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Investigation of field instrumentation in a preloading project

1 **Kazem Fakharian** MSc, PhD
Assistant Professor, Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran, Iran

2 **Amirhassan Mehdizadeh** MSc
Research Associate, Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran, Iran


Preloading with prefabricated vertical drains (PVDs) is one of the most successful methods for improvement of soft soils settlement. A preloading project is described in the southwest of Iran near the Persian Gulf, in which well-planned instrumentation was used. The purpose of the project was placement of an 8 m embankment with installation of PVDs to accelerate the consolidation of subsoil for construction of a concrete water tank, 100 m × 150 m with total volume of 100 000 m^3. The thickness of soft soil was about 15 m. This paper presents results of soil instrumentation to measure dissipation of excess pore pressure (EPP), differentiating settlement in different soil layers and a comparison of the measurements with predicted settlements and consolidation–time variations according to radial consolidation theory, the results of Settle3D analysis and the Asaoka method. In addition, smear zone effects caused by PVD installation are studied and suitable parameters of soil permeability and smear zone are proposed for future projects. The results clarify that if consolidation parameters are selected adequately based on laboratory tests, the maximum settlement and consolidation–time relations are estimated properly both by conventional theoretical solutions and by more detailed solutions such as Settle3D.

Notation

a	width of PVD
b	thickness of PVD
C_c	compression index
C_h	coefficient of horizontal consolidation
C_s	swell index
C_v	coefficient of vertical consolidation
c_u	undrained cohesion
D_e	effective diameter of drainage of PVD
d_m	diameter of mandrel
d_s	diameter of smear zone
d_w	diameter of equivalent circle of PVD
E	modulus of elasticity
e_0	initial void ratio
k_h	horizontal permeability
k_s	permeability of smear zone
k_v	vertical permeability
l	thickness of mandrel
S	PVD spacing
S_i	settlement at time t_i
S_{max}	maximum settlement
SP4-i	surface settlement instrument, i shows number of instrument
SSG4-i	magnetic extensometer, i shows number of instrument
t	time of consolidation
U, U_h, U_v	total, horizontal and vertical consolidation percentage, respectively
VPZ4-i,j	vibrating wire piezometer, i and j show number and depth of instrument
w	width of mandrel
Z	thickness of soil layer
β_0	intercept of trend line with S_i axis (Asaoka method)
β_1	slope of trend line (Asaoka method)
γ_t	bulk unit weight
ν	undrained Poisson ratio
ω	moisture content

1. Introduction

Preloading increases the shear strength of soft soils and decreases any future excessive settlements. There are various methods for preloading of soft soils, but the most common in Iran is the construction of an embankment. The dimensions of the embankment are determined according to the size of the foundation and the pressure from future structures. In addition, where deep deposits of low-permeability, fine-grained soils are present, prefabricated vertical drains (PVDs) are installed to reduce the consolidation time. PVDs create horizontal drainage paths which are shorter than the vertical drainage paths. Owing to disturbance of soil through sampling and laboratory testing, the determination of exact consolidation parameters is difficult. Therefore, predictions of maximum settlement and time of completion of consolidation in practical conditions are approximate. Instrumentation is an essential part in any preloading project. The initial design can be back-analysed during the works using the results from the instruments. Soil instrumentation not only helps designers to verify the end-of-consolidation process, but also contributes to providing more realistic predictions for future projects.

Many studies are available in the literature including laboratory work, field monitoring and numerical modelling of the consolidation process (e.g. Akagi, 1981; Bergado *et al.*, 1993a; Carillo, 1942; Ladd, 1991). Lorenzo *et al.* (2004) found that use of PVD in soil consolidation by surcharge preloading has accelerated the consolidation of clay considerably. Three full-scale test embankments constructed at the SBIA (San Bernardino International Airport) site revealed that the 10 m thick soft Bangkok clay reached 90% consolidation within 1 year after embankment construction, for PVD spacing ranging from 1·0 to 1·5 m. The PVD installation also contributed to a uniform rate of consolidation and settlement. Indraratna *et al.* (2005) studied the analytical and numerical solutions for soft clay consolidation using geosynthetic vertical drains. They found that lack of saturation of soil at the vertical drain boundary due to mandrel driving could delay the excess pore pressure dissipation in the early stages of the consolidation process. In addition, the rate of settlement and pore pressure dissipation associated with vertical drains are difficult to predict accurately. This difficulty may be attributed to the complexity of evaluating the soil parameters inside and outside the smear zone as well as the unsaturated zone at the soil–drain interface.

Sathananthan and Indraratna (2006) investigated the characteristics of the smear zone created by mandrel-driven PVDs. The extent of the smear zone for Moruya clay was found to be about 2·5 times the equivalent mandrel radius. However, the size of the smear zone may vary depending on the mandrel shape, installation speed and soil stiffness. Arulrajah *et al.* (2004) revealed factors affecting field instrumentation assessment of marine clay treated with PVDs. The authors found that the C_h value back-analysed by the Asaoka method is dependent on the time interval used for the prediction. The C_h value predicted by the Asaoka method decreases and converges to the actual value with a longer time of assessment and an increasing number of time intervals in the back-analysis. Dhar *et al.* (2011) have studied the preloading process of a container yard at Chittagong Port in Bangladesh. They investigated C_v, C_h, k_v and k_h based on laboratory tests and found that C_h/C_v and k_h/k_v values are 2·07 and 1·53, respectively.

In the geological formations of southwest of Iran, the most common ground improvement methods are deep foundations such as piles or micro-piles and preloading. The Special Economic Petrochemical Zone (PetZone) of Mahshahr required four water tanks to store the Karun River water for petrochemical industries in PetZone. The location of the project is in the Kuzestan province in northwest of Sarbandar City and the distance between the construction site and PetZone is about 20 km.

