Ground Improvement of Mongla Container Yard in Bangladesh

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ABSTRACT: In the context of constructing container yards on soft soil layers, it often becomes necessary to undertake ground improvement works to mitigate potential settlement caused by anticipated dead and live loads. In situations involving substantial accumulations of soft and compressible clay deposits, it becomes imperative to expedite the process of consolidation. The utilization of prefabricated vertical drains in conjunction with preloading is a commonly employed technique for ground improvement in such scenarios. In the context of ground improvement projects involving soft soil, it is necessary to determine the extent of improvement accomplished in the soft, compressible clay. This assessment assists in verifying if the soil has reached the desired level of consolidation, hence allowing for the removal of preloading measures. The analysis can be conducted using observational methods, wherein continuous records of ground behavior are monitored starting from the date of equipment installation. Field instruments are employed to validate the efficacy of soil improvement activities and to guarantee that the prescribed level of consolidation resulting from the sandfill and surcharge loading has been attained before the removal of the preloading. This paper presents a comparative analysis of different approaches used to assess the degree of consolidation in a case study conducted at the Mongla Port container yard project in Bangladesh.

KEYWORDS: Prefabricated Vertical Drain, Soft Clay, Observational Method, and Consolidation Settlement.

1. INTRODUCTION

The Bangladeshi coastline is abundant, with numerous small and large ports facilitating a substantial volume of trade. The Mongla Port Authority (MPA) is located in the coastal region near Rampal, on the Bagerhat side. This region consists of soft clays with a high degree of compressibility and a very low ability to support weight. There are currently six yards for containers that have been constructed on this soft clay. The first yard was created in the late 1980s. Ground improvement is necessary in this location to create a container terminal capable of handling heavy loads from containers. The predominant method employed is the utilization of prefabricated vertical drains (PVDs) in conjunction with preloading.

Prefabricated vertical drain (PVD) is the most cost-effective method among the various ground improvements used to accelerate the dissipation of excess pore pressure under embankments (Hansbo, 1979; Bergado & Patawaran, 2002; Yan & Chu, 2005; Chai et al., 2010; Mesri & Khan, 2012; Long et al., 2013; Indraratna 2010).

Reang et al. (2021) performed a case history at the Agartala-Bangladesh Railway Embankment. According to subsoil conditions, PVDs were installed in a triangular pattern with three different spacings, 0.7, 0.8, and 1.0 m, with depths of 15, 6, and 7.5 m. The maximum height of the embankment was 9 m. The study revealed that using PVDs reduces the consolidation time and construction time. According to Bergado et al. (2000), PVDs have been used as a ground improvement technique at Second Bangkok International Airport, Thailand. On soft Bangkok clay, three full-scale test embankments were constructed in stages. PVDs have been penetrated to 12 m thick, soft clay with a square pattern. The settlement rate and amount predicted by Asaoka's method proved to be in agreement with the observed values. Bergado et al. (1999) conducted a study on the Bangkok–Chonburi New Highway, which was 80 km long in Thailand. The embankment was constructed on improved soft marine clay. An explanation of how two full-scale test embankments made on soft clay deposits in China's eastern coastal region performed has been presented by Huang Wenxiong et al. (2006). The embankment was built on PVD improvement subsoil. In a triangle layout, the PVDs are driven to a depth of 19 m, with a 1.5 m c/c spacing. The finite element approach was employed to analyze the field performance of the two embankments. According to the study, PVD increased the average vertical hydraulic conductivity of soft subsurface by nearly 30 times compared to the original, untreated subsoil. To improve local traffic, the Ballina Bypass route in Ballina, New South Wales, Australia, was planned by Indraratna et al. (2018). The site's soil profile is composed of three different layers: a stiff clay layer, a soft silty clay layer, and a very thick clay layer that is more than 40 meters deep. To ensure ground stability and improvement, PVDs, as well as with surcharge and vacuum preloading, were used to consolidate the soft soils prior to construction. A trial embankment was constructed to the north of Ballina in order to assess the method's performance in this location. Bergado & Patawaran (2002) conducted research on PVDs in soft Bangkok clay as part of the construction of the new Bangkok International Airport. The comprehensive investigation confirmed that at a particular time, the magnitudes of consolidation settlements increased along with a commensurate reduction in PVD spacing. Lastly, the results of the full-scale study proved the effectiveness of PVDs in improving soft Bangkok clay. Indraratna et al. (1994), in “Performance of Embankment Stabilized with Vertical Drains on Soft Clay,” numerically studied the efficiency of vertical band drains on soft marine clay in Malaysia. The vertical band drains were installed at 1.3 m c/c with a triangular pattern and penetrated 18 m of the soft ground. The embankment was raised over the improved ground at 4.74 m in two stages. The numerical study was conducted by using the CRISP FEM code. The outcome of the numerical study was the influence of smear, and well resistance could be ignored if an efficient drain was modelled. Bergado et al. (1998), in “PVD Improvement of Soft Bangkok Clay with Combined Vacuum and Reduced Sand Embankment Preloading,” showed the improvement of undrained shear strength of soft Bangkok clay at Second Bangkok International Airport, Thailand. PVDs were installed with a triangular pattern at 1 m c/c spacing and penetrated 15 m. After the improvement of the soft clay, two full-scale embankments were constructed over it. Finally, the undrained shear strength was found to increase 1.5 to 2 times with respect to unimproved ground. Shukla & Kambekar (2007), “Working of Prefabricated Vertical Drain-A Case Study,” demonstrated the soil stabilization techniques for constructing a double-track broad-gauge railway embankment in the Belapur-Seawood-Uran areas of Navi Mumbai, India. The embankment has a length of 1.5 kilometres. The subsurface investigation report indicates that the soil layer comprises Yellowish firm clay to Greyish Marine Clay, with a depth of up to 8.5 m. The use of PVD with a triangle arrangement and a spacing of 1.5 m centre-to-centre is being evaluated as a treatment method for soft soil. They opted for a triangular layout instead of a square pattern in order to achieve a consistent consolidation process. The author's conclusion states that including PVD treatment not only decreases the consolidation time of the embankment but also minimizes its lateral movement. The study findings indicate that the predicted settlement closely aligns with the actual settlement. The study conducted by

