A Case History with Combined Physical and Vacuum Preloading in Colombia

D.G. Yanez, F. Massad

Abstract. The scope of this paper is the back analysis of soft soil treatment case studies in which prefabricated vertical drains together with physical and vacuum surcharge were applied. Initially, the vacuum surcharge systems in vertical drains used worldwide are reviewed. The radial consolidation theory specifically developed for vacuum and physical surcharges is presented, highlighting the influence of the vacuum efficiency parameter. This parameter represents whether or not there is a loss of vacuum pressure in depth. In the second moment, the local geological-geotechnical parameters of the construction are presented, from which data were used as case studies. This characterization was based on the study of regional geology, field geotechnical tests, the collection of undisturbed samples and further laboratory tests. Based on the specific consolidation theory and the geological-geotechnical characterization carried out, detailed analyzes are presented for 2 specific sites of the case study. The detailed analysis is composed of two steps of numerical modeling. The first step analyzes the vacuum loss in depth, as presented in the literature review and the second step is used to check the geotechnical parameters interpreted by the investigation campaign. At the end, comments and considerations are made about the vacuum consolidation theory and the soil characteristics in the case studies.

Keywords: consolidation theories, sample quality, settlement back analysis, soft soil, vacuum surcharge.

1. Introduction

There are several constructive methodologies for improving soft soil, such as partially or completely removing and substituting the soft soil layer; building light embankments; using stability berms; building piled embankments and embankments reinforced with geosynthetics; building in stages; preloading with temporary surcharge; installing Prefabricated Vertical Drains (PVDs) along with the use of preloading; and using PVDs together with both embankment and vacuum surcharge combined. The most adequate solution depends on: the geological-geotechnical characteristics of the deposits; the use of the constructed area and its vicinity; the time available for construction; and the cost of construction. (Almeida & Marques, 2010). Partially or completely removing and substituting the soft soil layer has been less and less applicable because of its high cost and the environmental impact given the need for both borrow and spoil banks. Staged construction is often an economical approach, but the time required for consolidation is often incompatible with the available schedule. PVDs can be installed to improve drainage conditions and geosynthetic reinforcement, i.e., geogrids or geotextiles, used to increase the embankment height in every stage, thus reducing the consolidation time, but sometimes it is not enough to meet modern construction restraints.

A current alternative to preloading in embankments on soft soil is vacuum surcharge, which results in negative pore pressure (suction) on the soil that is approximately 100 kPa next to sea level - a value that is lower in practice. Differently from a traditional embankment, suction does not lead to an increase in total stress, risking stability. Vacuum suction can also be used to accelerate consolidation time compared to conventional surcharge, even with PVD. The vacuum preloading technique was presented by Kjellman (1952) after initial tests in the 1940s. Kjellman’s system consisted in applying vacuum in a sand layer on the surface with an impermeable membrane over it. This impermeable membrane needs to completely isolate the area where vacuum is to be applied, therefore, it can be necessary to build a peripheral trench in the area, maintained with a water or slurry seal. Despite the knowledge available regarding the vacuum application theory (Holtz & Wager, 1975), it was only after the 1980s that this method became popular, especially in the Asian (Choa, 1989; Qian et al., 1992) and European geotechnical communities (Massé et al., 2001), due to advances in vacuum pump technology. The main advantages of vacuum surcharge (sometimes also called “virtual” surcharge) in relation to physical surcharge are the following, as presented by Pilot (1981): (i) there are no problems with embankment instability to be considered; (ii) it eliminates the need for material quarries for temporary surcharge, which are usually expensive and/or unavailable in modern projects; and (iii) the installation, application and removal of vacuum surcharge are quickly carried out. In opposition, vacuum surcharge adds more variables to the already complex problem of soft soil treatment with PVDs and preloading. So much is that Indraratna et al.
(2005) developed a specific radial consolidation theory for vacuum surcharge which can (or not) describe a linear suction pressure decrease with depth.

A vacuum suction system was used as preloading in a highway duplication in Colombia where alluvial soft soil is present. This deposit formation gives the soil a heterogeneous profile in which the stratigraphy includes erratic peat deposits with very low shear strength, eventual fine sands and the upper layers are affected by seasonal desiccation. Vacuum surcharge was especially attractive in this case because the existing highway lane’s operation could be compromised if the new embankment were to cause instability. Two sites 900 m apart from each other were chosen to be presented in this paper, namely A and B. SPT and piezocone testing were executed in the former and a complete investigation, including laboratory consolidation tests, was conducted at the latter. Better settlement predictions were expected to arise from the parameters measured at the latter site B but, given the heterogeneous nature of the soft soil, they did not. Thus, in order to provide reliable results and interpretations, back analysis was carried out to infer the vacuum efficiency and representative soil parameters.