Four tanks, $150\ \text{m} \times 100\ \text{m}$ each, with a maximum applied pressure of 100 kPa were considered. Two of the four tanks had already been constructed on micro-piles. The client decided to improve the subsurface soil of the other two water tanks by preloading using PVDs. Considering the dimensions of the tanks and their sensitivity to non-uniform settlements, monitoring the progress of preloading and achieving the required consolidation percentage was very important. Therefore, the client agreed to a widespread instrumentation plan to monitor the settlements and dissipation of excess pore pressure in different soil layers. Instrumentation of the project included surface settlement monuments, magnetic extensometers and vibrating wire piezometers. Surface settlement monuments are concrete-base benchmarks for settlement monitoring by surveying methods.

This paper presents instrumentation results for one of the tanks (tank no. 4) during construction of the embankment and up to 1 year afterwards. The consolidation percentage and maximum settlements are compared with predictions of conventional theoretical solutions and more detailed analytical solutions from Settle3D software. Finally, the consolidation and smear parameters evaluated by back-analysis and the Asaoka method (Asaoka, 1978) are presented and discussed.

2. Site and subsurface conditions

Mahshahr Port is located at the tidal zone in the northwest corner of the Persian Gulf. The layers of this area are mainly formed of two different geological formations, as described below.

(*a*) The upper formation is deltaic and estuarine deposits formed of frequent clay, silt and sand layers. These layers are deposited in marine condition and usually do not have an appreciable amount of pre-consolidation pressure. Layers I and II described later are of the upper formation.
(*b*) The lower formation is formed of dense to very dense sand and non-plastic silt along with stiff to hard silty clay. These layers are formed from bedrock erosion and decomposition. An evaluation of the geotechnical data indicates that the mechanical properties of the different layers gradually increase with depth.

The layers are described as follows.

(*a*) Layer I: this layer starts from ground level and continues to a depth of 15 m and it is mainly a cohesive lean clay; its consistency changes from soft to medium stiff. Sand and silt lenses are present at different depths, which cause an increase in standard penetration test (SPT) values. This layer was expected to provide the main contribution to the consolidation settlement during the preloading operations.

(*b*) Layer II: this layer starts from 15 m and continues to 22 m and is a cohesionless silty sand or sandy silt and there are sub-layers of lean clay with low thickness.

(*c*) Layer III: this cohesive lean layer starts from 22 m to the end of each borehole and its consistency changes from stiff to hard.

Table 1 presents a summary of the geotechnical parameters for these three soil layers. According to the geotechnical tests, the permeability of layer I varies between 10^{-6} and 10^{-10} m/s and water table elevation is an average of 1 m below the surface.

3. Project specifications and construction method

The upper 15 m (layer I) needed to be improved and preloading was selected in order to increase the bearing capacity and decrease the settlement. Owing to the low permeability of these fine-grained soils, the required time for consolidation would have been too high and hence PVDs were used to reduce the consolidation time. In addition, 100% of the required settlements could occur in a consolidation time corresponding to 80–85%, if the surcharge exceeds the design load by 35%.

There was no restriction on the type of embankment fill. A mixture of sand/gravel was selected for the surcharge based on the technical and cost evaluations. A 0·5 m thick gravel material was selected as a drainage blanket layer to cover the inclined natural soil to improve the hydraulic performance of discharged water. The drainage blanket was inclined such that it would be level after the predicted settlements had occurred. The pressure applied by a full tank is 100 kPa and the bulk weight of sand/gravel mixture (fill) of the embankment is around 17 kN/m^3. Therefore, the height of the embankment was selected to be 8 m (equivalent to 136 kPa) and it was constructed in five stages over 60 d.

4. Instrumentation plan

The soil instrumentation programme was planned to monitor the rate of settlement, the variations of excess pore-water pressure and the consolidation percentage and used surface settlement monuments, magnetic extensometers and vibrating wire piezometers. In total, 24 vibrating wire piezometers were installed at depths of 4–14 m below the ground surface. Fifteen surface settlement monuments and four magnetic extensometers were installed at different locations and depths within area of tank no. 4. The positions of instruments studied in this research are presented in Figure 1.

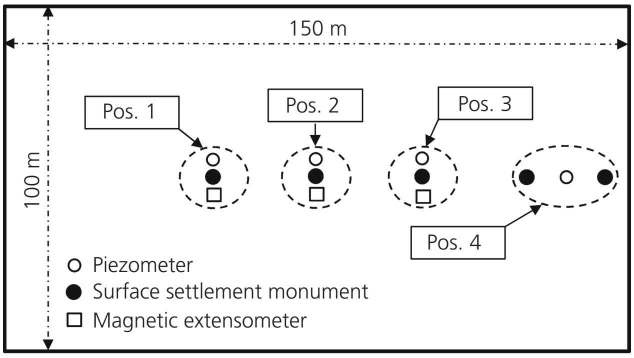

Figure 1. Layout of instruments

Layer / Parameter	Unit	Layer I			Layer II		Layer III
Thickness	m	0–3	3–13	13–15	15–18	18–23	>23
USCS classification	—	CL	CL	Cl	SM, ML	SM, ML	CL, ML–CL
SPT no.	—	5	6	10	25	36	40
γ_t	kN/m^3	19·5–20·5	19–20	19·5–20·5	19·5–20·5	20–21	20–21
Moisture content, ω	%	28	30	25	25	24	23
Undrained cohesion, c_u	kPa	45	30	35	0	0	150
Liquid limit (LL)	%	40	38	34	—	—	40
Plasticity index (PI)	%	18	18	15	—	—	23
C_c	—	0·2	0·22	0·22	—	—	0·18
C_s		0·026	0·03	0·028	—	—	0·03
Over-consolidation ratio	—		1–1·8		1–1·8		—
Modulus of elasticity, E	MPa	13	10	11	22	30	>40
Undrained Poisson ratio, ν	—	0·45–0·5	0·45–0·5	0·45–0·5	0·35	0·35	0·4

Table 1. Geotechnical parameters of soil layers

The combined results of the surface settlement monuments and the surface plates of magnetic extensometers are shown in Figure 2. The trends and magnitudes of settlements obtained from all instruments are reasonably close to each other. The slope of the diagrams can be divided into three parts. The first, second and third sections belong, respectively, to filling, end-of-filling to end-of-consolidation and fill removal.

