and opening sizes up to 35 mm.

6.5. Pore Water Pressures Variations

Pore pressures variations measured by piezometers under vacuum application are influenced, among other parameters, by the distance of the piezometer tips from the drain pipes due to deviations from the vertical of both instruments and drains. Other discrepancies can arise from the time lags of the instruments; the impact of different time responses is enhanced by complex loading histories i.e. when traditional gravity loadings are superimposed to vacuum preloading.

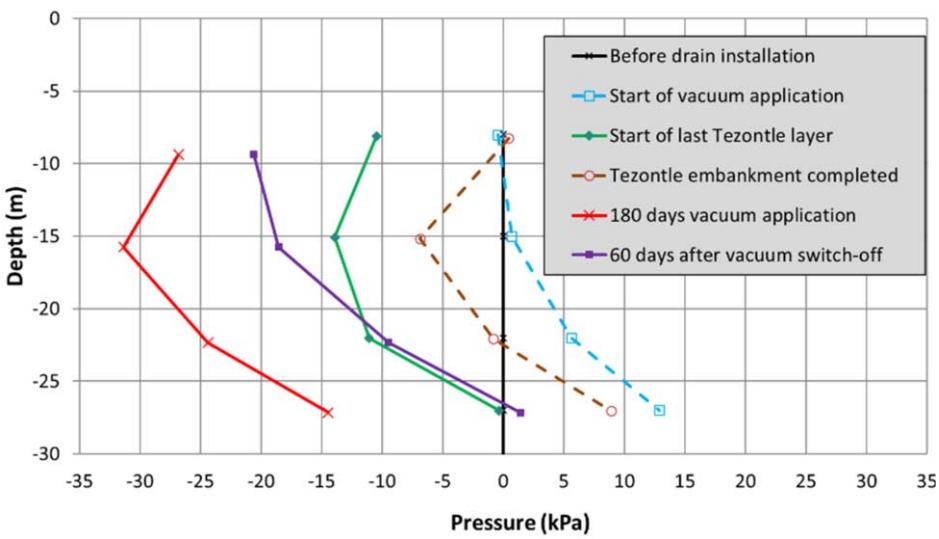

Figure 6. Variation of water pressure (average from Vibrating-Wire and Casagrande piezometers at the center of the embankment) during works. Settlements of piezometers are taken into account.

The profile with depth of pore pressures measured by piezometers installed at the center of fill can be observed in Figure 6. Pressure variations in this figure are corrected for the settlements of the piezometers tips. It is clearly evident that the excess pore pressures profile tends to move towards the positive values when the soil is loaded by

the Tezontle layers while it tends to move on the opposite direction when the vacuum pumps are switched on and during the whole consolidation period. Both the values and the shapes of the curves in the figure depict pore pressures in soil surrounding the drains and are not necessarily indicative of the vacuum distribution inside the drains.

6.6. Water Discharge from the Vacuum System

In a one-dimensional consolidation process a direct relationship exists between the settlement rate (ds/dt) of the ground surface and the flow rate (q_w) of water flushed out from the soil.

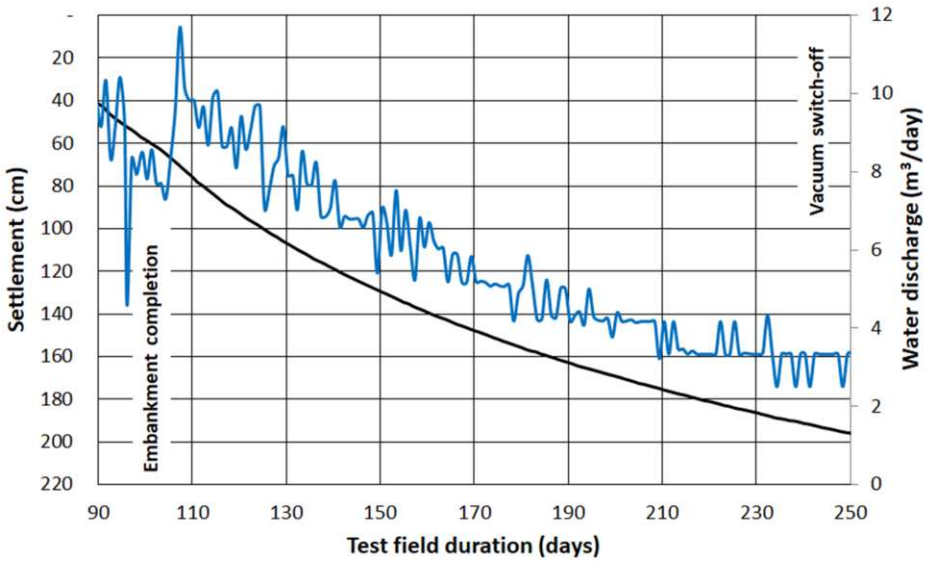

Figure 7. Embankment settlement at the center of the fill (black line) vs water discharge from pumping system (blue line).

Basing on such premise, the daily flow rates $q_w(t)$ of water extracted by the vacuum pump feeding the central portion of the embankment have been plotted in Figure 7 superimposed to the settlements of the center of the embankment.

