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Slope Stability of Embankments on Soft Soil Improved with Vertical Drains

F. Kassou [a,b*], J. Ben Bouziyane [a], A. Ghafiri [b], A. Sabihi [c]

[a] *Université Hassan II, Faculté des Sciences Ben Msik, Casablanca, Morocco.*

[b] *Ecole Hassania Des Travaux Publics, Casablanca, Morocco.*

[c] *Laboratoire Public d'essais et d'études, Casablanca, Morocco.*

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Abstract

The overloads of structures or embankments built on clayey soft ground are generally applied gradually, respecting a specific phasing. This phasing on construction allows the undrained shear strength of clay increasing over consolidation in order to avoid the risk of collapse during loading. In this work, the undrained shear strength of clay over the consolidation was estimated following SHANSEP method of which parameters proposed by eight researchers have been employed, as well as the slope stability analysis of embankments on soft soils during staged construction. Assessment of factor of safety for slope stability was conducted basing on the Bishop method. Additionally, the variations of undrained shear strength and factor of safety were presented. In order to validate the methods discussed in this study, slope stability analysis of five embankments constructed on clayey soft soils improved by the vertical drain technique in a high-speed railway construction project in Morocco was performed. For these embankments, field measurements about lateral displacement are presented. It was found that some of the adopted methods is in a good agreement with field measurements. Hence, generalization of these methods to many soft ground cases can be proposed.

Keywords: Shear Strength; Slope Stability; Soft Soil; Consolidation; Vertical Drains.

1. Introduction

The vertical drain system is an effective method for accelerating soil consolidation. It has several fields of application such as roads and railway embankments. These embankments are generally constructed following many phases to ensure the stability of the ground soil during construction. The undrained shear strength of foundation soils usually increases over consolidation, and it is very important to determine appropriately the gain in shear strength in the geotechnical engineering practice. Several researches have been conducted to determine undrained shear strength (Su) using field or laboratory tests. A series of research studies [1–3] have proposed correlations for the undrained shear strength of normally consolidated soils (Cu = Su) as a function of plasticity index (PI) or liquidity index (LI), or even liquid limit (LL), which implicitly shows the reduction of water content over consolidation. Janbu [4] has correlated (Su) with vertical effective stress σ'_v. Many researchers have predicted the increase in the undrained shear strength (Su) by the SHANSEP technique basing on correlation formula with (OCR) [5–8]. Other studies investigated instrumented test embankments or case histories to predict and determine undrained shear strength (Su) [9–13]. However, samples disturbance phenomenon has long been a problem and difficulties in obtaining undisturbed samples was discussed [14, 15]. Karlsson and Viberg (1967) found no unique relationship and concluded that there are several factors influencing

* Corresponding author: fouzia.kassou@gmail.com

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undrained shear stress [3]. In addition, Mesri and Cai (2017) [16] proposed that the relationship between Su and σ'_p can be constant, which the more recent study by D'Ignazio et al. proved [17]. Lechowicz et al. (2017) have performed dilatometer and laboratory tests on heavily preconsolidated boulder clays and Pliocene clays prevailing in the Warsaw region [18]. Empirical coefficients for multi-factor correlation to obtain undrained shear strength from dilatometer tests for boulder clays and Pliocene clays were determined. Lawane et al. (2018) have established correlations between shear strengths of disturbed samples and those of undisturbed samples of tropical soils of Burkina Faso using the Casagrande cell [19]. Most of these methods have been developed for specific soil cases. The possibility of generalizing them to other soil cases remains unexplored. Indeed, there still a need to study the slope stability in soft soils conditions. In order allow a general solution for many cases of soft soils under staged construction; it is important to have a comparative discussion and results of numerical calculation on all existing methods. To achieve this objective, this study compares eight analytical methods, for estimation the undrained shear strength over consolidation, then proposes a generalization to the case of five embankments on soft soils improved by vertical drains in Morocco.

2. Review of Existing Correlation Between (SU) and (OCR)

In order to estimate the shear strength increase during staged construction, several correlations using the SHANSEP approach are analyzed. This approach defines a normalized parameter by dividing (Su) to the effective vertical stress σ'_v. It considers that over consolidation ratio (OCR) has a significant influence on the strength of soil, by using a well-established relationship between normalized shear strength $\frac{S_u}{\sigma'_v}$ and (OCR). This relationship can be written as follows:

$$\frac{S_u}{\sigma'_v} = S(OCR)^m \tag{1}$$

$$\text{Or } \left(\frac{S_u}{\sigma'_v}\right)_{OC} = \left(\frac{S_u}{\sigma'_v}\right)_{NC} (OCR)^m \tag{2}$$

Where; S and m are SHANSEP parameters, which can be obtained from test data. OC and NC indicate normally consolidated and over consolidated, respectively. Many researchers obtained the values of S and m by using laboratory or field tests on undisturbed samples, for some particular soft soils. Table 1 presents the proposed values of S and m parameters according to several authors. In this table, it is indicated whether (Su) is divided by the original effective vertical stress (σ'_{v_0}) or the in situ effective vertical stress (σ'_v).