Desai et al. (2015), “Ground Improvement of Marine Clay for Highway Construction in Mumbai, India,” examined the behaviour of coastal clay treated with PVD beneath a 3.25-kilometre-long highway with a height of 4.3 metres. In order to assess the subsoil characteristics according to the design specifications of the highway, a total of five borings were conducted. These borings reached depths ranging from 6.30 to 14.40 metres below the original ground level. There are 650,000 PVDs built, with a spacing of 1.2 metres between each drain and arranged in triangular patterns. For the investigation of the marine clay foundation, monitoring instruments such as settlement markers, piezometers, and inclinometers have been deployed. The large displacement from the inclinometer was found immediately during preloading due to the low consolidation properties of marine clay up to 0.34 m. The study shows significant improvement without marine clay foundation shear failure. Singh et al. (2016), in “Ground Soil Improvement Work for the Construction of Udaipur Station Yard in the State of Tripura by Using PVDs,” studied the performance of PVD under a 6 m high railway embankment 1 km. The subsoil reports reveal that 9-13 m of soft clay below the EGL are comprised. They advised PVD to use a triangular pattern with 800 mm c/c spacing to construct this embankment. Chakraborty et al. (2017), in “Soft Ground Improvement at the Rampal Coal-Based Power Plant Connecting Road Project in Bangladesh,” studied the performance of a Vertical Sand Drain under the highway consisting of 4 lanes with two slow-moving lanes for 5.76 km. A total of 11 borings extending to the depth of 11-40 m depths have been performed for the design requirements of the laboratory test. In total, 1,10,000 numbers vertical sand drains (VSD) with 5 m in length, having diameter 0.125 m with 1 m c/c spacing installed. The Standard Penetration Tests (SPT) were performed to evaluate the shear strength gain of treated soil. The study result showed that the treated subsoil's SPT values significantly increased. An investigation was conducted on the settlement of a PVD-treated clay foundation of a railway embankment along the Gopalganj to Kashiani 32 km track Hore et al. (2020). In addition to PVD, lime treatment has been employed to mitigate the swelling potential of the topsoil. The recorded settlement time was deemed excessive in relation to the anticipated duration. The occurrence of this inaccuracy can be attributed to the construction of the smear zone, the absence of imposed surcharge load, and insufficient subsoil information. They recommend implementing PVD treatment for clay foundations beneath railway embankments as a cost-effective and time-efficient alternative. To ensure continuous monitoring of the foundation soil response to the fill load increments as well as the deformations of the embankment, South America mounted battery-operated geotechnical monitoring instruments at the time of ground treatment and staged embankment construction by Wu et al. (2019). The embankment had a 1:2 side slope, was 690 meters long, and 25 meters high. The backfilled foundation was installed with PVDs, and sand columns and staged embankment fill placement followed. To successfully use the Observational Method in embankment building, they came to three conclusions: planning and maintaining constant, accurate instrumentation measurements during the construction was essential. Organize and submit the monitoring data in a timely manner. The results will then be evaluated and interpreted by experienced geotechnical experts who will be in charge of the design, construction, and alternative design/remedial measures preparation in the event of a “best way out.” The efficiency of the OM would be severely compromised if any one of the three links broke. Gui et al. (2020) studied combined drainage consolidation-Preloading for the Colombo–Katunayake Expressway (CKE), Sri Lanka. At the site, up to 15 m, thick peat deposits existed. The embankment height was 1.7 m to 7.3 m. Sand piles, gravel piles, and PVDs with preloading are used as ground improvement techniques. This study showed that the consolidation effect of the sand and gravel piles was better than the PVDs. Imai et al. (2020) conducted a study to investigate the effectiveness of Sand compaction piles and PVDs for mitigation of liquefaction and consolidation settlement at the Dhaka Metro Rail Depot yard. The soft soil thickness was 5-15 m. The study result showed that the settlement of SCP areas was smaller than the PVD areas. Siddique et al. (2022) investigated the performance of PVDs with preloading at the Chittagong Port. The Port Park Area of Chittagong Port consists of a tidal plain at a narrow strip between Chittagong's hilly uplands and the Bay of Bengal. Fifteen boreholes were drilled to gather subsoil information for the site, which were distributed over the area. Based on the subsoil investigation reports, there was extremely soft to firm silty clay or clayey silt, fine-grained silty sand, and some decomposed materials close to the ground surface. These materials were found to be between 3.0 m and 7.0 m thick and extend to depths between 7.0 m and 8.5 m below the ground's surface. The geotechnical characteristics of the subsoil were evaluated using a series of laboratory tests. PVD was employed as a ground improvement technique to support the 56 kPa design load. PVDs penetrated 9 m of soft substrata, having 1 m c/c at the triangular pattern. Their study showed that the final settlement was 415 mm, which was very close to the designed settlement of 450 mm. Islam & Yasin (2013) conducted a case study regarding PVDs combined with preloading at Chittagong Container Terminal. The design load of 77 kPa from the container yard was taken as a 5 m equivalent surcharge load. PVDs penetrated 5 m at 1-1.2 m c/c spacing with a triangular pattern. After applying these ground improvement techniques, the subsoil's SPT increased drastically. Radhakrishnan (2011) presented a case history of PVD and Preloading performance for a major construction project in India. The subsoil condition of the project consisted of 13 m of soft soil underlying medium-dense silty sand. PVD combined with preloading was deployed as a ground improvement method. Three trial embankments with 4 m height were constructed to select optimum PVD spacing from 1.5 m, 1.25 m, and 2.5 m c/c. After observation of 330 days, confirmatory borings were executed. The bearing capacity was increased drastically.

The Observational Method (OM), proposed by the late professor (Peck, 1969) and based on the research of Terzaghi, offers numerous advantages compared to traditional design methods. These benefits include the possibility of time and cost savings, as well as enhanced safety assurance. Within the initially established OM framework, this method can be utilized in two ways: either as an initial approach from the beginning of the project (known as the ab initio approach) or as a contingency plan when unwelcome development or unacceptable incidents arise during construction and need to be addressed (referred to as the “best way out” approach). Although geotechnical professionals are familiar to Nicholson et al., 1999 and Wu, (2011) with the applications and benefits of the observational method (OM), its use is not yet common in the civil engineering profession. Only recently has its adoption in design codes started to occur (Hardy et al., 2018; Spross and Johansson, 2017).

Various techniques exist for determining the final settlement and degree of consolidation. Asaoka's observational technique (Asaoka, 1978) and the hyperbolic curve method (Thiam-Soon et al., 1991) are the most often used methodologies that have successfully anticipated the final settlement. In previous studies, both these methods had been used to predict ultimate settlement (Bo et al., 2007; Duy Nguyen & Pham, 2012; Poon et al., 2020).

This paper deals with a case study of 100 Ha yards over very soft clays for jetty 4&5 at MPA, improved with PVD. Both the Asaoka method and the Hyperbolic approach were employed in this project to calculate the eventual settlement.

2. PROJECT DESCRIPTION AND LOCATION

Mongla port is the second-largest seaport in the country, following the Chattogram port in Bangladesh. At present, the MPA (Mongla Port Authority) is set up with a total of five jetties. However, the implementation of the Public-Private Partnership (PPP) initiative will result in the addition of two more jetties. This expansion is expected to enhance the port's overall capacity significantly. The Mongla port exhibits substantial potential; yet it is noteworthy that around 90 percent of the containers are directed towards Dhaka. The opening of the Padma Bridge will result in a substantial reduction in the distance between Dhaka and Mongla. Saif Powertec is now engaged on the construction of a versatile jetty facility at the Mongla port, with the objective of accommodating vessels of both container and cargo types. The jetty has the capacity to accommodate about ten million metric tonnes of break-bulk cargoes and 350,000 twenty-foot equivalent units (TEUs) of containers at its maximum operational capability. The construction of a terminal, measuring 380 metres in length, is underway on a 25-acres plot adjacent to the pier of Mongla port in Figure 1.

