TECHNICAL PAPER

A Field Investigation to Improve Soft Soils Using Prefabricated Vertical Drain

Ripon Hore [1] · Sudipta Chakraborty [2] · Mehedi A. Ansary [1]

Abstract

This paper presents a case history of soft ground improvement technique using prefabricated vertical drain (PVD) with application to a railway track in Bangladesh. The method with preloading and staged construction was used for the construction of a 32.4-km-long new railway track at Kashiani–Gopalganj section. The soil profile at the site consisted of a 5m to 16m thick silty clay layer having water content between 20 and 24%. The shear strength of soft soil varied between 12 kPa and 38 kPa. The estimated consolidation of soil was between 1 and more than 5 years. Soil treated with PVD having 1.0 and 1.5 m spacing of triangular grid resulted in accelerated settlements as observed from the measured data. Through field monitoring, 524 to 731 mm settlement was observed. The observed time at the field was long compared to the estimated time due to three main reasons such as creation of a smear zone, not applying the surcharge load, and not having the exact soil data for settlement estimation.

Keywords Consolidation · Prefabricated vertical drain · Soil improvement · Railway embankment

✉ Mehedi A. Ansary
  ansary@ce.buet.ac.bd

  Ripon Hore
  riponhore@gmail.com

  Sudipta Chakraborty
  sudipta.ckr@gmail.com

[1] Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh

[2] Bangladesh Network Office for Urban Safety, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh

1 Introduction

Transportation plays a significant role in the progress of any country. In the construction of road, railway, and airway, the alignment may have to be fixed through the soils which may not bear the traffic loads. In most cases, ground improvement is needed. The ground improvement technique using prefabricated vertical drains (PVD) is one of the most suitable methods to overcome this problem. PVDs are typically used as ground improvement system in the construction of road, railway, embankment, airport and ports, industrial projects, and land reclamation projects. The sole purpose of the vertical drain system is to shorten the drainage path of the pore water from a low permeable layer to free water surface or to a pre-installed drainage layer, thereby accelerating the rate of primary consolidation or the process of settlement. Application of ground improvement method using PVDs coupled with surcharge or preloading can significantly shorten the period of primary settlement. PVD is a prefabricated material consisting of a plastic core covered by synthetic geotextile "filter jacket." PVDs are installed in either square or triangular patterns. PVD installation equipment can be developed to suit the soil condition, installation depth, specified scope of work, and required production rate.

The soft Bangkok clay foundations at the site of the Second Bangkok International Airport (SBIA) have been improved using PVD (Bergado and Patawaran 2000). Three full-scale test embankments were constructed in stages on soft Bangkok clay with PVDs installed to 12 m depth in a square pattern. The rate and amount of settlement has been predicted by Asaoka's method proved to be in excellent agreement with the observed values. Performance of an 80-km-long road embankment (Bangkok–Chonburi New Highway) was constructed on improved soft marine clay in Bangkok (Ruenkrairergsa et al. 2001). The site was evaluated during and after construction. PVDs were associated with preloading and found to be effective to accelerate the consolidation settlement on soft Bangkok clay. In general, soft clay thickness has been varied from 6 to 14 m. PVD spacing has been designed to be 1.2 m triangular grid pattern with varied length at each contracted section. Shen et al. (2005) have presented a case history of the performance of two full-scale test embankments constructed on soft clay deposit in the eastern coastal region of China. One embankment is constructed on natural subsoil and the other constructed on PVD-improved subsoil. The PVDs are installed to a depth of 19 m with spacing of 1.5 m in a triangular pattern. Field performance of the two embankments was analyzed using the finite element method. The analytical results show that PVDs have increased the bulk vertical hydraulic conductivity of soft subsoil by about 30 times compared to the original nontreated subsoil.

The vacuum–PVD system was applied for soft Bangkok clay combining capped PVD with vacuum pressure and embankment loading (Saowapakpiboon et al. 2008). The vacuum–PVD was installed into 10 m depth with spacing of 0.85 m and arranged in a triangular pattern. The final settlement of 0.91 to 1.21 m with a degree of consolidation of 66 to 80% has been achieved. The vacuum–PVD system has reduced the time of consolidation by more than 50%. The rail track of the Sandgate Rail Grade Separation Project (sandgate between Maitland and Newcastle, in the Lower Hunter Valley of New South Wales, Australia) was stabilized using short PVDs in the soft subgrade soil (Indraratna et al. 2011). The soil profile at the project site consists of soft

compressible soil (4–30 m), soft residual clay, and shale bedrock. The PVD was installed into 8 m depth with spacing of 2 m and arranged in a triangular pattern. The Ballina Bypass route in Ballina (New South Wales, Australia) was built to reduce local traffic jams (Indraratna et al. 2012). The soil profile at the site consists of very thick clay (more than 40 m), soft silty clay layer (20 m thick, water content 80–120%), and a firm clay layer. To ensure ground stability and improvement, PVDs combined with surcharge and vacuum preloading were utilized to consolidate the soft soils before the construction. A trial embankment was built to the north of Ballina to evaluate the effectiveness of the technique in this area.

The railway embankment along the alignment of the project for the construction of a double line track from Tongi to Bhairab Bazzar (including signaling on Dhaka–Chittagong main line) on soft ground between chainage 29 + 300 and 64 + 265 m was completed recently (BRTC (Bureau of Research, Testing and Consultation), 2013). Depending on the soft ground treatment, 29 sections were identified, among which 7 sections (for 10.86 km) of the embankment were used. The PVD was installed with spacing of 1.8 m c/c and arranged in a square pattern. In these three projects of Thailand—The North-South Expressway (NSEW) project at Dong Nai Province, the Cai Mep International Terminal (CMIT) project at Ba Ria-Vung Tau Province, and the three test embankments (TS1, TS2, and TS3) at Suvarnabhumi Bangkok International Airport (SBIA) project, PVD with or without vacuum consolidation is used for soft soil improvement (Long et al. 2013). In the NSEW project, PVD was installed with spacing of 0.9 m and arranged in a triangular pattern. In the CMIT project, PVD was installed with spacing of 0.9 and 1.2 m arranged in a square pattern. In the SBIA project, PVD was installed into 12 m depth with spacing of 1.5, 1.2, and 1.0 m. Reddy et al. (2013) have proposed finite element modeling (FEM) for a portion of new road at Kakkanad, Thankalam new road, India, where ground improvement (1000 m and road width is expected to be 1.2 m) was done by preloading with PVD. The primary consolidation of soil will continue for a longer period of time, and it was about 2.36 years from preliminary calculations which results in uneven settlement and severe cracking of the pavement surface. These PVDs were placed at a distance of 1.3 m spacing. After preloading, the consolidation time was reduced and the percentage reduction in time after preloading is up to 30%. As the settlement of soil was observed by preloading, hence, the bearing capacity of the soil was improved.

PVD combined with staged construction of embankment was considered as a common technique to reduce both the time for full consolidation and the excess pore water pressure (Al-Soud 2016). A 3D software was used to analyze the embankment and the underlying soft soil at Basra region. The numerical analysis has shown that the PVD is very effective in accelerating the consolidation process and increasing the long-term stability of the embankment besides installing the PVD (1 m spacing) and leads to a rapid dissipation of most of the excess pore water pressure at the end of each stage. The length of the PVD sheets significantly accelerated the consolidation more than the spacing. A case study of quality management of PVD materials in the mega Changi East land reclamation project in the Republic of Singapore was presented in a technical report by Bo et al. (2015). According to this report, a good quality management system in PVD improvement works depends on the good field performance, and quality management of PVD material installation in ground improvement works in land reclamation projects was a critical task for designers, contractors, and the like.

Recently, the Bangladesh Railway (BR) was taken up to a decision to construct a new 32.4-km-long railway track along Kashiani–Gopalgang–Tungipara section. Along this track, the soft soil thickness varies from 5 to 16 m. PVD in triangular pattern was used to obtain quick consolidation settlement due to soft soil. This paper aims to discuss the soil profile of the railway project and the techniques used for soil improvement and presents analysis of the monitored settlement data.

2 Objective and Scope

The objectives of this study were to (a) perform comparative analysis of settlement time without using PVD and with the PVD method; (b) analyze the in situ test (i.e., standard penetration test, SPT) along with laboratory soil investigation to evaluate the soil compositions and parameters; (c) select an optimum design for PVD insertion, including depth of installation, spacing, grid pattern, and total number of PVD installed; (d) execute the PVD installation program effectively; (e) monitor the settlement, after PVD installation; and (f) compare before and after settlement time after PVD installation. The specific purposes of the study were to evaluate the soft soil improvement process using PVD at the project site, analyze the monitored settlement data, and give a statement whether the study was feasible or not.

3 Non-Productive Workcode Project Description and Geology of the Site

3.1 Project Description

Bangladesh Railway had started construction work of a new BG track along Kashiani–Gopalganj section (BRTC, 2016). Construction of the 32.4-km railway embankment of the project site resting on soft soil requiring soil treatment was the major challenge for this project. The latitude and longitude of the site are between 23.20° N and 23° 13′ N and between 89.80° E and 89° 42′ E, respectively. The location and alignment of the site are presented in Figs. 1 and 2, respectively.

3.2 Geology

The geology of the project site comprises of Deltaic silt and Paludal deposits. Geologically, the site comprises of light-gray to gray, fine sandy silt to clayey silt. Fine overbank sediments are deposited by distributaries in flood basins. Moreover, this site consists of gray or bluish-gray clay, black herbaceous peat, and yellowish-gray silt. Alternating beds of peat and peaty clay are common in sills and large structurally controlled depressions; peat is thickest in deeper parts.

