Jordan Journal of Civil Engineering, Volume **18**, No. **3**, **2024**
DOI: https://doi.org/10.14525/JJCE.v18i3.01

Jordan Journal of Civil Engineering

Journal homepage: https://jjce.just.edu.jo

Consolidation Settlement by Observational and Conventional Methods Utilizing Results of Test Embankments from Algeria

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ARTICLE INFO

Article History:
Received: 7/11/2022
Accepted: 24/4/2024

ABSTRACT

The aim of this work is to predict the ultimate primary settlements of a highway embankments using four different methods. These methods include the Terzaghi, Asoaka, Chunlin and hyperbolic methods. Field data from three monitored test embankments in Algeria was used to assess the performance of each method in predicting ultimate primary settlements. Results indicated that the Terzaghi method, relying on laboratory-determined coefficient of consolidation, notably underestimated the intermediate settlement and overestimated the time required to achieve a 90% consolidation in comparison to the other methods. This underscores a significant limitation in solely employing laboratory data. Conversely, the Asaoka, Chunlin and hyperbolic methods, incorporating field measurements, provided more accurate predictions of both settlement and coefficient of consolidation. This study underscores the importance of employing field-based methods to obtain accurate coefficient of consolidation values, thereby enhancing the reliability of consolidation-settlement predictions for the engineering design of highway embankments.

Keywords: Initial settlement, secondary settlement, Consolidation settlement, Test embankments.

INTRODUCTION

Settlement of cohesive soils has always been an issue that engineers face in many projects of ground works. Predicting and calculating this settlement can be a challenging task due to the many variables involved, such as soil type, moisture content and loading conditions. Furthermore, cohesive soils exhibit time-dependent behavior, making it even more challenging to estimate their settlement accurately.

Terzaghi et al. (1996) developed a comprehensive method for predicting the ultimate primary settlement of cohesive soils. The consolidation settlement is the most challenging term to estimate, since it is dependent on time and occurs over a long period for cohesive soils. While this term has been estimated using traditional methods, recent studies have proposed novel approaches to improve the prediction accuracy.

For instance, Asaoka (1978) proposed an observational procedure to predict soil ultimate primary settlement and the *in-situ* coefficient of consolidation (Shukla et al., 2009; Mesri and Huvaj-Sarihan, 2009). Meanwhile, Tan et al. (1994, 1995, 1996) developed the hyperbolic method based on the rectangular-hyperbola method proposed by Sridharan et al. (1987). Li (2014) introduced a novel method for estimating settlement over time by introducing a new term, the potential settlement. In addition, the inflection-point method originally proposed by Cour (1971) has been extended by Sinha et al. (2007) to include soils treated with pre-fabricated vertical drains (PVDs). Additional contributions include the slope and settlement rate-settlement methods, which were reported by Al-Zoubi (2013). In addition, Guo et al. (2018 a) proposed a method for predicting soil ultimate primary settlement by fitting the settlement curve to the Chapman-Richards equation using linear regression. More recently, Guo et al. (2018 b) published a modification of the hyperbolic method by presenting a new chart.

This paper presents a comparative study of four different methods for estimating the ultimate primary settlement of highway test embankments. The methods considered are the classic Terzaghi method (Costet and Sanglerat, 1981; Holtz et al., 1981), the Asaoka method (1978), the hyperbolic method proposed by Tan (1995) and the method introduced by Li (2014). The study is conducted using observational data obtained from multiple cases of soil settlements under embankments.

Table 1. Key equations from the literature

Eq N°	Expression	Symbols
1	$\delta_{tot} = \delta_i + \delta_c + \delta_s$	
2	$\delta_i = \frac{q}{E} \cdot B \cdot I \cdot (1 - \nu^2)$	ν: Poisson's ratio B: total width of the loaded area I: influence factor. (Lutenegger et al., 1995)
3	$\begin{cases} E = \frac{2}{3} E_{oed} \\ E_{oed} = \frac{1+e}{C_c} \cdot \frac{\Delta\sigma}{log\left(1 + \frac{\Delta\sigma}{\sigma}\right)} \end{cases}$	σ: initial stress at h/2 of the compressible layer C_c: compression index e: void ratio (Costet and Sanglerat, 1981)
4	$\mathrm{E_{oed}} = 2.3 \cdot \sigma \cdot \frac{1+e}{C_c}$; when $\Delta\sigma$ is so small compared to σ	
5	$\delta_s = \frac{C_\alpha}{1 + e_0} H \cdot \log\left(\frac{t}{t_{100}}\right)$	t_{100}: time to end of primary consolidation C_α : creep coefficient H : thickness of the compressible layer $C_\alpha / C_c = 0.04 \pm 0.01$ (Mesri et al., 1985).
6	normally-consolidated soils ($\sigma'_{v0i} = \sigma'_{pi}$): $\delta_c = C_{ci} \frac{H_i}{1 + e_{0i}} log\left[\frac{\sigma'_{v0i} + \Delta\sigma_i}{\sigma'_{v0i}}\right]$	C_{ci}: compression index for layer i C_{si}: recompression index for layer i e_{0i}: void ratio for layer i $\Delta\sigma_i$: stress due to embankment load in the i[th] layer σ'_{v0i} : effective stress in the i[th] layer σ'_{pi} : pre-consolidation stress in layer i $\Delta\sigma = i \cdot q$ and $q = \gamma_r \times h_r$ h_r: embankment height γ_r: density of the embankment material I: influence coefficient for vertical stress under a very long embankment (Holtz et al., 1981)
7	over-consolidated soils: $\delta_c = C_{si} \frac{H_i}{1 + e_{0i}} log\left[\frac{\sigma'_{v0i} + \Delta\sigma_i}{\sigma'_{v0i}}\right]$; when $\sigma'_{v0} + \Delta\sigma_i < \sigma'_{pi}$	
8	over-consolidated soils: $\delta_c = C_{si} \frac{H_i}{1 + e_{0i}} log\left[\frac{\sigma'_{pi}}{\sigma'_{v0i}}\right] + C_{ci} \frac{H_i}{1 + e_{0i}} log\left[\frac{\sigma'_{v0i} + \Delta\sigma_i}{\sigma'_{pi}}\right]$ when $\sigma'_{v0} + \Delta\sigma_i > \sigma'_{pi}$	
9	$\delta_{n+1} = \beta_0 + \beta_1 * \delta_n$	β_0: the intercept

10	$\delta_i = \delta_{ult} - (\delta_{ult} - \delta_j) \times \beta_1^{i-j}$	β_1: slope of the resulted linear segment
11	$\delta_i = \beta_1^{i-j} \times \delta_j + \beta_0 \times \frac{1 - \beta_1^{i-j}}{1 - \beta_1}$	δ_{n+1} : settlement at time n+1 δ_n : settlement at time n δ_j: settlement at reference point time j δ_i: targeted settlement at time i
12	$C_v = -\frac{H^2 \ln \beta_1}{6\,\Delta t}$	C_v: coefficient of consolidation Δt: time interval between successive settlement data H: soil-layer thickness
13	$T_v = \frac{C_v}{H'^2} t$	H': drainage length

Theories on Settlement

When a soil deposit is loaded, such as an embankment construction, deformations will occur. The overall vertical displacements at the surface caused by this loading are termed "settlement" (Holtz et al., 1981). According to the theory of Terzaghi, the total or final settlement (δ_{tot}) of a soil consists of consolidation settlement (δ_c), secondary settlement (δ_s) (Chen et al., 2021) and immediate settlement (δ_i) (Holtz et al., 1981), noting that δ_{tot} can be expressed as in Eq.1 (Table 1). The immediate-settlement calculations are based on the elasticity theory and can be influenced by multiple factors as reported by (Pantelidis, 2020). In this paper, the immediate-settlement expression used by Skempton and Bjerrum (1957) is adopted (Eq.2, Table 1).