3. GEOLOGY OF THE SITE

The geological composition of the project site consists of Paludal Deposits (ppc) and Tidal Deltaic Deposits (dt). From a geological perspective, the site is characterized by the presence of tidal deltaic deposits. These deposits exhibit a range of colours, ranging from light to dark grey, and undergo weathering that results in a yellowish-grey hue. The composition of these deposits primarily consists of silt to clayey silt, with occasional occurrences of extremely fine to fine sand in the form of lenses. These sand lenses are typically found in both active and defunct stream channels, featuring crevasse splays. This site exhibits deposits characterized by the presence of brackish water. The region is characterized by a multitude of tidal creeks that intersect with one another, resulting in extensive areas that become underwater during spring tides (Chakraborty et al., 2017). Additionally, the composition of this site comprises of soft marsh clay, peat-grey or purplish grey clay, black herb peat, and yellowish grey silt. The presence of alternating layers of muck and peaty clay is a prevalent characteristic observed in bogs and significant depressions that are influenced by structural factors. It is worth noting that the thickness of peat tends to be greater in the deeper regions of these formations.

4. SUBSOIL INVESTIGATION

The soil test result has been compiled pursuant to a contractual arrangement between SAIF Port Ltd. and GEOSCAPE, a sub-soil investigation organization located in Dhaka (GCL, 2019). The sub-surface investigation involves conducting forty-five borings (Figure 2) that extend to depths ranging from 15.0 m to 30.0 m. This includes conducting necessary field and laboratory tests, assessing the bearing capacity, and ultimately providing recommendations for a suitable and secure foundation type based on the subsoil conditions. Nevertheless, this research includes nineteen boreholes ranging from 15.0 m to 30.0 m in depth, which are illustrated in Figure 3 to Figure 7. These boreholes were conducted to cover the preloading area. The wash boring technique is employed to drill boreholes subsequent to the installation of the casing pipe. Laboratory studies were conducted to evaluate the soil properties, including grain size analysis, specific gravity, and direct shear tests. Disturbed soil samples are often obtained during the execution of Standard Penetration Tests (SPT). These samples are obtained by disturbing the normal soil structure during the collection process. The samples represent the composition and the mineral content of the soil. The top formation of soil existing roughly to the depth 1.5 m (NBH-6), 4.5 m (ABH-33 and NBH-13), 6 m (ABH-47 and ABH-36), 7.5 m (ABH-46, ABH-32, ABH-34, ABH-40, NBH-10 and NBH-9), 9 m (ABH-43 and ABH-39), and 10.5 m (NBH-16 and ABH-42) is predominated by plastic nature soft silty clay. Further below, a layer of silty sand 4.5 m (NBH-16), 7.5 m (NBH-10), 10.5 m (NBH-13), 13.5 m (NBH-9), 19.5 m (NBH-6 and NBH-42), 21 m (ABH-42 and ABH-39), 22.5 m (ABH-46, ABH-32, ABH-34 and ABH-40), 24 m (ABH-47 and ABH-36), and 25.5 m (ABH-33) measured from the existing ground level of investigated boreholes.

Unconfined compressive test value varies between 11-23 kPa for the underlying soft clay. According to laboratory test results, the specific gravity of the investigated soil ranges from 2.67-2.73. The values of internal friction angle obtained from the performance of the direct shear tests vary from 21°-23°. Several consolidation tests were carried out in a depth of 5 m at different locations of the container yard. Following data have been obtained from one-dimensional consolidation tests: initial void ratio ranges between 0.877 to 0.926, compression index ranges from 0.309 to 0.326 and coefficient of consolidation ranges from 0.7 to 7 m^2/year.

5. DESIGN OF GROUND IMPROVEMENT

One layer has been considered for the design of ground improvement. The thickness of the layer is 10 m (DSL, 2020). The unit weight, initial void ratio, and compression index are 20 kN/m^3, 0.924, and 0.326, respectively. The location of the groundwater table has been considered at the existing ground level. The equivalent surcharge load of the yard area is 67 kPa. The primary settlement can be calculated by Equation 1.

$$S_c = \frac{C_c \times H}{1+e_0} \times \log\left(\frac{\Delta\sigma}{\sigma_0'}\right) \tag{1}$$

The C_v to C_h ratio has been taken as 3 for this subsoil. Without ground improvement, 3.02 years are required to accomplish the 90% consolidation. It is too long period from the viewpoint of container yard operation. In this case, PVD has been employed as ground improvement technique to reduce the long consolidation time. In triangular pattern, 1.5 m c/c PVD improvement will take 62 days for completing the 90% consolidation. The total primary consolidation settlement has been calculated as 629 mm. In these 62 days, without ground improvement, only 188 mm settlement would be occurred. The time required for U_h = 90% consolidation due to radial consolidation has been evaluated by Equation 2.

$$t_{90} = \left(\frac{D_e^2}{8 \times C_h}\right) \times \left(\ln\left(\frac{D_e}{d_w}\right) - 0.75 + (k_{ratio} - 1) \times \ln\left(\frac{d_s}{d_w}\right) + F_r\right) \times \ln\left(\frac{1}{1-U_h}\right) \tag{2}$$

where, F_r is the well resistance factor, k_{ratio} is the horizontal coefficient of permeability to vertical coefficient of permeability ratio. Equation 3 is used to calculate well resistance factor.

$$F_r = \pi \times z \times (L - z) \times \frac{k_h}{q_w} \tag{3}$$

$$F_r = \pi \times 5 \times (10 - 5) \times \frac{4.41 \times 10^{-9}}{50} = 0.218$$

where, z is the distance from the drainage end of the drain, L is the length of the drain when one-way drainage and it will be halved at double drainage, k_h is the horizontal coefficient of permeability, and q_w is the discharge capacity of the drain. The equivalent diameter of the drain, d_w is evaluated by Equation 4.

$$d_w = \frac{2 \times (t+a)}{\pi} \tag{4}$$

$$d_w = \frac{2 \times (3+100)}{\pi} = 66\ mm$$

where, d_w is the equivalent diameter of the drain, t is the thickness of the band-shaped drain cross-section, a is the width of the band-shaped drain cross-section,

$$D_e = 1.05 \times S \tag{5}$$

$$D_e = 1.05 \times 1.5 = 1.58\ m$$

From Equation 5, D_e is the diameter of the equivalent cylinder, and S is equal to 1.05 for the triangular pattern. Equation 6 is used for the calculation of mandrel properties.

$$d_m = \sqrt{\left(\frac{4}{\pi} \times w \times l\right)} \tag{6}$$

$$d_m = \sqrt{\left(\frac{4}{\pi} \times 45 \times 150\right)} = 93\ mm$$

where, w and l are the dimension of mandrel.