4 Geotechnical Subsurface Condition

The subsurface investigation work included execution of 52 borings extending to the depth of 15.0 to 19.5 m, performance of the required field and laboratory tests,
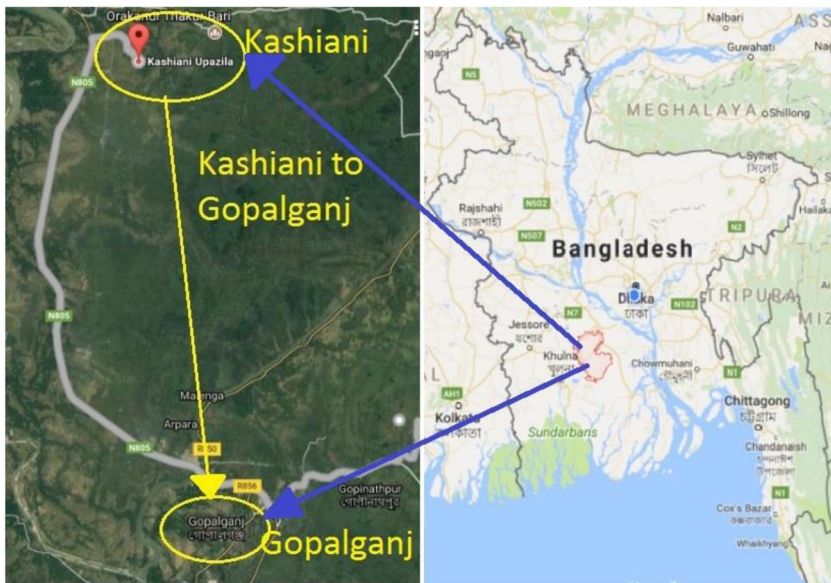
Fig. 1 Project locations on the map

evaluation of the bearing capacity, and finally recommending for the safe and appropriate type of foundation suited to the subsoil conditions. Boreholes were drilled vertically using the wash boring technique. A detailed laboratory investigation carried out on disturbed and undisturbed tube samples collected from the 52 boreholes is described in Table 1. The density and stiffness characteristics of the subsoil layers in the boreholes were measured by performing the SPT. The SPT value with soil composition of all the bore logs along the alignment is presented in Table 2. The thickness of soft compressible layers (silty clay, clayey silt, and fine silty sand) varied from 4.5 to 16.0 m as can be seen from Table 3. Four bore log samples at different depths (15, 16.5, 18, and 19.5 m respectively) along the alignment are shown in Fig. 3. Based on the provided elevations (RL) of existing ground and embankment top, the height of the embankment fill was found to vary from 0.9 to 10.74 m. Considering the variation of fill height and the thickness of the soft compressible layer, total length of the track alignment was divided into 18 sections for the design of PVD spacing and stage construction of embankments.

By comparing the laboratory-measured values of index and swelling properties of the subsoil samples with various recommended criteria outlined, the relative degree of expansion of the subsoil samples was evaluated. Based on the values of liquid limit and plasticity index as recommended by the United States Army Corps of Engineers Waterways Experiment Station (Johnson and Snethen 1978), the degree of expansion of samples, in general, was low to high. Based on the values of linear shrinkage data of Bangladesh soil as recommended by Hossain (1983), the degree of expansion of subsoil samples was low to high. Based on the values of free swell as proposed by IS: 1948 (1970), the degree of expansion of the samples was low. Based on the values of free swell index proposed by IS: 2911 (1980), the degree of expansion of undisturbed samples was low. Based on the
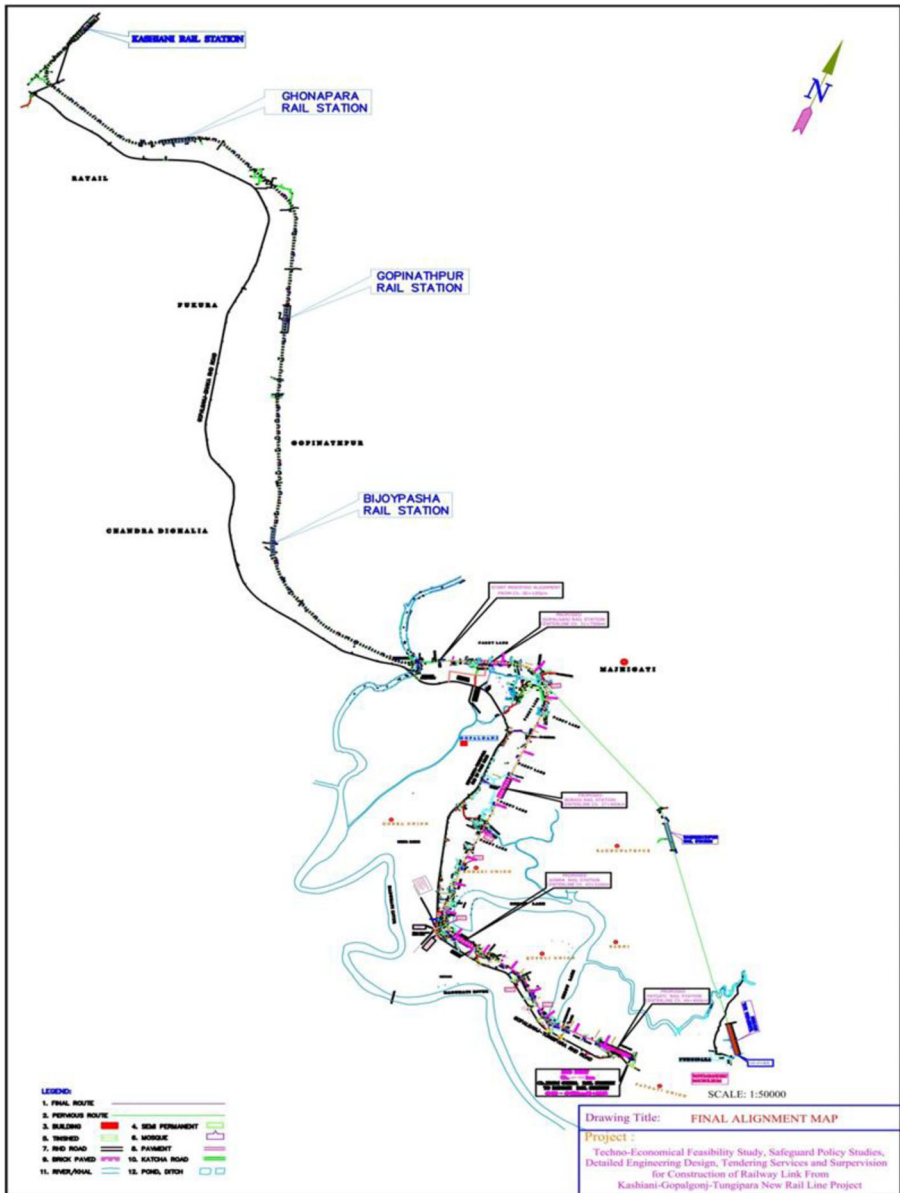
Fig. 2 The alignment of the project section

values of the swelling pressure as proposed by Chen (1975), the degree of expansion of undisturbed samples was low. Based on the values of swelling potential as recommended by Seed et al. (1962), the degree of expansion of the laboratory-compacted samples was medium. Based on the predicted values of swelling potential as recommended by Chen (1975), the degree of expansion of the samples, in general, was low to high. On the basis of the abovementioned findings, it was concluded that the overall degree of expansion of the samples was

Table 1 Detailed laboratory test results of 52 numbers of boreholes

Parameters	Sample no.	Experimental value	Remarks
Liquid limit (LL)	25	~31 and 94	
Plasticity index (PI)	25	8~60	
Shrinkage limit (SL)	12	12 to 20%	
Linear shrinkage (LS)	20	4 to 18%	
Specific gravity (G_s)	20	~2.65 and 2.76	
Organic matter content	4	~3.5 and 42.7%	
Undrained shear strength (q_u)	3	~12 and 38 kN/m^2	Very soft in consistency
Axial strain at failure (ε_f)		~2 and 6%	
Undrained shear strength (q_u)	3	~45 and 99 kN/m^2	Soft to firm in consistency
Axial strain at failure (ε_f)		~5 and 15%	
Undrained shear strength (q_u)	2	129 and 159 kN/m^2	Firm in consistency
Axial strain at failure (ε_f)		10 and 13%	
Dry density (γ_d)	6	6.64~15.41 kN/m^3	
Effective cohesion (c')	6	0.0~16 kN/m^2	
Internal friction (ϕ')	6	17.2~43.7°	
Moisture content (m/c %)	20	20.7~24.0%	
Maximum dry density (γ_{dmax})	20	14.32~15.0 kN/m^3	
Swelling potential (S_p)	25	2.0~4.0%	
Swelling pressure (P_s)	5	3.4~14.5 kPa	One samples was non-swelling type
Compression index (C_c)	12	~0.1 and 0.67	Nine samples were around 0.3 and above
Swell index (C_s)	12	~0.03 and 0.14	
Initial void ratio (e_0)	12	~0.76 and 1.69	
Coefficient of consolidation for vertical flow (C_v)	4	~1.3 and 30.0 m^2/year	
Coefficient of consolidation for horizontal flow (C_h)	8	~1.0 and 33.4 m^2/year	
Coefficient of vertical permeability (k_v)	4	5.6×10^{-11}~8.3×10^{-9} m/s	
Coefficient of horizontal permeability (k_h)	8	5.3×10^{-11}~9.5×10^{-9} m/s	

low to high. Because of the presence of subsoil of high expansion potential at different locations, modification of the subsoil was essential to reduce the degree of expansiveness. The upper soil layer up to a depth of about 1 m needs to be stabilized before placement of embankment fill. Although soil stabilization was done to achieve various objectives, the purpose of soil stabilization here was to reduce the shrinkage/expansion potential. Mixing of lime about 4 to 5% by weight of dry in situ soil may be adopted. Powdered quick lime (CaO) was spread over an area and then mixed with the soil by harrows. However, this requires procurement of a huge amount of lime. Alternatively, granular material, such as fine sand or

