

# Control of geotechnical behavior after surcharge installation using prefabricated vertical drains and settlement monitoring

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## ABSTRACT:

Gas To Energy (GTE) project is one of the most impactful projects in Guyana. The project aims to build the first NGL facility dedicated to generating electricity and delivering a reliable, clean, and affordable energy source for the people of Guyana.

Ground improvement using prefabricated vertical drains (PVD) and surcharge was employed to accelerate soil consolidation, enhance stability and anticipate creep settlement over an area of approximately 22 hectares. This process involved the installation of more than 300 000 PVD reaching an average depth of 23 m.

The surcharge installation phasing and the consolidation time required for each phase were initially designed based on the soil parameters obtained in the geotechnical investigation. Thanks to the real-time monitoring of settlement and other instrumentation, the geotechnical parameters were recalibrated, and soil behavior was predicted using Asaoka and Hyperbolic methods. These approaches enabled precise determination of the optimal timing for surcharge removal, ensuring that the defined residual settlement criteria were met with confidence.

**KEYWORDS:** Soil improvement, Prefabricated Vertical Drains (PVD), surcharge, consolidation, settlement, monitoring, creep, gas to energy (GTE), Guyana.

## 1 INTRODUCTION

The Gas To Energy (GTE) Project is aimed to transport natural gas from the offshore Stabroek block field, located at 190 km of the coast of Guyana in the Atlantic Ocean, to an onshore facility for processing and electricity generation (Figure 1). The project includes a pipeline, a 300 MW combined cycle power plant and a gas processing plant (NGL plant).

The NGL plant is located approximately 20 km south of Georgetown in Guyana and covers a total surface area of about 40 Ha. Figure 1 shows the general site location.

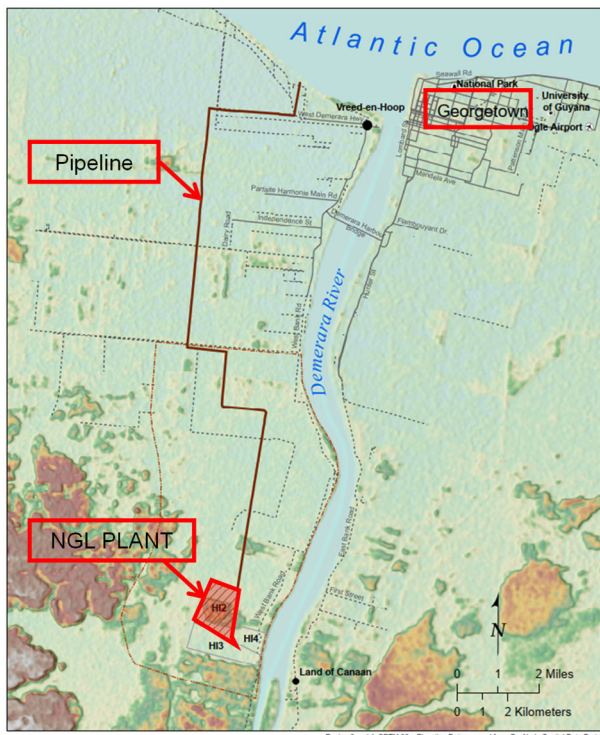


Figure 1. General site location.

The project area to be treated covered 21.8 ha, with an original ground level ranging from +1.0 m to +2.0 m MSL (average +1.5 m MSL), and a groundwater table located at approximately the same elevation. A working platform was first established at +1.8 m MSL, and the final platform level was set at +4.5 m MSL to enable the installation of various plant facilities on piles foundation with ensuring adequate flooding protection. However, placing this fill over the site would have imposed significant additional vertical stress on the underlying soft clay, resulting in excessive post-fill-placement settlement and negative skin friction on the piles. To mitigate these risks, appropriate soil improvement using PVD and combined with surcharge preloading were implemented prior to the commencement of the foundation works.

Menard was awarded the execution of the PVD installation, as well as the design of the PVD layout and the determination of the required surcharge load to meet the project's long-term settlement criteria. These criteria were defined according to the type and sensitivity of the structures:

- < 5 cm for process-related areas and,
- < 10 cm for buildings and offsite areas.