6. CONSTRUCTION SEQUENCES

The whole project area at the site has been levelled by cutting and filling the earth. A geotextile has been laid all along the area. Over the geotextile, a 500 mm layer of Sylhet sand has been placed. The PVDs have been driven at 1.5 m c/c spacing at a triangular pattern (Figure 8). A total of 445000 running meter PVD at 15000 points have been driven. After that, the designed depth of the PVDs was driven by using a rig-machine. Above the existing

Figure 1 Project location

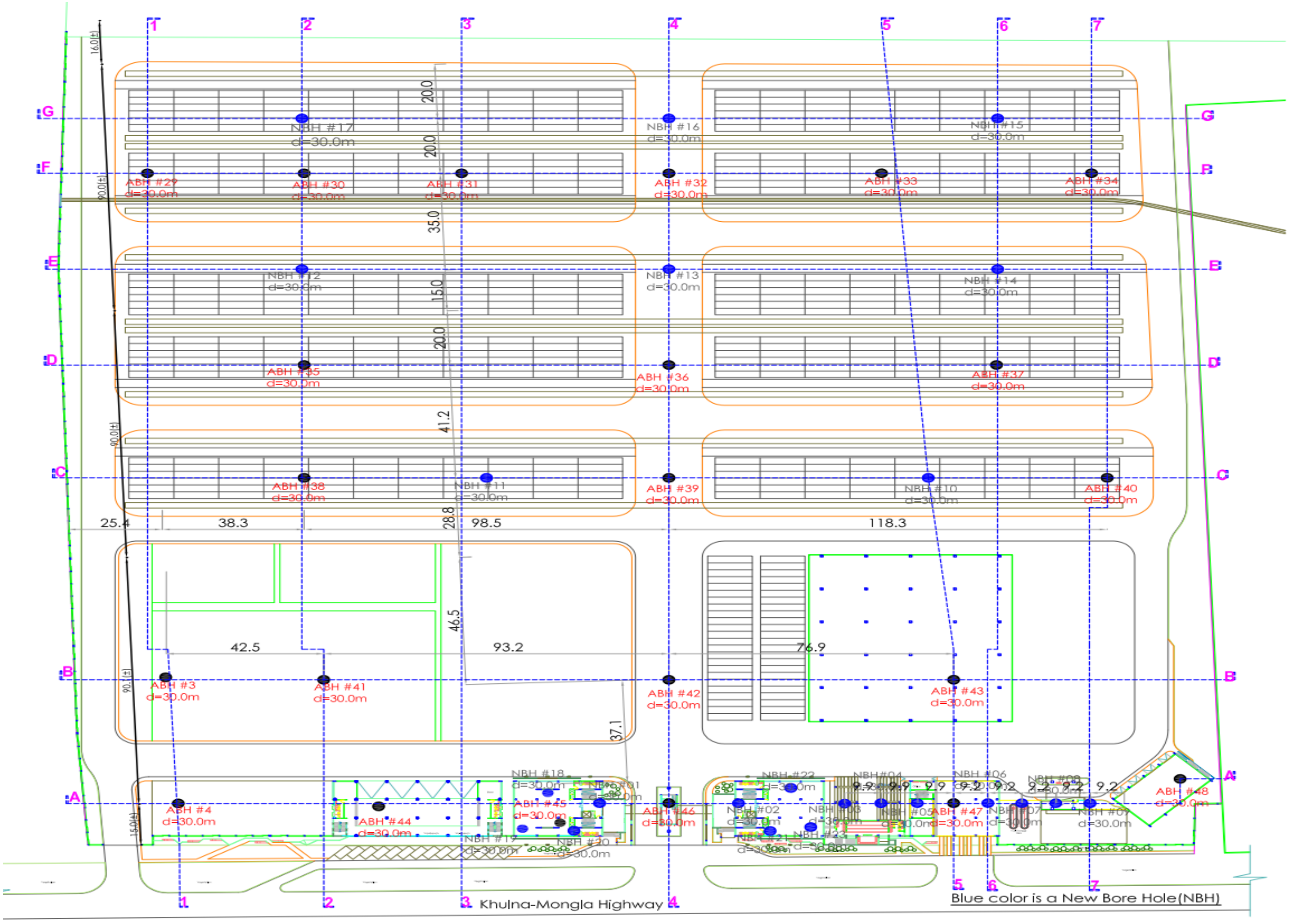


Figure 2 Layout of Jetty 3 & 4

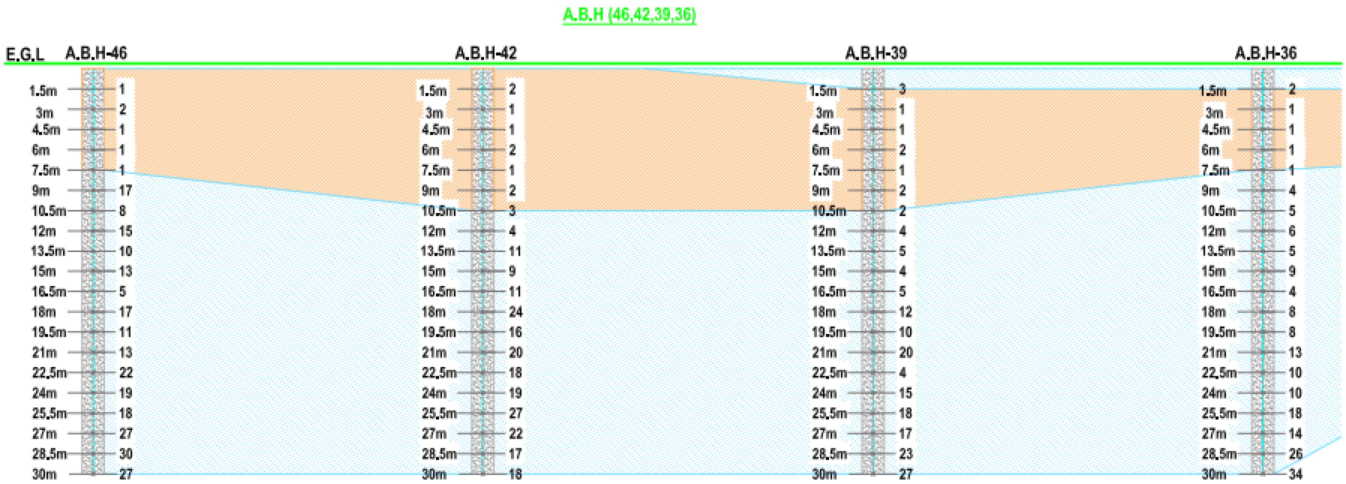


Figure 3 Subsoil profile at section 4-4 (Part-1)