Table 2 SPT value with soil compositions of 52 numbers of boreholes

No.	Ch. (m)	Soil type along depth (m)	SPT
01.	0 + 800	Gray soft silty clay trace fine sand medium plastic (1.5 m)	4
		Light brown to gray medium stiff clayey silt with fine sand medium compress (1.5–7.5 m)	4–7
		Gray medium dense to very dense silty fine sand trace mica (7.5–15 m)	25–50
02.	1 + 800	Light brown to gray medium stiff silty clay trace fine sand high plastic (3 m)	3–5
		Gray medium dense silty fine sand trace mica (3–4.5 m)	14
		Gray very loose fine sand with clay (4.5–7.5 m)	2–3
		Gray medium dense to very dense silty fine sand trace mica (7.5–15 m)	12–50
03.	2 + 550	Light brown medium stiff silty clay trace fine sand high plastic (2.5 m)	5
		Gray loose silty fine sand trace mica (2.5–6 m)	3–9
		Gray medium dense to very dense silty fine sand trace mica (6–15 m)	12–42
04.	3 + 300	Light brown soft silty clay trace fine sand high plastic (1.5 m)	3
		Gray loose fine sand with clay (1.5–3 m)	8
		Gray very soft silty clay trace fine medium plastic (3–7.5 m)	1
		Gray dense silty fine sand trace mica (7.5–9 m)	43
		Gray medium dense silty fine sand trace mica (9–12 m)	11–14
		Gray dense to very dense silty fine sand trace mica (12–15 m)	49–50
05.	3 + 800	Light brown medium stiff silty clay trace fine sand high plastic (1.5 m)	5
		Gray very soft silty clay trace fine sand medium plastic (3–7.5 m)	1
		Gray loose fine sand with clay (7.5–9 m)	5
		Gray medium dense to very dense silty fine sand trace mica (9–15 m)	13–50
06.	4 + 300	Gray soft to medium stiff silty clay trace fine sand medium plastic (4.5 m)	2–5
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	14–48
07.	5 + 050	Gray very soft silty clay trace fine sand medium plastic (4.5 m)	1
		Gray loose fine sand with clay (4.5–7.5 m)	6–9
		Gray medium dense silty fine sand trace mica (7.5–19.5 m)	14–27
08.	5 + 300	Gray very soft to soft silty clay trace fine sand high plastic (10.5 m)	1–2
		Gray medium dense silty fine sand trace media (10–18 m)	15–29
09.	5 + 800	Light brown very soft silty clay trace fine sand high plastic (1.5 m)	1
		Dark gray very soft silty clay with organic matter (1.5–3 m)	1
		Gray very soft to soft clayey silt with fine sand medium low compress (3–10.5)	1–6
		Gray dense silty fine sand trace mica (10.5–15 m)	34–37
10.	6 + 300	Gray soft to medium stiff silty clay trace fine sand medium plastic (3 m)	2–5
		Gray loose fine sand with clay (3–10.5 m)	5–9
		Gray medium dense to dense silty fine sand trace mica (10.5–15 m)	20–31
11.	6 + 550	Light brown soft to medium stiff silty clay trace fine sand high plastic (1.5 m)	2–6
		Gray soft silty clay trace fine sand medium plastic (3–6 m)	2–3
		Gray loose fine sand with clay (6–10.5 m)	7–9
		Gray dense to very dense silty fine sand trace mica (10.5–15 m)	46–50
12.	6 + 800	Light brown soft to medium stiff silty clay trace fine sand high plastic (3 m)	3–7
		Gray loose fine sand with clay (3–4.5 m)	10
		Gray medium dense to dense silty fine sand trace mica (4.5–15)	7–47
13.	7 + 800	Light brown soft silty clay trace fine sand medium plastic (1.5 m)	3
		Gray medium dense to dense silty fine sand trace mica (1.5–15 m)	10–32
14.	8 + 300	Light brown to gray soft silty clay trace fine sand medium plastic (4.5 m)	2–3

Table 2 (continued)

No.	Ch. (m)	Soil type along depth (m)	SPT
		Gray medium dense silty fine sand trace mica (4.5–6 m)	15
		Gray loose silty fine sand trace mica (6–9 m)	7–8
		Gray medium dense to dense silty fine sand trace mica (9–15)	13–30
15.	8 + 550	Light brown soft silty clay trace fine sand medium plastic (1.5 m)	2
		Gray soft to very soft silty clay trace fine sand medium plastic (1.5–7.5 m)	1–4
		Gray loose silty fine sand trace mica (7.5–10.5 m)	5–8
		Gray medium dense to dense silty fine sand trace mica (10.5–15 m)	13–48
16.	8 + 800	Gray soft silty clay trace fine sand medium plastic (1.5 m)	2
		Gray loose silty fine sand trace mica (1.5–4.5 m)	8–10
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	15–37
17.	9 + 800	Gray very loose silty fine sand trace mica (2.5 m)	3
		Light brown loose sandy silt (2.5–3.5 m)	5
		Gray medium dense to dense silty fine sand trace mica (3.5–9 m)	13–45
		Light brown medium dense sandy silt (9–10.5 m)	36
		Gray dense silty fine sand trace mica (10.5–15 m)	38–44
18.	10 + 800	Light brown medium stiff clayey silt with fine sand medium compress (1.5 m)	6
		Gray medium stiff silty clay trace fine sand high plastic (1.5–3 m)	7
		Gray stiff clayey silt with fine sand medium compress (3–4.5 m)	9
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	15–31
19.	11 + 550	Light brown soft silty clay trace fine sand high plastic (1.5 m)	3
		Gray soft to medium stiff silty clay trace fine sand high plastic (1.5–6 m)	2–6
		Gray medium dense to very dense silty fine sand trace mica (6–15)	10–45
20.	11 + 800	Light brown to gray soft to medium stiff silty clay trace fine sand high plastic (3 m)	3–5
		Gray medium dense to dense silty fine sand trace mica (3-15 m)	10–47
21.	12 + 800	Gray soft silty clay trace fine sand medium plastic (1.5 m)	2
		Gray loose fine sand with clay (1.5–3 m)	5
		Gray medium dense to dense silty fine sand trace mica (3–15 m)	22–39
22.	13 + 300	Gray soft silty clay trace fine sand medium plastic (1 m)	18
		Gray medium dense to dense silty fine sand trace mica (1–15 m)	17–42
23.	13 + 800	Gray soft silty clay trace fine sand medium plastic (1 m)	2
		Gray loose fine sand with clay (1–3.5 m)	4–5
		Gray medium dense to dense silty fine sand trace mica (3.5–15 m)	15–40
24.	14 + 050	Gray soft silty clay trace fine sand medium plastic and gray loose silty fine sand trace mica (2 m)	10
		Gray very loose fine sand with clay (2–4.5 m)	2–3
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	15–34
25.	14 + 300	Gray loose silty clay trace fine sand medium plastic (1 m)	3
		Gray loose silty fine sand trace mica (1–6 m)	8–9
		Gray medium dense to dense silty fine sand trace mica (6–15 m)	13–42
26.	15 + 300	Light brown soft silty clay trace fine sand medium plastic (1.5 m)	3
		Gray medium dense silty fine sand trace mica (1.5–4.5 m)	11–17
		Gray soft to very soft silty clay trace fine sand (4.5–10.5 m)	1
		Gray very dense silty fine sand trace mica, dark gray medium dense silty fine sand with mica, and gray dense to very dense silty fine sand (10.5–18 m)	10–50
27.	15 + 800	Gray soft clayey silt with fine sand medium compress (1.5 m)	4
		Gray soft silty clay trace fine sand medium plastic (1.5–4.5 m)	2

Table 2 (continued)

No.	Ch. (m)	Soil type along depth (m)	SPT
		Gray loose silty fine sand trace mica (4.5–7.5 m)	6–7
		Gray medium dense to dense silty fine sand trace mica (7.5–15 m)	13–36
28.	16 + 800	Black soft organic clay (2.55 m)	2
		Gray loose to medium dense silty fine sand trace mica (2.55–15 m)	9–26
29.	17 + 300	Gray soft clay silty clay trace fine sand medium plastic (1.5 m)	2
		Black soft organic clay (1.5–3 m)	2
		Gray soft silty clay trace fine sand medium plastic (3–4.5 m)	2
		Gray medium dense silty fine sand trace mica (4.5–15 m)	11–28
30.	17 + 800	Gray soft silty clay trace fine sand high plastic (1.5 m)	3
		Gray medium dense to dense silty fine sand trace mica (1.5–15 m)	12–27
31.	18 + 300	Black very soft organic clay (1.5 m)	1
		Gray soft silty clay trace fine sand medium plastic (3–4.5 m)	3–4
		Gray loose silty fine sand trace mica (4.5–6 m)	9
		Gray medium dense to dense silty fine sand trace mica (6–15 m)	16–35
32.	19 + 300	Black very soft organic clay (1.5 m)	1
		Gray soft silty clay trace fine sand medium plastic (1.5–3 m)	2
		Gray loose fine sand with clay (3–4.5 m)	5
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	10–37
33.	20 + 050	Gray very soft to soft silty clay trace fine sand medium plastic (6 m)	1–3
		Gray soft clayey silt with fine sand medium compress (6–7.5 m)	4
		Gray medium dense to dense silty fine sand trace mica (7.5–15 m)	15–30
34.	20 + 800	Black very soft organic clay (1.5 m)	1
		Gray very soft silty clay trace fine sand medium plastic (1.5–3 m)	1
		Gray loose fine sand with clay (3–4.5 m)	3
		Gray soft silty clay trace fine sand medium plastic (4.5–6 m)	2
		Gray medium dense silty fine sand trace mica (6–15 m)	10–28
35.	21 + 300	Gray very soft silty clay trace fine sand high plastic (4.5 m)	1–2
		Gray medium stiff silty clay with fine sand (4.5–7.5 m)	7–8
		Gray soft silty clay with fine sand medium plastic (7.5–12 m)	3–4
		Gray loose to dense silty fine sand trace mica (12–18 m)	9–31
36.	22 + 300	Gray medium stiff clayey silt with fine sand medium compress (1.5)	6
		Gray loose silty fine sand trace mica (1.5–4.5 m)	6–10
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	20–42
37.	23 + 300	Gray medium stiff clayey silt with fine sand medium compress (1.5 m)	6
		Light brown to gray loose silty fine sand trace mica (1.5–4.5 m)	8–10
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	12–30
38.	23 + 800	Gray soft silty clay with fine sand medium plastic (2.5 m)	3
		Gray medium dense to dense silty fine sand trace mica (2.5–15 m)	9–32
39.	24 + 300	Gray soft silty clay trace fine sand high plastic (4.5 m)	2–4
		Gray soft silty clay with fine sand medium plastic (4.5–9 m)	2–3
		Gray medium dense to dense silty fine sand trace mica (9–15 m)	13–33
40.	24 + 800	Light brown to gray soft silty clay trace fine sand high plastic (9 m)	3–4
		Gray medium dense to dense silty fine sand trace mica (9–15 m)	13–32
41.	25 + 800	Gray very soft silty clay trace fine sand medium plastic (6 m)	1–2
		Gray loose medium dense silty fine sand trace mica (9–15 m)	8–28
42.	26 + 300	Gray soft silty clay trace fine sand medium plastic (1.5 m)	3