## 2 GEOTECHNICAL DATA

Based on the geotechnical investigation and thanks to the high density of boreholes carried out within the project area, the soil conditions were accurately defined.

Figure 2 shows the CPT data ( $q_c$  and  $R_f$  versus depth) carried out in the area studied here. The CPT profile indicates soft clays deposits extending to a depth of down to 22 m, characterized by an average  $q_c$  value of about 0.4 MPa, classifying it as very soft clay. Below this layer, dense sand and stiff to hard clay have been identified and considered as stable and not posing any geotechnical risk.

Several soil profiles were identified according to the thickness of the soft clay layers (Figure 3). Table 1 summarizes the soil strata encountered for the profiles within the project area subject to ground improvement (Areas A and B).

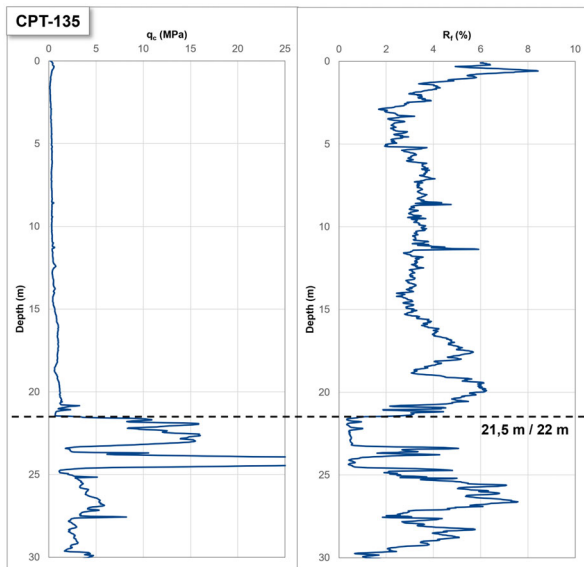


Figure 2. CPT data –  $q_c$  and  $R_f$  versus depth.

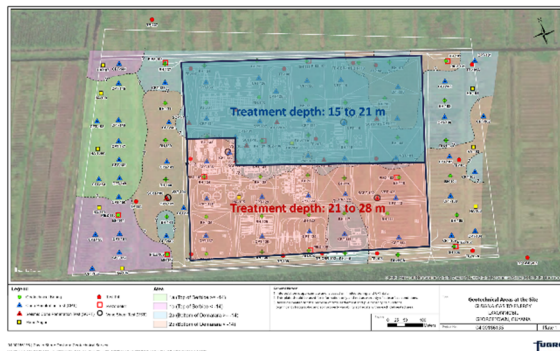


Figure 3. Soil conditions and reached treatment depth.

Table 1. Soil layers elevations.

Layer	Area A			Area B		
	$Z_{sup}$ (m MSL)	$Z_{inf}$ (m MSL)	H (m)	$Z_{sup}$ (m MSL)	$Z_{inf}$ (m MSL)	H (m)
Dem. 1	+1.5	-4	5.5	+1.5	-4	5.5
Dem. 2	-4	-14	10	-4	-14	10
Dem. 3	-	-	-	-14	-21	7
Coro. 1	-14	-21	7	-	-	-
Coro. 2	-21	-30	9	-21	-30	9
Coro. 3	-30	-50	20	-30	-50	20

Remark: Dem. = Demerara Clay; Coro. = Coropina Clay.

Table 2 and Table 3 show the compressibility and the consolidation parameters obtained for each layer. Note that:

- a ratio of 3 between the coefficient of radial consolidation and the coefficient of vertical consolidation ( $C_r/C_v$ ) was initially retained.
- no information was provided regarding the secondary compression index. As the ratio  $C_{ae}/C_c$  is usually between 3 to 5 % for soft clays, an average value of 4 % was considered.

Table 2. Soil Compressibility parameters.