Table 1. Values for SHANSEP parameters S and m by several researches

Author (Year)	Investigated Soil	S	m	σ'_{v_0} or σ'_v
Seah (2003)	Bangkok clay	0.265; 0.245; 0.27	0.75; 0.89; 0.75	σ'_v
		0.265; 0.245	-	
Bergado (2002)	Bangkok clay	0.22	0.8	σ'_{v_0}
Jamal Mohd et al. (1997)	Bukit Raja clay	0.259	0.78	σ'_{v_0}
		0.19 to 0.23	-	
Jamiolkowski et al. (1985)	-	0.19 to 0.27	0.8	σ'_{v_0}
Ladd (1991)	Sensitive marine clays	0.2	-	σ'_v
	Homogeneous clays of low to moderate sensitivity	0.22	0.8	
	Northeastern US clays	0.16	0.75	
	Sedimentary deposits of silts and organic soils and clays with shells	0.25	0.88	
Roy & Singh (2008)	Sensitive, soft and compressible soil in Kolkatta	0.34; 0.15	0.62; 1.08	σ'_v
Indraratna et al. (1992)	Malaysian Muar clay	0.2	0.8	-

3. Method for Estimating the Gain in SU During Consolidation

For over consolidated clayey soil, the settlement equation can be written in two ways depending on preconsolidation pressure σ'_p, the original effective vertical stress σ'_{v0} and total surcharge value $\Delta\sigma$. Therefore, for $\sigma'_{v0} + \Delta\sigma < \sigma'_p$, Equation 3 is used to determine the settlement:

$$Sc = H_0 \frac{C_s}{1+e_0} \log \frac{\sigma'_{v0}+\Delta\sigma}{\sigma'_{v0}} \tag{3}$$

While for $\sigma'_{v0} + \Delta\sigma > \sigma'_p$, the settlement is determined by using Equation 4:

$$Sc = H_0 \frac{C_s}{1+e_0} \log \frac{\sigma'_p}{\sigma'_{v0}} + H_0 \frac{C_c}{1+e_0} \log \frac{\sigma'_{v0}+\Delta\sigma}{\sigma'_p} \tag{4}$$

Where Sc: Consolidation settlement; H_0: Total height of the soil layer; e_0: void ratio of the soil layer; Cs: recompression index; Cc: compression index; σ'_p: preconsolidation pressure; σ'_{v0}: original effective vertical stress; $\Delta\sigma$: total surcharge value.

To compute settlements corresponding to the sequential construction of the embankment, Equations 5 and Carillo [20] Equation 6 are used:

$$s_c(t) = U_t \times S_c \tag{5}$$

$$U_t = (1 - U_v)((1 - U_r) - 1 \tag{6}$$

Where; $S_c(t)$: Consolidation settlement for a given time (t); U_t: the degree of consolidation at a given time (t); Sc: Total consolidation settlement; U_v and U_r: are respectively vertical and radial degree of consolidation considered separately.

It is very important to mention that sample disturbance, caused either during sampling or during the preparation of the specimens, has a significant effect on the value of preconsolidation pressure as measured with laboratory oedometer test. Therefore, Equation 1 is used here to obtain σ'_p from the initial undrained shear strength (Cu_0) of the foundation, which is correlated with several field tests properties. Hence, σ'_p can be expressed as:

$$\sigma'_p = (\sigma'_{v0} \times \frac{Cu}{S \times \sigma'_{v0}})^{\frac{1}{m}} \tag{7}$$

Following are three major correlations between initial undrained shear strength (Cu_0) and in situ parameters.

3.1. Pressuremeter Test

The empirical relationship by Ménard [21], $C_{u0} = \frac{p_l - p_0}{5.5}$, is usually used for determining undrained shear strength (Cu0), where Pl: limit pressure P0: at rest horizontal pressure. (Cu_0) is also derived from pressuremeter parameters by Amar and Jéséquel [22] using the following equation: $C_{u0} = \frac{p_l - p_0}{a} + b$, where $a = 5.5; b = 0$ for $p_l - p_0 \leq 0.3\ MPa$; $a = 12; b = 0.03$ for $0.3 \leq p_l - p_0 \leq 1\ MPa$ and $a = 35$; $b = 0.085$ for $1\ MPa \leq p_l - p_0$. Cassan [23] suggested that $C_{u0} = \frac{p_l - p_0}{10} + 0.025$ for $0.3 \leq p_l - p_0 \leq 1\ MPa$. Baguelin and Jéséquel [24] proposed the following correlation $C_{u0} = 0.21\ {p_l}^{0.75}$.