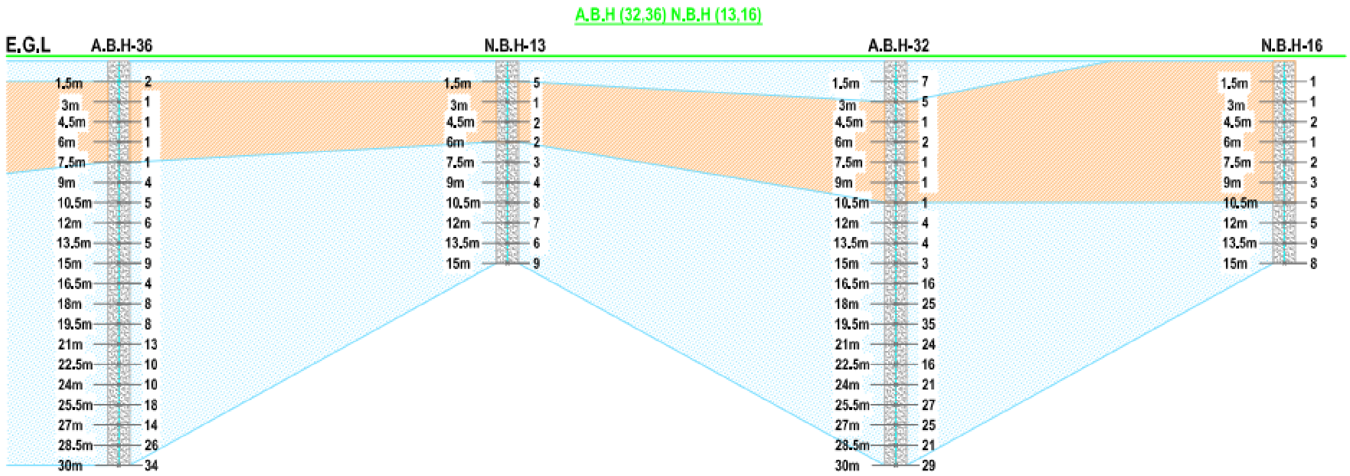


Figure 4 Subsoil profile at section 4-4 (Part-2)