Layer	$\gamma$ ( $\text{kN/m}^3$ )	$e_0$ (-)	$C_c$ (-)	$C_s$ (-)	OCR (-)
Dem. 1	15.5	1.8	0.6	0.080	1.0
Dem. 2	15.5	1.8	0.7	0.100	1.1
Dem. 3	18.0	1.8	0.7	0.100	1.1
Coro. 1	18.0	0.65	0.3	0.045	1.8
Coro. 2	19.0	0.65	0.3	0.045	1.8
Coro. 3	19.0	0.65	0.3	0.045	1.1

with:

$\gamma$	Unit weight	$C_s$	Recompression index
$e_0$	Initial void ratio	OCR	Overconsolidation ratio
$C_c$	Compression index		

Table 3. Consolidation parameters.

Layer	$C_v$	$C_r$	$C_v$	$C_r$
	( $\text{m}^2/\text{year}$ )		( $\text{m}^2/\text{s}$ )	
Dem. 1	2	6	$6.3 \times 10^{-8}$	$1.9 \times 10^{-7}$
Dem. 2	1.2	3.6	$3.8 \times 10^{-8}$	$1.1 \times 10^{-7}$
Dem. 3	1.2	3.6	$3.8 \times 10^{-8}$	$1.1 \times 10^{-7}$
Coro. 1	1.8	5.4	$5.7 \times 10^{-8}$	$1.7 \times 10^{-7}$
Coro. 2	1.8	5.4	$5.7 \times 10^{-8}$	$1.7 \times 10^{-7}$
Coro. 3	1.8	5.4	$5.7 \times 10^{-8}$	$1.7 \times 10^{-7}$

with:

$C_v$	Coefficient of vertical consolidation	$C_r$	Coefficient of radial consolidation
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### 3 PROCEDURE & RESULTS

#### 3.1 Design principle

The principle of ground consolidation by PVD and surcharge is to sufficiently reduce the void ratio by occurring settlement during consolidation phase in order to meet the required post-construction (residual) settlement. The adopted design approach was carried out in several steps:

- Assess the total settlement, including both primary and secondary (creep) settlements, over the design life of the structures, from the initial geotechnical parameters.
- Determine the required surcharge load and calculate the necessary degree of consolidation to meet the long-term criteria.
- Define the vertical drain layout (mesh spacing) based on the allocated consolidation time and the target degree of consolidation.

Note that the total surcharge includes an additional temporary surcharge intended to achieve the targeted primary settlement during the consolidation phase, with a degree of consolidation below 100 % (approximately 80 – 85 %). This additional surcharge also helps pre-consume a significant portion of the expected secondary settlement by inducing a higher level of primary settlement. Once this temporary surcharge is removed, the soil behaves as an overconsolidated soil with limited creep settlement. This approach, known as the pre-ageing method, is illustrated Figure 4.

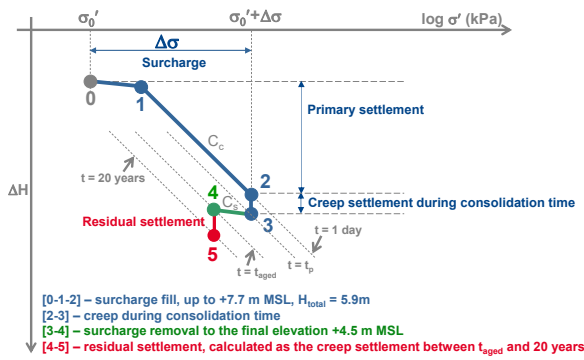


Figure 4. Pre-aging concept.

The primary settlement was estimated using Eq. (1) for over-consolidated soils and Eq. (2) for normally consolidated soils:

$$\Delta H = H \frac{C_s}{1 + e_0} \log \left( \frac{\sigma'_p}{\sigma'_{v0}} \right) + H \frac{C_c}{1 + e_0} \log \left( \frac{\sigma'_{v0} + \Delta \sigma'}{\sigma'_p} \right) \quad (1)$$

$$\Delta H = H \frac{C_c}{1 + e_0} \log \left( \frac{\sigma'_{v0} + \Delta \sigma'}{\sigma'_{v0}} \right) \quad (2)$$

with:

H	Soil thickness	$\sigma'_p$	Preconsolidation pressure
$\sigma'_{v0}$	Overburden pressure	$\Delta \sigma'$	Applied additional stress (surcharge)

The creep settlement was calculated as follows:

$$\Delta H = H \frac{C_{ae}}{1 + e_0} \Delta[\log t] \quad (3)$$

with:

$C_{ae}$	Secondary compression index
t	Time at which the secondary settlement is calculated

The vertical degree of consolidation was calculated using Terzaghi's theory (Eq. (4), Terzaghi and Peck (1948)), and the radial degree of consolidation was determined based on Barron (1948)'s theory (Eq. (5) and (6)). Finally, the overall consolidation ratio was obtained using Carrillo (1942)'s theory (Eq. (7)).

$$U_v(t) = \left( 1 + \frac{1}{2T_v^3} \right)^{-1/6} \text{ where } T_v = \frac{C_v}{H_d^2} t \quad (4)$$

$$U_r(t) = 1 - \exp \left( \frac{-8 \cdot T_h}{F(n)} \right) \text{ where } T_h = \frac{C_r \cdot t}{D^2} \quad (5)$$

$$F(n) = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2} \text{ where } n = \frac{D}{d} \quad (6)$$

$$(1 - U) = (1 - U_r)(1 - U_v) \quad (7)$$

with:

$T_v$	Time factor	t	Time
$C_v$	Coefficient of vertical consolidation	$C_r$	Coefficient of radial consolidation
$H_d$	Longest drainage path	D	Equivalent diameter of the unit cell
d	Drain diameter		

Once the additional surcharge aimed to accelerate primary settlement during the consolidation phase and to consume sufficient creep settlement was removed, the equivalent soil age can be obtained using the following formula:

$$t_{aged} = t_p \times \left( \frac{\sigma'_{v, \text{before removal}}}{\sigma'_{v, \text{after removal}}} \right)^{\frac{C_c - C_s}{C_{ae}}} \quad (8)$$

The long-term residual settlement is then calculated as the creep settlement occurring between the time of surcharge removal  $t_{aged}$ , obtained from Eq. (8), and the project's design life (20 years), using the following equation:

$$\Delta h_{res} = H \times \frac{C_{ae}}{1 + e_0} \log \left( \frac{t_{aged} + 20 \text{ years}}{t_{aged}} \right) \quad (9)$$

with:

$t_p$	Preloading time
$\sigma'_{v, \text{before removal}}$	Soil effective vertical stress before surcharge removal
$\sigma'_{v, \text{after removal}}$	Soil effective vertical stress right after surcharge removal

### 3.2 Preliminary design

Based on the geotechnical parameters provided below and the target final platform level to be achieved (+4.5 m MSL), the surcharge installation schedule was developed for each soil profile in accordance with the settlement criteria (5 cm for sensitive zones and 10 cm for typical zones). Table 4 presents the construction and consolidation phases defined for sensitive areas (5 cm residual settlement) located within Area B, including:

- the required number of 45 cm-thick fill layers to achieve the long-term residual settlement criterion, installed in two phases to ensure fill slope stability during the surcharge placement.
- the installation rate (45 cm per n days).
- the required consolidation time at the end of each phase.
- Geogrid bands required to ensure fill slope stability during placement.

Table 4. Preliminary construction and consolidation phasing (sensitive area, soil profile B).

Step	Final Elev. (m MSL)	Initial Elev. (m MSL)	$\Delta h_{fill}$ (m)	$\Delta t$ (day)	t cumulated (day)	Comment
Step 1	1.80	2.25	0.45	2	2	1 geotextile 600 kN/m: +2 m MSL
Step 2	2.25	2.70	0.45	2	4	
Step 3	2.70	3.15	0.45	2	6	
Step 4	3.15	3.60	0.45	2	8	
Step 5	3.60	4.05	0.45	2	10	Fill construction H = 3.6 m Rate: 45cm per 2 days
Step 6	4.05	4.50	0.45	2	12	<b>Phase 1</b>
Step 7	4.50	4.95	0.45	2	14	
Step 8	4.95	5.40	0.45	2	16	
-	5.40	5.40	-	28	44	Consolidation time
Step 9	5.40	5.85	0.45	4	48	
Step 10	5.85	6.30	0.45	4	52	
Step 11	6.30	6.75	0.45	4	56	Fill construction H = 2.25 m Rate: 45cm per 4 days
Step 12	6.75	7.20	0.45	4	60	<b>Phase 2</b>
Step 13	7.20	7.65	0.45	4	64	
-	7.65	7.65	-	56	120	
Surcharge removal						