The phenomenon of secondary settlement is a deferred deformation over time which could be significant (Mendoza et al.., 2022), particularly occurring after the completion of primary consolidation (Oliveira et al., 2023). According to Terzaghi et al. (1996), the secondary settlement is given by the logarithmic formula in Eq.5 (Table 1). This work focuses particularly on the consolidation settlement, which is more difficult to estimate accurately, as it depends on time (Chung and Kweon, 2021). In one-dimensional consolidation, according to the stress-strain curve, δ_c can be calculated using equations Eq.6, Eq.7 and Eq.8 (Table 1) (Costet and Sanglerat, 1981; Feng et al., 2020).

Consolidation Settlement According to the Asaoka's Method

Asaoka's method has been studied and compared to other methods, especially by Tan and Chew (1996) and Li (2014). It is considered an "observational procedure" which uses the data of the first settlements to predict the ultimate primary settlement and the *in-situ* one-dimensional consolidation coefficient C_v. This method is based on successive settlements $\delta_0, \delta_1, \delta_2$ ...etc., respectively at times 0, Δt, $2\Delta t$, ... etc., where δ_0 is the initial settlement. Asaoka (1978) expressed the graphical representation of the relationship between successive settlements as in Eq.9 (Table 1).

While the ultimate settlement δ_{ult} represents the stable settlement state; $\delta_{n+1} = \delta_n = \delta_{ult}$, it can be predicted using Eq.14. Hence, the settlement at any given time, t, is estimated by Eq.15.

$$\delta_{ult} = \frac{\beta_0}{1 - \beta_1} \quad (14)$$

$$\delta(t) = \left(\frac{\beta_0}{1 - \beta_1}\right) - \left[\frac{\beta_0}{1 - \beta_1} - \delta_0\right]\beta_1^j \quad (15)$$

In Eq.15, δ_0 must be determined first. Different values of δ_0 lead to different values of $\delta(t)$. This is why the accuracy strongly depends on the initial-time choice. However, the initial-time determination is different according to various authors, which can cause dissimilarity in settlement estimation. The settlement-prediction accuracy is significantly influenced by the time-increment (Δt) selection. Furthermore, accurate predictions are achieved by using settlement data from higher consolidation degrees and by employing larger time increments (Huang et al., 2021).

Consolidation Settlement According to the Hyperbolic Method (Tan, 1995)

According to Tan and Chew (1995), the hyperbolic method is a technique that has its origins in the rectangular-hyperbolic method proposed by Sridharan et al. (1981, 1987), in which the consolidation coefficient C_v is computed from oedometric tests by adjusting the laboratory-consolidation results to the linear segment between U_{60} and U_{90} points of the theoretical hyperbolic curve $T_v/U = f(T_v)$ (T_v: *vertical factor of time*).

Furthermore, in the consolidation theory of Terzaghi (1943), there is a single graph expressing the average degree of consolidation as a function of time $U = f(Tv)$. The graph connecting T_v / U and T_v (Fig.1) shows a first part concave up to U_{60}, followed by a linear segment between U_{60} and U_{90}. This linear part can be expressed by an equation of a rectangular hyperbola (Tan and Chew, 1996):

$$\frac{T_v}{U} = \alpha T_v + \beta \tag{16}$$

where: α is the slope of the segment between U_{60} and U_{90}, β is the intercept at the origin and U is the consolidation degree in both vertical and horizontal directions.

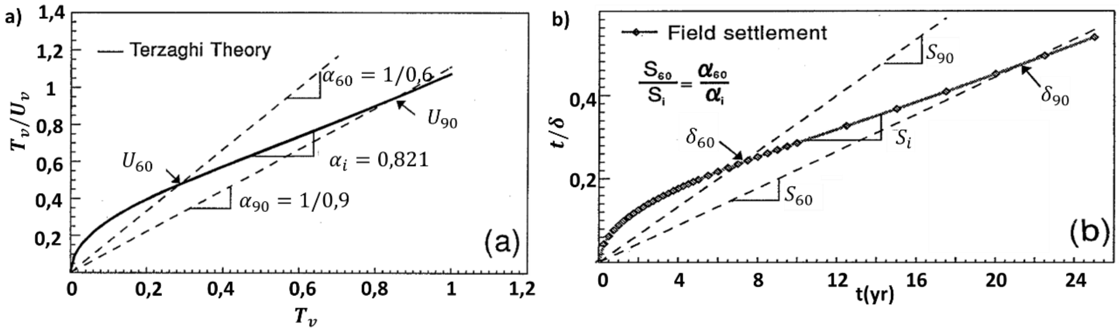

Figure (1): Hyperbolic curves of Terzaghi (a) and *in-situ* measurements (b) (Tan and Chew, 1996)

Tan and Chew (1995) have also shown that *in-situ* measurements of soil settlement as a function of time, represented in Eq.17 as $t/\delta = f(t)$ (Fig.1 b) showed the same shape as the curve of Terzaghi (Fig.1 a). Consequently, this property can be extrapolated to identify 60% and 90% of settlement levels for any settlement dataset. The slopes s_{60} and s_{90} of the radiating segments to the 60% and 90% settlement points can be calculated using Eq.18 and Eq.19, respectively.

$$t/\delta(t) = s_i \times t + \beta' \tag{17}$$

$$s_{60} = s_i \frac{\alpha_{60}}{\alpha_i} \tag{18}$$

$$s_{90} = s_i \frac{\alpha_{90}}{\alpha_i} \tag{19}$$

where: s_i is the slope of the initial linear segment of the graph $t/\delta = f(t)$ and α_i is the slope of the theoretical graph $T_v/U = f(T_v)$ (Fig.1 a).

Tan and Chew (1995) have shown that the ultimate primary settlement can be predicted as follows:

$$\delta_{ult} = \frac{\alpha_i}{s_i} = \frac{\delta_{60}}{0.6} = \frac{\delta_{90}}{0.9} \tag{20}$$

Nevertheless, according to Li (2014), the limitation of the hyperbolic method is in the determination of the initial time, since this method is based on the initial slope of settlement. The difference in initial time may have the effect of the difference in settlement. Another limitation is that the slope is not entirely constant between 60% and 90% consolidation as reported by Aparna et al. (2019) and thus, the accuracy is not guaranteed (Huang et al., 2021). The constant-load condition has been assumed in the hyperbolic method, because during the loading period, the settlement rate is variable and the initial slope cannot be judged. Hence, the settlement before the end of loading cannot be predicted.

Consolidation Settlement According to Chunlin's Method (Li, 2014)

Li (2014) criticizes the Asaoka (1978) and Tan (1995) methods, particularly the choice of initial time, which he found subjective. Li (2014) starts with calculating the settlement over time from the one-dimensional consolidation equation of Terzaghi by writing:

$$\delta_t = \delta_{ult}\left(1 - \frac{8}{\pi^2} e^{bt}\right) \tag{21}$$

where: b is an unknown parameter that can be obtained graphically or from the Asoaka (1978) method.

