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Settlements and Consolidation Rates under Embankments in a Soft Soil with Vertical Drains

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ABSTRACT

In this paper, a study was carried out in order to estimate settlements and consolidation rates under embankments constructed on Moroccan soft soils. Settlement measurements in several embankments in High Speed Railway project between two Moroccan cities, Tangier and Assilah, were analyzed. The objective of this study is to estimate settlement values and settlement rates, with sufficient precision, in soft soils under embankment loadings. It was found that the elastic method using the pressuremeter modulus results in more accurate settlement values than the oedometric method. Furthermore, settlement rates could be determined with fair accuracy by using a correlation between the vertical coefficient of consolidation and the liquid limit, and also by considering an isotropic behavior of soil.

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1. INTRODUCTION

Settlements under embankments constructed on soft soils are usually excessive and take a long period to be stabilized. In many cases, ground improvement techniques are used to decrease the consolidation period and or settlement values. Among these techniques, prefabricated vertical drains (PVD) technique is used to accelerate consolidation rates. Numerous research articles related to settlements or consolidation rates had been published since 1936. The most commonly used solutions to analyse consolidation rates in soils improved with vertical drains are Barron [1] and Hansbo [2] solutions. These methods require the determination of both vertical and radial coefficients of consolidation by using one-dimensional consolidation test, and several available curve-fitting procedures [3, 4]. Usually, field measurements often show that the values of Cv and Cr obtained by the curve-fitting procedures are not satisfacory. Hence, it is necessary to correlate these coefficients with some simple index property. For this reason, US-Navy [5] have proposed a correlation between Cv and the liquid limit. Sridharan and Nagaraj [6] have found that Cv has a better correlation with the shrinkage Index. Solanki and Desai [7] have suggested new correlations to obtain Cv from soil plasticity characteristics, especially the liquid limit. Asma et al. [8] have proposed a relation between Cv and Wl of undisturbed silty clay in central and Southern regions of Iraq. Devi et al. [9] have found that Cv has a better correlation with the liquid limit Wl. The present work compares two existing methods for settlement calculations, and suggests a recalibration method which aims to estimate Cv and Cr. This method is then applied to four embankments on PVD improved subsoil of High Speed Railway (HSR) project in Morocco. It was shown that for all the studied cases, the method yielded good results.

2. DESCRIPTION

The study has been carried out whithin the framework of the High Speed Railway project linking Tangier and

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Assilah, 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. This work is concerned with the behavior of four embankments constructed on soft soils with vertical drains. During and after construction, field measurements showed that the values of settlements determined by classical methods are overestimated. Moreover, the settlement rates found by the same methods are underestimated. Thus, two basic strategies were used in order to assess settlement and consolidation rates, which are: 1) the use of elastic method with pressuremeter modulus for settlement calculation. 2) the use of US-Navy correlation between the coefficient of vertical consolidation Cv and limit liquid Wl, and the assumption Cv/Cr=1 (isotropic behavior of soil).

2. 1. Site The geological map of Tangier region describes the different geological components (Figure 1). The current site/ region is characterized by the abundance of mudstone, silt, alluviums and colluviums. Moreover, large valleys which are present in this area, are basically near to small cost rivers estuaries. This area of investigation is subjected to wheather variations that affect water level on the ground. Nevertheless, the highest level of water reached in this site could be compared to the natural conditions on the ground.

2. 2. Embankments Four preloading embankments, named 2288, 3058, 3078 and 3119 (Figure 2), were constructed over a period of one year. The maximum completed height of embankments varied from 6 to 11 meters. The soil foundation subjected to embankments loading consists of moderately plastic clay, generally fairly firm, but with clearly softer and mucky passages.