### 3.3 Monitoring & Settlement prediction

The instrumentation and monitoring program is essential for the successful execution of this type of ground improvement in such soil conditions, as it enables accurate assessment of actual soil settlement and verification of consolidation performance. It also allows comparison of observed soil behavior with design predictions and facilitates timely adjustments, when necessary, to ensure compliance with the project's settlement criteria.

The implemented instrumentation system (see Figure 5) comprised:

- Multi-point piezometers to monitor the dissipation of excess pore water pressure within the soft clays.
- Earth pressure cells to measure the vertical stress induced by the surcharge installation.
- Settlement plates installed on the working platform, to record total vertical ground movement.
- Inclinometers along the boundaries of the embankment to monitor lateral soil displacements and detect potential slope failure mechanisms.

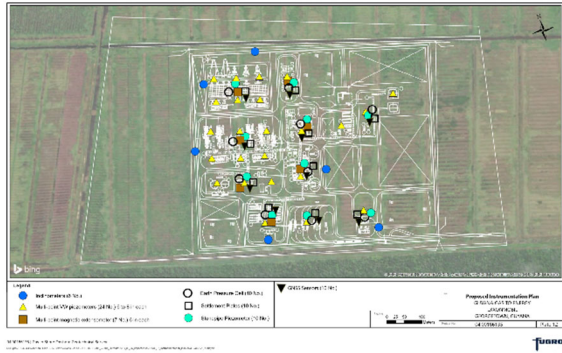


Figure 5. Proposed instrumentation system.

Figure 6 shows the pore water pressure change versus time. An increase in pore pressure is observed during the surcharge construction phase, followed by a decrease in pore pressure under constant vertical stress during the consolidation phase.

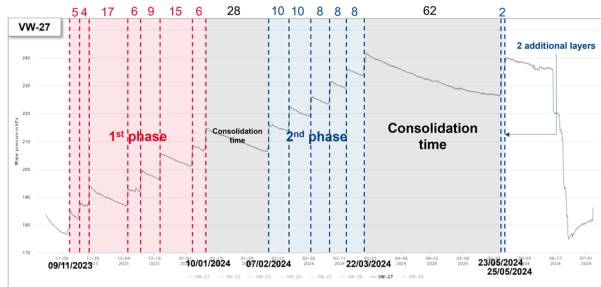


Figure 6. Piezometer example.

Using settlement plate and piezometer data obtained from consolidation phase 1 (period under constant surcharge pressure), the final settlement was predicted according to the Hyperbolic method proposed by Tan (1995) and the Asaoka graphic approach (Asaoka, 1978, Magnan, 1980). Accurate estimation of the ultimate settlement is possible once sufficient data have been collected and clear linear trends are observed, as shown in Figure 7 for the hyperbolic method and Figure 8 for the Asaoka approach. This corresponds to a degree of consolidation  $U(\%) = s(t)/s_{\infty}$  exceeding 70 %, at which point a stabilisation in the calculated final settlement is achieved, allowing a confident prediction of the ultimate value.

The results show that the final settlements predicted by both methods are very close, 81 cm and 77 cm for the hyperbolic method and the Asaoka method, respectively. This indicates that the actual soil settlement is much smaller than the one initially estimated from the original compressibility parameters (~ 140 cm). Furthermore, the settlement-time curves reveal that the observed consolidation rate is slower than that obtained using a  $C_r/C_v$  ratio of 3. Consequently, the compressibility and consolidation parameters ( $C_c$  and  $C_r/C_v$ ) were hence recalibrated to align both with the measured final settlement and the overall shape of the settlement–time curve

(see Figure 9). This calibration resulted in a  $C_r/C_v$  ratio of about 2, allowing a more accurate estimation of the consolidation rate and the final settlement expected for phase 2.