Table 2 (continued)

No.	Ch. (m)	Soil type along depth (m)	SPT
		Gray soft clayey silt with fine sand medium compress (1.5–3 m)	3
		Gray medium dense to dense silty fine sand trace mica (3–15 m)	13–39
43.	26 + 550	Gray medium stiff silty clay trace fine sand medium plastic (1.5 m)	8
		Gray soft clayey silt with fine sand medium compress (1.5–6 m)	2
		Gray medium dense to dense silty fine sand trace mica (6–15 m)	14–42
44.	27 + 300	Gray medium stiff clayey silt with fine sand (1.5 m)	7
		Gray loose silty fine sand trace mica (1.5–3 m)	9
		Gray medium dense to dense silty fine sand trace mica (3–15 m)	12–44
45.	27 + 800	Gray medium stiff silt with fine sand (3 m)	8
		Dark gray loose fine sand with clay and organic matter (3–6 m)	6
		Gray dense to very dense silty fine sand trace mica (6–15 m)	33–50
46.	28 + 800	Gray soft silty clay trace fine sand high plastic (4.5 m)	2–3
		Gray medium dense to dense silty fine sand trace mica (4.5–15 m)	14–34
47.	29 + 300	Light brown soft silty clay trace fine sand high plastic (1.5 m)	3
		Gray soft to medium stiff silty clay trace fine sand medium plastic (1.5–7.5 m)	2–6
		Gray loose to medium dense silty fine sand trace mica (7.5–13.5 m)	7–20
		Gray silty clay trace fine sand medium plastic (13.5–15 m)	6
		Gray medium dense to dense silty fine sand trace mica (15–19.5 m)	17–34
48.	29 + 670	Light brown soft silty clay trace fine sand high plastic (1.5 m)	3
		Gray very soft to soft silty clay with fine sand medium plastic (1.5–13.5 m)	1–3
		Gray medium dense silty fine sand trace mica (13.5–18 m)	14–28
49.	30 + 300	Silty clay trace fine sand (0.45 m)	
		Gray loose silty fine sand trace mica (0.45–9 m)	6–9
		Gray medium dense silty fine sand trace mica (9–15 m)	12–26
50.	30 + 800	Gray medium stiff clayey silt with fine sand medium compress (3 m)	6–7
		Gray soft clayey silt with fine sand low compress (3–6 m)	3
		Gray loose to medium dense silty fine sand trace mica (6–15 m)	7–23
51.	31 + 800	Gray very soft to medium stiff clay with fine sand medium plastic (10.5 m)	1–6
		Gray medium dense to dense silty fine sand trace mica (10.5–15 m)	23–31
52.	32 + 050	Gray very soft to soft clayey silt with fine sand low compress (12 m)	1–4
		Gray medium dense silty fine sand trace mica (12–16.5 m)	20–33

dredge sand (50 to 60% of sand by weight of in situ dry soil), may be mixed with the existing in situ top soil. After mixing, the loosened material have to be leveled and compacted over which embankment fill can be constructed in compacted layers. In order to assess the appropriate proportion of sand to be mixed, a sample with relatively high linear shrinkage value was mixed with fine sand (% finer no. 200 sieve = 4.9; fineness modulus, F.M. = 1.17) in proportions of 20, 40, and 60% of dry weight of in situ soil, and linear shrinkage tests were conducted on the three sand mixed samples. It was found that due to mixing of 60% of sand, the linear shrinkage was reduced from 18 to 12%, i.e., a reduction of 33%. Therefore, it was recommended that the top 1 m of subsoil should be mixed with at least 60% by weight of fine sand (% finer no. 200 sieve ≤ 5, F.M. ≥ 1).

Table 3 Thickness of the soft compressible layer and final height of embankment and the settlements (mm) for different segments of the embankment

Segment	Chainage (km)	Thickness of the soft compressible layer (m)	Final height of embankment (m)	Height including surcharge load (m)	Required base width (m)	Predicted settlements, S_c (mm)	Required time (days) [year]	PVD spacing in triangular pattern (m)	Predicted settlements using PVD, S_c (mm)	Required time using PVD (days)
1.	0+500 to 2+53-5	4.5	0.90	–	–	–	–	–	–	–
2.	2+535 to 5+00-0	8	5.20	8.5	31.65	1057	956 [2.62]	1	1009	41
3.	5+000 to 7+00-0	12	4.84	8.14	30.21	1280	2271 [6.22]	1	1230	48
4.	7+000 to 8+00-0	5	4.30	7.6	28.05	774	336 [0.92]	1	724	28
5.	8+000 to 9+57-5	10	5.20	8.5	31.65	1193	1547 [4.23]	1	1145	47
6.	9+575 to 11+0-00	5	2.7	6.0	17.65	704	323 [0.88]	1.5	655	80
7.	11+000 to 12+0-00	8	4.0	7.3	22.85	987	936 [2.56]	1.5	938	94
8.	12+000 to 13+5-00	5	6.0	9.3	34.85	835	345 [0.94]	1.5	785	87

Table 3 (continued)

Segment	Chainage (km)	Thickness of the soft compressible layer (m)	Final height of embankment (m)	Height including surcharge load (m)	Required base width (m)	Predicted settlements, S_c (mm)	Required time (days) [year]	PVD spacing in triangular pattern (m)	Predicted settlements using PVD, S_c (mm)	Required time using PVD (days)
9.	13 + 500 to 16 + 5-00	10	5.7	9	33.65	1219	1556 [4.26]	1.5	1177	123
10.	16 + 500 to 18 + 0-00	5	5.5	8.8	32.85	818	343 [0.94]	1.5	770	84
11.	18 + 000 to 20 + 5-00	6	6.84	10.14	38.21	968	523 [1.43]	1.5	918	89
12.	20 + 500 to 22 + 0-00	9	6.6	9.9	37.25	1208	125 [0.35]	1.5	1158	116
13.	22 + 000 to 23 + 5-00	5	5.13	8.43	31.37	805	341 [0.93]	1.5	756	84
14.	23 + 500 to 25 + 5-00	10	6.16	9.46	35.49	1254	1568 [4.3]	1.5	1204	121
15.	25 + 500 to	7	6.68	9.98	37.57	1046	729 [2]	1.5	998	111

Table 3 (continued)

Segment	Chainage (km)	Thickness of the soft compressible layer (m)	Final height of embankment (m)	Height including surcharge load (m)	Required base width (m)	Predicted settlements, S_c (mm)	Required time (days) [year]	PVD spacing in triangular pattern (m)	Predicted settlements using PVD, S_c (mm)	Required time using PVD (days)
	27 + 0-00									
16.	27 + 000 to 28 + 5-00	10	10.0	13.3	54.85	1454	1618 [4.45]	1.0	1406	59
17.	28 + 500 to 30 + 5-00	16	10.74	14.04	57.81	1929	4418 [12.1]	1.0	1881	89
18.	30 + 500 to 32 + 3-65	11	6.56	9.86	37.09	1345	1932 [5.3]	1.5	1296	133
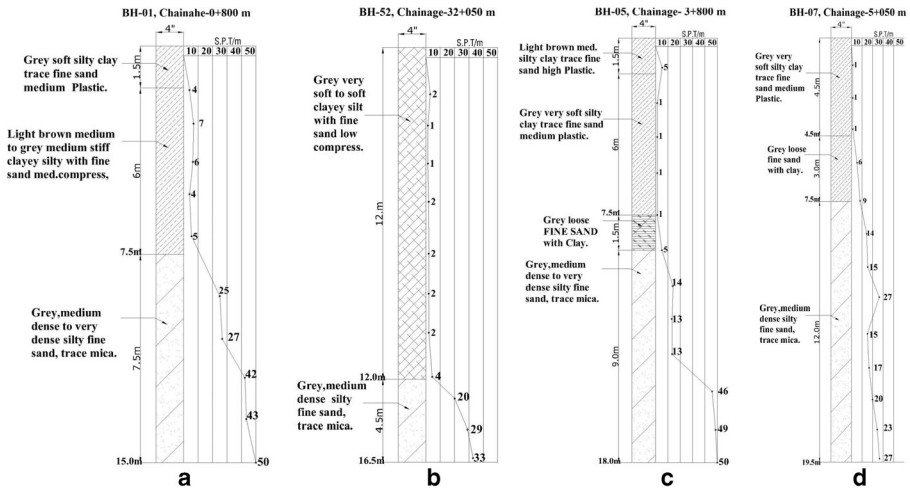
Fig. 3 a–d Soil profile with SPT values of boreholes

5 Consolidation Settlement

A total of 32.4-km railway embankment was divided into 18 segments according to the variation of the thickness of soft compressible layer. In the first 2.535 km, the thickness of the soft compressible layer was 4.5 m; 60 kPa surcharge pressure was considered due to axle load. The considered unit weights, initial void ratio (e_0), and compression index (C_c) were 18 kN/m³, 1.25, and 0.35, respectively. For the design of subsoil improvement, the average thickness of the soft compressible layer and the average height of embankment and estimated settlements (both before and after using PVD) along different segments are presented in Table 3. The 1D consolidation theory of Terzaghi (1943) was used for the calculation of the consolidation settlements due to full design load and time for consolidation.

The 1D consolidation settlement according to theory is given by the following (Terzaghi 1943):

$$S_c = \frac{C_c}{1 + e_0} H \log \frac{\sigma'_0 + \Delta\sigma}{\sigma'_0} \tag{1}$$

Where S_c = the consolidation settlement, e_0 = initial void ratio; H = thickness of layer; $\Delta\sigma$ = increase in total vertical stress at the center of the layer, and σ'_0 = effective vertical stress at the center of the layer.