Also, in order to simplify calculations, Li (2014) introduced a new variable called « potential settlement » as:

$$\ln\left(\delta_p \frac{\pi^2}{8\delta_{ult}}\right) = b \times t \tag{22}$$

where: δ_p is the potential settlement to take place in the future ($\delta_p = \delta_{ult} - \delta(t)$).

Based on the expression in Eq.22, the relationship $\ln\left(\delta_p \frac{\pi^2}{8\delta_{ult}}\right)$ *vs.* time yields a straight line of slope b. According to Li (2014), parameter b represents the drainage condition in Terzaghi's consolidation equation. It can be obtained using the coefficient of consolidation C_v and the length of drainage according to several types of drainage (Table 2).

Table 2. Mathematical expression of b under different drainage conditions (Li, 2014)

Type of drainage	Mathematical expression
Radial	$8C_h/F(n)d_e^2$
Vertical	$\pi^2 C_v/4H^2$
Vertical-Radial	$\frac{8C_h}{F(n)d_e^2} + \pi^2 C_v/4H^2$

Note: C_h and C_v are the radial and vertical consolidation coefficients, respectively, H is the vertical drainage length, n is the ratio of d_e to d_w, d_e is the effective diameter of well, d_w is the diameter of well and F is an unknown coefficient correlated with n.

Studied Cases

Algeria has recently initiated an ambitious program concerning basic infrastructure equipment like motorways and railways. So far, the biggest project in this grandiose program is unquestionably the East-West Highway. This strategic motorway with a length of 1216km connects the Algerian-Tunisian border to the Moroccan-Algerian border. This is the case of the economic pole of Bejaia and its main port on the Mediterranean coast. To cater to the fast-growing region, a project to construct a nearly 100km-long penetrating motorway is currently underway (Fig.2). In this paper, three different cases were studied, the first and the second cases are both part of the project of the highway penetrating "Bejaia", while the third case is for a railway embankment in the project of the line 'Blida-Khemis Meliana'.

Two of the studied cases concern a highway of "Bejaia" province; both cases are situated within the section that follows the "Soummam" river and passes through swampy agricultural areas requiring, sometimes, the use of massive backfill. The main problem posed by these embankments is the settlement of the foundation soils. This is why test embankments were carried out and instrumented over a period of 360 days for the first case (PK 12+970) and over a period of 250 days for the second case (PK 13+170); in order to study the settlement evolution.

The third case concerns an embankment of a railway line, the 'Blida-Khemis Meliana' line, passing through the 'Cheliff basin.' The studied section is situated in an agricultural zone, where the groundwater table is at a depth of 15 meters. Settlement under the embankment at PK 122+180 was monitored for 200 days.

Layout Characteristics

for the first two cases, the studied segments are parts of section 1 (Pk0 to Pk21+400) between the port of "Bejaia" and the East-West Highway (Fig. 2). Moreover, the considered section crosses an agricultural wetland.

The layout of the lane runs entirely in the embankment; hence, the unique symmetrical cross-type profile is shown in Fig. 3.

Experimental Embankments

To facilitate a comparative analysis of consolidation settlement, test embankments were constructed and monitored for varying durations at each study location: 360 days for the first case (PK12+970), 250 days for the second case (PK13+170) and 180 days for the railway embankment at PK122+180. The settlement data collected from these instrumented embankments is illustrated in Fig.6.

The dimensions of the studied embankments are presented in the diagram of Fig.3. The instruments used for the 360-day survey are presented in Fig. 4.

The geotechnical *in-situ* survey has been carried out at Pk12+970. It mainly consisted of reconnaissance wells, core drills, piezometric drills and pressure-meter and penetrometer tests. The foundation is mainly composed, top-down, of:

- A thin layer of topsoil with a thickness of 0 to 1m.
- A layer of wet and slightly plastic clay with a variable thickness from 1m to 3m.
- A layer of saturated and poorly plastic clay exceeding 15m of thickness.