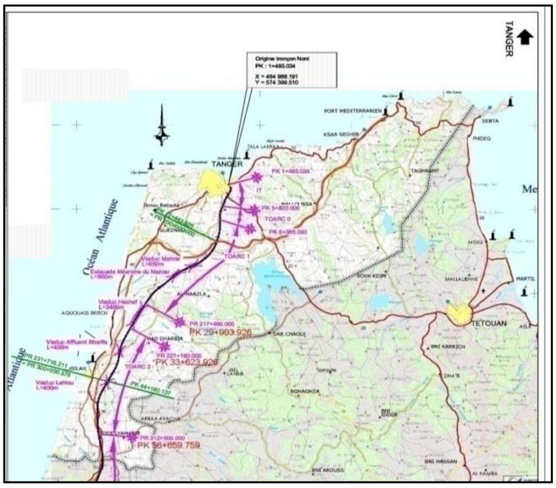

Figure 1. HSR Project plan

Figure 2. Site of embankment 3119

The thickness of compressive layers varies between 6m and 22 meters. The ground profil and some soil properties at the sites of embankments are given in Table1. Underneath embankments, vertical band drains were installed in a square pattern varying from (1.3m x 1.3m) to (2.5m x 2.5m), with 5cm of equivalent diameter and 8 m to 20 m length.

2. 3. Soil Foundation Several laboratory and field tests were performed on the entire area of each embankment by a certified laboratory. These tests include 2 field vane tests; 24 drill holes; 24 pressuremeter tests; 13 static and dynamic cone penetrometer tests.

In addition, a considerable amount of laboratory tests were performed. These included oedometer consolidation tests and triaxial tests. For the stability, compressibility and consolidation analysis, the relevant soil parameters were deduced from laboratory tests. However, to avoid sampling disturbance, many researchers have suggested correlations between undrained shear strength Cu and field tests or laboratory parameters (see Table 2).

TABLE 1. Soil profil and soil properties of the four embankments

Embankment	Height (m)	Soil Layer	Thickness (m)	Unit Weight (KN/m^3)
2288	9	overconsolidated clay	2	20
		stiff clay	3	20
		mud	5	19
		silty clay	3	20
		pelitic clay	3	19
3058	6.5	brown clay	3	18
		marl clay	4	18
		altered pelite	10	20
3078	10.5	fallen rock	6,5	20
		altered pelite	7	20
3119	7.5	fallen rock	6	18
		altered pelite	10	18

In this work, Cu was correlated with available field tests parameters derived from pressurmeter test, field vane test and static cone penetrometer test. The lowest value provided by all these correlations is then considered for the analysis. Results are shown in Table 3.

TABLE 2. Correlation of undrained shear strength with some soil parameters

Tests	Formula	conditions	observations
Field vane test	$C_u = \mu \times Su$	μ = 0.95 for IP = 20 μ = 0.90 for IP = 30 μ = 0.85 for IP = 40 μ = 0.80 for IP = 50 μ = 0.75 for IP = 60 μ = 0.70 for IP = 70	μ is the coefficient of Bjerrum [10]
Triaxial test	$C_u = (\sigma'h \times \sin\Phi_{cu}) / (1 - \sin\Phi_{cu}) + (C_{cu} \times \cos\Phi_{cu}) / (1 - \sin\Phi_{cu})$	Theoretical formula	$\sigma'h = K0 * \sigma'v$ & $K0 = 1-\sin\Phi'$ (jaky [11])
Static cone penetrometer	$C_u = q_c/15$ pour qc < 1 MPa	for qc < 1 MPa	correlation for clay soils [12]
pressuremeter	$Cu = (pl–p_o)/5.5$	$(pl-p_o)<0.3MPa$	Cassan [13]
	$C_u = (pl–p_o)/12+ 0.03$ MPa $C_u=(pl-p_o)/10+ 0.025$ Mpa	$0.3MPa \leq (pl-p_o) \leq 1MPa$	
	$C_u=(pl-po)/35+0.085$ MPa	$1MPa \leq (pl-p_o) \leq 2.5$ MPa	
	$C_u = 0.21\ pl^{0.75}$	Cu and pl in bars	Baguelin and Jezequel [14]
Compression test	$C_u= Rc/2$	Theoretical formula	
Oedometer test	$C_u= (0.3$ à $0.35) \times \sigma'_p$		Leroueil et al. [12]
Plasticity index (Ip)	$C_u=(0.11 + 0.0037\ I_p) \times \sigma'_{v0}$	For normally consolidated soils	Skempton and Henekel [15]