The time for consolidation settlement (t) is given by (Terzaghi 1943):

$$t = \frac{T_v H^2}{C_v} \tag{2}$$

Where T_v = time factor, H = length of the drainage path, and C_v = coefficient of consolidation for vertical flow.

The maximum and minimum settlements due to the design load of the project site were estimated to be 1929 mm for a 16-m-thick layer and 704 mm for a 5-m-thick layer of compressible soil, respectively. These settlements are too high from serviceability consideration of the railway embankment. It was therefore considered necessary to preconsolidate the soil before construction of the embankment. However, the time required for the consolidation using preloading was a major concern in the design of the preconsolidation. With the consolidation coefficient estimated from the laboratory tests (i.e., $C_v = 7.0$ m²/year), the time required for 90% consolidation was estimated to range from 0.88 year (323 days) to 12.1 years (4418 days) for 5 and 16 m layers of soft soil, respectively. Preloading with vertical drains (sand drains or prefabricated vertical drains) was therefore considered for the design of the yard to accelerate the consolidation process. A brief description of the design of the ground improvement method using preloading with vertical drains is outlined below.

6 Soil Improvement

The project stretch of the new railway line passes through mostly paddy fields and the subsoil varies from soft silty clay to clayey silt up to 6 m depth from OGL and beyond 6 m; up to 10 m depth, there was very loose to loose fine sand with SPT (N) values ranging from 1 to 10. It clearly indicates that the ground cannot bear loads arising from the structure or even from embankment formation. Further, the ground was going to experience substantial settlements due to its soft state of condition. The project alignment requires the construction of embankment with an average height of 5 to 8 m all along the alignment. Since the underlying soil was found to be soft silty clay, ground improvement by installation of PVD also called as band drains/wick drains followed by preloading had been proposed. In the foundation level of the railway embankment, a total number of 4,402,326 PVDs was used. Soil improvement works for 18 segments of embankment had been considered based on the available geological profiles and the results of laboratory tests conducted under additional geotechnical investigations. The design of the soil improvement works involved some steps such as determination of the maximum thickness of soft layer from the bore log data and estimation of undrained shear strength of soft soil and the embankment fill. Values of undrained shear strength of soft soil were assumed to vary with depth. Settlements resulting from the long-term consolidation of cohesive soils often create serious problems in foundation engineering. In many cases, precompression of the foundation soils, by applying a surcharge load prior to the placement of the final permanent load, was used to reduce settlement problems. As the consolidation process was governed by the rate of excess pore pressure dissipation, shortening the length of pore water flow paths greatly reduces the consolidation time. PVDs were artificially created drainage paths installed for the purpose of shortening drainage paths.

6.1 Design Assumptions

PVDs were used to improve the ability of the soil to support construction and accelerate the rate of settlement. Most soil deposits had greater permeability in the horizontal direction than in the vertical direction. Therefore, PVDs help in accelerating the rate of

consolidation settlement. Because of their ease of installation, flexibility, reliability, and lower cost, PVDs had almost totally replaced conventional sand drains for a majority of applications. The center to center spacing (S) between drains was $S = D/1.05$ (for triangular spacing) and $D/1.13$ (for rectangular spacing), respectively. Here, D is the diameter of the cylinder of influence for the drains. In designing PVD, the value of the consolidation coefficient for horizontal (C_h) flow can be estimated 2 to 3 times the value of the consolidation coefficient for vertical flow (C_v). The values of C_h were sometimes considerably larger than C_v, particularly in sedimentary deposits of a continuous nature. For design purposes, it was reasonable to calculate the equivalent diameter ($d = 2(a + b)/\pi$), where a is the drain width and b is the drain thickness (Hansbo 1979).

6.2 Design of PVD

The design of soil improvement works was carried out for the 17 soft ground segments of embankment on the basis of detailed subsoil investigation. The laboratory test results on samples collected from 52 boreholes were also considered. In segment 1 (chainage 0 + 550 to 2 + 535 km), subsoil improvement was not to be considered. A summary of the thickness of soft subsoil layers at different segments of embankment and height of embankment fill is presented in Table 3. The maximum thickness of the soft soil layer was 16 m, considered for the design. Vertical drains were designed to install down to a depth of approximately 16 m below the ground level to cover the full depth of the soft clay layer.

Table 4 shows five different options of vertical drains initially estimated. Time for 90% consolidation with 250 mm diameter vertical sand drains at 1 and 1.5 m center-to-center (c/c) spacings in a triangular pattern was calculated to be 85–145 and 95–160 days. Moreover, the times for 90% consolidation with PVDs at 1.0, 1.5, and 2.0 m spacings were assessed to be 28–89, 80–133, and 159–259 days, respectively. Approximate costs for each of the options are also shown in Table 4. Based on the comparison, PVDs (width = 100 mm, t = 5 mm) at 1.0 and 1.5 m c/c in a triangular pattern were selected.

Table 4 Approximate cost and consolidation time for five options of soil improvement

| Options	Consolidation time (days)	Material cost (USD/m²)	Construction cost (USD/m)
250 mm diameter sand drain @ 1.0 m c/c in a triangular pattern	85–145	2.5	1
250 mm diameter sand drain @ 1.5 m c/c in a triangular pattern	95–160	2.0	
PVDs (width = 100 mm, t = 5 mm) @ 1.0 m c/c in a triangular pattern	28–89	5.3	4
PVDs (width = 100 mm, t = 5 mm) @ 1.5 m c/c in a triangular pattern	80–133	3.9	
PVDs (width = 100 mm, t = 5 mm) @ 2.0 m c/c in a triangular pattern	159–259	2.7	

The values of different soil parameters, embankment, and PVD dimensions and criteria were adopted for the PVD design. Axle load, slide slope, berm width, and crest width of embankment were 60 kPa, 2 m, 2 m, and 6.85 m respectively. The values of undrained shear strength of embankment fill (C_m), undrained angle of internal friction (ϕ_m), unit weight of fill (ɣ), and CT had been considered 0 kN/m^2, 35°, 18 kN/m^3, and 12 kN/m^2, respectively. Again, the values of ΔCT and D_c were considered as 0. The values of initial void ratio of soft subsoil layer (e_0), compression index (C_c), coefficient of consolidation for vertical flow (C_v), coefficient of consolidation for horizontal flow (C_h), and well resistance (k_h/q_w = ratio of horizontal permeability of the soft subsoil and discharge capacity of the PVD) were adopted as 1.25, 0.35, 7 m^2/year, 14 m^2/year, and 0.01, respectively. The width and thickness of PVD and the allowable settlement of embankment after improvement were 100, 3.2, and 50 mm, respectively. The desired degree of consolidation in various stages was adopted as 80–90%. The value of angle of friction (ϕ) of subsoil for calculation of ΔCu after each stage loading was 15°. Minimum factor of safety (FS) for stability against foundation failure for each stage loading was considered as 1.1. Embankment height was varied 4 to 8 m and 8 to 12 m for one and two numbers of berm on each side. In all the analysis, the soil was assumed to be normally consolidated due to the presence of very soft subsoil deposits.

Additional surcharge height, equivalent to the design axle load of 60 kPa, was considered in the design to reduce future potential settlements under service conditions. This additional surcharge height was taken as 3.3 m (= 60 kPa/18 kN/m^3) of fill height. Fill height in various stages was determined with consideration to ensuring a minimum factor of safety of 1.1 against stability failure. The estimated total consideration settlement at the center of embankment under design height and axle load at various segments was found to vary from about 700 to 1929 mm. In the analyses, it was considered that 50 mm of settlement of the embankment may be allowed to occur in the future, and the rest of the amount of the total consolidation settlement was completed at the end of the subsoil improvement and before placement of the railway track. A summary of the analyses containing estimated total settlement and required time before and after PVD was used, PVD spacing, etc. is presented in Table 3.

6.3 Stage Loading

Stability analysis at different depths of a soft soil layer involves determination of the values of factor at different depths of the soft soil layer. A soil layer was assumed to be safe against bearing capacity failure when the calculated factor safety at each depth of the soil layer was at least 1.1. If the stability of the subsoil layer under the loading of full embankment height and surcharge, if any, was adequate, then single-stage loading was decided. Otherwise, double- or multiple-stage loading was proposed.

In segment 2 to segment 5 (chainage 2 + 535 to 9 + 575 km), PVD had already been installed at 1 m spacing in a triangular pattern, and in the analysis for that stretch of the embankment, stage construction and waiting time for 1 m PVD spacing had been assessed as shown in Fig. 5. PVD layout plan at chainage 2 + 529 to 2 + 600 km is presented in Fig. 4. Settlement of the fill had been monitored (record RL) at least twice a week. After the required maintenance period of preloading and surcharging, the surcharge was removed and construction of the railway track was started. The cross-section of the embankment showing the proposed subsoil improvement scheme is
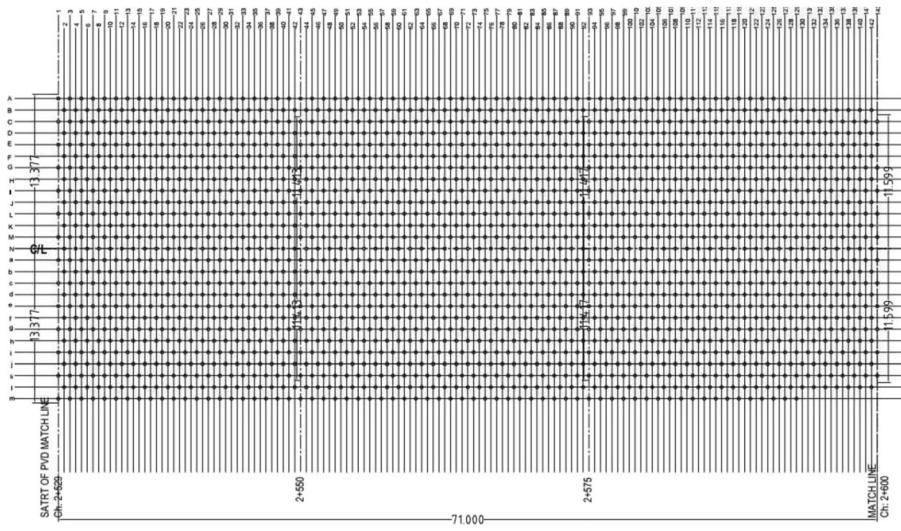
Fig. 4 PVD layout plan at chainage 2 + 529 to 2 + 600 km

presented in Fig. 7. The plan and section of embankment showing the locations of the settlement plates and alignment stakes is displayed in Fig. 8. It was found from the analysis that three-stage loading with waiting times was necessary to ensure foundation stability. Total waiting times for embankment loading in the three stages were found to be in the range of 28 to 48 days (Figs. 4, 5, 6, 7, and 8).