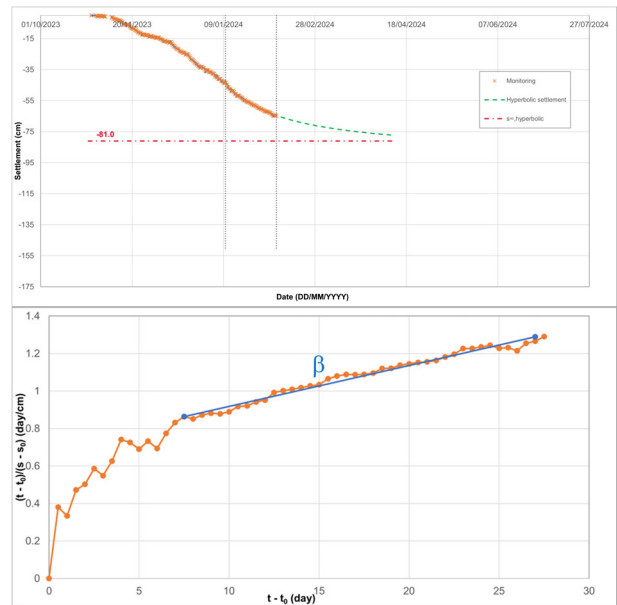


Figure 7. Ultimate settlement estimation – Hyperbolic approach – Phase 1.

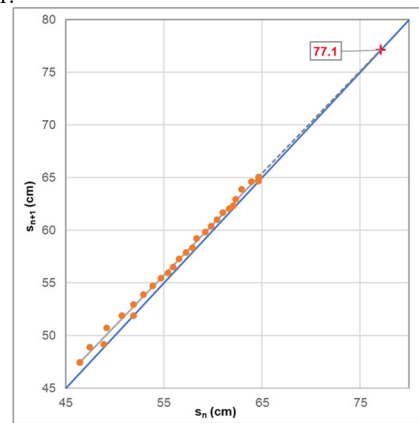


Figure 8. Ultimate settlement estimation – Asaoka approach – Phase 1.

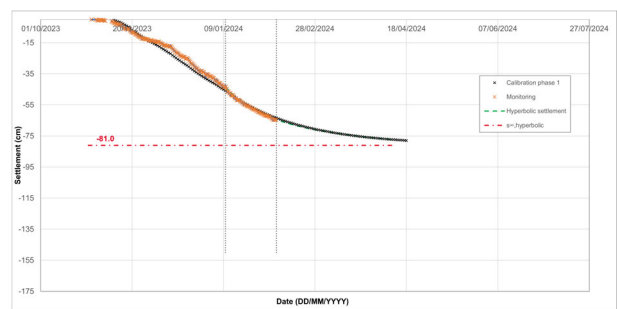


Figure 9. Parameters recalibration – Phase 1.

Based on the recalibrated parameters, the surcharge construction sequence and consolidation time for Phase 2 were revised (see Table 5 for more details) to satisfy the long-term residual settlement criteria and maintain fill slope stability during installation.

Table 5. Phasing adjustment.

Fill Construction and Consolidation Phasing						
Area 5 Central		Sensitive areas				
Typical / Sensitive	Final Elev. (m MSL)	Initial Elev. (m MSL)	$\Delta h_{(m)}$	$\Delta t$ (day)	$t_{(cumulated)}$ (day)	Comment
Step 1	1.8	2.25	0.45	4	4	1 geotextile 600 kN/m - 2 m MSL
Step 2	2.25	2.7	0.45	4	8	
Step 3	2.7	3.15	0.45	7	25	
Step 4	3.15	3.6	0.45	6	31	
Step 5	3.6	4.05	0.45	8	39	Fill construction H = 3.6 m
Step 6	4.05	4.5	0.45	14	53	
Step 7	4.5	4.95	0.45	6	59	Phase 1
Step 8	4.95	5.4	0.45	1	59	
	5.4	5.4	-	27	86	Consolidation time
Step 9	5.4	5.85	0.45	7	93	Phase 2
Step 10	5.85	6.3	0.45	7	100	
Step 11	6.3	6.75	0.45	7	107	Fill construction H = 2.7 m
Step 12	6.75	7.2	0.45	7	114	
Step 13	7.2	7.65	0.45	7	121	Rate: 45cm per 7 days
Step 14	7.65	8.1	0.45	7	128	
	8.1	8.1	-	60	188	Consolidation time