3.2. Cone Penetration Test

$c_{u0} = \frac{q_c}{15}$ for $q_c \leq 1\ MPa$ this correlation is established for clayey soils.

3.3. Field Vane Test

The measured vane strength S_u has to be corrected prior to use. The corrected undrained shear strength (Cu_0) is given by $c_{u0} = \mu\, S_u$ where μ is an empirical vane shear correction factor that has been related to plasticity index (PI) by Bjerrum [25].

To account for changes in stress history during construction, it is logical to try to find a value of effective vertical stress $\sigma'_v(t)$ for any time during consolidation process. This effective vertical stress is derived based on Equations 3 or 4 and 5. Hence, for $\sigma'_{v0} + \Delta\sigma < \sigma'_p$.

$$\sigma'_v(t) = {\sigma'_{v0}}^{(1-U)}\ (\sigma'_{v0} + \Delta\sigma)^U \tag{8}$$

While for $\sigma'_{v0} + \Delta\sigma > \sigma'_p$ there is two possible cases, if $\sigma'_v(t) < \sigma'_p$, then $\sigma'_v(t)$ can be established using Equation 9:

$$\sigma'_v(t) = {\sigma'_{v0}}^{(1-U)}\ {\sigma'_p}^{U(1-\frac{C_c}{C_s})}\ (\sigma'_{v0} + \Delta\sigma)^{U\frac{C_c}{C_s}} \tag{9}$$

And if $\sigma'_v(t) > \sigma'_p$ then $\sigma'_v(t)$ can be obtained by Equation 10:

$$\sigma'_v(t) = {\sigma'_{v0}}^{\frac{C_s}{C_c}(1-U)}\ {\sigma'_p}^{(1-U)(1-\frac{C_s}{C_c})}\ (\sigma'_{v0} + \Delta\sigma)^U \tag{10}$$

After $\sigma'_v(t)$ is determined, the subsequent increase in undrained shear strength Cu(t) of subsoil due to consolidation is simply calculated by combining Equation 1 and 7. Note that in Equation 1, $Su = Cu(t)$ and $\sigma'_v = \sigma'_v(t)$. Hence:

For $\sigma'_v(t) < \sigma'_p$:

$$C_u(t) = C_{u0}\left(\frac{\sigma'_v(t)}{\sigma'_{v0}}\right)\left(\frac{\sigma'_{v0}}{\sigma'_v(t)}\right)^m \tag{11}$$

While for $\sigma'_v(t) > \sigma'_p$;

$$C_u(t) = C_{u0}\left(\frac{\sigma'_v(t)}{\sigma'_{v0}}\right)\left(\frac{\sigma'_{v0}}{\sigma'_p}\right)^m \tag{12}$$

Figure 1, summarizes the method explained above for obtaining the gain in shear strength while consolidation.

For $\sigma'_{v0} + \Delta\sigma < \sigma'_p$

$$Sc = H_0 \frac{C_s}{1+e_0} \log \frac{\sigma'_{v0} + \Delta\sigma}{\sigma'_{v0}}$$

$$s_c(t) = U_t * S_c$$

$$\sigma'_v(t) = {\sigma'_{v0}}^{(1-U)} (\sigma'_{v0} + \Delta\sigma)^U$$

$$C_u(t) = C_{u0}\left(\frac{\sigma'_v(t)}{\sigma'_{v0}}\right)\left(\frac{\sigma'_{v0}}{\sigma'_v(t)}\right)^m$$

For $\sigma'_{v0} + \Delta\sigma > \sigma'_p$

$$Sc = H_0 \frac{C_s}{1+e_0} \log \frac{\sigma'_p}{\sigma'_{v0}} + H_0 \frac{C_c}{1+e_0} \log \frac{\sigma'_{v0} + \Delta\sigma}{\sigma'_p}$$

$$s_c(t) = U_t * S_c$$

$$\text{if } \sigma'_v(t) < \sigma'_p : \sigma'_v(t) = {\sigma'_{v0}}^{(1-U)} {\sigma'_p}^{U(1-\frac{C_c}{C_s})} (\sigma'_{v0} + \Delta\sigma)^{U\frac{C_c}{C_s}}$$

$$\text{if } \sigma'_v(t) > \sigma'_p : \sigma'_v(t) = {\sigma'_{v0}}^{\frac{C_s}{C_c}(1-U)} {\sigma'_p}^{(1-U)(1-\frac{C_s}{C_c})} (\sigma'_{v0} + \Delta\sigma)^{U}$$

$$C_u(t) = C_{u0}\left(\frac{\sigma'_v(t)}{\sigma'_{v0}}\right)\left(\frac{\sigma'_{v0}}{\sigma'_p}\right)^m$$