2. Practical Systems for Applying Vacuum Currently in Use

There are currently two well-known systems for applying vacuum on soft soil: the “membrane” and the “drain-to-drain” systems, as shown in Fig. 1. The former, an older system, uses an impermeable membrane mounted on a sand blanket over a peripheral trench in the whole area to be treated, just like the system created by Kjellman (1952), as is commercially employed in Europe and several Asian countries, especially China and Japan. In some cases, it is necessary to dig an impervious slurry wall below the peripheral trench in order to isolate sand lenses and permeable layers, whereas the more recent system allows intermediate sealing sheets to be used (Chai et al., 2008). The second system connects vacuum straight to the PVDs and was developed in the Netherlands in the early 1990s (Sandroni et al., 2012). In both systems, it is common to temporarily lower the water table during the operation of vacuum, which leads to superficial dehydration and occasionally to tension cracks.

One of the advantages of the “drain-to-drain” system in comparison with the “membrane” one is that the former can be used in terrains where there are intermediate sand layers in the soil. In this case, any sand layers below as well as above the water level must be isolated from the vacuum by blind stretches of the drains in order not to lose efficiency of the vacuum, avoiding the need for the impervious slurry wall around the area to be treated in the “membrane” system. If the lower layer is a draining one, the PVD must remain approximately 1.0 m above this stratum, forming a lower impermeable zone. Contrarily, one possible disadvantage of the “drain-to-drain” method regards its suction reach. In the “membrane” system, vacuum pressure is applied in the sand blanket and to the soil through the vertical drains, while in the “drain-to-drain” system vacuum is only applied directly inside the vertical drains. Hence, as Li et al. (2011) verified in laboratory with samples whose diameter was 30 cm and 1 m high, the “membrane” system was slightly more efficient than the “drain-to-drain” one, possibly due to the application of vacuum not only in the vertical drain, but also on the surface, resulting in both radial and vertical suction combined. However, when the soft soil is too thick, this effect is negligible.

The highest vacuum pressure that can be applied is limited by atmospheric pressure, which is higher than 90 kPa up to an altitude of 1,000 m. In practice, however, the highest feasible pressures are between 60 and 80 kPa, due to vacuum losses in the system (in the vacuum pumps, connections, hoses, vertical drains, etc.) and on the soil. A nominal suction ($p_v$) of 80 kPa is usually associated with vacuum consolidation and, when the necessary surcharge

![Figure 1 - Systems for applying vacuum on the field (a) with membrane, and (b) drain to drain (or “membraneless”). Source: Indraratna (2010).](image-url)
for the construction is higher than 80 kPa, physical pre-loading is also employed. When the vacuum pump is located above the level where suction is applied, part of the pressure dissipates throughout the suction height \((H_s)\). It is possible to observe the relationship between atmospheric pressure, suction height - or the difference between the elevation of the vacuum pump and the water table level - and the vacuum pressure effectively applied on the soil, as shown in Eq. 1 and Fig. 2.

\[
\sigma_{\text{vac}} = p_v - H_s \cdot \gamma_w \tag{1}
\]

3. Radial Consolidation Theory with Vacuum, Focusing on its Distribution in Depth

Indraratna et al. (2005) presented a theory of radial consolidation with physical and vacuum surcharge through the overlapping of their effects. Their theory stems from the radial consolidation described by Barron (1948) and Hansbo (1979) and adopts a distribution of vacuum pressure according to what is presented in Fig. 3, where vertical and radial vacuum pressure can be affected by the reduction factors \(k_1\) and \(k_2\), respectively. Corresponding pore pressure excess is shown in Eq. 2. These factors, as well as spacing ratio \((n = D/d_w)\), define the efficiency of vacuum preloading \((G)\) indicated in Eqs. 2, 3 and 4. This vacuum distribution was adopted based on laboratory studies, with large-scale consolidation equipment and vertical drains, which indicated vacuum losses of up to 25% in 1 m of drain, from the top to the bottom of the sample. Other authors, such as Massé et al. (2001), consider that vacuum pressure as constant with depth, being limited only by the maximum depth at which PVDs can be installed, i.e., approximately 45 m.

\[
\Delta u = \sigma_{\text{vac},0} \cdot G(n) + [q - \sigma_{\text{vac},0} \cdot G(n)] \cdot U_v \tag{2}
\]

or

\[
\Delta u = (1 + k_1) n(1 + 2 k_2) + (2 + k_2) \frac{6(n + 1)}{G(n)} \tag{3}
\]

\(G(n)\) efficiency of vacuum preloading, \(q\) conventional surcharge, \(n\) spacing ratio, \(D\) diameter of the effective influence zone of the drain, \(d_w\) diameter of the drain (well), \(\sigma_{\text{vac},0}\) vacuum pressure applied at the top of the drain \((\sigma_{\text{vac},0} < 0)\), \(k_1\) vacuum reduction factor in the vertical direction, \(k_2\) vacuum reduction factor in the horizontal direction.