Instruments were monitored for around 1 year. About 90% of consolidation occurred in 220 d and the maximum, average and minimum settlements monitored by the surface instruments before removing the fill were 822, 742 and 675 mm, respectively. At the end of consolidation, 3 m of embankment materials were removed for backfilling of the surrounding area, during which a 15 mm heave was observed.

Magnetic extensometer results are shown in Figure 3 and illustrate differential settlements between various sub-soil layers. Ground surface elevation is +4·00 m and the elevation of the beginning of the second layer is around −11·00 m (depth 15 m). Figure 3 shows that most of the settlements are attributed to the upper 15 m (layer I), as expected. Maximum, average and minimum settlements are measured as 836, 782 and 707 mm, respectively. Both types of settlement instruments have recorded differences of 120 to 150 mm between upper and lower limits of measured settlements. Such a difference seems to be attributed to variability of sub-soil strata across the preloading area. The average results of the four magnetic extensometers indicate that 585 mm (75%) of total average settlement of 782 mm has occurred within layer I, 133 mm (17%) within layer II and only 62 mm (8%) within layer III.

Figure 4 compares filling, soil settlement and excess pore pressure measured at several locations over time. A jump in excess pore pressure curves is observed at the beginning of filling of each stage before dissipating when the surcharge remains constant. In addition, a slight increase of excess pore pressure at the end of consolidation probably results from suction, which appears through removing the surcharge. Some piezometers were installed 4 m below the surface. These 4 m deep piezometers did not measure any noticeable excess pore pressure during filling. This is probably due to the over-consolidated near-surface layers.

Ideally, for normally consolidated soils the excess pore pressure (EPP) should be equivalent to the surcharge pressure of each filling phase. However, owing to the spacing of the vertical drains, the existence of sand or silt lenses, over-consolidation of soil and gradual filling of each stage, the increase in EPP is observed to be considerably smaller than the applied surcharge. All diagrams exhibit a high rate of settlements and increase in EPP during filling, followed by a rapid dissipation of EPP and a decelerating rate of settlement during constant surcharge. This observation is in contrast to Hansbo *et al.* (1981), who found that the rate of dissipation of the piezometers was frequently slower than the rate of consolidation settlements. They considered that the settlement observations were more reliable than the pore pressure observations, and recommended the use of settlement as a measure of the process of consolidation. It is likely that some piezometers have been installed near PVDs, causing the EPP to dissipate faster than expected in this study. It is concluded that a coincidence between dissipation of EPP and constant readings of settlements prove completion of consolidation, but in cases of time difference observation between the two, the completion of settlement is a more reliable criterion.

As can be observed in Figure 4, surface settlement monuments and magnetic extensometers exhibit a very good correlation in the different stages of filling, constant load and fill removal. The EPP variations compare to settlement variations quite well qualitatively. In all four positions, the peak EPP is observed at the end of filling, beyond which it has dissipated rapidly. The rate of settlement variations starts to reduce after this peak point and EPP has also dissipated completely when the settlement readings have stopped.

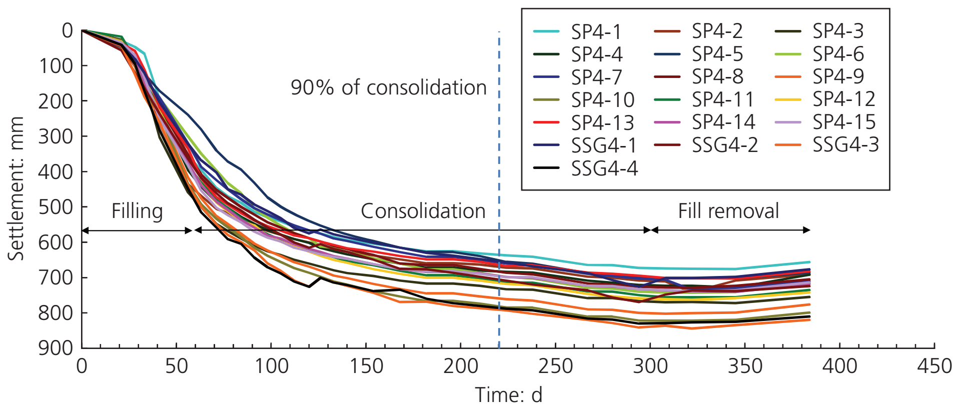

Figure 2. Surface settlements plotted against time from different instruments across the preloaded area