TABLE 3. Mechanical properties of soil foundation

embankment	In-situ parameters										Compressibility parameters			
	Soil layer	δ (kN/m^3)	Wl (%)	pl* (MN)	EM (Mpa)	α	Qc (Mpa)	Cu (kPa) correlated with Pl*; CPT or Su	σ'p (kPa) Laboratory test	σ'p (kPa) formula (1)	e0	Cc	Cs	Cv (m^2/s)
2288	Clay	20		0,8	4	0,67	1,5	96		320	0,64	0,31	0,078	$4,3x10^{-8}$
	Clay	20	44-48	0,5	3,3	0,67	1,2	72	195	240	0,64	0,31	0,078	$4,3x10^{-8}$
	Muck	19	40-55	0,29	1	0,67	0,4	26	35-130	87	1,03	0,48	0,12	$8,2x10^{-9}$
	Clay	20	59	0,8	3,4	0,67	1	66	200	220	1,03	0,24	0,075	$2,1x10^{-8}$
	Alluvioms	20	59	1,8	10	0,25	3	-	-		Pressurmeter calculation			
	Silts	19		3,7	25	0,5		-	-		Pressurmeter calculation			
	Clay	19	66	2,33	25,3	0,67		110	250	366	0,82	0,19	0,072	1,2x10-7
	Mudstone grise							-	-		Incompressive layer			
3058	Clay	18	24-79	0.41	4,56	0,67	2,5	70	120-200	233	1,165	0,26	0,06	$4x10^{-8}$
	Clay	18	46-60	0.75	7,7	0,67	2,3	70	130-260	233	1,041	0,33	0,09	$4x10^{-8}$
	Mudstone altérées	20	35-87	1.2	18	0,67		100	210-310	333	-	-	-	$6,4x10^{-8}$
	Mudstone										Incompressive layer			
3078	Clay	20	50-96	0,64	8,10	0,67		81	80	270	0,922	0,28	0,069	$5,72x10^{-8}$
	Clay	20	62-81	1,03	14,31	0,67		111	210	370	0,171	0,36	0,099	$7,97x10^{-7}$
	Mudstone										Incompressive layer			
3119	Clay	18	31-47		5.01	0.67		83	61	276	0.674	0.216	0.01	$6.11x10^{-8}$
	Alluvioms lâches	19			4.81	0.67		80						
	Alluvioms denses	22			10	0.67		105		350	0.344	0.2	0.036	$8x10^{-8}$
	Clay	18	42-74		20.23	0.67		115	195-200	383	0.769	0.26	0.022	$1.3x10^{-7}$
	Mudstone		59-65								Incompressive layer			

Furthermore, since specimens disturbance has also a significant effect on preconsolidation pressure Ϭ'p of soft soils, Ϭ'p was correlated with undrained shear strenght Cu using Equation (1) [12]. The undrained shear strenght Cu, preconsolidation pressure Ϭ'p, field test parameters and compressibility parameters are summarized in Table 3.

$$\sigma' \mathrm{p} = \frac{\mathrm{Cu}}{0.36} \quad (1)$$

3. DETERMINATION OF SETTLEMENTS AND CONSOLIDATION RATES

3. 1. Settlements For settlement calculations, the embankment load as well as the railway operating load of 30 kPa were taken into account. In order to assess the impact of the corrected preconsolidation pressure showed in Table 3, total vertical stresses are compared to both corrected and non corrected Ϭ'p.

As a result, normally consolidated soil foundation showed an overconsolidated behavior after correction of Ϭ'p. This greatly affects settlement calculation when oedometric method with re-compression index were used. The second method which was used to estimate settlements in soft soils is the elastic method by injecting the pressuremeter mdulus Em. Therefore, settlements could be determined by considering the embankment as a strip footing and using the formula below:

$$\mathrm{S} = \int_0^h \frac{\alpha(z).\Delta 6(z)}{E(z)} \mathrm{dh} \quad (2)$$

where:
ΔϬ(z): permanent vertical excess pressure due to embankment at depth z,
E(z): pressuremeter modulus at depth z,
α(z): rheological coefficient depending on type of soil at depth z,
h: layer thickness.

Table 4 summarizes the overall settlement values obtained by both methods: oedometric method with re-compression index and elastic method within pressuremeter modulus. These values were then compared to measured settlements. The results showed that the second method is the most efficient and provide and excellent match with the field measurments.