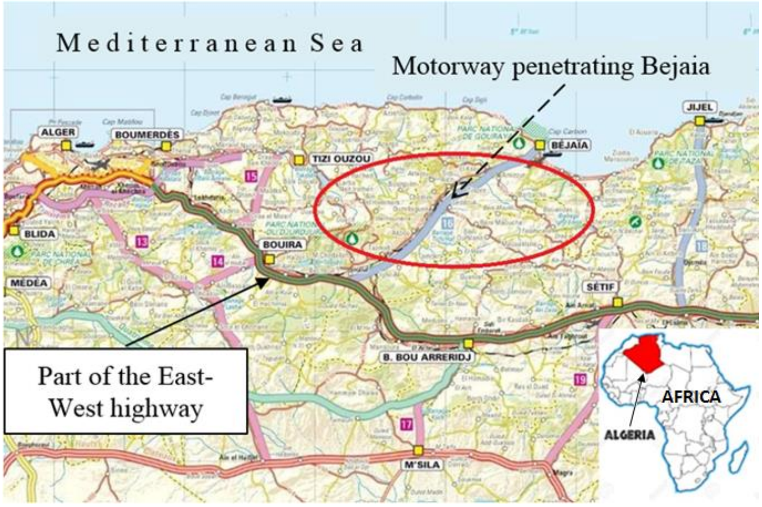

Figure (2): Synoptic map of the motorway project penetrating Béjaia

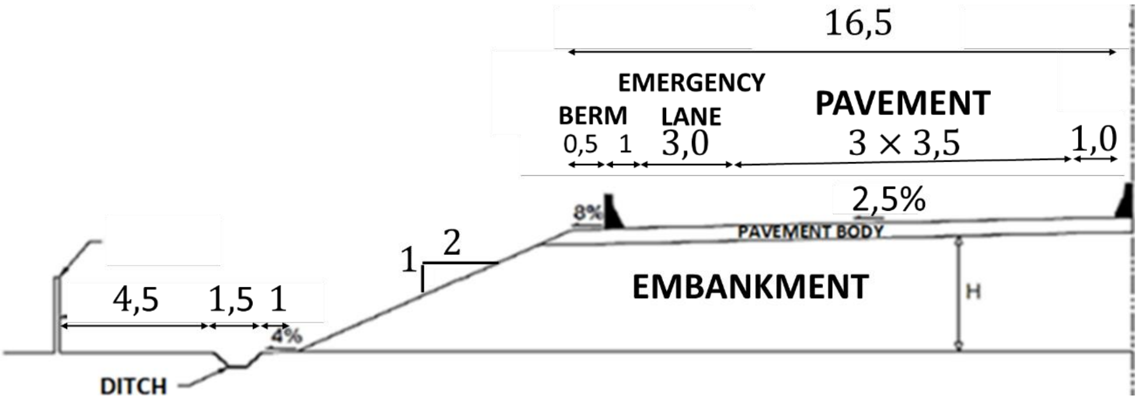

Figure (3): Typical cross-section of the project "Bejaia" highway

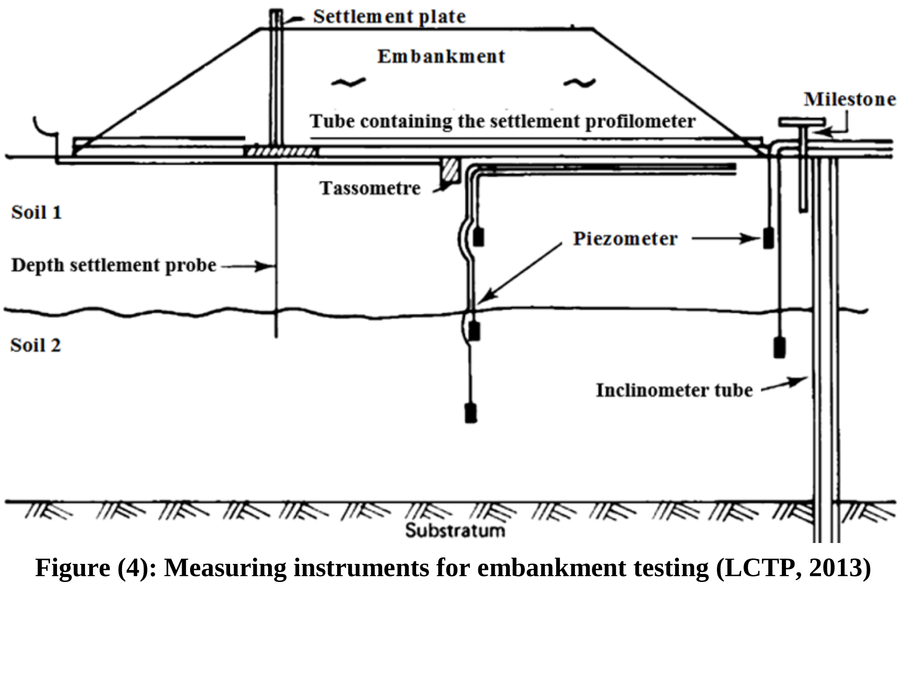

Figure (4): Measuring instruments for embankment testing (LCTP, 2013)

Table 3. Results of the main tests on the ground under the embankment test

Exploration wells (Bejaia Highway project)				
well N°	PK	Depth (m)	Description	Compressibility
RSC12-4	PK12+925	0-1	Vegetal soil	/
		1-15.6	Poorly plastic clay	Moderately compressible
Penetration test using core-drilling machine				
well N°	PK	Depth (m)	Description	
SPT12-4	PK12+925	0-1	Vegetal soil	
		1-15.6	4-6	Loose
Water-table depth found at 2.16m				
Exploration wells (case of Railway line)				
well N°	PK	Depth (m)	Description	Compressibility
SS70	PK122+180	0-2.5	Compressible clay	/
		2.5-4	Firm clay	/
		4-6.3	Moderately firm clay	/
Water-table depth found at 15 m				

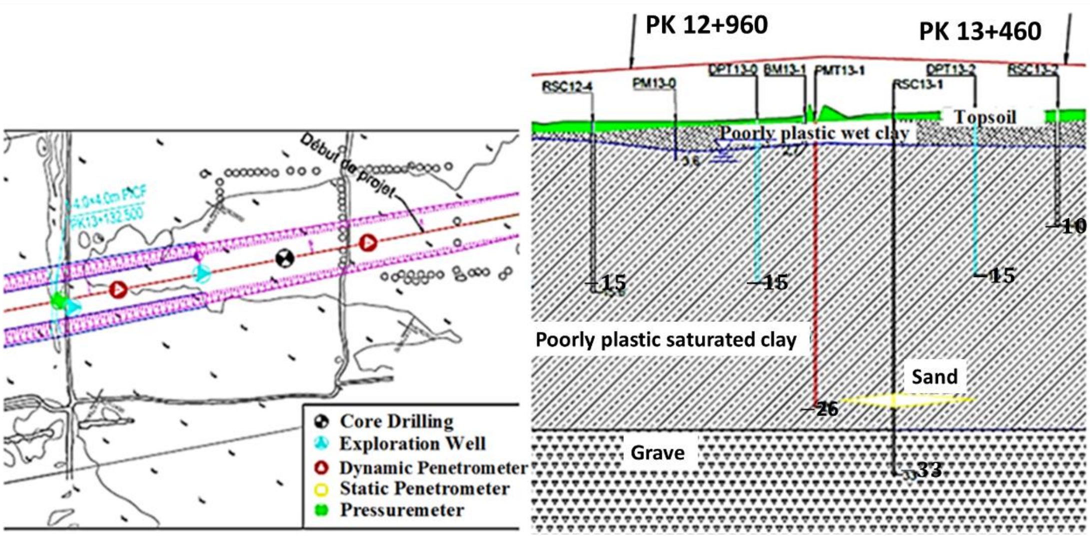

Figure (5): Plan projection (left) and longitudinal profile (right) for conducted tests at PK13+000