For segment 6 to segment 15 (chainage 9 + 575 to 27 + 000 km) and segment 18 (chainage 30 + 500 to 32 + 365 km), it was found that construction of the embankment was to be done in three stages to avoid stability failure. For these segments, minimum waiting times for the three stages of construction were calculated for PVD spacings of 1.5 and 2.0 m. It was found that total waiting time for the three stages varies from 80 to 133 days for PVD spacing of 1.5 m as presented in Table 3.
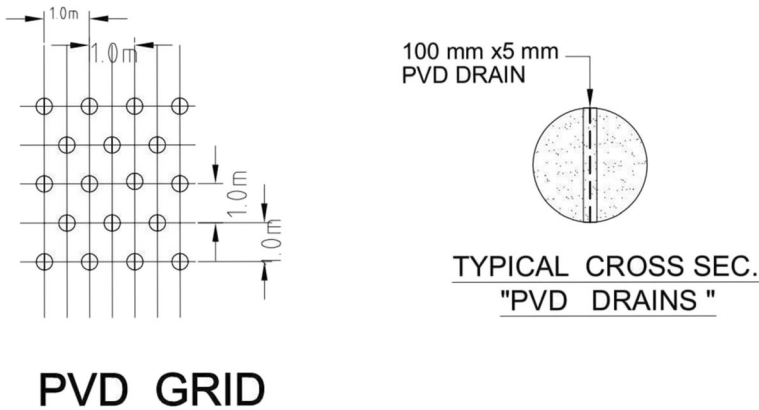
Fig. 5 PVD grid and typical cross-section of PVD drains
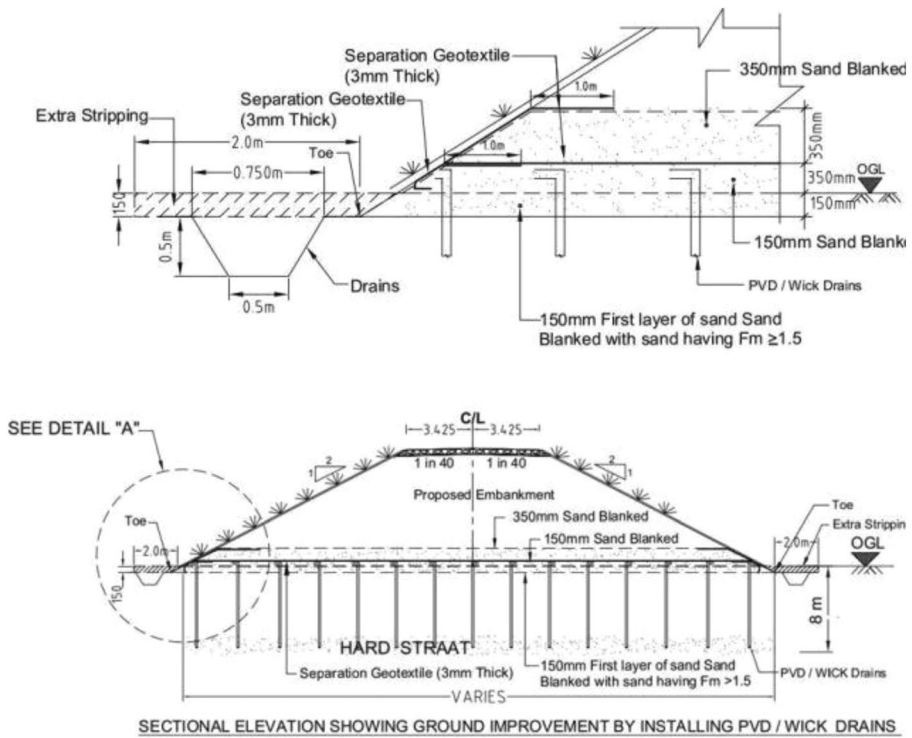
Fig. 6 Sectional elevation of ground improvement by installing PVD at chainage 02 + 529–8 + 000

In segment 16 (chainage 27 + 000 to 28 + 500 km) and segment 17 (chainage 28 + 500 to 30 + 500 km), both embankment height and compressible soft soil layer thickness were large. Analysis shows that for these segments, four-stage construction was required with appropriate waiting time in each stage to ensure foundation stability. For these segments, minimum waiting times for the four stages of construction were
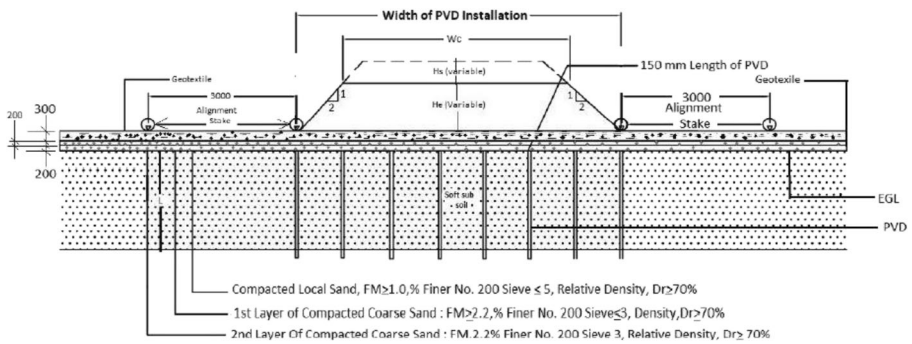
Wc = Width of embankment crest at final height
Hs = Height of soil surcharge
He = Final height of embankment
L = Thickness of soft compressible sub-soil layer

Fig. 7 Cross-section of embankment for subsoil improvement scheme
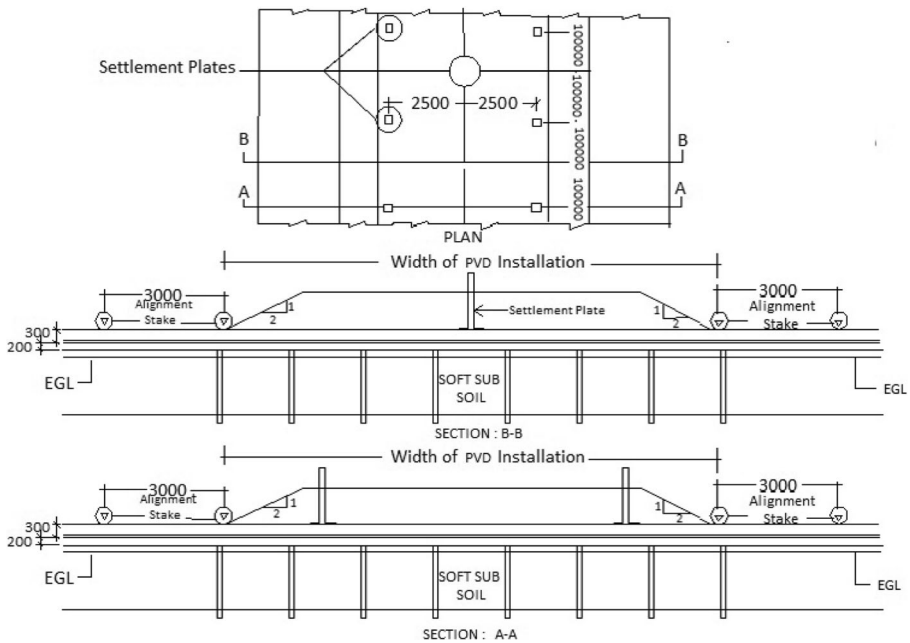
Fig. 8 Plan and section of embankment for the locations of settlement plates and alignment stakes

calculated for PVD spacing of 1.0 and 1.5 m. It was found that total waiting time for the four stages varies from 59 to 89 days for PVD spacing of 1.0 m as presented in Table 3, whereas for PVD spacing of 1.5 m, the total waiting time for the four stages varies from 155 to 221 days as presented in Table 3.

It was to be noted that, in addition to the waiting times for different stages, additional duration was required to attain the fill heights in stages and removal of additional surcharge. While placing the fill, proper compaction had to be ensured up to the design height of the embankment in the respective segment. The rest of the fill height can be placed without compaction. It was also to be noted that the required base width for placement of fill up to the surcharge height, with a side slope of 1 (V):2 (H), was larger than the design base width of the embankment sections. The required base widths are shown in Table 3. The fill for surcharge loading and additional material on the sides had to be removed after desired improvement occurred. Predicted settlements at the end of each stage load and waiting time are shown in Table 3. However, actual waiting times for the different stages were governed by field monitoring of settlement using settlement places. In addition to settlement plates, alignment stakes were placed at specified locations to monitor any unusual lateral displacement of the foundation soil. The completed rail way track is captured in Fig. 9f.

6.4 Geotextile

Needle-punched geotextile was used for PVD construction. The specification of non-woven needle-punched geotextile was given here. The values of mass per unit area (ASTM D 5261), thickness (under a pressure of 2 kPa, ASTM D 5199), effective opening size (AOS/EOS, ASTM D 4751), and vertical permeability at 20 °C (under

Fig. 9 **a**, **b** The installation of PVD. **c** The installed PVD. **d** First layer of sand blanket spread over the top. **e** Sand blanket is compacted with a vibratory roller. **f** The completed railway track

2 kPa normal stress, DIN) had been estimated as ≥ 400 g/m^2, ≥ 3.5 mm, < 75 μm, and $\geq 3.0 \times 10^{-3}$ m/s, respectively. The values of horizontal permeability (ASTM D 4716), grab tensile strength (ASTM D 4632), grab tensile elongation (ASTM D 4632), wide width tensile strength (ASTM D 4595), wide width tensile elongation (ASTM D 4595), and CBR puncture resistance (ASTM D 6241) at 20 °C were tested as $\geq 3.0 \times 10^{-3}$ m/s, ≥ 1500 N, $\geq 85\%$, ≥ 20 kN/m, $\geq 80\%$, and ≥ 3500, respectively.