Figure 1. Proposed method for obtaining the increase in shear strength

4. Case of Moroccan Embankments

The study has been carried out within the framework of the High Speed Railway project linking Tangier and Asilah, two cities in the north of Morocco. The project crosses several soft soils with different ground improvement techniques such as preloading technique, vertical drains, rigid inclusions or stone columns. The analysis is concerned with the behavior of five embankments constructed on soft soils improved by vertical drains.

4.1. Embankments and Soil Condition

Five preloading embankments, named 2288, 3058, 3078, 3089 and 3119, were constructed over a period of one year. The maximum completed height of embankments varied from 6 to 11 meters. The ground soil subjected to embankments loading contains moderately plastic clay, generally fairly firm, but with clearly softer and mucky passages. The thickness of compressive layers varies between 6 and 22 meters. A detailed soil profile including some soil properties was given by Kassou et al. (2017) [26]. Hence, only measured initial undrained shear strength (Cu_0) and initial effective vertical stress σ'_{v0} are presented in Table 2. Underneath embankments, vertical band drains were installed in a square pattern varying from (1.3×1.3 m) to (2.5 ×2.5 m), with 5 cm of equivalent diameter and 8 to 20 m length. Figure 2 shows the installed vertical drains for embankment 3119.

Figure 2. Installed vertical drains before embankment 3119

Table 2. Soil parameters

Embankment	Soil layer	Depth (m)	σ' (kN/m^3)	σ'_v (kPa)	Cu_0 (kPa)
R2288	Over Consolidated clay	2	10	10	50
	Slightly consistant clay	3	10	35	38
	Muck	5	9	72.5	29
	Silty clay	3	10	110	76
	Alluviums	3.5	10	145	-
	Mudstone clay	3	9	173.5	110
	Mudstone	Substratum			
R3058	Clay	3	8	12	70
	Marly clay	3.5	8	38	70
	Altered pelite	10	10	102	100
	Mudstone	Substratum			
R3078	Fragments	6.5	10	32.5	81
	Altered mudstone	7	10	100	111
	Mudstone	Substratum			
R3119	Fragments	6	9	27	84
	Altered mudstone	10	9	99	115
	Mudstone	Substratum			
R3089	Fragments	1	8	4	50
	Very altered mudstone	4.5	8	26	90
	Altered mudstone	5	9	66.5	100
	Mudstone	Substratum			

4.2. Comparing Numerical Results

In this section, comparison of undrained shear strength variations while consolidation is performed by combining the correlations presented in Table 1 to Equations 11 and 12, for normally consolidated and over consolidated soil respectively. Then the slope stabilities of embankments are assessed by the bishop method. The study focuses on the results obtained for soft grounds that are loaded by five embankments as mentioned in the previous paragraph. For each embankment, the most representative profile was selected in order to confirm the conclusions of the current study. For all the profiles, the gain in shear strength was investigated in every soil layer in the soft ground. For example, detailed information for all the soil layers under embankment 2288 are given in Table 3. In this table, the correlation used for Cu variation is obtained from Jamiolkowski (1985) studies [6]. It was noticed a very much closeness for some analyzed soil layers and it was conducted to distinguish three particular cases: over consolidated soil; normally consolidated soil and over consolidated soil that becomes normally consolidated after loading. Hence, estimated values (Cu and Fs) were shown for three selected soil layers among the analyzed ones as can be seen in Figures 3 to 5.

4.2.1. Undrained Shear Strength

Figure 3 compares the undrained shear strength curves computed using correlations described before. In this case, we notice that the subsoil remains overconsolidated after construction. There is a close agreement between the undrained shear strength obtained by Jamiolkowski (1985), Ladd (1991), Indraratna et al. (1992) and D'Ignazio and Länsivaara (2016) [6, 8, 9, 13]. Correlations from Mohd Amin et al. (1997) and David Suits et al. (2003) [10, 12] results in a slightly higher undrained shear strength compared with the others, while Roy and Singh (2008) overpredicts the undrained shear strength substantially [11].

Table 3. Numerical values for Cu over consolidation.