To assess vacuum distribution in real practice, Indraratna et al. (2005) used field data to compare their theory of radial consolidation with vacuum considering four cases:

- Case A: vacuum pressure is constant in both directions \((k_1 = k_2 = 1)\);  
- Case B: vacuum pressure is constant throughout the vertical drain and varies linearly down to 0 in the radial direction \((k_1 = 1, k_2 = 0)\);  
- Case C: vacuum pressure is constant in the radial direction and varies linearly down to 0 throughout the vertical drain \((k_1 = 0, k_2 = 1)\); and  
- Case D: vacuum pressure varies linearly down to 0 in both vertical and radial directions \((k_1 = k_2 = 0)\).

The one that came closer to the data measured in the field was Case C, in which the vacuum pressure adopted is constant in the radial direction and varies linearly up to 0 throughout the vertical drain \((k_1 = 0, k_2 = 1)\), therefore \(G(n) = 0.5\). In Case A, the vacuum pressure adopted is constant in both the vertical and radial directions, so \(G(n) = 1.0\). In Eq. 2 and 3 it is easy to observe that the efficiency of vacuum preloading \((G(n))\) directly determines the amount of negative pressure that remains at the end of the vacuum consolidation process, therefore also determining the difference in pore pressure that dissipates during settlement. It is possible to notice that when \(G(n) = 1.0\) (Case A), Eq. 2 is
the same as presented by Hansbo (1979). Despite the difference in the amount of pore pressure dissipation in the soil among the cases assessed, the rate of radial consolidation with vacuum, expressed by the relation \( U_r \times T_r \), is the same as the traditional radial consolidation by Hansbo (1979) and Barron (1949).

4. Local Geological-Geotechnical Characteristics of the Case Study

4.1. Alluvial origin of the soft soil

The construction case history analyzed in this paper is the duplication of the Ruta del Sol highway in Colombia, officially called Ruta Nacional 45. The highway runs through five Colombian departments: Cundinamarca, Boyacá, Santander, Cesar and Magdalena, between the cities of Villeta (Cundinamarca) and Ciénaga (Magdalena). More details on the geotechnical parameters and the soft soil treatment can be seen in Yanez (2016). Regional and local geology are described according to CONSOL (2015).

During construction, alluvial soil deposits were found in the low areas next to the Magdalena River, where drainage is insufficient and organic matter and fine compressible soil accumulate. Figure 4 presents the kinds of soil that are formed in these geological conditions. Alluvial soils can also be highly dehydrated during periods of drought.

The Magdalena River begins in southwestern Colombia and runs throughout the country, ending in the Caribbean Sea. It is about 1,550 km long and its average streamflow amounts to approximately 7,000 m³/s. In comparison, the Tietê River, in Brazil, has an average streamflow of about 2,500 m³/s. Differently from marine soft soil, alluvial soils are generated according to the action and intensity of the waters that flow in a river. Rivers that meander in lowlands undergo a random sedimentation process throughout time, varying according to the streamflow and rain intensity over a period of hundreds of years.

The aforementioned construction is located near the equator, in the Intertropical Convergence Zone with a biannual rain pattern. Average annual precipitation amounts to 2,150 mm. Temperatures in the area vary between 27 and 35 °C and relative humidity varies between 75% and 80%. The Magdalena River Basin valley is located between 185 and 800 m altitude in relation to sea level. Rocks from the Paleogene and Neogene periods (both from the Tertiary) of continental origin are found in this area. They usually correspond to conglomerates, sandstones and argillites with the addition of other unconsolidated deposits from the Quaternary period. The region’s relief is moderate, with mounts and hills formed due to the erosion that acted on the sedimentary rocks of the Neogene and landscapes smoothed by recent alluviation, resulting from weathering in the highest areas.

Alluvial deposits originated from the Magdalena River dynamics form terraces elevated some meters above the original streambed, characterized by a lowland morphology slightly inclined in the direction of the body of water. These terraces present slopes that are between 3 and 5 m high on the river banks. On the surface, they are mainly comprised of brown clayey silt to sandy silt and an occasional bedding of rounded gravel and sand with some fines. These regions where drainage is insufficient are frequently flooded and often accumulate fine materials and decomposing vegetation, which generate the soft soil identified in the case study.

4.2. Geotechnical investigations

All borings were located according to the highway stationing, called ST and each ST corresponds to one kilometer along the axis. Initial investigations were carried out by SPT borings. Disturbed samples were collected from all SPT borings in every meter to carry out characterization tests in laboratory (grain size by sieving, water content, specific gravity and Atterberg Limits). Samples with \( N \)
(number of blows) between five and ten blows were found in the middle of a larger soft soil stratum in several borings, which might have been influenced by the presence of twigs and decomposing organic matter in the compressible alluvial layer. Additional geotechnical tests were carried out in sites where the soft soil was thicker than 6 m. Undisturbed samples were collected with 4” Shelby tubes for laboratory consolidation tests, and piezocone tests were conducted with pore pressure dissipation in more than one depth.