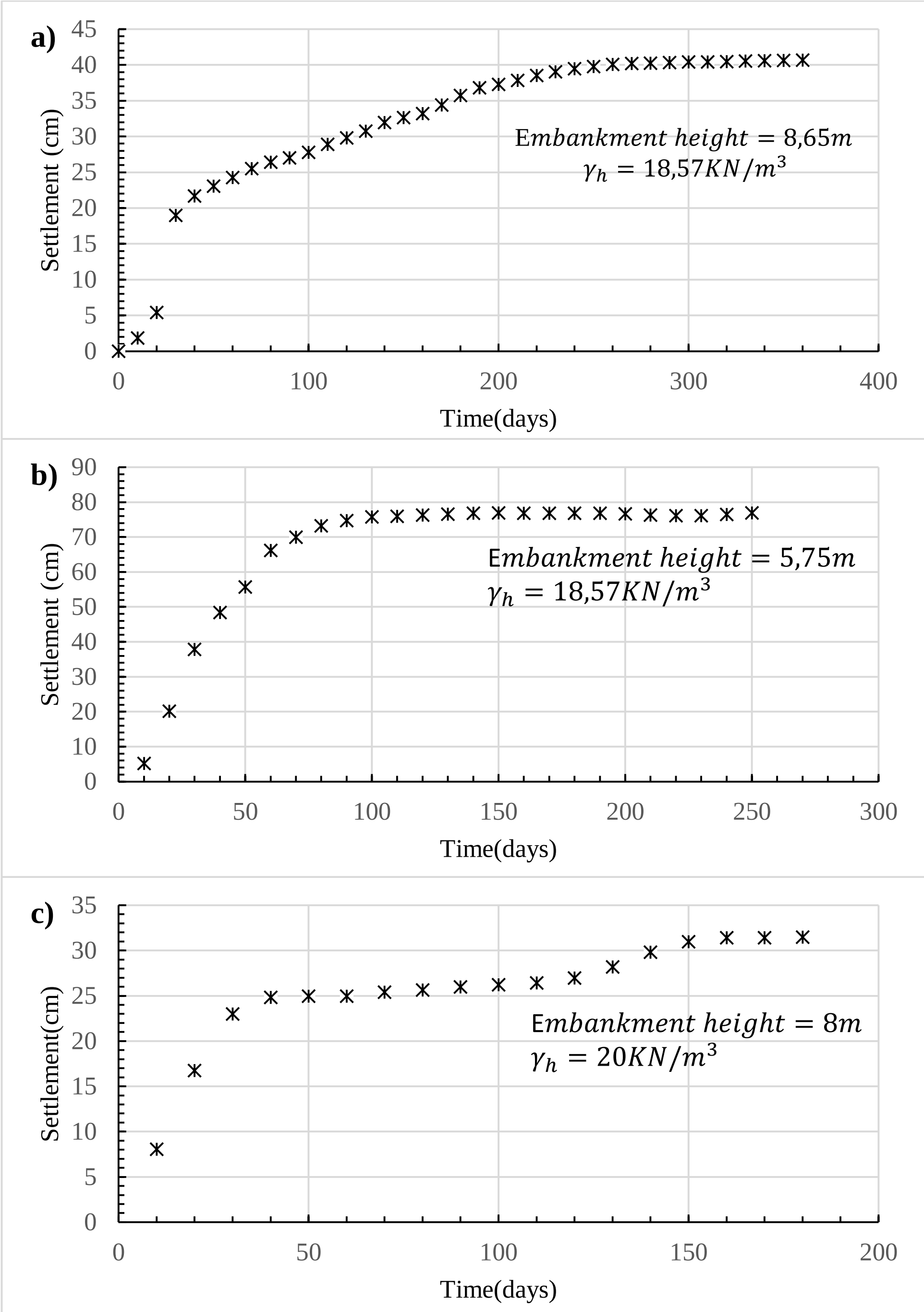

Figure (6): Settlement-observation curves at a) PK 12+970 b) PK 13+170 and c) railway PK122+180

Settlement Predictions

Terzaghi (1943) Method

The consolidation settlements for all cases were computed based on the parameters in Fig.7, Equations from Eq.1 to Eq.8 and by considering the correction of Skempton and Bjerrum (1957) μ=0.7 for both cases of Bejaia highway (PK 12+970 and PK 13+170) and a correction coefficient of 0.8 for the third case of railway embankment at PK 122+180. This was found at 25.55, 59.01 and 18.64 for embankment of PK 12+970, PK 13+170 and PK 122+180, respectively.

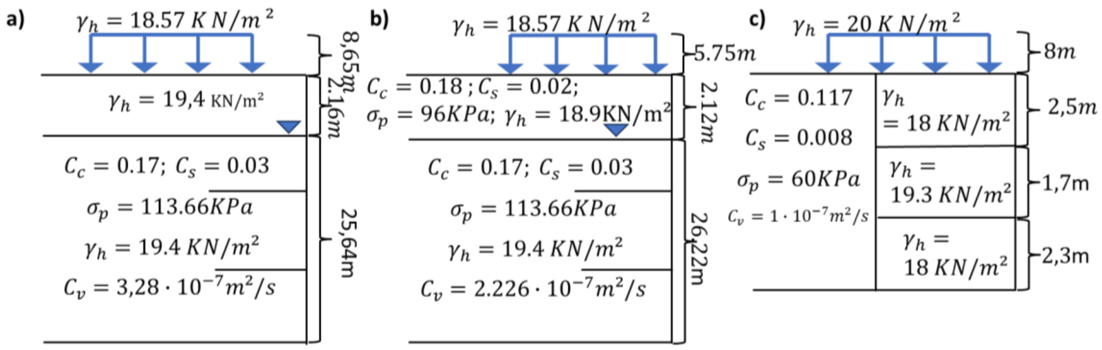

Figure (7): Embankment profiles a) Bejaia highway (PK 12+970).
b) Bejaia highway (PK 13+170) and c) Railway (PK 12+970)

The consolidation time factor (T_v) for each studied case can be calculated from Eq.13 (Table 1) using C_v values obtained from oedometer laboratory measurements presented in Fig.7. T_v values were calculated for each studied case at the time corresponding to the end of field measurements, Notably, all cases exhibited T_v values of less than 0.2. Consequently, Casagrande's approximation (Eq.23) was employed to estimate the degree of consolidation for each case. Therefore, the settlement value for each case was estimated using Eq.24, for the first case (PK 12+970) T_v(360days) = 0.024 and the estimated settlement at 360 days is 19 cm. As for the second case (PK13+170) T_v(250days) = 0.024. Therefore, δ (250) =22cm. Finally, for the third case (railway embankment at PK122+180), the calculated settlement at 180 days was found to be 12.3cm.

$$U = 1.128\sqrt{Tv} \tag{23}$$

$$\delta_{360} = U \times (\delta_c) + \delta_i \tag{24}$$

However, employing coefficient of consolidation values that better align with the observed settlement curves enhances the estimation accuracy; for demonstration purpose, average values were used. In particular, for the first case with a C_v value of 0.86 m²/days, the estimated primary settlement at 360 days is 37.55cm. In the second scenario, utilizing a C_v value of 2.5m²/days yields an estimated primary settlement of 70.68 cm at 250 days. In the third case, a C_v value of 0.255 m²/days results in an estimated primary settlement of 25.86 cm.

Asaoka's Method (Asaoka, 1978)

Fig.8 shows the relationship between successive settlements δ_{n-1} and δ_n obtained from field observations. Asaoka's method is mainly a graphical experiment using the curve $\delta_{n+1} = f(\delta_n)$ written as in Eq.9. So, from Eq.14, the predicted ultimate primary settlement for the first case (PK 12+970) is 43.15cm.

In the same manner, for the second case of the test embankment at (PK 13+170), the predicted ultimate primary settlement is 77 cm. As for the third case, the predicted settlement by Asaoka's method is 42.93 cm. The settlement at a given time can be estimated by Eq.15, but a value of δ_0 is needed. In order to overcome this issue, Eq.10 and Eq.11 (Table 1) can be used. It is recommended that the chosen δj would be in the range of the dataset used for the prediction. As for the coefficient of consolidation, Asaoka (1978) reported that it can be estimated by Eq.12. So, for the highway test embankment at Pk12+970, the estimated coefficient of consolidation is 0.9749 m²/days, for PK13+170 it is estimated to be 6.36 m²/days, while for the railway embankment at (PK122+180), the estimated C_v value is 0.025 m²/days.