Embankment		Soil layer	Consolidation degree U (%)					
			50	60	70	80	90	100
R2288	Cu values by Jamiolkowski et al. [6] (kPa)	Overconsolidated clay	60	66	73	79	86	93
		Slightly consistent clay	48	50	53	57	60	64
		Muck	45	49	54	59	65	71
		Silty clay	59	62	66	69	72	77
		Alluviums	0	0	0	0	0	0
		Mudstone clay	78	81	85	88	91	94

R3058		Mudstone	79	87	96	104	112	120
		Silty clay	65	70	75	80	85	91
		Altered Mudstone	76	80	84	88	92	96
R3078		Fragments	77	84	90	96	103	109
		Altered Mudstone	85	90	94	99	104	108
R3119		Fragments	84	92	99	107	114	122
		Altered Mudstone	90	95	100	105	110	115
R3089		Fragments	100	112	125	139	153	169
		Altered Mudstone	121	133	145	157	170	182
		Very altered Mudstone	117	127	136	146	155	165

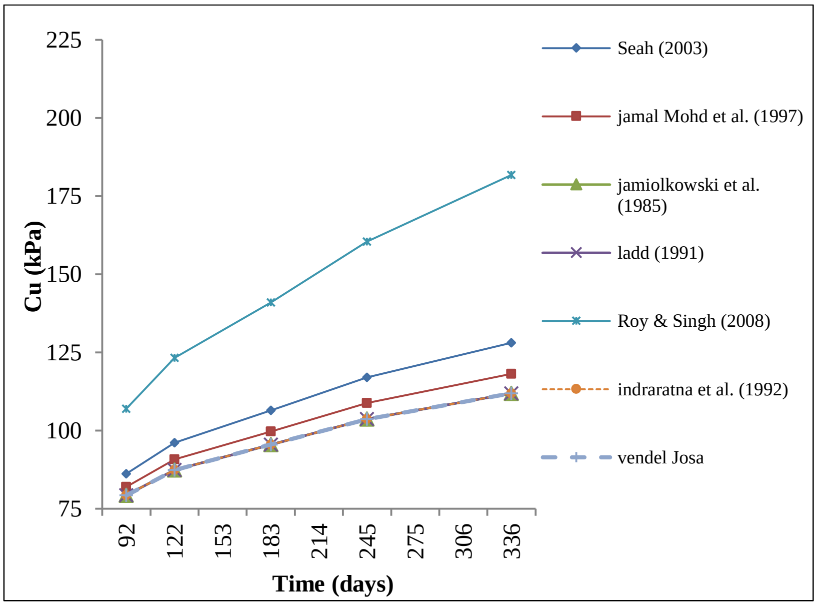


Figure 3. Comparison of Cu curves for over consolidated clay, R3058

Figure 4 illustrates the variation of undrained shear strength of normally consolidated subsoil. It indicates that Ladd (1991), Mohd Amin et al. (1997) and David Suits (2003) [8, 10, 12] correlations resulted in a slightly higher undrained shear strength compared to Jamiolkowski (1985) results [6]. The maximum difference on S_u is 11%. Figure 4 also shows that Indraratna (1992) and D'Ignazio and Länsivaara (2016) [9, 13] correlations yielded slightly lower undrained shear strength than Jamiolkowski (1985) [6]. This doesn't exceed 13%. Once again, it can be observed that Roy and Singh (2008) [11] method is widely larger compared to all other methods.