Overall, 51 consolidation tests were performed with the undisturbed samples collected ranging from ST 68 to 112 and grouped in a database parameter. Geotechnical assessment was based on all in situ testing and restricted to fair, good, very good and excellent undisturbed samples. Quality analysis of the samples is described in item 4.2.1 ahead and the geotechnical parameters that raised from it described in item 4.2.3.

4.2.1. Quality of the consolidation test samples

Undisturbed sample quality was assessed according to Lunne et al. (1997). This method is based on Eq. 5, that relates the initial void ratio of the sample \( e_0 \) and the void ratio measured in the consolidation test in the effective stress in its natural state \( e \). Table 1 presents the sample disturbance criterion.

\[
\frac{\Delta e}{e_0} = \frac{e_0 - e}{e_0} \tag{5}
\]

In most cases, sample disturbance between collection and transportation to the laboratory were so significant that caused unusable test results. It is possible to observe the analysis of the quality of the 4” Shelby samples submitted to consolidation testing in Fig. 5.

4.2.2. Stress history through piezocones

Preconsolidation stress measured by piezocones (CPTu), according to Mayne & Brown (2003), was compared to the preconsolidation stress obtained through laboratory tests. Figure 6 presents this calibration comparison performed in three sites.

In the laboratory, preconsolidation stress was obtained using the method by Silva (1970). Preconsolidation stress estimates presented by the company that owns the piezocone equipment, by Mayne & Brown (2003), were fairly similar to the values obtained in the consolidation tests, as can be observed in Fig. 6. In general, the stress history observed in the alluvial soil in the case history and presented in the sequence is like what is shown in Fig. 6: an occasional top weathered crust (possibly by dehydration) followed by normally consolidated compressible soil.

4.2.3. Geotechnical parameters

Yanez et al. (2015) obtained preliminary geotechnical parameters from the parameter database comparing the resulting water content and measured compressibility at the laboratory. Poor and very poor samples were not considered in this assessment. It is possible to notice that data regarding compressibility \( C_s/(1 + e) \) as a function of water content are very sparse (Fig. 7b), differently from data concerning \( C_s \) as a function of \( w \) (Fig. 7a).

Table 2 shows regional geotechnical parameters representative of the soil in the case history divided into two groups, as presented by Yanez et al. (2015). The first group consists of clayey silt, with soft to medium consistency and water content \( w \) between 30% and 100%. The second group is considerably more compressible and peaty, with very soft consistency and higher water content. Geotechnical parameters of soil from the alluvial lowlands in the city of São Paulo are presented for comparison.

4.2.4. Representative borings

In Figs. 8 and 9 it is possible to observe representative borings of two sites in the case history. The borings also contain data on natural water content and Atterberg limits. The SPT-336 (Fig. 8a) and the CPTu-5 were carried out close to each other.

At Site A, two pore pressure dissipation tests were performed, analyzed according to Houlsby & Teh (1988). Figure 10 presents the pore pressure dissipation test done 3.1 m deep in CPTu-5.
The result of the pore pressure dissipation tests with piezocones can be observed in Table 3, which summarizes data from 3 dissipation tests conducted in Site A and 6 in Site B. It is possible to notice that the consolidation ratio measured using piezocones (Table 3) is consistent with the one measured in laboratory tests (Table 2) for Group 1, clayey silts.

5. Embankment Instrumentation

Many instruments were installed at the case study site to monitor the construction of the embankments. Namely, settlement plates (marked as PR), inclinometers and standpipe piezometers were widely used. Settlement plates were installed to monitor vertical displacements during embankment construction. These settlement instruments consist of 1 m square metal plates connected to threaded rods and placed on top of the original ground before the embankment heightening. PVC pipes were installed around the rods to prevent friction between the rods and the compacted fill. Benchmarks were also installed several meters away from the settled area.

Standpipe piezometer measurements were not reliable due to dehydration of the instruments by vacuum suc-
tion and poor installation procedures. Furthermore, inclinometers and piezometers readings are not discussed in this article. More details on the geotechnical instruments and monitoring can be seen in Yanez (2016).