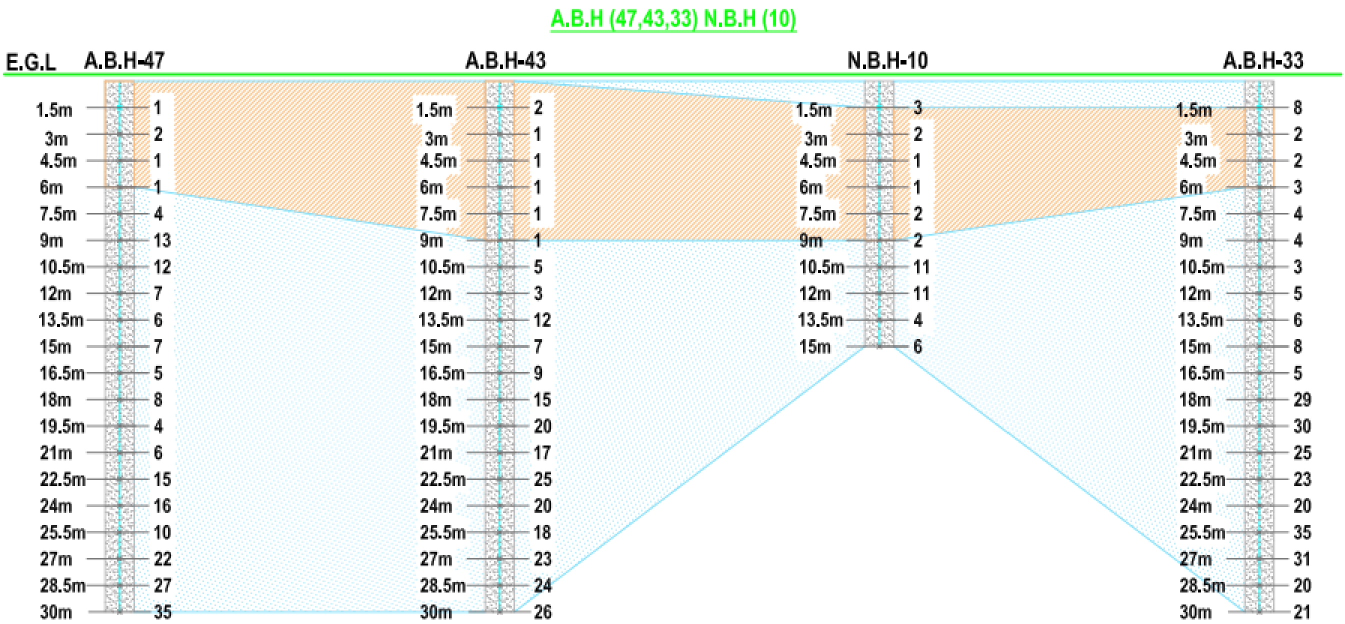


Figure 5 Subsoil profile at section 5-5

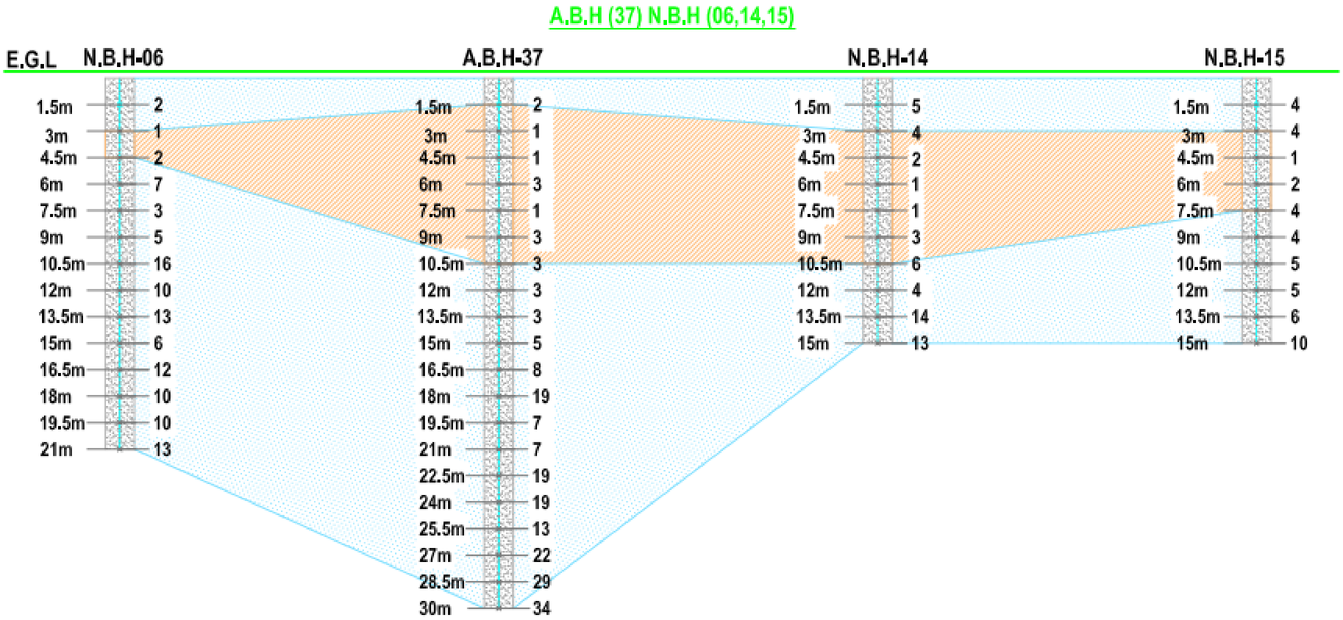


Figure 6 Subsoil profile at section 6-6

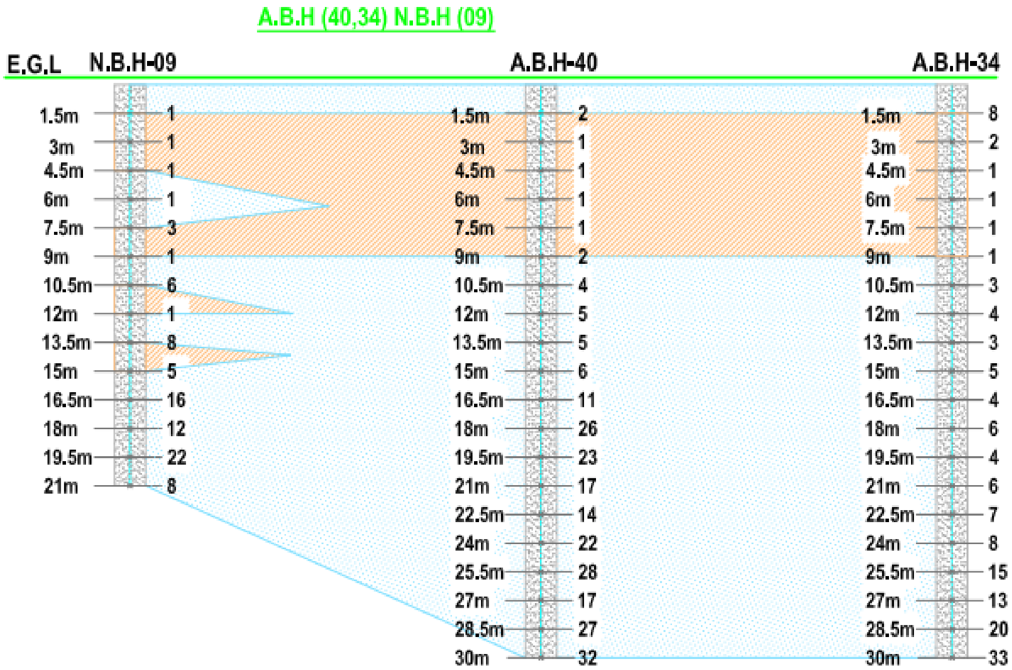


Figure 7 Subsoil profile at section 7-7

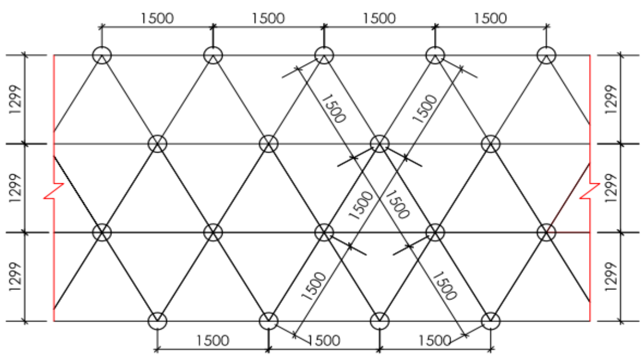


Figure 8 PVD layout at 1.5 m c/c triangular spacing

ground level, 450 mm PVD have been cut after driving at each point. Placement of another layer of geotextiles throughout the area to make a sand blanket sandwich formation. A piezometer has been installed at the locations by boring at the point. The surface settlement plate

Figure 9 PVD cutting at the design level

has been installed at the EGL. A surcharge load has been applied by dredging sand from the Pussur river. To reach the design surcharge height, 45 days were taken. The surcharged load remained until the consolidation reached 90%. All the sequential construction stages have been drawn in Figure 9 to Figure 13.

Figure 10 PVD driving at a triangular pattern

Figure 11 At the final height of the preloading level

Figure 12 Taking the settlement plate reading

Figure 13 Removing of preloading materials

7. SETTLEMENT MONITORING

To verify the design assumption, a total of 10 settlement plates have been placed on the ground surface. With respect to the trial benchmark's level, every day the level of settlement plates has been taken (CRTS, 2021). The levelling data have been collected from the field about 120 days. All the collected data have been plotted in computer for the generation of time-settlement graphs. The graphs have been depicted in Figure 13.

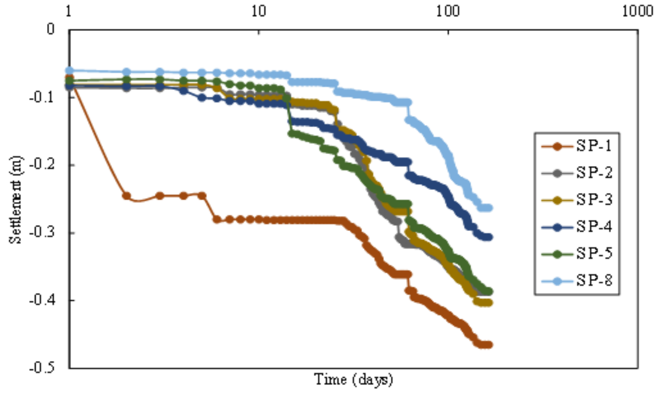
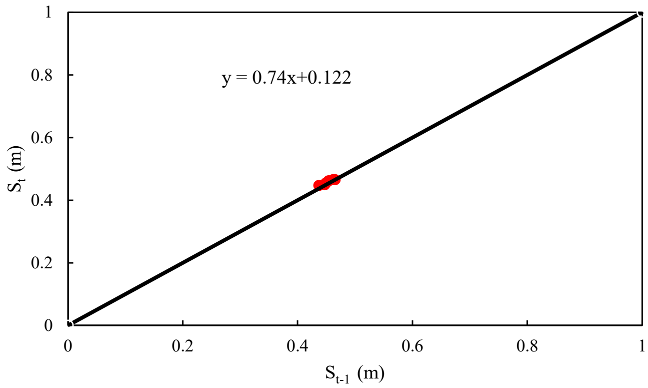
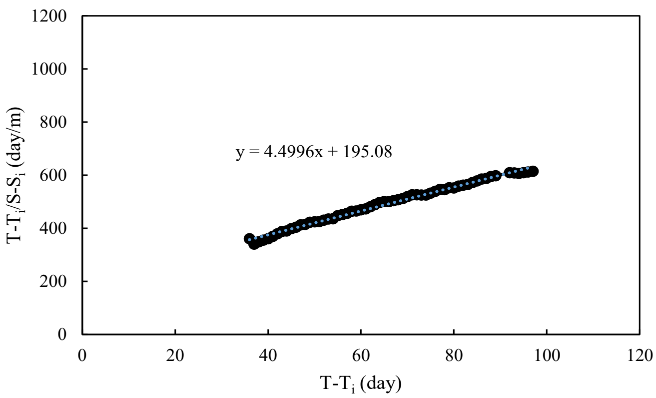