3. 2. Consolidation Rates At the early stages of Moroccan HSR project, settlement rates were determined using Terzaghi [16] or Barron's solution. Two cases were analysed: preloading without vertical drainage and preloading associated to vertical drainage. In the first case, the most popular method of settlement rates calculation was suggested by Terzaghi [16], which was developed in one-dimensional consolidation theory.

To extract Cv from oedometric curves, the Taylor [4] is more efficient than Casagrande fitting procedure [3]. In the case of a multilayer model, the calculation was performed by using the equivalent consolidation coefficient Cvm which have been defined by Absi [17]. In the areas where the preloading was associated with the vertical drainage, there were no information about horizontal and vertical permeabilities of soil. Therefore, it was not possible to assess the radial consolidation coefficient Cr correctly. To evaluate the time factor Tv, Terzaghi [16] and Barron [1] abacuses were used. In this study, a recalibration method was proposed for the case of preloading associated to vertical drainage. This method is based on the corrlelation between the coefficient of vertical consolidation Cv and the liquid limit Wl proposed by the US-Navy [5] as shown in Figure 3.

In this graph, three curves are plotted, one is the upper limit for Cv for completely remolded samples. The second curve is for Cv-nc for normally consolidated soil. The third curve is the lower limit for Cv-oc: the coefficient of vertical consolidation for overconsolidated soil.

TABLE 4. Settlement values with elastic and oedometric methods

Embankment	Profil	S.elastic (cm)	S.oedo (cm)	S.actual (cm)
2288	PR228+500	7	21	5,7
	PR228+600	25	43	47
	PR228+750	76	65	55,6
	PR228+850	91	75	43,5
	PR228+920	95	81	46,4
3058	PR305+680	15	28	18,6
	PR305+760	20	31	20,7
	PR305+900	15	27	12,6
	PR306+000	15	27	14,3
	PR306+100	17	27	14,5
	PR306+200	19	27	12,9
	PR306+320	16	25	12,3
	PR306+400	14	25	11,9
3119	PR310+500	14.2	19.5	19
	PR310+700	10	15	9,5
	PR310+900	26	25	14,1
	PR311+060	19	23	15
	PR311+300	7	13	9,1
	PR311+500	8	14	9,5
	PR311+700	10	14	9.5

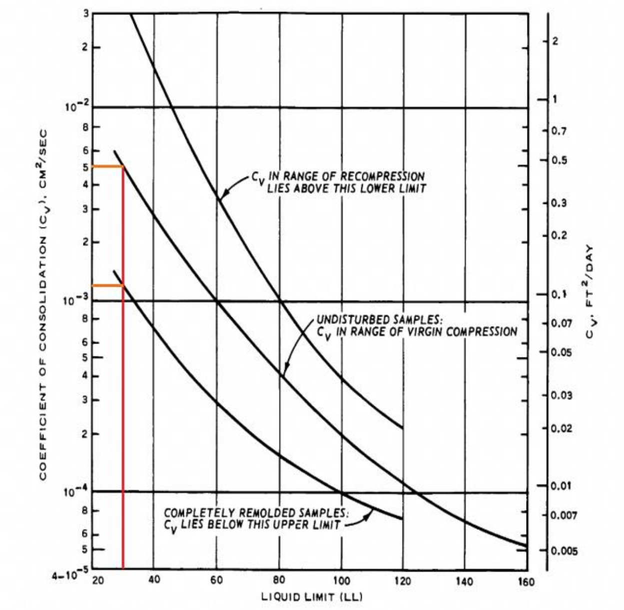

Figure 3. variation of Cv with Wl (according to US Navy 1982)