Table 2 - Geotechnical parameters of the alluvial soft soil. Source: Yanez et al. (2015, p. 1602).

| Characteristics | Case history | Lowland soil in the city of São Paulo* |
|-----------------|--------------|--------------------------------------|
| Thickness (m)   | 6-21         | < 5                                  |
| Consistency     | Very soft to medium | Very soft                            |
| \( \sigma'_{v} - \sigma'_{cr} \) | 0-20 | 0-40 | N/A |
| SPT             | 0-6          | 0-2                                  | 0-4 |
| LL (%)          | 40-80        | 70                                   | 30-100 |
| PI (%)          | 10-35        | 30                                   | 10-35 |
| % < 5 \( \mu m \) | -            | -                                    | 30-75 |
| \( \gamma \) (kN/m³) | 26.0 | 19.0 | N/D |
| \( \gamma_{isc} \) (kN/m³) | 14.5-16.5 | 11-15 | 11.0-18.0 |
| \( w \) (%)     | 30-100       | > 200                                | 30-300 |
| \( e_{s} \)     | 1-2          | > 6                                  | 1-6 |
| \( S_{s} \) (kPa) | 20-40 | 10-20 | 5-25 |
| OMC (%)         | 1-10         | > 20                                 | N/A |
| \( C_{v} \) (%) | -            | -                                    | 3 |
| \( C_{vzo} \) (cm³/s) | (1-50).10⁻⁴ | (1.5-3.0).10⁻⁴ | (30-50).10⁻⁴ |
| \( C/(1+e_{s}) \) | 0.10-0.35   | 0.40-0.45                            | 0.15-0.35(0.25) |
| \( C/C_{v} \) (%) | 10-25       | 12-15                                | 10 |

*According Massad, F. (2003). Obras de Terra: Curso Básico de Geotecnia. Oficina de Textos, São Paulo, 170 p.

Figure 8 - Example of representative borings in Sites A and B, respectively.
6. Sites Analyzed

6.1. Site A

In Figure 11 it is possible to observe Site A plan with the location of the three SPT borings and the two CPTu tests carried out. In this specific site, no undisturbed samples were collected for additional laboratory tests.

Figure 12 shows the geological-geotechnical profile of this site and the location of the settlement plates (PR-01 to PR-06).

The stress history interpreted using piezocones can be seen in Fig. 13. From these tests, the soft soil next to CPTu-5 (Fig. 13(b)) was interpreted as preconsolidated approximately 20 kPa above the effective initial stress and normally consolidated next to CPTu-4 (Fig. 13(a)).

Between stationing ST 104 + 040 and ST 104 + 400 of the new lane, the soft soil was treated with PVDs installed in a triangular pattern, 1.30 m distant from each other and vacuum preloading. Geotechnical instruments began to be read in June 2014. Vacuum surcharge was applied for approximately five months.

6.2. Site B

Site B is located about 1 km away from Site A. PVD and vacuum surcharge was applied in this site between stations ST 102 + 780 and ST 103 + 240. As can be observed in Figure 9 - Piezocone test CPTu-4 carried out in Site A.

Figure 10 - Pore pressure dissipation at 3.1 m in CPTu-5, Site A in the Case history.

Table 3 - Coefficients of consolidation measured in Sites A and B using piezocones.

| Parameter | Coefficients of consolidation (m²/s) |
|-----------|-------------------------------------|
|           | Site A  | Site B   |
| c<sub>e</sub> (NC) | 2.3E-7  | 4.7E-7  |
| c<sub>e</sub> (NA)  | 1.1E-7  | 2.3E-7  |
| c<sub>e</sub> (Piezocone) | 1.5E-6  | 3.1E-6  |

Figure 12 shows the geological-geotechnical profile of this site and the location of the settlement plates (PR-01 to PR-06).

The stress history interpreted using piezocones can be seen in Fig. 13. From these tests, the soft soil next to CPTu-5 (Fig. 13(b)) was interpreted as preconsolidated approximately 20 kPa above the effective initial stress and normally consolidated next to CPTu-4 (Fig. 13(a)).

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Figure 11 - Site A plan and its corresponding geotechnical tests.

Figure 12 - Geological-geotechnical profile of Site A.

Figure 13 - Preconsolidation and effective vertical stresses in depth interpreted from the piezocone tests in Site A (a) CPTu-4, (b) CPTu-5.
Fig. 14. treatment with PVDs was interrupted about 20 m away from three buried piping (oil and gas pipelines) and a bridge abutment. The figure also presents the location plan of the five SPT borings and the three CPTu tests performed, as well as the plan location of the undisturbed samples collected next to borings SP-240 and SP-316.

Figure 15 presents the geological-geotechnical profile of the south lane axis and the location of the settlement plates in the new lane axis.

In this site, five undisturbed samples were collected for laboratory tests. Geotechnical instruments began to be read in August 2014. Vacuum surcharge was applied for approximately 5.5 months. PVDs were installed in a triangular pattern, 1.30 m distant from each other.

The stress history interpreted using piezocones can be seen in Fig. 16. The results in Fig. 16 (b) and (c) indicate a top weathered crust about 2 m thick and great soil variability below this depth, which can be interpreted as normally consolidated. The previously mentioned Figs. 6(a) and (b) also show the stress history measured at this site using piezocones, compared with the value measured in laboratory tests.