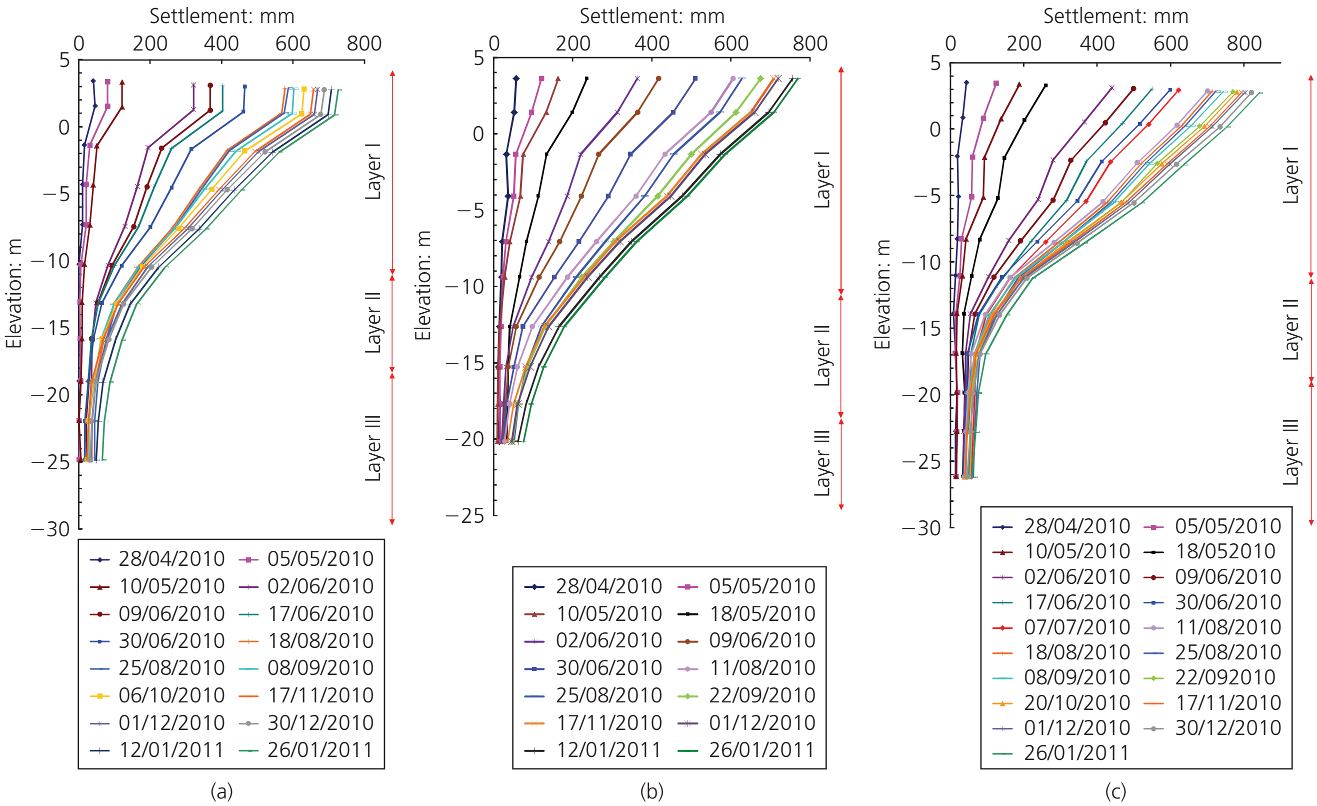

Figure 3. Measured settlement profiles plotted against depth at different times for three magnetic extensometers: (a) SSG4-1, position 1; (b) SSG4-2, position 2; (c) SSG4-3, position 3

5. Total settlement estimation

Total settlement of fine-grained soils includes immediate, consolidation and secondary consolidation components. The geotechnical records of the study area indicate that the secondary consolidation component is insignificant. The geotechnical parameters of soil classification do not highlight the importance of creep settlements. Besides, in the study area, on which numerous industrial structures have been constructed within the past few decades, no major long-term (more than a few years) settlements have been reported on comparable strata. Therefore, immediate and consolidation settlements are more effective at the study area, as explained below.

5.1 Immediate settlement

Theoretically, immediate settlement is expected to occur repeatedly when the tank fills up. Immediate settlements, usually simplified to elastic settlements, are not eliminated by preloading. Preloading, however, causes an increase in the modulus of elasticity (E) and, therefore, immediate settlements reduce during the service life of the tank.

Immediate settlement of layer I was estimated based on Boussinesq, Westergaard and 2(vertical):1(horizontal) stress distributions, and was calculated to be about 200 mm with an elastic modulus of 10 MPa under a surcharge of 136 kPa.

5.2 Consolidation settlement

The maximum settlement of layer I in a normally consolidated condition was estimated to be 780 mm. A comparison of a normally consolidated settlement with the results of the instruments (around 580 mm) suggested that the soil was over-consolidated and thus the consolidation parameters needed to be modified.

6. Consolidation time

The consolidation process in the preloading method with PVDs includes vertical and radial consolidation components. The instrument results show that 90% of total consolidation occurred in 220 d. Based on a one-dimensional (1D) consolidation formula, 22% of total consolidation is in the vertical direction and, considering Equation 1, the rest of the settlement (87%) is due to radial consolidation.