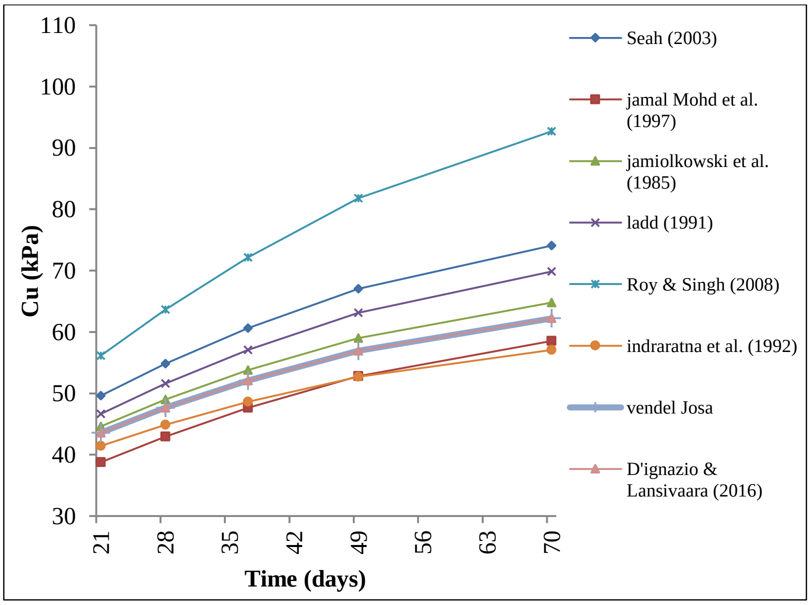


Figure 4. Comparison of Cu curves for normally consolidated mucky soil, R2288

Figure 5 shows the variation of undrained shear strength for over consolidated subsoil that becomes normally consolidated after construction. It can be observed that Mohd Amin et al. (1997) [10] results in S_u curve that decreases after 52 days of construction. This indicates that soil is completely restructured during the transition to the normally consolidated state. It is important to remind that, in normally consolidated state, effective shear strength is zero. Although effective friction angle is significant, its effect remains marginal until effective shear strength becomes non-zero. Therefore, more time is required to allow increase in undrained shear strength.

The principle results that can be concluded from Figure 3 to 5 are that the predictions from Roy and Singh (2008) [11] may not be appropriate for this case, while that the one from Jamiolkowski (1985) [6] can predict much better the behavior of the analyzed soft soil. For this reason, it is suggested to consider m = 8 in the SANSHEP procedure, which leads to the best trend of Cu curve in representing the ground soft soil behavior.

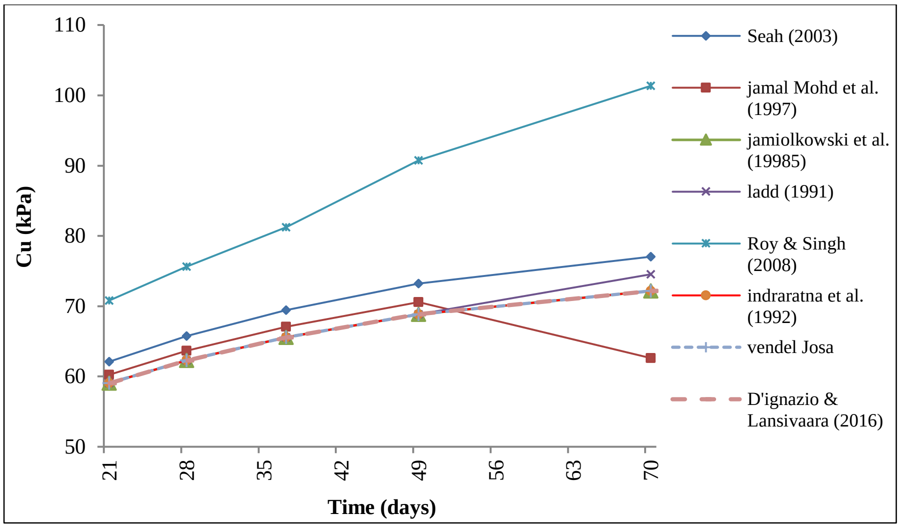


Figure 5. Comparison of Cu curves for over consolidated silty clay that becomes normally consolidated after loading, R2288

4.2.2. Variation of the Factor of Safety during Consolidation

To study the variation in overall stability of the embankment during consolidation, the factor of safety at various time intervals is investigated. Time beginning is considered at the end of construction, which matches with 50% of consolidation [26]. Figure 6 and 7 show the variation of factor of safety with time for both embankments R3058 and R3089, respectively. It can be seen from Figure 6 and 7 that up to 26 days from the end of construction, factor of safety increases rapidly with time. Factor of safety increases then slowly, and it is stabilized about 224 days after construction. It can also be seen from both figures that correlations proposed by Jamiolkowski (1985), Ladd (1991), Indraratna et al. (1992) and D'Ignazio and Länsivaara (2016) [6, 8, 9, 13] are in a good agreement, whereas, that predicted by Mohd Amin et al. (1997) [10] and David Suits et al. (2003) [12] respectively, is moderately higher, respectively lesser. However, Roy and Singh (2008) [11] correlation is extremely higher than other methods. Basing on the conclusion in the previous section regarding the SANSHEP parameter which best represents the studied case, correlation from Jamiolkowski (1985) [6] (m = 0.8) is found again to be the most convenient among the eighth analyzed correlations. For this, the correlation from Jamiolkowski (1985) [6] is used bellow in this section for computing Cu.