At first glance, it seems that CPTu-2 [Fig. 16(b)] indicates an underconsolidated soil; the values of $\sigma_{v0}$ (dashed line) were figured out by the executor of the test assuming water table at a 1.2 m depth. But, supposing water table at the surface, as indicated by boring SP-240 [see Figs. 8(b) and 14], the $\sigma_{v0}$ would be represented by the line with dashes and dots of Fig. 16-b.

7. Numerical Modeling

7.1 Software used in numerical modeling

RocScience’s Settle3D v. 2 software was used to calculate time-settlement curves considering embankment construction stages and stratigraphy according to the available borings.

Firstly, effective stress in the specified terrain is calculated by the software algorithm and afterwards it computes the resulting deformations according to the type of material adopted for each soil layer. Terzaghi’s unidimen-
sional consolidation theory is used for vertical flow and Hansbo (1979) radial consolidation theory for radial flow (RocScience, 2009). Pore pressure dissipation is obtained through a numerical approach with finite differences, according to the boundary conditions imposed in the model. A nonlinear model for consolidation settlement that accurately considers the relation $e\cdot\log\sigma_v$ is adopted by the software for calculating both compressibility and pore pressure dissipation. Eq. 6 is thus used to calculate permeability as a function of depth. Also, it is possible to specify different values for the coefficient of consolidation in the normally consolidated and overconsolidated branches.

$$k = \frac{C_v \cdot C_e \cdot \gamma_v}{2(1 + e_o) \cdot \sigma_{v0}}$$  \hspace{1cm} (6)

By incorporating Eq. 6 in its algorithm, the software adopts the same assumptions of the consolidation theory by Janbu (1965) and Mikasa (1965), in which settlement develops faster than in the Terzaghi’s traditional theory.

### 7.2. Numerical modeling assumptions

Numerical modeling considered the following assumptions:

- In terms of compressibility an equivalent single and homogeneous soil layer was adopted to account for the profile variability.
- Different coefficient of consolidation values were specified for the normally and overconsolidated branches.
- Compressible soft soil thickness in each model was determined based on the SPT borings next to the settlement plates analyzed.
- Two vacuum distributions in depth were considered in the numerical simulations:
  - constant with depth: $G(n) = 1.0$, widely used; and
  - linear decrease with depth: $G(n) = 0.5$, according to Indraratna et al. (2005), as shown in Fig. 3.

A surcharge at the surface of the terrain equal to the vacuum pressure effectively applied on the top of the vertical drain, as shown in Eq. 1, and multiplied by the efficiency of vacuum preloading, given by Eq. 3. Nominal vacuum pressure ($p_v$) for this construction was specified as 54 kPa, indicated in Table 4, and 2.0 m suction height ($H_s$) considered. Therefore, $\sigma_{v0} = 34$ kPa.
- PVDs length was adopted 1 m shorter than the compressible soil layer.
- By lack of specific information, the ratio of horizontal and vertical coefficients of consolidation was adopted 2.0 and equal to the ratio of horizontal and vertical permeability, indicated in Table 4.
- Embankment buoyancy effect and stress correction due to the layer thickness reduction was considered in all numerical analyses.
- Noncohesive and sandy material, occasionally with gravel, was used for the compacted embankment. Its unit

### Table 4 - Values adopted in the numerical modeling.

| Parameter | Value | Unit |
|-----------|-------|------|
| $k/k_s$   | 2.0   | -    |
| $C/C_e$   | 0.15  | -    |
| $d_s$ (m) | 0.028 | m    |
| $d/d_s$   | 2.5   | -    |
| $k/k_s$   | 2.5   | -    |
| $\gamma_{clay}$ | 15.5 | kN/m$^3$ |
| $\gamma_{embankment}$ | 20  | kN/m$^3$ |
| $p_v$     | 54    | kPa  |
weight varied from 19 to 21 kN/m$^3$, according to the quarries used. Since there were no specific records available of the deposits used in each site, 20 kN/m$^3$ was adopted as its unit weight.

- Secondary consolidation settlement was not considered in the numerical simulations.

Settle3D v2.0 considers horizontal soil profiles with no spatial variation of the soil layer thickness. Therefore, stratigraphic variations in the transversal sections were not considered. Embankments were modeled according to data provided by the instruments, regarding transversal width, slope angle and corresponding construction stages over time. Other values of geotechnical parameters adopted are indicated in Table 4.