1. $$(1 - U) = (1 - U_{\mathrm{v}})(1 - U_{\mathrm{h}})$$

Barron (1948) and Hansbo (1979) developed equations for

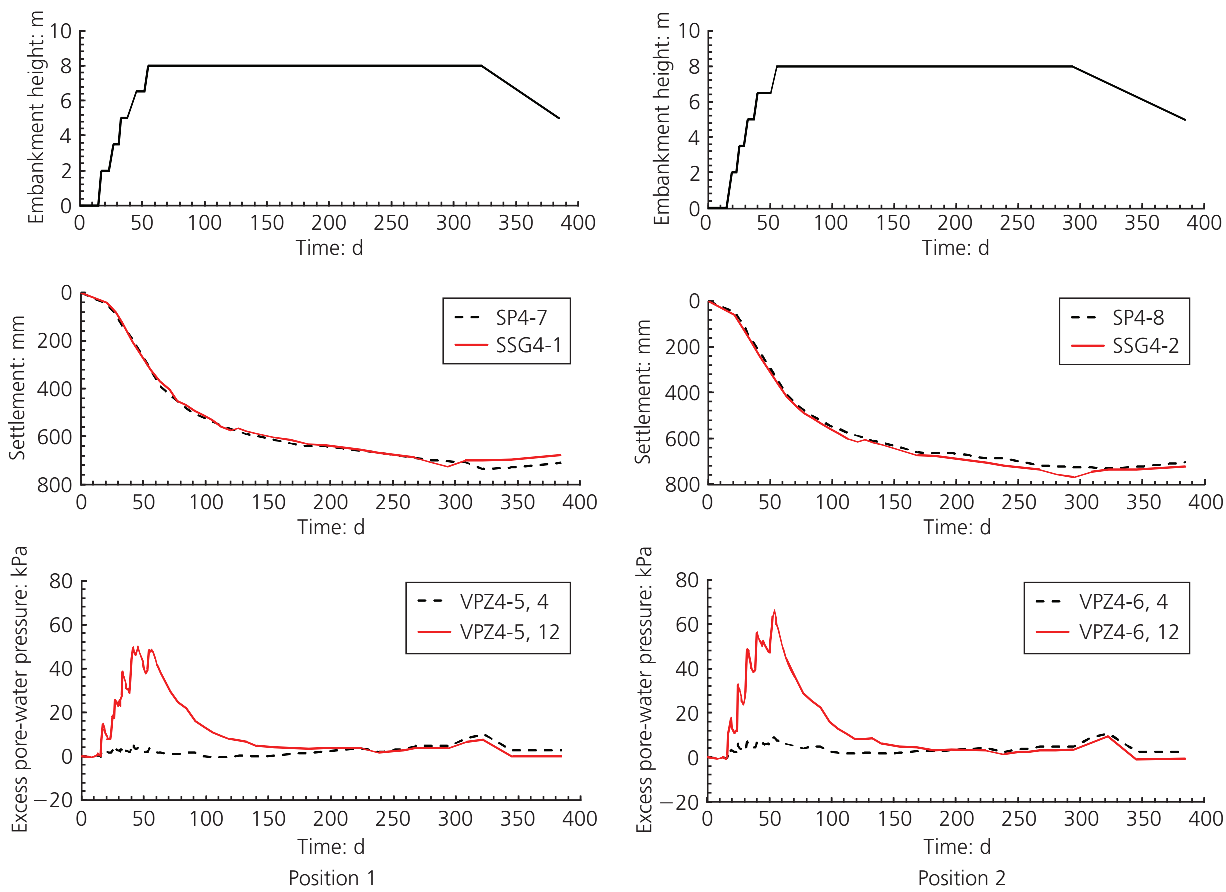

Figure 4. Monitoring results of various instruments and their comparisons at four different positions (continued on next page)

estimating consolidation time when vertical drains are used. Simplifying assumptions related to the mandrel dimensions, characteristics of PVDs and effects of PVD installation are explained in these equations. Despite the fact that much research has been carried out, measurement of C_h, k_s and d_s is still difficult.

Considering the time limits of the project construction and the required depth for improvement, PVDs were installed down to 15 m in a triangular pattern with 1·5 m spacing. The required time for 87% horizontal consolidation without smear effects and well resistance is presented in Table 2.

Figure 5 shows consolidation percentage variations with time based on measured settlements of each instrument. All instruments demonstrate that the rate of consolidation progress has practically reduced to zero after about 300 d. The average trend line shows that 90% of consolidation has occurred after 220 d. In ideal conditions (without smear zone) and based on radial consolidation formula, 87% of consolidation occurs in 147 d. The difference between the calculated 147 d and measured 220 d is an indication that the smear zone has formed. Comparison between the instrument results and the calculations of Table 2 proves that the smear zone has been formed and hence the consolidation parameters need to be revised.

7. Back-analysis interpretations

Comparing the initial estimates of time and maximum settlements with the instrument results clarified that there was no reasonable agreement between estimated magnitudes and site measurements. Thus, the consolidation parameters from geotechnical data were varied in Settle3D software and theoretical formulas to match the consolidation settlement and percentage with the instrument results. Settle3D is a three-dimensional analytical software (published by Rocscience Inc.) for estimating the immediate and consolidation settlements under embankments or surface loads with or without PVDs. Modelling of stage construction and fluctuations of ground water table are feasible in the software too. Based on previous diagrams, the predicted maximum settlement was around 800 mm at 100% consolidation, including immediate settlement and settlements of the layers underlying layer I. The magnetic extensometers show that approximately 25% of total settlement is attributed to the underlying layers including immediate and consolidation settlements.

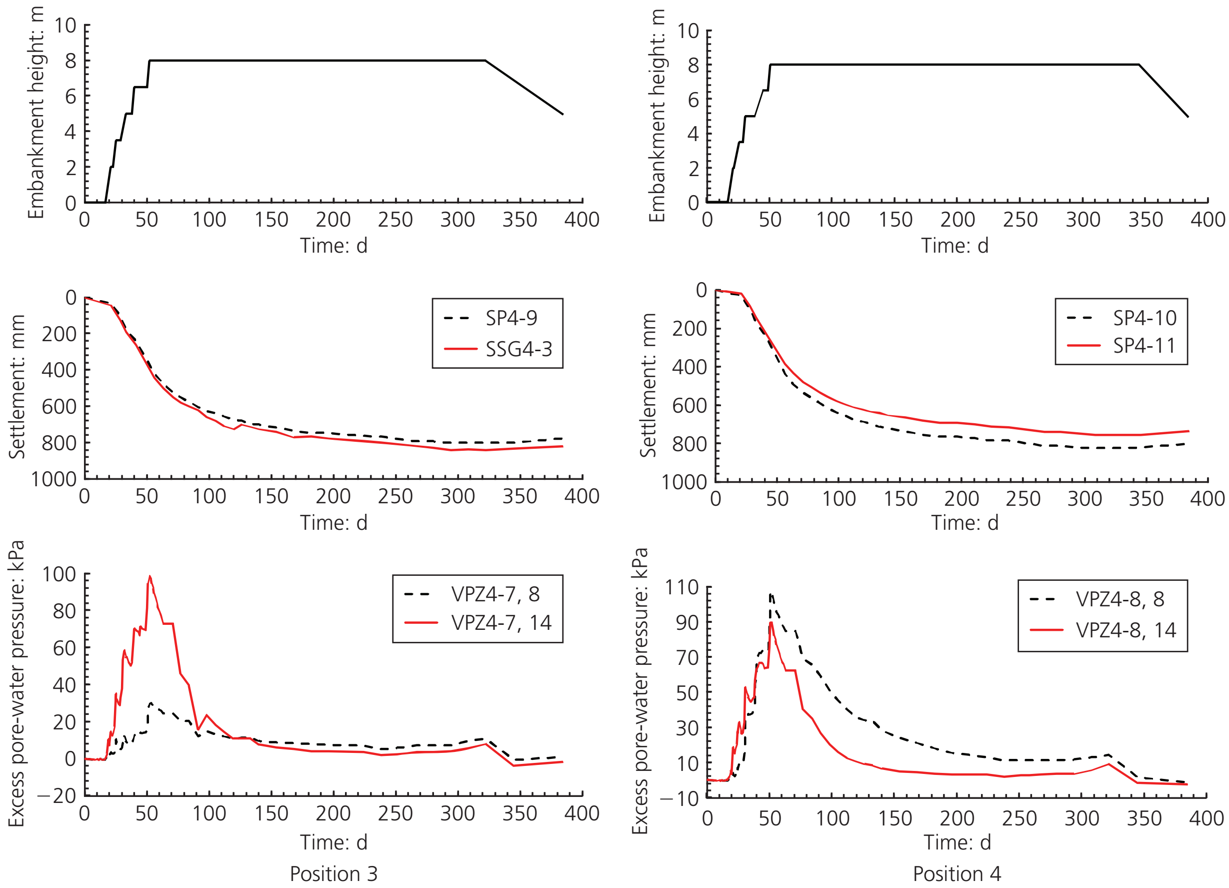

Figure 4. (continued)