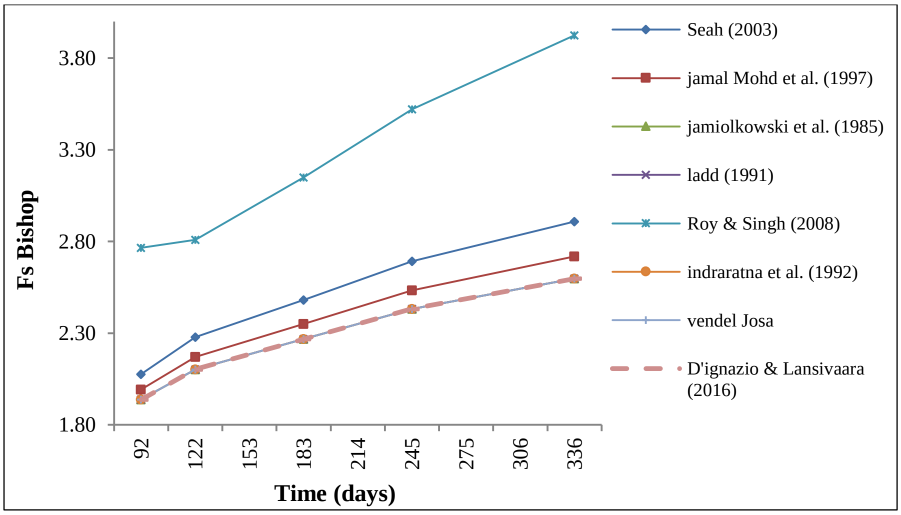


Figure 6. Variation of factor of safety with time R3058

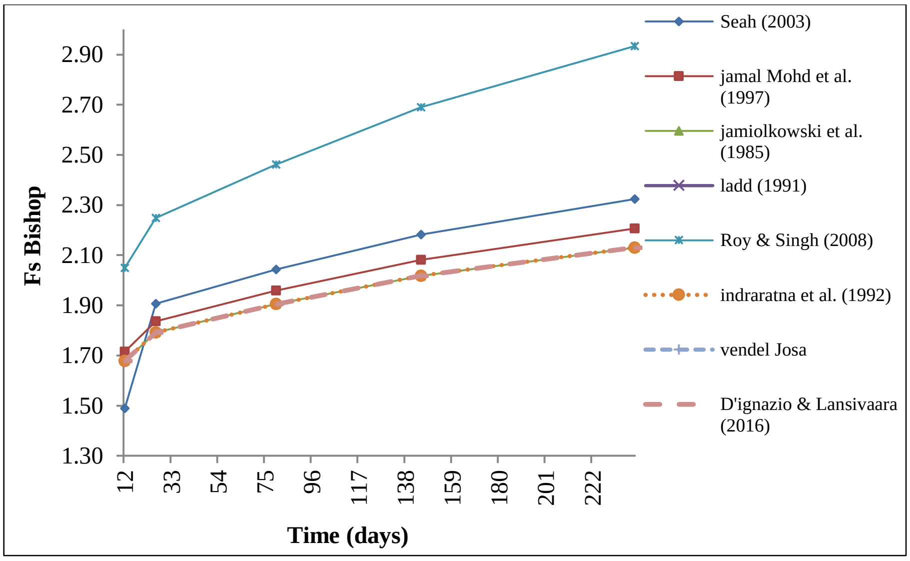


Figure 7. Variation of factor of safety with time R3089

Figure 8 shows critical slip surface after 12 days of construction of R3089 embankment, based on the correlation obtained from [6]. The value of the factor of safety obtained at this time interval is about 1.61. In Morocco, the railway projects standards require a minimum of 1.5. Therefore, staged construction adopted for embankment R3089 was fundamental in order to avoid the risk of collapse. Also, it can be observed from this figure that critical slip circle is located at about 11m from the ground surface.