8. Settlement Analysis

8.1. Analysis with the parameters of the geotechnical investigations

8.1.1. Site A

For this site, the following measured geotechnical parameters and conditions were assumed:

- void ratio approximately equal to 1 for silts and between 4 and 6 in the thin and randomly occurring peat strata;
- top and bottom drainage;
- vertical coefficient of consolidation, in the normally consolidated and overconsolidated ranges, of 1.1E-7 m$^2$/s and 7.5E-7 m$^2$/s, respectively, based on pore pressure dissipation tests by piezocones (Table 3);
- 20 kPa preconsolidated stress margin shown in CPTu-5, concerning plates PR-01 through PR-04 and normally consolidated soil as shown in CPTu-4, concerning plate PR-06;
- soft soil thickness based on borings and surveying:
  - PR-01: 16 m;
  - PR-02: 17 m;
  - PR-04: 15 m; and
  - PR-06: 12 m
- a 0.12 compression ratio $C_c/(1 + e_0)$ was adopted for the equivalent homogeneous soil layer because it is the arithmetic average of the compressibility obtained in the regional parameter database of the consolidation tests (see item 4.2.3) for clayey silts, the predominant soil in this site; and
- water level 0.5 m below the surface, based on the SPT and piezocone borings performed.

The result of the numerical simulations for the first four settlement plates can be observed in Figs. 17 and 18, in comparison with the measured settlement.

Settlement plates PR-01 and PR-02 had their stress history based on CPTu-4, next to them. It is possible to notice that total settlement estimated using numerical modeling was fairly coherent in relation to the results measured through instrumentation. Contrarily, measured settlement in plate PR-04 was higher than estimated by numerical models. It is possible that stress history around PR-04 is different than plates PR-01 and PR-02. PR-06 measured settlement was also higher than estimated by numerical analyses, even considering vacuum constant with depth.

8.1.2. Site B

The following conditions and measured geotechnical parameters were considered in Site B based on the geotechnical surveys:

![Figure 17 - Settlement measured and calculated - Site A - PR-01 and PR-02.](image-url)
void ratio approximately equal to 1 for silts and between 3 and 6 in the peat strata, of thinner thickness; top drainage, based on the SPT borings; vertical coefficient of consolidation $2.3 \times 10^{-7} \text{ m}^2/\text{s}$ and $1.5 \times 10^{-6} \text{ m}^2/\text{s}$, in the normally and overconsolidated ranges, respectively, based on pore pressure dissipation tests indicated in Table 3; 30 kPa preconsolidated stress margin down to 2 m deep and normally consolidated soil above this depth; soft soil thickness based on nearby borings:
- PR-04: 17 m;
- PR-10: 20 m; and
- PR-11: 11.5 m
- compression ratio $C_c/(1 + e_0)$ equal to 0.16 based on the good to fair quality consolidation test performed with an undisturbed sample collected at this site; and
- water level 0.5 m below the surface, based on the SPT and CPTu borings.

The result of the numerical simulations for these settlement plates can be observed in Figs. 19 and 20, also in comparison with measured settlement.

In plates PR-10 and PR-11 (Fig. 20), additional embankment heights of 2.76 m and 1.94 m were constructed about 100 and 165 days after vacuum pressure ceased, respectively. These new loadings resulted in additional consolidation settlement, as can be seen in the measured and presented data. The later embankment loading was about 55 and 38 kPa for plates PR-10 and PR-11, respectively. Numerical simulations with $G(n) = 1.0$ assumed a temporary surcharge of 34 kPa for vacuum loading, hence the numerical simulation for plate PR-11 does not show the development of significant settlement after 345 days, disagreeing with what was registered by the settlement plate.

All plates in Site B presented higher settlement than estimated by numerical analyses, including the ones considering vacuum constant in depth. In like manner, plates PR-04 and PR-06 from Site A also measured higher settlement than modeled. It is interesting to observe that the accuracy of the predictions in Site B was lower than that in Site A, despite the larger amount of geotechnical investigations available, including a good quality consolidation test.

### 8.2. Parameters obtained by back analyzing settlements

Based on the numerical models described in item 8.1, it was possible to adjust parameters $C/(1 + e_0)$ and $c,$

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**Figure 18** - Settlement measured and calculated - Site A - PR04 and PR06.

**Figure 19** - Settlement measured and calculated - Site B - PR04.
through back analysis to the best fit between calculated and measured data. A twofold adjustment was performed for each settlement plate. In the first step, parametric analyses regarding soil compressibility were carried out until total modeled settlement was best adjusted to measured data. The next step was a parametric analysis of the soil coefficient of consolidation looking for best adjustment regarding the shape of the curve. Settlement plates were adjusted for both considerations of vacuum distribution in depth: constant when $G(n) = 1.0$ and decreasing when $G(n) = 0.5$.

8.2.1. Site A

Stress history in the top layer was revised from pre-consolidated to normally consolidated in plate PR-04. The other plates stress history was not revised. Resulting adjustments performed can be observed in Figs. 21 and 22.