In Figure 4, an approximate correlation of Cv-nc with the limit liquid is presented. In this work, Cv-nc or Cv-oc are determined from Figures 3 and 4 depending on whether the soil is normally consolidated or overconsolidated. Moreover, unlike the classical method, the assumption Cr = Cv was considered as valid even though most the studied soils were overconsolidated. Indeed, theoretically, the ratio between these two coefficients is equal to the ratio between the horizontal and vertical soil permeabilities. In other words, it is directly related to soil anisotropy. Mariotti [18] showed different structures of clayley soils with equal compactness. He indicated that soil structure due to consolidation process can be either ordered , or unordered or even heterogeneous. For the tested soils, the overconsolidation could be related to the dessication effect, since they are considered as recent deposits. They can be considered, according to Mariotti [18], as unordered and pseudo-isotropic with a low consolidation ratio. Hence, the prediction of Cr = Cv can be regarded as reliable for these soils. To verify the isotropic characteristic for tested soils, several calibration assays were taken into consideration by varying Cr / Cv ratio from 2 up to 12. Then, different curves for the variation of [$t50_{anisotropic}$ / $t50_{isotropic}$] with kh/kv were plotted, where $t50_{anisotropic}$ and $t50_{isotropic}$ are respectively the anisotropic and isotropic half consolidation time, and Kh/Kv are respectively the horizontal and vertical soil permeabilities. In the absence of reliable laboratory or field data, Kh/Kv is assumed equal to Cr / Cv.

Figure 5 show the plotted curves predicted by the calibration assays compared with that of US-Navy [5]. It can be deduced from Figure 5 that, for any value of Cr/Cv between 2 and 12, the corresponding consolidation half time t50 is very short compared with that of US-Navy. It means that the consolidation rate is overestimated. Therefore, only the value Cr / Cv = 1 can be reliable for studied cases.

In the case of overconsolidated soil, it is considered at a first approach that 50% of the total settlement occur during the construction of the embankment. The residual settlement occurs at a rate imposed by the vertical consolidation coefficient Cv. The value of this coefficient depends on whether the soil remains overconsolidated or becomes normally consolidated under the project load. Hence, when the soil remains overconsolidated after the embankment loading, settlement rates calculations are carried out by considernig the coefficient of vertical consolidation for overconsolidated soil Cvoc. Otherwise, the coefficient of vertical consolidation for normally consolidated soil Cvnc is used. Table 5 summarizes the predicted preloading durations compared to measued ones.

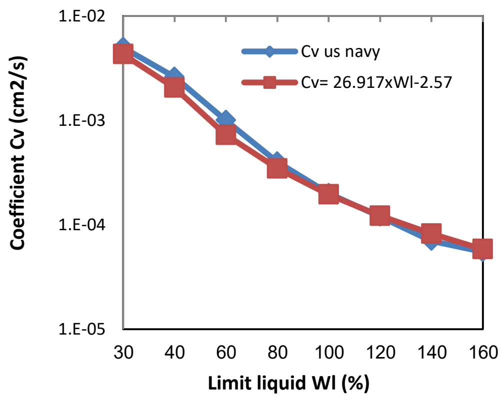

Figure 4. Approximate correlation between Cv and Wl

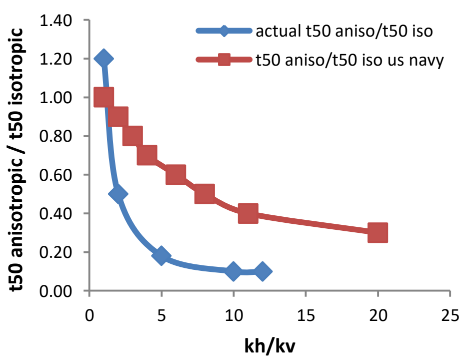

Figure 5. $t50_{anisotropic}$/$t50_{isotropic}$ curves versus kh/kv

TABLE 5. Preloaling period respecting the project specifications

Embankment	Profil	Embankment high (m)	Drains pattern (mxm)	Scour (m)	Preloading period (months)
	PR 228+500	2,72	(2x2)		4,5
	PR 228+600	10,8	(1,3x1,3)		2
2288	PR 228+750	10,8	(1,3x1,3)		2
	PR 228+850	9,8	(1,3x1,3)		2
	PR 228+920	10,5	(1,3x1,3)		2
	PR305+680	7	(1,7x1,7)		8
3058	PR305+760	7	(1,7x1,7)		8
	PR305+900	6	(1,7x1,7)		8
3078	PR 307+886	10,5	(1,5x1,5)	2,7	1
	PR 310+500	5,34	(2,5x2,5)		2
	PR 310+700	7,44	(2,5x2,5)		2
	PR 310+820	8,66	(2,5x2,5)	1	2
	PR 310+900	9,51	(2,5x2,5)		2
	PR 311+067	11,5	(2,5x2,5)	5	2
3119	PR 311+300	7,5	(2,5x2,5)		2
	PR 311+500	5	(2,5x2,5)		2
	PR 311+700	4,48	(2,5x2,5)		2
	PR 311+800	11		5	2
	PR 311+950	11		5	2
	PR 312+187	11	(1,6x1,6)	4	2

The predicted durations must satisfy the design criteria of the Moroccan rail network. It is 10 cm of residual settlement over 25 years, and less than 1 cm of settlement per year, immediately after the end of construction.