PVD dimensions			Mandrel dimension							
a: mm	*b*: mm	d_w: m	*w*: mm	*l*: mm	d_m: m	C_h: m^2/year	*S*: m	D_e: m	U_h: %	*t*: d
100	3·5	0·066	72	142	0·114	3·8	1·5	1·575	87	147

Table 2. Prediction of consolidation time in ideal conditions from conventional theories

The average maximum measured settlement is 760 mm on the 300th day, out of which 580 mm has occurred in layer I. In the back-analysis procedure, layer I was divided into 15 sub-layers. Based on lab consolidation tests on samples recovered from different depths, specific e_0, C_c, C_s and over-consolidation ratio (OCR) were selected for each layer. The parameters were then varied to obtain the best match between theoretical settlements and the results from the instruments. Final parameters of consolidation achieved from back-analysis are presented in Table 3. This table shows that most of the sub-layers of layer I are over-consolidated and hence the assumption of nearly normally consolidated clay in the initial design phase was not correct.

The OCR parameters originally reported by the geotechnical laboratory have in fact been lower than those obtained from the procedure above. This is attributed to the quality of samples taken for the odometer tests. Usually it is difficult to recover good-quality undisturbed samples from soft soils of the upper layers using conventional equipment. Unless special measures are used, for example large block samples, conventional thin-walled samplers have been shown to cause considerable disturbance either during field sampling or removal and handling in the lab.

After adjusting the parameters shown in Table 3, they were used as input for the Settle3D software. Comparison between conven-

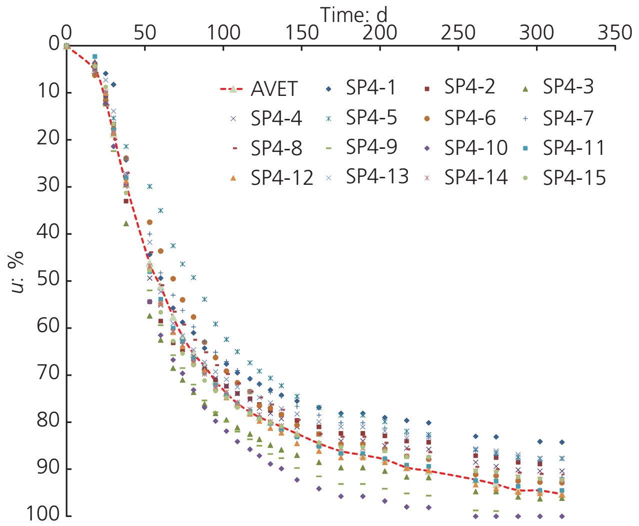

Figure 5. Consolidation percentage variations with time based on different instrument measurements

Layer no.	Z: m	C_c	C_s	e_0	γ_t: kN/m^3	OCR
1 (Surf.)	1	0·20	0·025	0·80	20	8·0
2	1	0·20	0·025	0·80	20	7·5
3	1	0·20	0·025	0·83	20	4·3
4	1	0·22	0·030	0·83	20	2·6
5	1	0·22	0·030	0·83	20	2·6
6	1	0·22	0·030	0·83	20	2·1
7	1	0·22	0·030	0·83	20	2·1
8	1	0·22	0·030	0·80	20	2·1
9	1	0·22	0·030	0·80	20	1·1
10	1	0·21	0·030	0·80	20	1·1
11	1	0·21	0·030	0·80	20	1·1
12	1	0·21	0·030	0·80	20	1·1
13	1	0·20	0·028	0·80	20	1·1
14	1	0·20	0·028	0·80	20	1·1
15	1	0·20	0·028	0·80	20	1·1

Table 3. Soil consolidation parameters resulted from back-analysis

tional theories, Settle 3D and the instrument results of the total settlement of layer I plotted against time are presented in Figure 6. Conventional theories include basic soil mechanics theories of 1D and radial consolidation formulas. The input parameters for both conventional theory and Settle3D methods are the adjusted parameters from instrument results.

The correlations between conventional theories and instrument results during the first 60 d are not good. This is attributed to ignoring the stage construction of the embankment in the theoretical calculations. In conventional theoretical analysis, it is assumed that all of the surcharge pressure is applied at once.

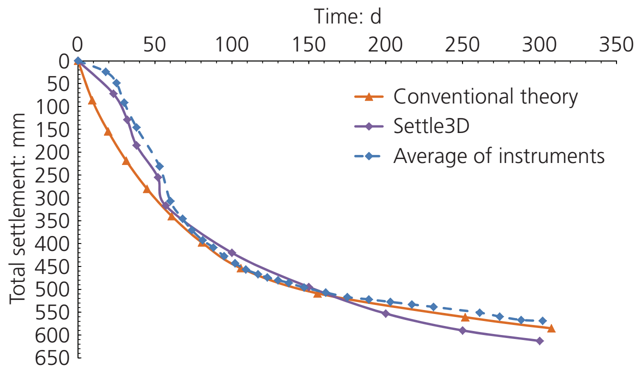

Figure 6. Calculated total settlement of layer I plotted against time based on conventional theory, Settle3D back-analysis and average of instrument results

Therefore, larger immediate and consolidation settlements are obtained during the construction time. Incorporating the stage construction feature in Settle3D software has provided a better match in prediction of the consolidation settlement component (Figure 6).