In Site A, the adjustment tended towards higher compressibility parameters than the ones measured by geotechnical investigations. On the other hand, the coefficient of consolidation was adjusted for a lower number than measured by dissipation tests.
Based on the coefficient of consolidation back analyzed, it was possible to calculate the time necessary to reach 95% of primary consolidation if no PVDs were installed. Using Terzaghi’s theory for vertical consolidation and considering Taylor’s solution for embankment construction time (variable loading), maximum and minimum times required would be 52.6 and 16.5 years for settlement plates PR-02 and PR-06, respectively. It is thus possible to notice that the installation of PVDs was effective regarding the time required for primary settlement to develop.

8.2.2. Site B

Stress history was not revised for any plate in Site B. Likewise Site A, compressibility adjustment through back analysis in Site B tended towards higher parameters than ones measured through geotechnical surveying. Resulting adjustments performed are presented in Figs. 23 and 24.

Plate PR-10 compressibility parameter analyzed was higher than expected for local organic silts and closer to expected for peat. Furthermore, coefficient of consolidation was adjusted to a lower figure than measured by dissipation tests, as was the case at Site A.

Again, using Terzaghi’s theory for vertical consolidation and considering Taylor’s solution for embankment construction time, the maximum and minimum times required for 95% of the primary consolidation to be concluded if no PVDs were installed at Site B would be 32.5 and 181.8 years for settlement plates PR-11 and PR-10, respectively.

9. Final Remarks

For the development of this paper, firstly an investigation campaign with 51 laboratory consolidation tests were analyzed regarding regional geotechnical parameters. Afterwards, in 2 different sites, 7 settlement plates were chosen for a more thorough analysis of vacuum consolidation theory and settlement predictions. Corresponding geotechnical parameters were inferred from field investigations and calibrated through back analysis. Concerning the analyses carried out, the following conclusions were reached:

- Geotechnical soil parameters were inferred by tests that presented a good or higher quality and calibrated through
back analysis of the measured settlement. Results obtained showed that back analyzed compressibility might have been influenced by the random presence of peat and organic matter, since it was systematically higher than what was obtained through consolidation tests.

- Preconsolidation stress estimates presented by the owner company of the piezocone equipment adjusted fairly well to the values obtained in the consolidation tests. Observed stress history was usually normally consolidated but also presented an occasional top crust with a low preconsolidation stress margin. Due to the loading imposed by the compacted embankment and vacuum combined pressures, stresses beyond the preconsolidation pressure may have been reached.

- The coefficient of horizontal consolidation \( (c_h) \) of the soft soil, inferred through back analyses based on the geotechnical instrumentation, resulted in values systematically lower than the ones measured using piezocones. This might be related to the alluvial composition of the soil, which presents sandy and silty fractions that could have affected the measurements made using piezocones. This hypothesis is related to the quality of the investigations and to the representativeness of this coefficient \( (c_h) \) for the equivalent soil as a whole.

- When the present research was being developed, two different theories of consolidation with vacuum surcharge \( (\sigma_{vac}) \) were compared, namely \( \sigma_{vac} \) constant with depth - more widely used - and \( \sigma_{vac} \) decreasing with depth - according to Indraratna et al. (2005). In Site A, the more widely used theory resulted in better accuracy, whereas in Site B the results were basically the same.

- The assessment of the theory that considers vacuum decreasing with depth depends on the availability of piezo-
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List of Symbols

- $B$: CPTU pore pressure ratio
- $C$: compression index
- $c$: coefficient of horizontal consolidation
- $CR$: compression ratio
- $C$: swelling index
- $c_u$: undrained shear strength
- $c_v$: coefficient of vertical consolidation
- $D$: diameter of the effective influence zone of the drain
- $d$: diameter of the smear zone
- $d_d$: diameter of the drain (well)
- $e$: void ratio
- $e_i$: initial void ratio
- $F(n)$: drain spacing factor
- $F_d$: drainage resistance factor
- $f_c$: cone lateral friction
- $F_i$: soil disturbance factor
- $G(n)$: efficiency of vacuum preloading
- $H$: compressible layer thickness
- $k_h$: horizontal permeability in the undisturbed zone
- $k_p$: horizontal permeability in the disturbed or smear zone
- $k_v$: vertical permeability
- $n$: spacing ratio
- $q_c$: cone tip resistance
- $q_d$: drain discharge capacity
- $R$: radius of the cone
- $t$: time
- $T_n$: horizontal time factor
- $T_v$: vertical time factor
- $U$: percentage or degree of average primary consolidation
- $w$: pore pressure
- $u_i$: initial pore pressure
- $z$: depth
- $\gamma$: natural specific weight of the soil
\( \gamma \): specific gravity weight
\( \gamma' \): submerged weight
\( \gamma_w \): specific weight of water

\( \sigma'_{pc} \): preconsolidation pressure
\( \sigma'_{ve} \): effective vertical stress
\( \sigma'_{vo} \): initial effective vertical stress