4. MEASUREMENT RESULTS

4. 1. Location of Measuring Instruments

The settlements beneath the embankments were measured with the aid of several extensometers and settlement gages. Figure 6 shows the schematic layout of the extensometers installation.

4. 2. Comparison Between Actual and Calculated Settlements

Figures 7 up to 17 illustrate the measured and determined settlement-time curves using either the elastic method or the oedometric method. Both methods were coupled with the method suggested in this paper to take into account the settlements variation with time. The time origin used in all figures matches the end of construction of the embankment. Which means after dissipation of 50% of the final settlement.

For embankment 2288, it can be seen from Figures 7 to 10 that the field measurements approach 90% consolidation at the end of 100 days, and its magnitude is 45cm.

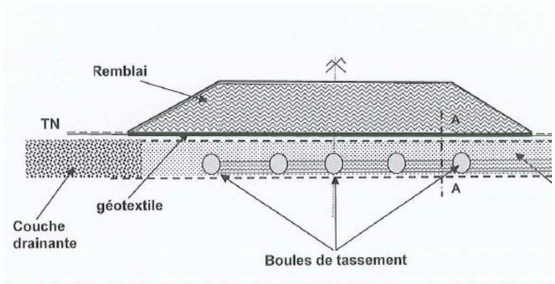

Figure 6. location of extensometers

On the contrary, low settlements are observed for embankments 3058, 3078 and 3119, which is due to the fact that foundation soil is relatively stiff compared to 2288.

In Figures 8, 9, 15 similar variations with time of actual and elastic settlements are noticed. Moreover, final settlements of actual and elastic methods represent nearly half of oedometric settlements. Their maximum value reaches 80 cm for PR228+750 and 100 cm for 228+850. These high values are due to the high thickness of the compressive layer.

However, in Figures 7, 13 actual and elastic settlements vary similarly but are almost two times higher than oedometric settlements.

In Figures 10, 11, it can be seen that actual, elastic and oedometric settlements vary closely, but with some discripancies. In addition, all settlement values don't exceed 20 cm.

Two particular cases were observed in the variation with time of the actual settlement. Either this variation is very fast as seen in Figure 13, or it is very slow as shown in Figure 15.

Overall, small differences were reported between theoretical and experimental settlement curves, especially when the elastic method was used. In contrast, these differences were higher in the oedometric method. Furthermore, through the variation of settlements with time in several profiles, it was showed that the assumption Cr=Cv was valid for all studied soils.