For calculating the consolidation time, the parameters of the smear zone and horizontal consolidation need to be investigated. Many researchers have studied the size of the smear zone as a function of mandrel dimension. Basu and Prezzi (2007) and Dey (2008) have summarised the previous observations and findings as explained below.

Bergado *et al.* (1991, 1993b), Chai and Miura (1999), Hansbo (1986, 1987, 1997a, 1997b), Holtz *et al.* (1991), Holtz and Holm (1973), Jamiolkowski *et al.* (1983), Madhav *et al.* (1993), Mesri *et al.* (1994) and Onoue *et al.* (1991) suggested the radius of smear zone may vary between one and four times the equivalent mandrel radius. In addition, Barron (1948) showed that if the thickness of smear zone was 1/6 of the drain radius, the time required to achieve a specific degree of consolidation would be increased by about 20%. If the thickness of the smear zone is increased to twice the drain radius, then consolidation time doubles. Cassagrande and Poulos (1969) also found that the permeability of the smear zone could be less than 1/10 of the undisturbed soil and even possibly as little as 1/1000.

To apply the effect of smear zone in this study, back-analysis was used in ranges of 1–3 for k_h/k_s and d_s/d_m and 0·5–3 for C_h/C_v based on previous research and other field experiences. Owing to high discharge capacity and short length of PVDs, the well resistance is ignored in the consolidation time calculations. The predicted consolidation percentage variations with time using different methods (Settle3D and conventional theories) and their comparisons with average measured results are shown in Figure 7. The 'prediction with smear zone' has shown the best match. Table 4 describes the back-calculated parameters for proper consolidation–time estimates.

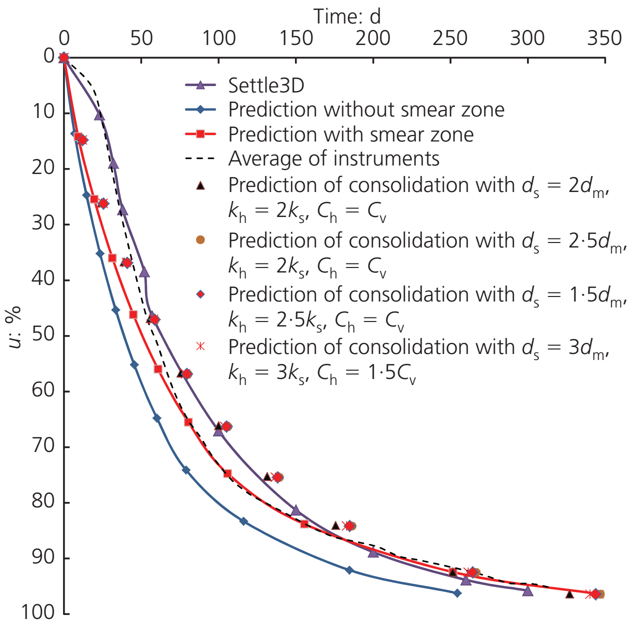

Figure 7. Consolidation percentage plotted against time based on conventional theory, Settle3D and average of instrument results

U_h: %	C_h/C_v	k_h/k_s	d_s/d_m
10	1·1	1·5	2
20	1·1	1·5	2
30	1·1	1·5	2
40	1·1	1·5	2
50	1·1	1·5	2
60	1·1	1·5	2
70	1·1	1·5	2
80	1·0	1·5	2
90	0·9	1·5	2
95	0·85	1·5	2

Table 4. Back-calculated time-related consolidation parameters of layer I

It should be remembered that a reasonable ratio adopted for either of k_h/k_s or C_h/C_v could contribute to proper estimation of consolidation time. It is noticed, however, that proper selection of C_h/C_v is more effective in calculating adequate variations of consolidation time.

8. Practical implication

The proper estimation of d_s and k_s before starting the construction is difficult. The question then arises of how could the end-of-consolidation time be estimated beforehand. On the basis of the results presented in this paper, it is proposed to adopt d_s and k_s from previous local experience. At the end of embankment construction, d_s and k_s have to be adjusted using the back-calculation from instrument results to fine tune the end-of-consolidation time estimate. Figure 7 shows that the U%–time curves estimated from back-calculation show a good correlation with instrument measurements after about 50% of consolidation is completed. Therefore, the interpretations of initial construction stage instrument results and back-calculations can be used to more precisely estimate the 80% or 90% consolidation time. If a surcharge of over 35% of the design pressure is applied, practically achieving 80% consolidation shall be equivalent to the end-of-consolidation for the design load.

9. Application of Asaoka method

In this section, the end-of-consolidation settlement and C_h are estimated by the Asaoka method. The Asaoka method (Asaoka, 1978) can be used for the prediction of the maximum consolidation settlement and C_h based on field settlement observations. This method is simple, but uncertainties are involved in the predictions because the soil properties and field pore pressure variations are not included. According to Asaoka's theory, 1D consolidation settlements at certain time intervals are described as shown in Equation 2.

$$2. \quad S_i = \beta_0 + \beta_1 S_{i-1}$$

where S_1, S_2, . . ., S_i are settlements measured by instruments. S_i presents the settlement at time t_i. All the settlement data are displayed in the form of S_i plotted against S_{i-1} diagrams and then trend lines are plotted. The values of β_0 and β_1 are identified by intercept of the trend line with the S_i axis and the slope, respectively. The maximum settlement and C_h are shown in Equations 3 and 4 (Asaoka, 1978).

$$3. \quad S_{max} = \beta_0/(1-\beta_1)$$

$$4. \quad C_h = (1-\beta_1)D_e^2\mu/(8\beta_1\Delta t)$$

where $\Delta t = (t_i - t_{i-1})$ is time interval and $\mu = [\ln(D_e/d_w) - 0{\cdot}75] + [(k_h/k_s) - 1]\ln(d_s/d_w)$

For this project, graphs of S_i plotted against S_{i-1} are drawn in 90, 80, 70 and 60 consolidation percentages for each settlement instrument. These diagrams show in what consolidation percentage the Asaoka method is more accurate. Figure 8 illustrates Asaoka graphs for the average settlements of all instruments (Avet) for different consolidation percentages. Table 5 shows results of the Asaoka method for a maximum settlement (S_{max}) and C_h. For calculation of C_h, it is assumed that d_s and k_h are 2 and 1·5 times of d_m and k_v, respectively, and C_h is constant across all the layers.