7. Numerical Modeling

7.1. Numerical Code Adopted

Numerical analyses were carried out by ILLICON code developed at Department of Civil and Environmental Engineering of the University of Illinois at Urbana-Champaign (Funk and Mesri [11]). The code is a finite differences (FDM) software based on the unit cell approach that adopts the nonlinear one-dimensional consolidation theory for clay proposed by Mesri and Rokhsar [26] and further by Mesri & Choi [27]. The key features of the code are exhaustively described in Khan [1] and Mesri and Khan [28], [23].

7.2. Model Assumptions

Initial pore water pressure profile and overburden stress profile were assumed as shown in Figure 1. The complete compression curves were directly input in the calculations.

Compressibility properties in the smear zone was assumed corresponding to fully remolded samples defined by the intrinsic compression line proposed by Burland [29]. According to Chai et al.[14] for one-way drainage, a trapezoidal vacuum pressure distribution inside the drains was assumed to account for the presence of the upper sealed portion; after a linear trend near the top, a constant vacuum pressure distribution was considered in the remaining part with a value corresponding to the vacuum pressure measured on top of drains at connections with the vacuum lines: 60 kPa in stabilized conditions.

7.3. Predictions of Settlements and Consolidation Degrees by Illicon Code.

Figure 8 shows the trend with time of surface settlements in the center of fill predicted by numerical analyses along with those evaluated at the elevations at which magnet extensometers were installed. The estimate can be considered as fully satisfactory.

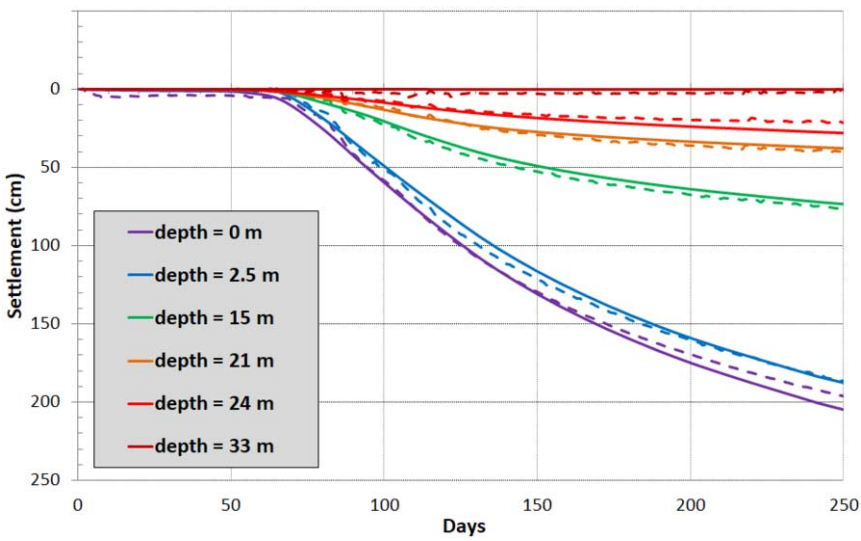

Figure 8. Evolution of settlement with time: comparison between numerical predictions (continuous lines) and field measurements (dotted lines) from extensometers at different depths (left).

Illicon code computes also an overall consolidation degree, expressed in terms of settlements (β), averaged against the depth; the final value at the time of vacuum pumps switch-off was approximately β = 0.7.

8. Closing Remarks

The main purpose of the trial test was to verify the capability of a drain-to-drain vacuum application method combined with a preloading fill to be successfully applied in the

construction area of the New International Airport of Mexico City. An intensive monitoring program was carried out to control the effects induced in the subsoil by the consolidation process. Monitoring comprised all the period of construction and a period of 60 days after vacuum switch off. The following conclusions can be drawn:

- the drain- to-drain vacuum system proved able to maintain the vacuum pressure almost constant, on average 70 kPa at the pump manifolds, being 78 kPa the atmospheric pressure at the test site elevation, for the whole duration of the trial field. This value is almost equivalent to the placement of a 3 m layer of basalt;
- the vacuum assisted preloading fill with drains was very effective, inducing settlements that, at the center of fill, reached about 2 m, with an average consolidation degree approximately equal to 70%. A much longer time would have been necessary for a traditional preloading system without vacuum to cause the same settlements (as demonstrated by results gathered by a traditional surcharge embankment tested before in the same area);
- settlements at the fill boundaries varied linearly with the settlements at the center of fill, in a ratio of approximately 0.42 (at the corners) to 0.64 (on the center lines) with respect to the latter one. Significant settlements (of the order of some centimeters) were still detected at a distance of 40 m from the edge of the embankment.
- lateral displacements outside the fill were influenced by construction history and vacuum application, but they remained relatively small and directed towards the fill. The maximum values observed at ground surface on the inclinometers located nearest to the embankment ranged between 170 and 270 mm. A linear relationship was observed between the ground surface movements detected close to the boundary of the fill and the vertical settlements measured at the center of the fill. Significant values were still observed at approximately 20 m from the embankment edge.
- pore water pressures variations measured by Casagrande and vibrating wires piezometers were in acceptable agreement, the latter showing a prompter response.
- the trend with time of the flow rate of water extracted by vacuum pumps was in good relationship with the corresponding trend of settlements.
- ground surface settlements and deep settlements predicted by numerical analyses were in good agreement with the corresponding ones measured at the center of fill.
- pore water pressures predictions were of poorer quality; possible reasons were mainly identified in operational uncertainties such as deviation from the verticals of both piezometers tips and drains during their installation, kinking and buckling of the flexible PVD drains due to the very large settlements that they experienced in the subsequent consolidation period, influence of different time lags etc.

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