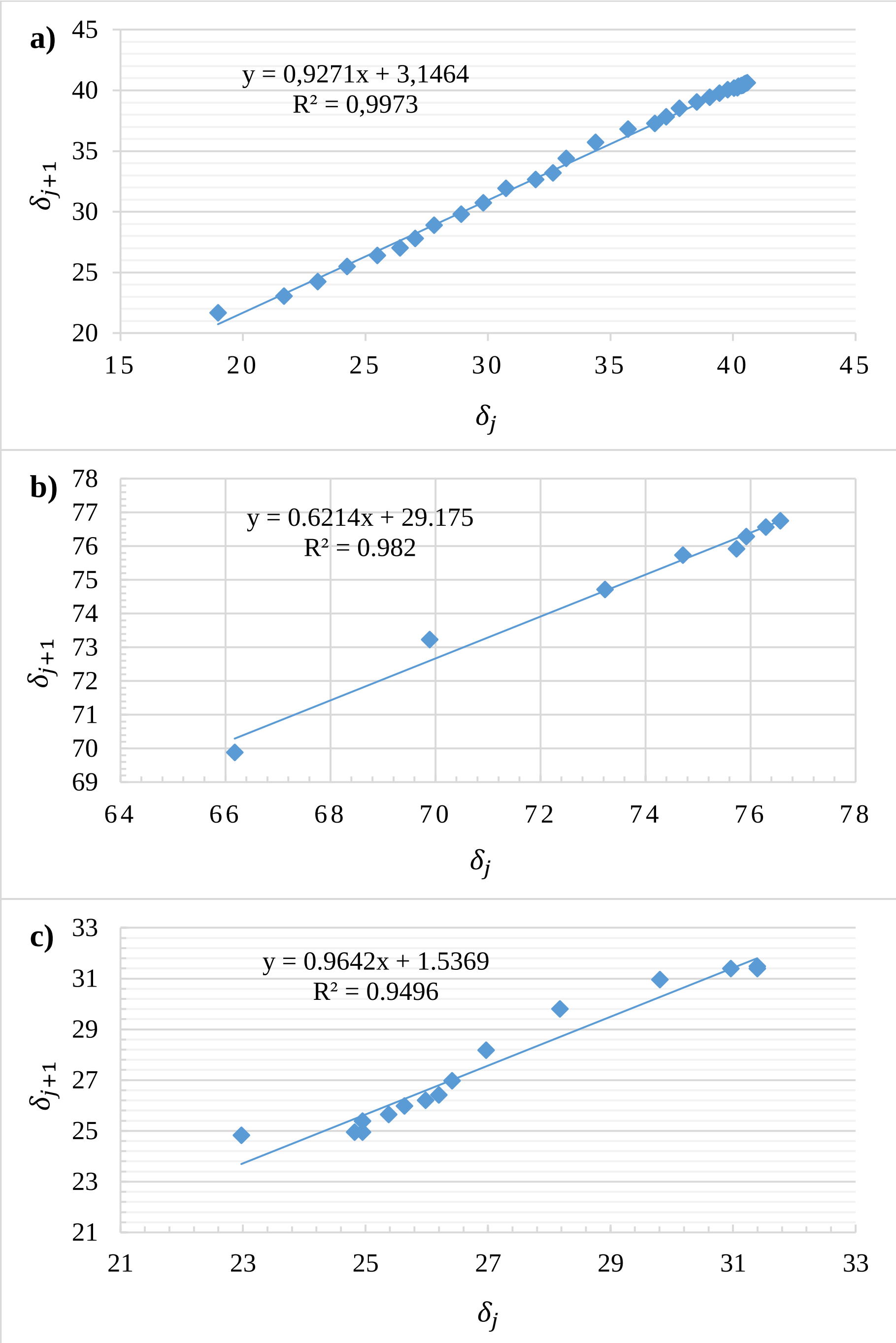

Figure (8): Asaoka curve of the embankment test a) PK 12+970 b) PK 13+170 and c) PK 122+180

The Hyperbolic Method (Tan, 1995)

Using settlement data illustrated in Fig.6, the plot of t/δ *vs*. time t is illustrated in Fig.9. The ultimate primary settlement can be calculated by Eq.20, the theoretical slope value for the Terzaghi theory without vertical drains is 0.82 (Tan, 1995). Therefore, δ_{ult} = 40cm for the first case (PK 12+970). As for the second case of highway embankment at PK 13+170 and for the railway embankment at PK 122+180, the respective values of δ_{ult} are 68 cm and 28.45cm. Therefore, from Fig.9 and by using Eq.16, the estimated settlement at 360 days for the first case is 41.37cm, for PK 122+180 δ (250 days) is 78.61cm and for the last case δ (180 days) is 30.61 cm.

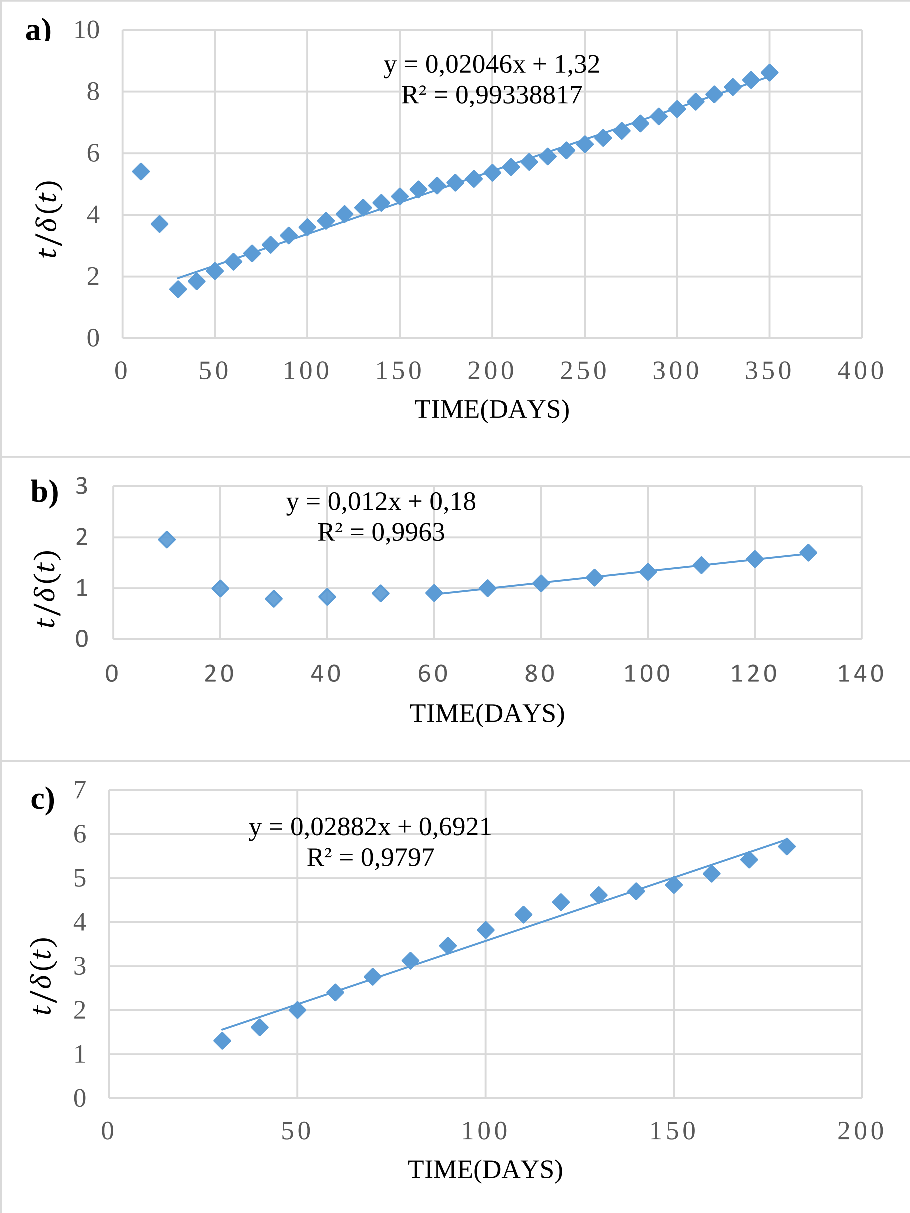

Figure (9): Hyperbolic curve of the embankment test
a) PK 12+970 b) PK 13+170 and c) Railway embankment PK122+180

Time of the end of primary consolidation according to Al-Shamrani (2005) can be estimated by the following expression:

$$t_p = \frac{\alpha_i \cdot \beta'}{(1-\alpha_i)S_i} \quad (25)$$

Hence, time of the end of primary consolidation for the first case is 293 days, for the second case is 68 days and 109 days for the third case.