Table 5 shows that the suitable consolidation percentages for prediction of S_{max} and C_h are 80 and 90% for the soil of the study

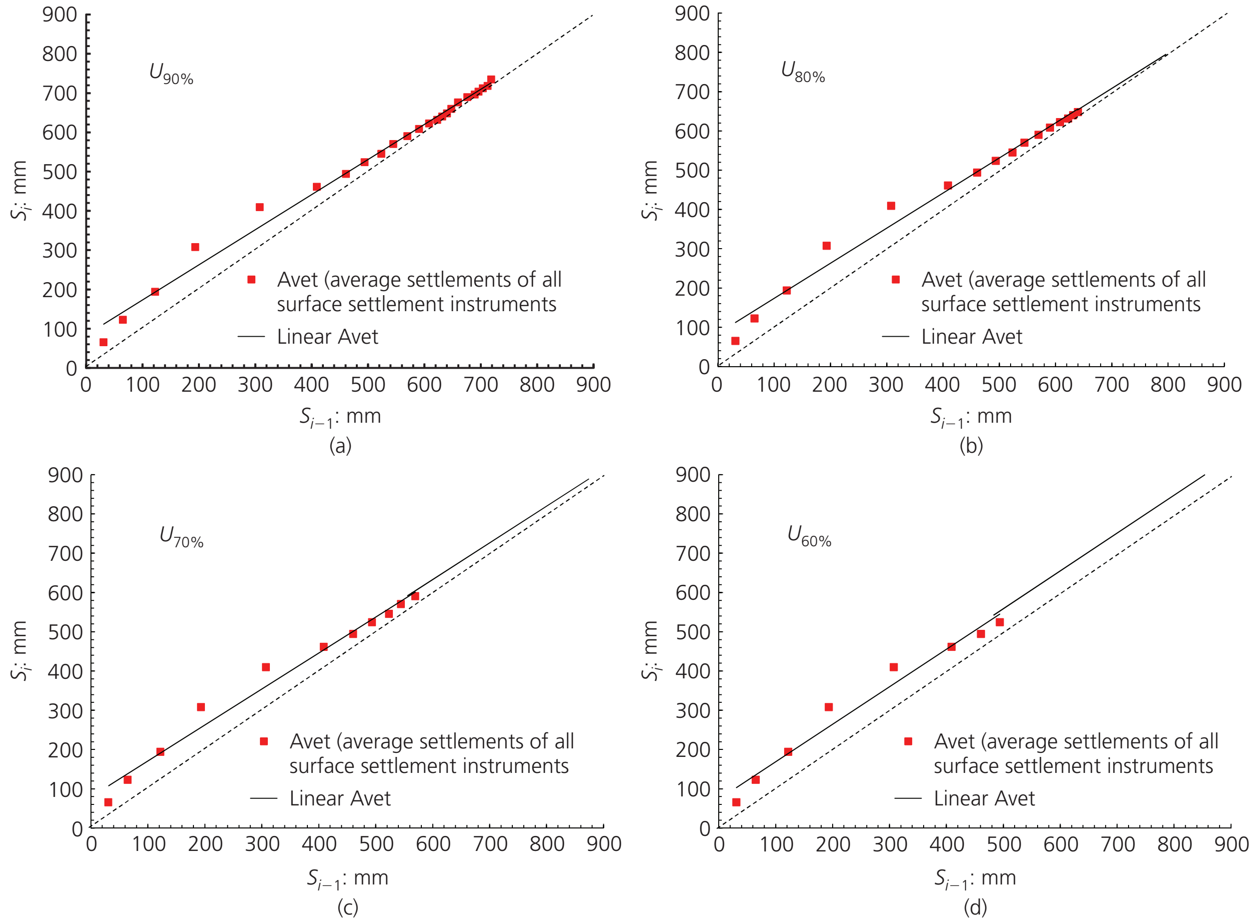

Figure 8. Prediction of final settlement and C_h based on Asaoka method in: (a) 90%; (b) 80%; (c) 70%; (d) 60% consolidations

site. The S_{max} values for 60 and 70% consolidations shown in Table 5 are not reliable values.

10. Conclusion

The results of a well-instrumented highly supervised preloading project with vertical drains in the southwest of Iran are presented. Applied stress of embankment was 136 kPa constructed in five stages. Length and spacing of PVDs were 15 and 1·5 m, respectively. Results of all instruments are shown and compared with theoretical formulas, Settle3D software and the Asaoka method.

U_h: %	S_{max}: mm	C_h: m^2/year
60	1384	1·9
70	844	3·2
80	761	3·7
90	759	3·7

Table 5. Prediction of S_{max} and C_h by Asaoka method

In addition, consolidation parameters and smear effects are investigated. The most important findings of this case study can be summarised as follows.

- Trend and magnitudes of all settlement instruments (magnetic extensometers and surface settlement monuments) have shown reasonably good agreement with each other.
- Magnetic extensometers show that most of the settlement has occurred within compressible layer I, as expected. Approximately 25% of total settlement is attributed to underling layers, including immediate and consolidation. This ratio is observed to be applicable during the construction stages too.
- The 4 m deep piezometers did not measure noticeable excess pore pressure during filling. The over-consolidation effects of upper layers near the surface have contributed to this observation, as revealed from back-calculations.
- If soil parameters are selected adequately, especially OCR, conventional theoretical formulas and Settle3D provide reasonable estimates of the settlements and consolidation time.

- Based on the Asaoka method, prediction of S_{max} and C_h are reasonable when 80% of the consolidation has occurred.

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