Surcharge removal down to FGL = +4.5 m MSL.

Once sufficient monitoring data were available, the ultimate settlement after placement of the total required surcharge was predicted using the same methodology (see Figure 10 and Figure 11). The results again showed good agreement between the Hyperbolic and Asaoka methods with an average ultimate settlement of approximately 168 cm. However, the residual settlement calculated using the pre-ageing concept detailed in Section 3.1 was slightly higher than the allowable 5 cm after the proposed consolidation period of two months. Considering the project’s time constraints, two additional 50 cm surcharge layers were placed over a two-week period to induce further settlement and confidently achieve the required residual settlement target.

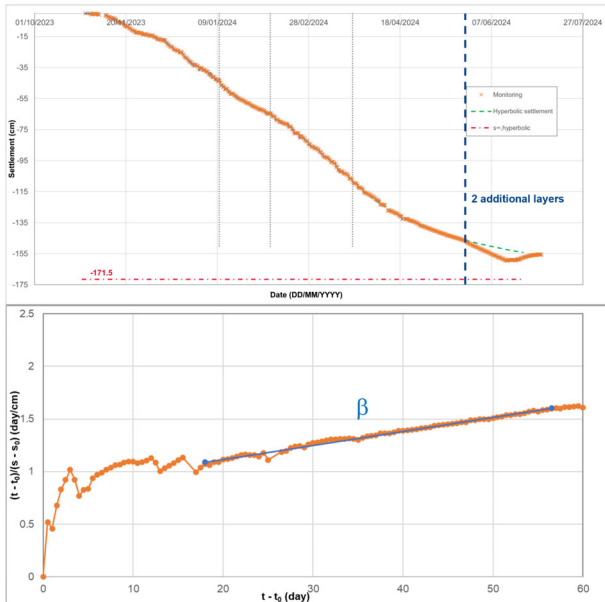


Figure 10. Ultimate settlement estimation – Hyperbolic approach – Phase 2

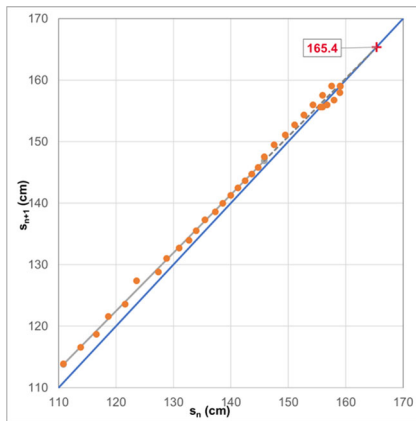


Figure 11. Ultimate settlement estimation – Asaoka approach – Phase 2.

Figure 12 shows a slope failure observed in approximately 4m-high white sand fill material placed in the storage area without ground improvement by PVD. This incident highlighted the importance of closely monitoring horizontal displacements, particularly through real-time inclinometer readings. For the treated area, the maximum horizontal displacement recorded under a surcharge height of 6.3 m was about 56 cm (see Figure 13), corresponding to a horizontal-to-vertical displacement ratio of 38 %. This value remains below the critical threshold of 45 – 50 %, beyond which surcharge installation must be halted to prevent slope instability. Managing the challenging soil conditions to avoid such failures was one of the most critical aspects of the project. The failure observed in a storage area without ground improvement in Figure 12 served as a clear reminder of these risks.



Figure 12. Photos of a circular failure in untreated storage area.