Figure 13 Variation of settlement over time of various settlement plates

To evaluate the degree of consolidation, Asaoka and the hyperbolic method have been used. After reaching the final height of the preloading, these methods have been applied. Figure 15 to Figure 20 depict the Asaoka and the Hyperbolic plot of each settlement plates. The prediction of ultimate settlement based on the monitoring data by two methods have been tabulated in Table 1.

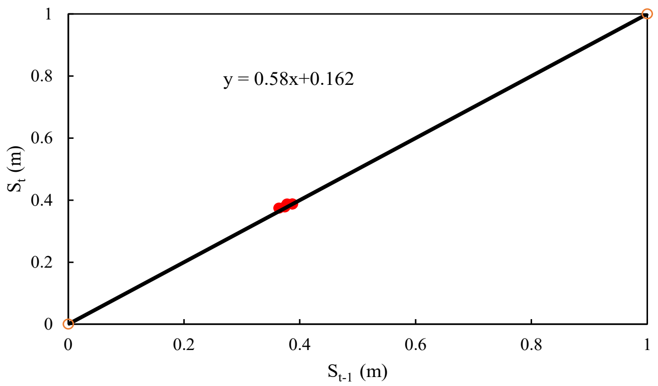


(i) SP-1

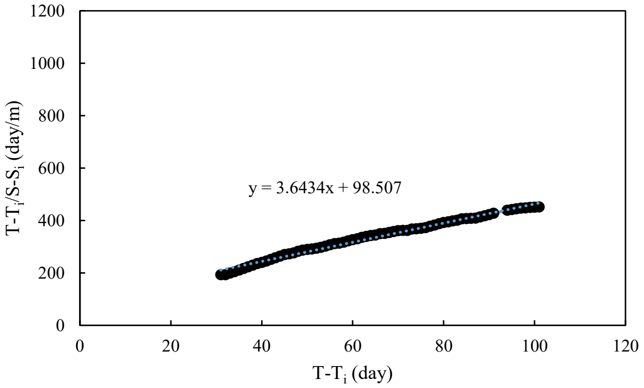


(ii) SP-1

Figure 14 Ultimate settlement prediction at SP-1 (i) Asaoka method and (ii) Hyperbolic method

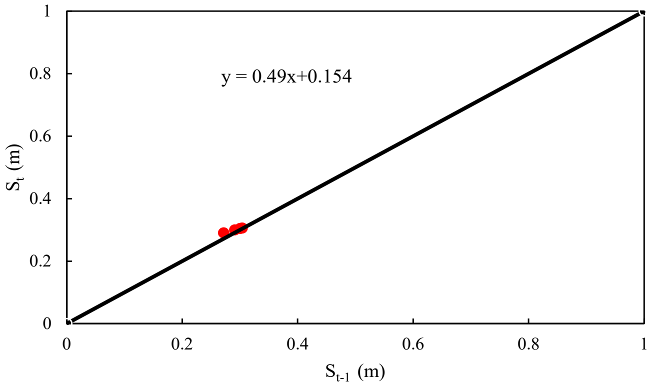


(i) SP-2

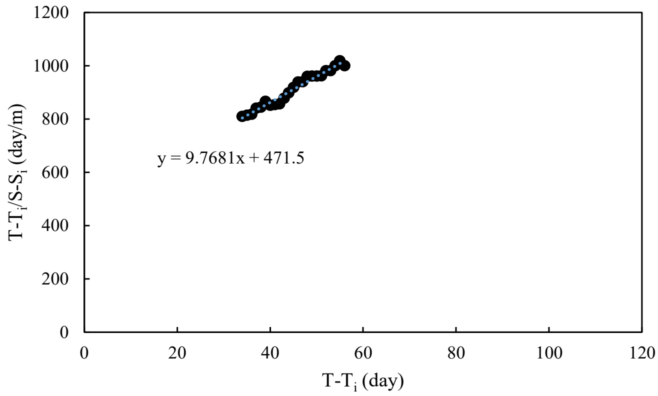


(ii) SP-2

Figure 15 Ultimate settlement prediction at SP-2 (i) Asaoka method and (ii) Hyperbolic method

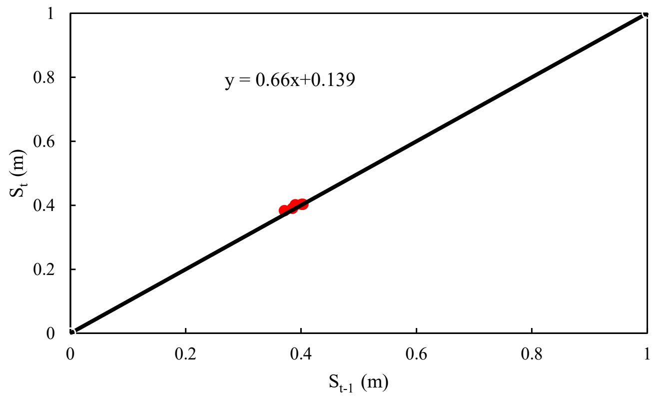


(i) SP-4

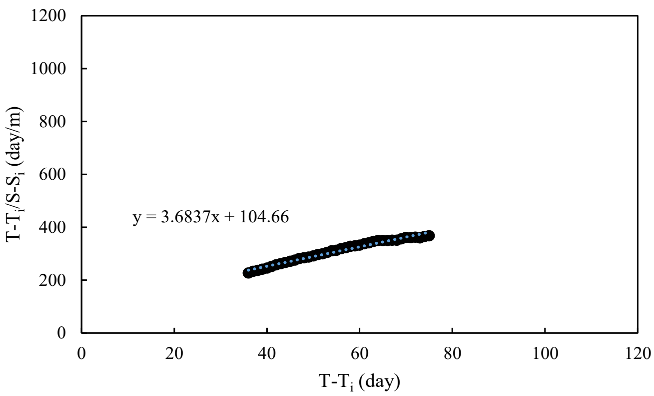


(ii) SP-4

Figure 17 Ultimate settlement prediction at SP-5 (i) Asaoka method and (ii) Hyperbolic method