7 Method of Construction

PVD was installed with approved hydraulics stitched as shown in Fig. 9a and b respectively, which caused a minimum of disturbance to the soil during installation and maintained the mandrel in a vertical position. The installed PVD without disturbance to the soil is captured in Fig. 9c. The PVDs were installed using a mandrel or sleeve that advanced through the compressible soils to the required depth using constant load, or constant rate of advancement. There were some steps that were followed for the improvement of subsoils. The existing ground surface was prepared by stripping of top vegetation and grubbing and leveling at the desired elevation. After clearing the surface soil including vegetation and vegetation roots up to a depth of 150 mm, there was no separate payment for clearing of vegetation and vegetation roots from surface soil, a sand blanket of medium to coarse sand of 500 mm compacted

thickness was laid over the top of the compacted formation, and the project at least 150 mm above NGL and 5.0 m on all sides is shown in Fig. 9d. The blanket was spread over the entire area of treatment. In case any pits were observed, the same was also filled with sand. The blanket was laid in layers of 175 mm (maximum compacted thickness) and compacted to 95% relative density by means of vibratory rollers as displayed in Fig. 9e.

In order to remove the water from the consolidating system, proper drainage arrangement including pits, channels, and pumping arrangement was provided; 200-mm-thick compacted layer of sand (F.M. ≥ 1.0; % finer no. 200 sieve ≤ 5; relative density, Dr $\geq 70\%$) was placed. The first layer of 200-mm-thick compacted coarse sand layer (F.M. ≥ 2.2; % finer no. 200 sieve ≤ 3; relative density, Dr $\geq 70\%$) was also placed as shown in Fig. 6. PVD was installed at specified spacing. In areas where wind was prevalent, fabric installation was started at the upwind side of the project and proceeds downwind. The leading edge of the fabric was secured at all times with sandbags or other means sufficient to hold it down during high winds. Sandbags or rubber tires were used as required to hold the fabric in position during installation. Tires with exposed steel cords or other sharp edges may snag or cut the fabric. Material equipment or other items were dragged across the fabric or allowed to slide down the slopes on the fabric. The installed geotextile had been covered by preloading fill. Surcharge was used as preloading surcharge. The material was earth locally available and fit for earth-filling purposes. The earth was spread in layers not more than 250 mm compacted thickness. Earth fill below the designed formation level and up to 500 mm above the designed formation level was compacted up to 95% proctor density/80% relative density as applicable. The remaining height of the earth fill was compacted to 80% proctor density/60% relative density. Then, a nonwoven needle-punched geotextile fabric was placed over the first coarse sand layer as presented in Fig. 7. A second layer of 300-mm-thick compacted coarse sand layer (F.M. ≥ 2.2; % finer no. 200 sieve ≤ 3; relative density, Dr $\geq 70\%$) was also placed as shown in Fig. 6.

The settlement plates were allocated with their base at the top of the second coarse sand layer to monitor the vertical settlement of subsoil. Then, embankment fill was positioned. Alignment stakes at and beyond the embankment toes were inserted to monitor the horizontal movement of ground.

8 Monitoring of Settlements

Ground settlement was monitored using the settlement plates at seven chainage locations (for the first 15 km of the project) during consolidation under embankment load to validate the design assumption and to ensure preconsolidation before construction of PVD. Practically, in the field, it was a challenging and tough job to manage the application of surcharge load which was considered during the design phase. During the implementation phase of the project, no surcharge load (60 kPa) was placed, and only embankment load was used as preloads. Settlement monitoring was started immediately after placement of the materials to the embankment height and continued until consolidation was completed.

Settlement versus time curve of these chainage points is presented in Fig. 10. At chainage point 3 + 800 km, 486 mm settlement in 256 days was observed at the end of

the first stage and 652 mm settlement was observed at 376 days (which was monitored until 586 days) at the end of the second stage. According to the estimation, 90 days was required to achieve 629 mm settlement with PVD and without surcharge as presented in Table 5. However, it took 346 days to achieve 638 mm settlement. In 90 days, only 260 mm settlement was observed.

During stage loading at chainage point 5 + 050 km, the first and second stages were completed with 490 mm settlement in 220 days and 720 mm settlement in 366 days, respectively, as shown in Fig. 10. According to the estimation, 133 days was required for 683 mm settlement, whereas 366 days was observed to achieve 720 mm settlement at the field.

A total of 716 mm settlement was monitored in 339 days at chainage point 5 + 300 km. However, 127 days was needed to achieve 721 mm settlement according to the calculation using PVD as shown in Table 5. It can be seen from Fig. 10 that only 323 mm settlement was achieved in 136 days. The first and second stages were completed with 470 mm settlement in 220 days and 638 mm settlement in 346 days (which was monitored until 489 days), respectively.

According to the prediction, 115 days was required to accomplish 529 mm settlement with and without using PVD and surcharge load at chainage point 5 + 800 km. However, only 240 mm settlement was observed in 115 days in the field. The estimated settlement and the required time of the first and second stages were 475 mm in 225 days and 643 mm settlement in 365 days, respectively.

At chainage point 6 + 300 km, a total 633 mm settlement was observed in 371 days, which did not fulfill the design estimation (521 mm settlement in 105 days). Moreover, 230 mm was monitored in 105 days as depicted in Fig. 10.

Figure 10 shows the first and second stages were finished with 361 mm settlement in 180 days and 618 mm settlement in 368 days, respectively, at chainage point 8 + 300 km. The observed total settlement at this point was 618 mm in 368 days. However, it took 258 days more than the estimated time period.

As per estimation, 125 days was required for 459 mm settlement, whereas 367 days was observed to achieve 524 mm settlement at chainage point 11 + 800 km. Moreover, 129 days was monitored to achieve 258 mm settlement only. However, the duration of the first and second stages was 190 and 342 days, respectively. The predicted and observed settlement with percentage of error in prediction for the specific seven chainage points is also presented in Table 5.

The discussion made in this section shows that estimated time did not match with the observed consolidation period. Moreover, the observed time was too long from the estimated time. There could be three possible reasons behind this phenomenon. These are smear effect, not using surcharge load, and not having the exact soil data for estimation. In the design phase, smear effect was not considered. A smear zone (disturbed zone) could have developed during the installation of PVD using a mandrel, and this zone retards the horizontal consolidation of soft clays in the vicinity of PVD (Xie 1987; Sharma and Xiao 2000; Basu and Prezzi 2007; Dhar et al. 2011; Tran-Nguyen and Edil 2011). Actually, in the design phase, the settlement was calculated with respect to surcharge load, both with and without PVD. Hence, it was recommended to apply 60 kPa surcharge load during the implementation. However, during the
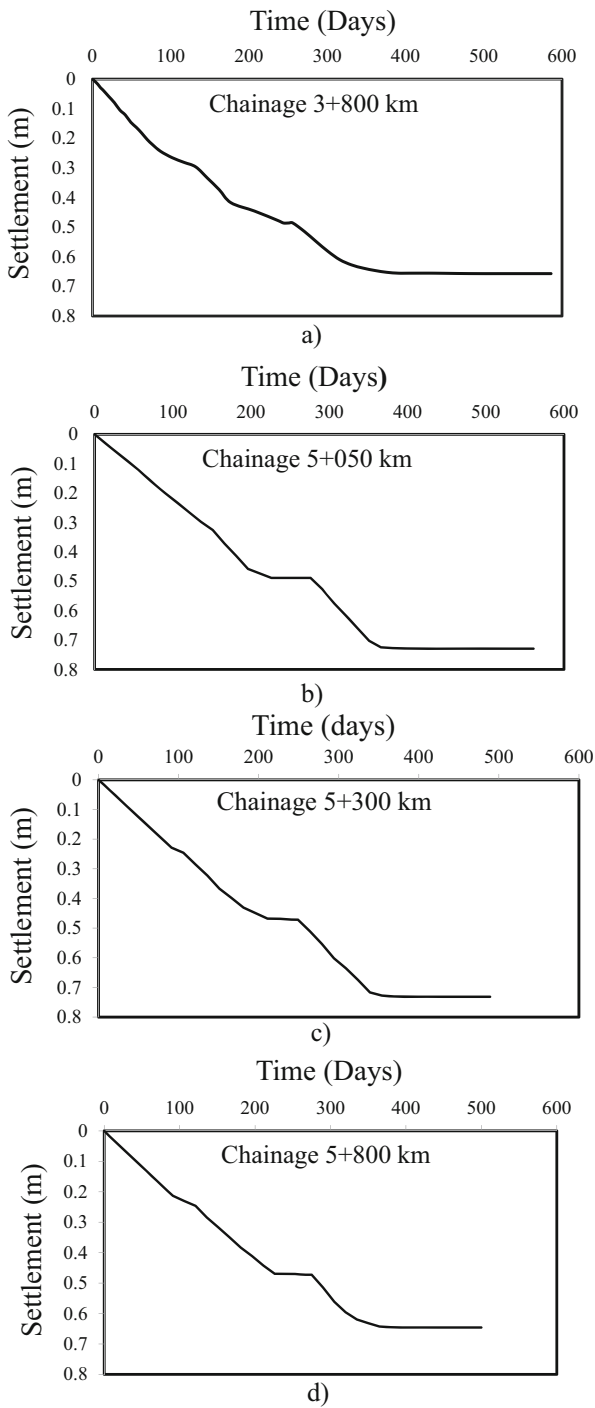
Fig. 10 **a**–**g** Settlement versus time curve at eight chainage points
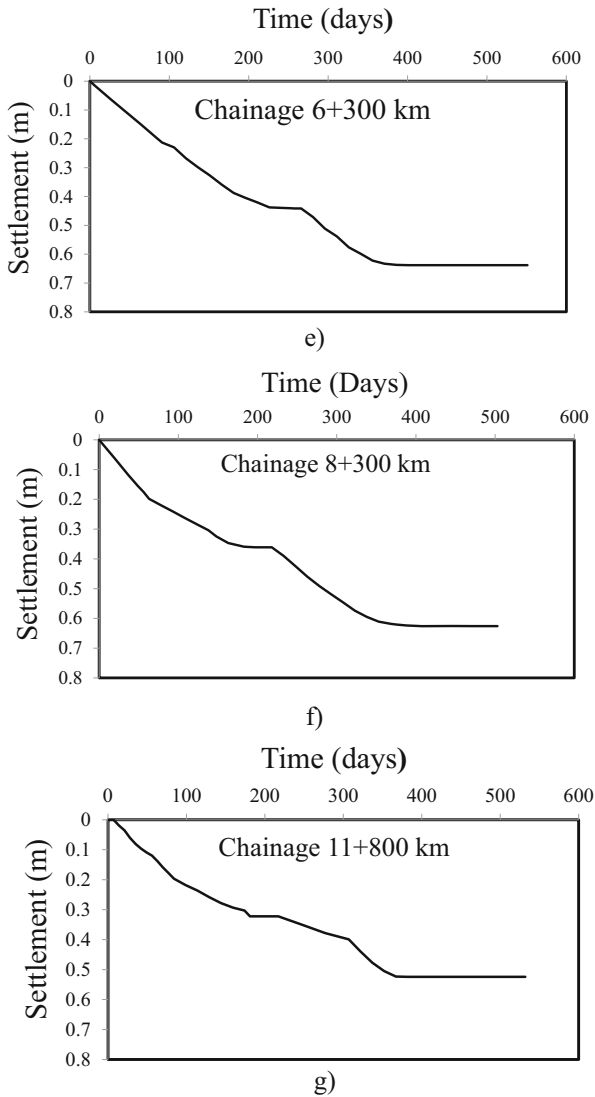
e)