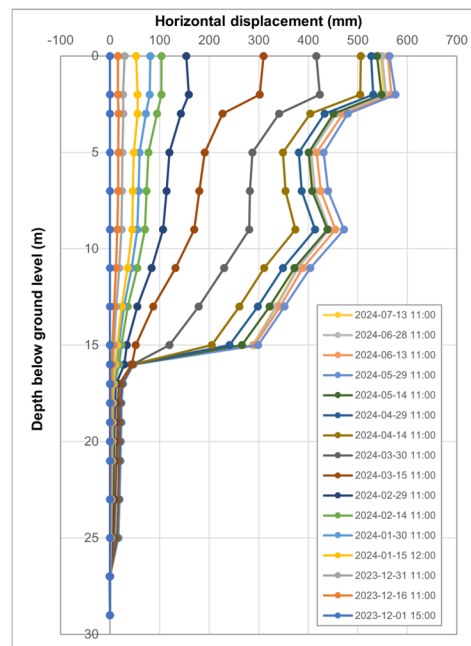


Figure 13. Example of inclinometer measurements.

#### 4 PVD EXECUTION

The PVD installation works covered a total surface area of approximately 218 000 m<sup>2</sup>. The drains were installed using a uniform 0.9 m × 0.9 m square grid spacing. A total linear length of approximately 7.0 million meters of PVD was installed across the site, reaching an average depth of 23 meters. To meet the production targets, up to five machines were progressively mobilised on site. Each machine achieved a daily production rate of approximately 500 PVDs per 10-hour shift, representing 11 500 linear meters per shift.

The works started in early September 2023 with the deployment of a single machine and were successfully completed by mid-March 2024, at which five (5) machines were operating.



Figure 14. PVD installation.

#### 5 CONCLUSIONS

Raising the platform level from the original ground elevation of +1.5 m to the final elevation of +4.5 m, over very soft clay deposits with 23 m average thickness, presented a significant challenge in controlling settlement prior to constructing the various facilities of the NGL plant for the GTE project. To address this, ground improvement using prefabricated vertical drains (PVD) combined with surcharge preloading was successfully implemented to limit long-term residual settlement and ensure slope stability.

A preliminary design was developed based on the available geotechnical parameters, and an extensive instrumentation program was installed to monitor soil behavior during surcharge placement and the consolidation period. Thanks to this monitoring system, which included, among others, settlement plates and inclinometers, the soil behaviour in terms of both vertical settlement and horizontal displacement was closely tracked.

The collected monitoring data enabled accurate prediction of the actual final settlement using the Hyperbolic and Asaoka methods, and then the back-calculation of the real compressibility and consolidation parameters of the ground. The residual settlement was estimated using the pre-ageing concept, and the additional temporary surcharge was removed with confidence once sufficient settlement had taken place during the consolidation phase.

#### 6 ACKNOWLEDGEMENTS

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#### 7 REFERENCES

- Asaoka, A. 1978. Observational procedure of settlement prediction. *Soils and Foundations* 18(4), 87–101.
- Barron, R.A. 1948. Consolidation of fine-grained soils by drain wells. *Transactions of the American Society of Civil Engineers* 113, 718–742.
- Carrillo, N. 1942. Simple two- and three-dimensional cases in the theory of consolidation of soils. *Journal of Mathematics and Physics* 21(1–4), 1–5.
- Magnan, J.P. 1980. Analyse graphique des tassements observés sous les ouvrages. *Bull. Liaison Labo P. et Ch.*, 109, sept-oct. 1980, Réf, 2478.
- Tan, S.A., Lee, S.L., Yong, K.Y., and Chew, S.H. 1991. Hyperbolic method for consolidation analysis. *Journal of Geotechnical Engineering*, ASCE 117(11), 1723–1737.
- Tan, S.A. 1995. Validation of hyperbolic method for settlement in clays with vertical drains. *Geotechnical Engineering* 26(1), 91–103.
- Terzaghi, K., and Peck, R.B. 1948. *Soil mechanics in engineering practice*. New York: John Wiley & Sons.