From Eq.16 and Eq.13, the consolidation coefficient can be estimated by the following expression:

$$C_v = T_v \cdot H'^2 \cdot \frac{1-U\times\alpha}{U\times\alpha} \cdot \frac{S_i}{\beta'} \quad (26)$$

So, when considering a consolidation degree of 90%, the values of the coefficients of consolidation from Eq.26 are 0.9031 m²/days,4.029 m²/days and 0.5296 m²/days, respectively, for the highway embankments at PK12+970, PK 13+170 and for Railway embankment at PK122+180.

Chunlin Method (Li, 2014)

This method is based on Eq.22, which shows a linear relationship between the logarithm $\ln[\delta_p\times\pi^2/(8\times\delta_{ult})]$ and the time t. The slopes b of the resulted straight lines are given graphically in Fig.10. The latter gives a value of b = -0.007828 for the first case and a value of -0.034 and -0.019 for the second case and the third case. From Table 2 and by using the values of the slopes in Fig.10, the estimated coefficients of consolidation are 0.6129 m²/days, 2.766 m²/days and 0.325 m²/days, respectively, for PK12+970, PK13+170 and Railway embankment at PK122+180.

It is taken into account that Li (2014) uses the ultimate primary settlement predicted by the method of Asaoka. In this case, the settlement values measured at the end of instrumentations are used. According to Eq.21, the estimated settlement for the first case, after 360 days, is given by:

$$\delta_{c\,(t=360)} = 40.64 * \left(1 - \frac{8}{\pi^2} e^{-0.007828\times t}\right) = 38.6\text{cm} \quad (27)$$

Clearly, the settlement after 360 days is underestimated. The accuracy of the estimation can be improved by using the second term of the straight-line equation, which leads to the modification of Eq.28, to be:

$$\delta_{c\,(t=360)} = 40.64 * \left(1 - \frac{8}{\pi^2} e^{-0.007828\times t - 0.2066}\right) = 39\ cm \quad (28)$$

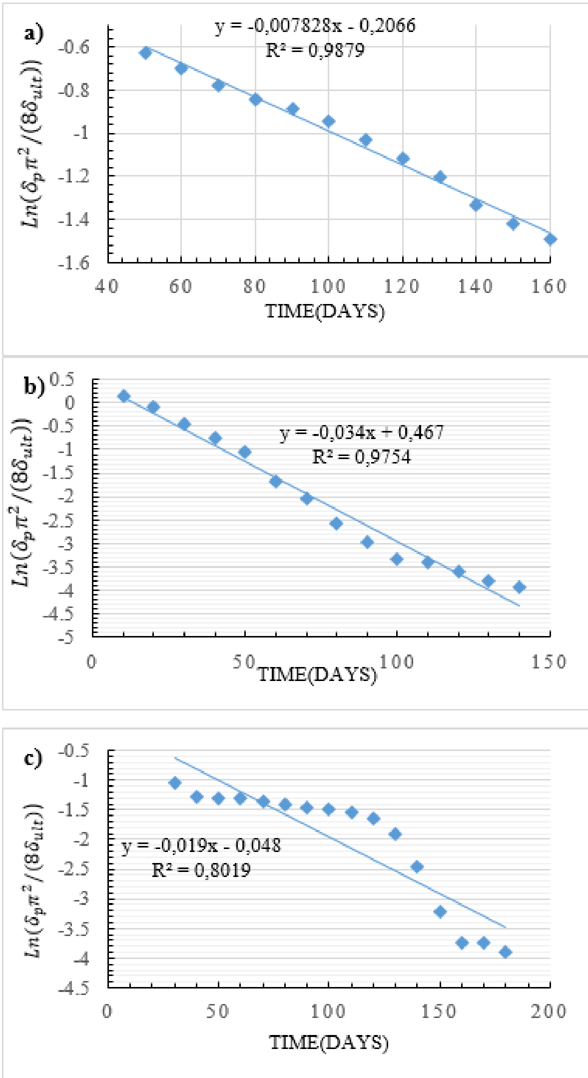

Figure (10): Chunlin curve a) Embankment at PK 12+970 b) Embankment at PK 13+170 and c) Railway embankment at PK 122+180

It doesn't seem much of an improvement, but overall, it represents a minimization of the error of settlement estimation. Fig.11.b shows the error of settlement estimation by Chunlin's method and the estimation by Eq.28. in Fig.11.b, the improvement of the accuracy can be noticed when using Eq. 28.

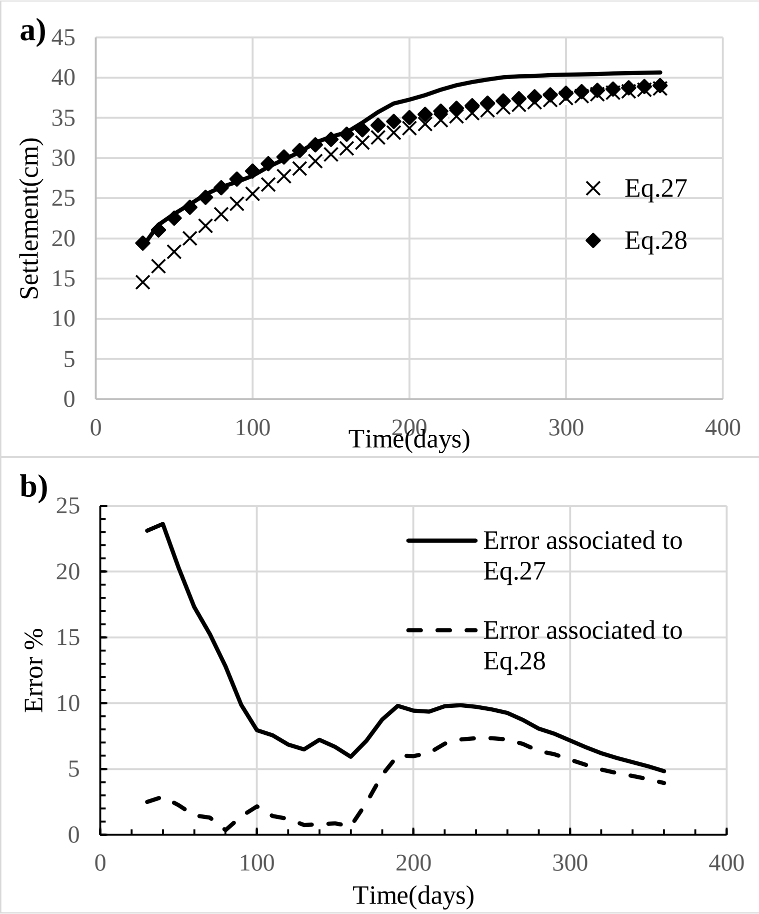

Figure (11: a) Comparison between measured settlement, estimated settlement by Chunlin's method using Eq.27 and Eq.28 and b) Errors associated

RESULTS AND DISCUSSION

Based on the findings presented in Fig.12 and the settlement measurements illustrated in Fig. 6, it is evident that the Terzaghi's method significantly overestimates the time (t_{90}) needed to attain 90% consolidation. And since the coefficients of consolidation C_v are determined in the lab, this leads to the underestimation of the intermediate settlement. Additionally, it can also be noted that Terzaghi's method underestimated the ultimate primary settlement. On the other hand, when using a value of the coefficient of consolidation that better represents the field measured settlements, it becomes evident that the estimated time required to achieve 90% consolidation has been improved, compared to that estimated by the coefficient of consolidation determined from the laboratory.