f)

g)

Fig. 10 (continued)

implementation period, no surcharge load was applied due to some construction-related issues (such as transportation of surcharge load). However, in the field, only the embankment load was used as preload for a long time. Moreover, this phenomenon may be due to not having the exact soil profile and index properties at these seven points for settlement estimation. Thus, the consolidation process was very slow in the field and it took more time to achieve the desired settlement. The percentages of error in prediction vary between 0.81 and 22.4% as shown in Table 5. Currently, the construction work of 15 km of railway embankment was completed and the rest of the 17.365-km railway embankment was under the construction stage.

Table 5 Thickness of the soft compressible layer and predicted and observed settlements (mm) for different chainages of the embankment

Chainage (km)	Thickness of the soft compressible layer (m)	Height of embankment (m)	Predicted settlement without PVD and without surcharge (mm)	Required time with PVD and without surcharge (days)	Observed settlement with PVD (mm)	Observed time (days)	% error in prediction
3 + 800	7.5	5.4	629	90	657	586	4.5
5 + 050	4.5	5.9	683	133	730	561	6.9
5 + 300	10.5	5.3	721	127	731	489	1.38
5 + 800	3	5.6	529	115	644	500	21.8
6 + 300	3	5.4	521	105	638	551	22.4
8 + 300	4.5	4.8	621	110	626	503	0.81
11 + 800	3	4	459	125	524	532	14.16

9 Summary and Conclusions

The soil improvement details of the Kashiani–Gopalganj railway project, which passed through mostly paddy fields, are described in this paper. Specific details with respect to the challenges encountered during the selection of optimum design regarding soft soil improvement, stage loading, PVD design, PVD material selection, and installation are also discussed.

A comprehensive laboratory investigation was executed in this project for determining the geotechnical design parameters for analysis of consolidation with and without prefabricated vertical drains. Based on the assessment, PVDs (width = 100 mm, $t = 5$ mm) at 1.0 and 1.5 m c/c in a triangular pattern were selected for the project due to the lowest consolidation time. However, in this project, a total number of 4,402,326 PVDs was installed. Stage loading (single, double, and multiple) was also applied with PVD for getting the required stability against the failure of foundation.

Settlement estimates including the selected soil consolidation properties in the analyses are discussed and compared to the actual settlement measured during construction. The observed time was too long from the estimated time at the field. There could be three reasons behind this. These are smear zone creation, not applying the surcharge load, and not having the exact soil data for settlement estimation. In the design period, the smear effect was not considered. Thus, the smear zone which was developed during the installation of PVD could have reduced the consolidation time. Moreover, the recommended surcharge load (60 kPa) was not applied in the field due to some construction-related issues. However, there was lack of exact soil data and essential data were adopted from the representative borehole of that segment.

The concept of soil improvement using PVD beneath the railway embankment was not only cost-effective but also safe, reliable, and time-saving as shown through the success of the project.

Compliance with Ethical Standards

Conflict of Interest   The authors declare that they have no conflict of interest.

References

Al-Soud, M.S.: Numerical analysis of prefabricated vertical drains improved soft soil beneath an embankment during staged construction. J Eng Dev. **20**(1), 151–163 (2016)

Basu, D., Prezzi, M.: Effect of the smear and transition zones around prefabricated vertical drains installed in a triangular pattern on the rate of soil consolidation. Int J Geomech. **7**(1), 34–43 (2007)

Bergado, D. T., and Patawaran, M. A. B. Recent developments of ground improvement with PVD on soft Bangkok clay. In Proc. Intl. seminar on geotechnics in Kochi, pp. 1-6 (2000)

Bo, M.W., Arulrajah, A., Horpibulsuk, S., Leong, M.: Quality management of prefabricated vertical drain materials in mega land reclamation projects: a case study. Soils Found. **55**(4), 895–905 (2015)

BRTC (Bureau of Research, Testing and Consultation): Report on review of design report for the treatment of soft soil areas of embankment for the proposed construction of double line track from Tongi to Bairab Bazar including signalling on Dhaka-Chittagong main line. Department of Civil Engineering, April 2013, Bangladesh University of Engineering and Technology, Dhaka (2013)

BRTC (Bureau of Research, Testing and Consultation): Report on design of foundation treatment, preloading with staged construction and monitoring scheme for the construction of railway embankment from Chainage 0+550 km to 32+365 km of Kashiani-Gopalganj section. Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka (2016)

Chen, F.H.: Foundation on Expansive Soils. Development in Geotechnical Engineering Vol. 12, First edn. Elsevier, Amsterdam (1975)

Dhar, A.S., Siddique, A., Ameen, S.F.: Ground improvement using pre-loading with prefabricated vertical drains. International Journal of Geoengineering Case Histories. **2**(2), 86–104 (2011)

Hansbo, S.: Consolidation of clay by band shaped prefabricated drains. Ground Eng. **12**(5), 16–25 (1979)

Hossain, M. (1983). Swelling properties of selected local soils. Unpublished M.Sc. thesis, BUET, Dhaka, Bangladesh, pp. 1–168

Indraratna, B., Rujikiatkamjorn, C., Vinod, J.S., Nimbalkar, S.: Physical and Chemical Ground Improvement for Sustainable Transportation Infrastructure under Cyclic Loads, pp. 140–156. Centre for Geomechanics and Railway Engineering, School of Civil, Mining and Environmental Engineering, University of Wollongong, Wollongong City, NSW 2522, Australia (2011)

Indraratna, B., Rujikiatkamjorn, C., Kelly, R., Buys, H.: Soft soil foundation improved by vacuum and surcharge loading. Ground Improv. **165**(G12), 87–96 (2012)

Johnson, L.T., Snethen, D.R.: Prediction of potential heave of swelling soil. Geotech Test J. **1**(3), 117–124 (1978). https://doi.org/10.1520/GTJ10382J. ISSN 0149-6115

Long, P.V., Bergado, D.T., Nguyem, L.V., Balasubramanium, A.S.: Design and performance of soft ground improvement using PVD with and without vacuum consolidation. Geotechnical Engineering Journal of the SEAGS & AGSSEA. **44**(4, December 2013, ISSN 0046-5828), 36–51 (2013)

Reddy, S.K., Moosani, Divya, P.V.: Improving engineering properties of soft soil using preloading and prefabricated vertical drains. International Journal of Research in Civil Engineering, Architecture & Design. **1**(2, ISSN Online: 2347-2855, Print: 2347–8284), 08–12 (2013)

Ruenkrairergsa, T., Lin, P.C., Sunantapongsak, S.: Performance of PVD road embankment on soft Bangkok clay. In: Proceedings of the International Conference on Soil Mechanics and Geotechnical Engineering, vol. 3, pp. 2179–2182. AA Balkema Publishers (2001)

Saowapakpiboon, J., Bergado, D.T., Chai, J.C., Kovittayanon, N., De Zwart, T.P.: Vacuum-PVD combination with embankment loading consolidation in soft Bangkok clay: a case study of the Suvarnabhumi Airport Project. In: Geosynthetics in Civil and Environmental Engineering, pp. 440–449. Springer, Berlin (2008)

Seed, H.B., Woodward Jr., R.J., Lundgren, R.: Prediction of swelling potential for compacted clays. J Soil Mech Found Div Am Soc Civ Eng. **88**(SM3, June), 53–87 (1962)

Sharma, J.S., Xiao, D.: Characterization of a smear zone around vertical drains by large-scale laboratory tests. Can Geotech J. **37**(6), 1265–1271 (2000)

Shen, S.L., Chai, J.C., Hong, Z.S., Cai, F.X.: Analysis of field performance of embankments on soft clay deposit with and without PVD-improvement. Geotext Geomembr. **23**(6), 463–485 (2005)

Terzaghi, K.: Theoretical Soil Mechanics, pp. 11–15. Wiley, New York (1943)

Tran-Nguyen, H.H., Edil, T.B.: The characteristics of PVD smear zone. In: Geo-Frontiers 2011: Advances in Geotechnical Engineering, pp. 748–757 (2011)

Xie, K. H. (1987). Consolidation theories and optimisation design for vertical drains. Doctor of Philosophy thesis, Zhejiang University, China

Publisher's Note Springer Nature remains neutral with regard to jurisdictional claims in published maps and institutional affiliations.