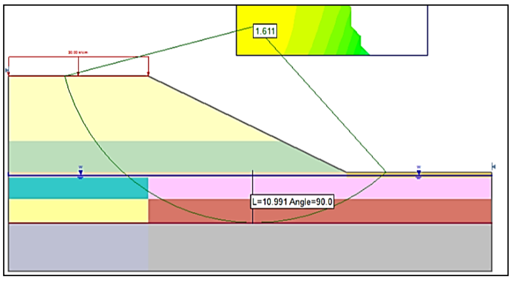
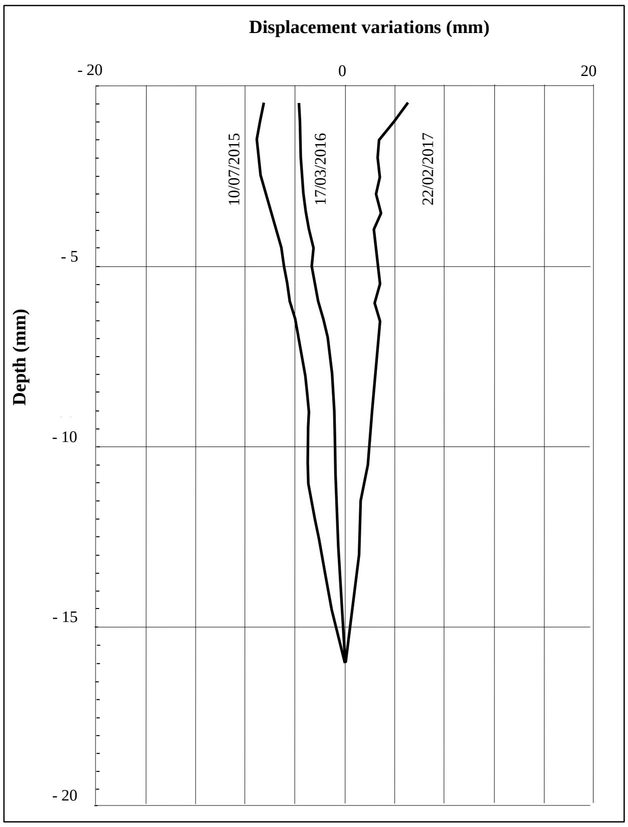
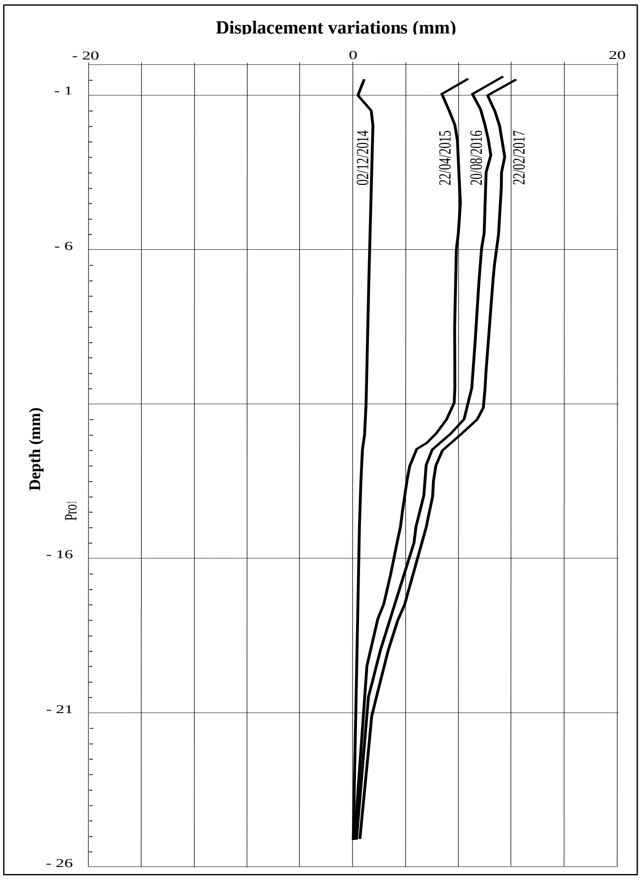


Figure 8. Critical slip surface after 12 days of construction R3089



(a)



(b)

Figure 9. (a) Lateral displacements at various time periods R3089, PR308+830; (b) Lateral displacements at various time periods R3089, PR308+920

The variation of measured lateral displacement over time given by inclinometer installed under embankment R3089 is shown in Figure 9. It is observed from Figure 9(a) that the altered crust up to the depth of 12.5 m has undergone a monolithic displacement, which indicates the existence of a reactivated shear plane at this depth. Lateral displacements of this upper crust are considerably larger than any deeper lateral movements. Therefore, it is clear that the critical slip surface should be located at this depth. This is well confirmed by the result obtained from Figure 8.

Figure 9(b) shows the embankment band load effect on the lateral displacement. It can be observed from this figure that the vertical settlement of the embankment center line generates negative lateral displacements immediately at the end of construction. Positive lateral movements can be observed only after 210 days, which indicate the beginning of delayed lateral creep. However, embankment R3058 doesn't show any lateral displacement.

5. Conclusion

This study compares several methods for estimating undrained shear strength and its variation while consolidation of soft soils under staged loading. Then slope stability analysis is conducted and the variation of the factor of safety during consolidation is analyzed. In this work, it is focused on the case of soft soils that are under embankments during staged construction. In order to reach this objective, the methods were applied to the case of five embankments on soft soil stabilized with vertical drains. For this case, estimation of undrained shear strength of soft soil was conducted by comparing numerous correlations between this parameter and over consolidation ratio. The gain in undrained shear strength during consolidation was then determined by combining the proposed correlations with available in situ test parameters. It can be observed that some correlations provide a very good match with field measurements. It can be concluded that these correlations are an excellent tool to predict the gain in undrained shear strength of soft soils over consolidation. Therefore, generalization of the methods listed before can be possible, as performed in the current study.

6. Conflicts of Interest

The authors declare no conflict of interest.

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