Regarding the hyperbolic method, the estimated settlements at the end of the observation period are quite accurate compared to field measurements and upon comparing the estimated t_{90} with the one from the monitored settlement, it can be said that the hyperbolic method showed mere 5% underestimation in terms of t_{90} for the first case. As for the second and the third cases, the predicted values of t_{90} were about a half of the instrumented results.

In the case of Asaoka's method, the predicted settlements closely matched field measurements at the end of the observation period, as well as for the ultimate settlement and t_{90} estimates in the first and second cases. However, in the third case, the method overestimated the ultimate primary settlement, leading to a higher t_{90} value.

As for Chunlin's method, which employed measured settlements in the settlement-prediction process for all three cases, the estimated time required to reach 90% settlement exhibited some accuracy variations, with errors of less than 20% for the second and third cases, suggesting that the estimated coefficients of consolidation by Chunlin's method are acceptable for the studied cases.

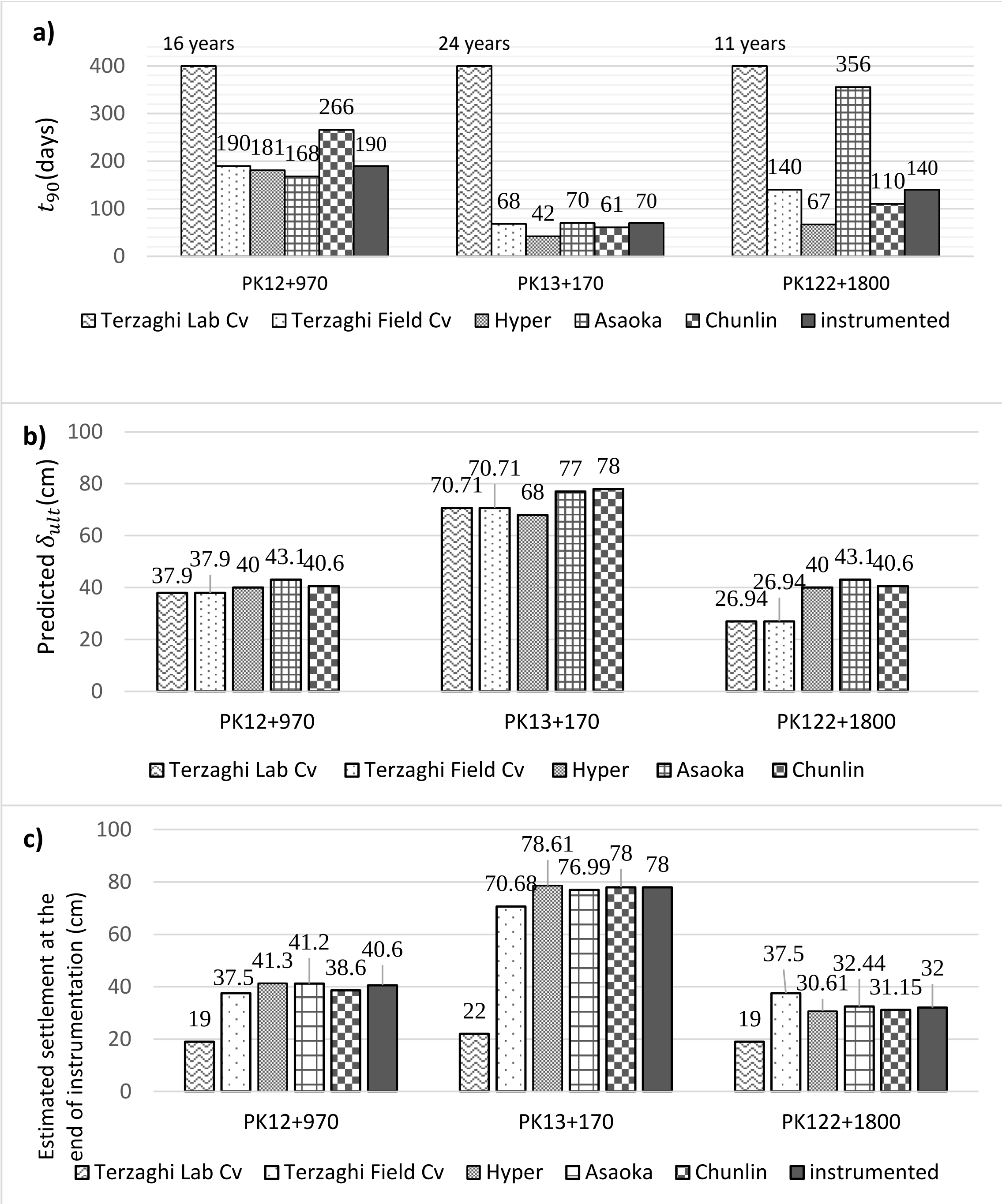

Figure (12): Result comparison by a) Time to achieve U_{90} b) Predicted δ_{ult} and c) Estimated settlement at the end the observation period

Concerning the predicted settlement at a given time by the hyperbolic method, it should be noted that Eq.16 was used. According to Al-Shamrani, (2005) and Tan (1991), such expression is used to predict the total settlement at a given time; so, if the predicted settlement at the specific time is greater than the predicted ultimate primary settlement, that only means that the time at the end of primary consolidation has already passed.

Finally, the previous results lead to the realization of the importance of the filed data-based prediction methods in the process of pursuing an accurate estimation of the consolidation coefficient and how it can affect the overall estimation of the consolidation settlement. Especially, since it was found that the values of the coefficient of consolidation provided by the lab measurements were considerably lower than the values predicted by the four methods, this explains the underestimation of Terzaghi's method.

Similar results were found by Benamghar and Boudjellal (2017) and Lakkoju et al. (2020).

CONCLUSIONS

The objective of this work was conducting a comparative study of four methods for estimating consolidation settlement in soft ground for three different cases, where the field settlement was monitored for over 6 months.

The predicted ultimate primary settlements by the method of Terzaghi (1943) were underestimated for the three studied cases. A significant overestimation of t_{90} is also observed. As for the estimated settlements at time of the end of observation periods, by the hyperbolic method (Tan, 1995; Asaoka, 1978; Li, 2014), the results were similar for each studied case. The difference lies in the prediction of the coefficient of consolidation, which leads to different values of t_{90} for each case separately. It was noticed that the values provided by the lab measurements were considerably lower than the values from the field-based methods and based on the results, the hyperbolic method yielded the more accurate values of the coefficient of consolidation.

Moreover, an expression for the settlement estimation at a given time for Asaoka's method was featured, where the settlement can be estimated from a known settlement value at a given time, without the need for the value of the initial settlement. In addition, it was proposed to use all the terms resulting from Chunlin's method for more accurate estimations.

In summary, this study underscores the critical importance of precisely estimating the coefficient of consolidation. Such accuracy significantly influences the predicted soil consolidation settlement and, consequently, holds implications for engineering assessments.

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