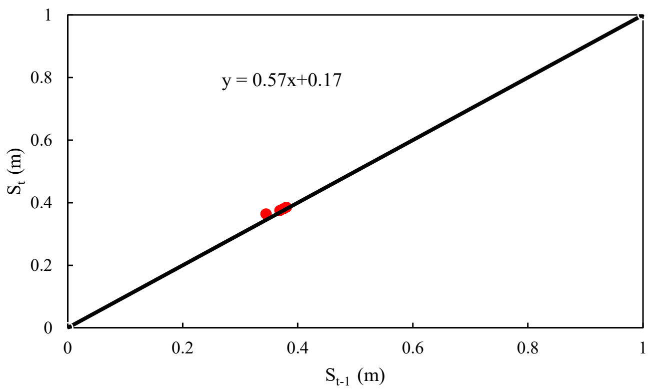


(i) SP-3

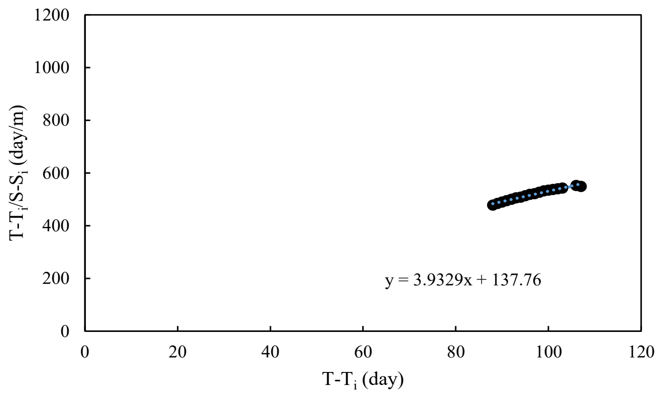


(ii) SP-3

Figure 16 Ultimate settlement prediction at SP-3 (i) Asaoka method and (ii) Hyperbolic method

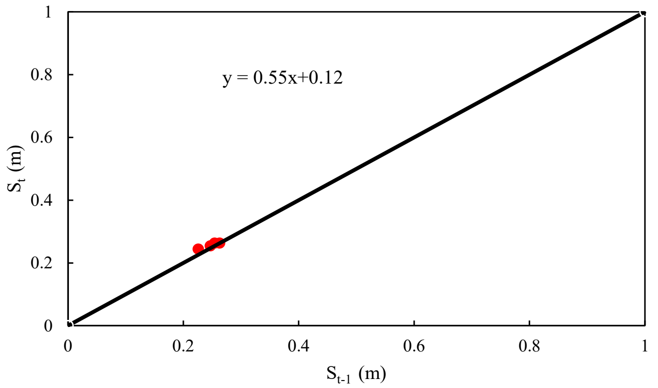


(i) SP-5

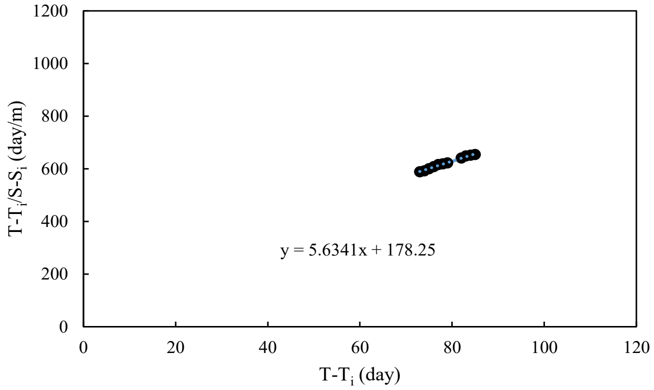


(ii) SP-5

Figure 18 Ultimate settlement prediction at SP-5 (i) Asaoka method and (ii) Hyperbolic method



(i) SP-8



(ii) SP-8

Figure 19 Ultimate settlement prediction at SP-8 (i) Asaoka method and (ii) Hyperbolic method

Table 1 Prediction of ultimate settlement using Asaoka and Hyperbolic methods

	ASAOKA METHOD			HYPERBOLIC METHOD			
Plate ID	m	S_{ult} (mm)	U%	b	S_{ult} (mm)	U%	S_t (mm)
SP-1	0.740	470	98.9	4.50	508	91.5	465
SP-2	0.583	389	99.5	3.64	424	91.3	387
SP-3	0.658	421	95.7	3.68	454	88.8	403
SP-4	0.499	351	87.2	9.76	335	91.3	306
SP-5	0.570	416	92.8	3.93	399	96.7	386
SP-8	0.554	267	98.5	5.63	275	95.6	263

From Table 1, it is clear that: the field settlements do not exceed the design settlement. Hence, these outcomes confirm the validity of laboratory tests and the design assumptions. When the differences between 3-6% of the degree of consolidation have been observed (Duy Nguyen & Pham, 2012), then the coefficient of the radial consolidation has been calculated by the following Equations:

The coefficient of radial consolidation C_h is evaluated as Equation 7.

$$\frac{\ln(\beta_1)}{\Delta t} = \frac{8C_h}{\mu\, D_e^2} \tag{7}$$

where, Δt is the time increment and β_1 is the slope of the fitted line with 45° at the Asaoka plot.

$$\mu = \frac{n^2-1}{n^2}\ln(n) - \frac{3n^2-1}{4n^2} \tag{8}$$

where, drain spacing ratio

$$n = D_e/d \tag{9}$$

where, d is the equivalent diameter of the of drain and

$$D_e = 1.05 \times \text{PVD spacing} \tag{10}$$

Figure 20 Variation of C_h with different settlement locations

During the initial design stage, the C_h value adopted as 21 m^2/year. Asaoka method yields mobilized value of C_h equal to 6-12 m^2/year as depicted in histogram shown in Figure 20. The range of C_h values are well below the design value. This evidence indicates the creation of smear zone around the PVD and took longer time to reach the primary consolidation. Similar findings have been reported by Stark et al. (2018).

8. CONCLUSIONS

The application of preloading combined with PVD has been presented and discussed in this paper. Regarding the time consumption aspect and values of required work, PVD is the preferable technique. In this yard, PVD has been installed in a triangular pattern at 1.5 m c/c spacing. Due to installation, the time of consolidation has been reduced. The field settlement was measured to be found within the design limit. However, the observation time was too long than the design time. There could be the following reason: the creation of a smear zone, not having exact data for the geotechnical investigation data. During the yard construction, negligible settlements have been observed and no maintenance have been required. The project provided invaluable knowledge to the neighbourhood experts and contractors, contrasting with the established standard processes.

9. ACKNOWLEDGMENTS

The authors express their gratitude to Saif Powertec for granting them permission to utilise the field monitoring and other pertinent data of the ground development project.

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