4. 3. Embankment 2288

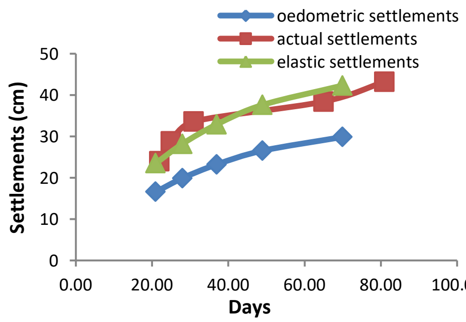

Figure 7. Theoretical and actual settlements versus time PR228+600

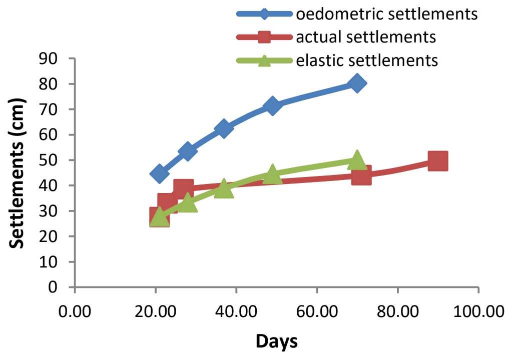

Figure 8. Theoretical and actual settlements versus time PR228+750

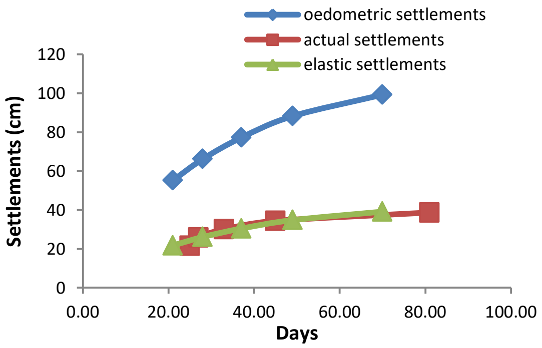

Figure 9. Theoretical and actual settlements versus time PR228+850

4. 4. Embankment 3058

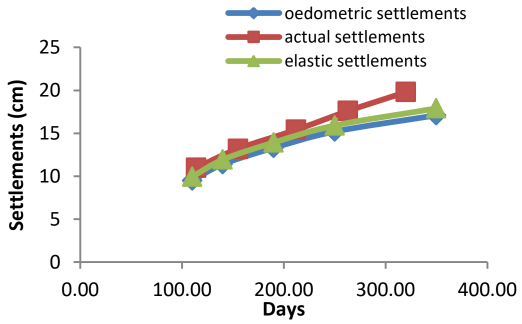

Figure 10. Theoretical and actual settlements versus time PR305+760

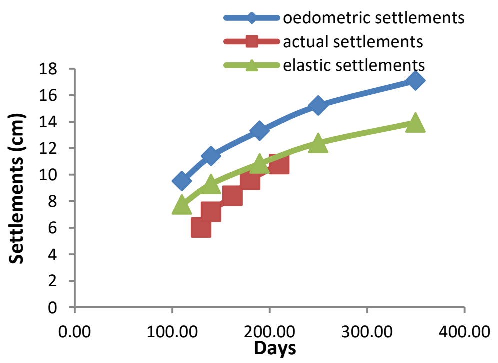

Figure 11. Theoretical and actual settlements versus time PR305+900

4. 5. Embankment 3078

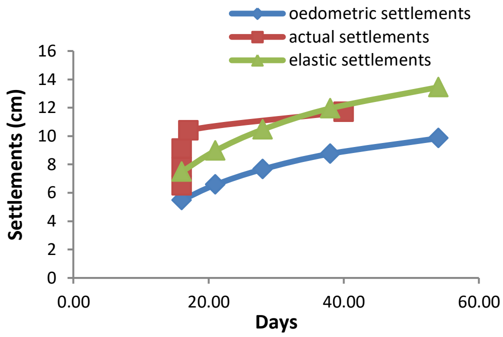

Figure 12. Theoretical and actual settlements versus time PR307+886

4. 6. Embankment 3119

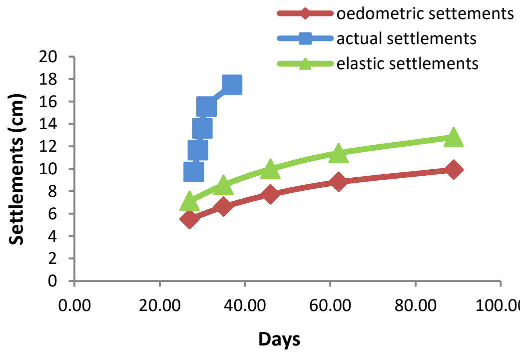

Figure 13. Theoretical and actual settlements versus time PR310+500

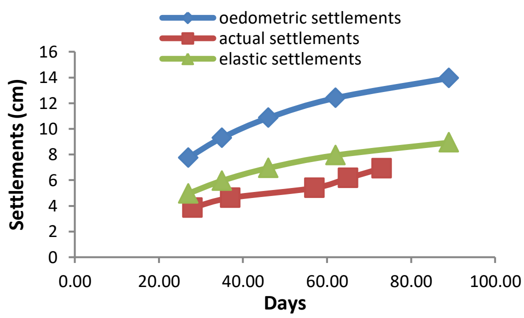

Figure 14. Theoretical and actual settlements versus time PR310+700

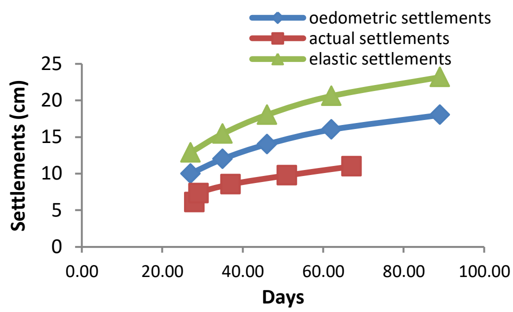

Figure 15. Theoretical and actual settlements versus time PR310+900

4. 3. Comparison Between Actual and Calculated Settlement Rates A comparison was made between our results and Asaoka method [19], based on the radial consolidation coefficient, in order to confirm the accuracy of settlement rates. The Asaoka procedure consists of plotting settlement data points taken at regular intervals after the load has been added. Each settlement data point Tn at time n is plotted against the settlement point Tn-1 of time n-1. Figure 17 shows settlements versus time for PR311+500. Figure 18 shows the application of the Asaoka procedure in interpreting this settlement data. The coefficient Cr is related to the Asaoka line slope given by the following formula:

$$a1 = -\frac{\Delta t}{\ln \beta 1} = \frac{1}{\frac{8Cr}{D^2\, F(n)} + \frac{\Pi^2\, Cv}{4\, H^2}} \qquad (3)$$

where:

Δt: Interval of time taken in the stripping of settlement curves

β: Slope of Asaoka line.

H: Length of vertical drainage path.

D: diameter of drains impact area (equal to the distance L between drains multiplied by 1,05 or 1,13 depending on whether the mesh is triangular or square).

$$\boldsymbol{F(n)} = \frac{\boldsymbol{n^2}}{\boldsymbol{n^2-1}}\ln(\boldsymbol{n}) - \frac{\boldsymbol{3n^2-1}}{\boldsymbol{4n^2}}$$

n = D/d: Ratio of the diameter D of drains impact area to their diameter d.

Cv : vertical consolidation coefficient.

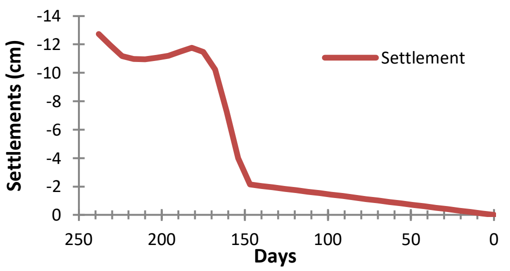

Figure 16. settlements versus time, PR311+500

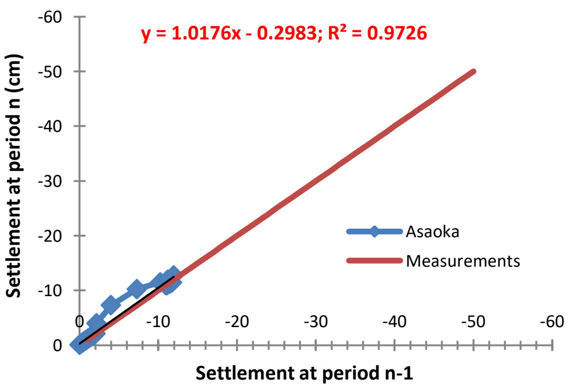

Figure 17. Asaoka line, PR311+500

Table 6 gives, for each instrumented cross section, the values of the radial consolidation coefficient which were determined by the methode suggested in this paper and those determined by using the Asaoka method. Therefore, agreement between the suggested method and asaoka method is relatively acceptable and the discrepancies between evaluated Cr coefficients in both methods were very low.

TABLE 6. Comparison between Cr of the proposed method and Asaoka method

embankment	Profil	Cv_{method} (m^2/s)	Cr_{method} (m^2/s)	Cr_{Asaoka} (m^2/s)
R2288	PR 228+500	2,72E-07	2,72E-07	
	PR 228+600	2,72E-07	2,72E-07	1,42E-07
	PR 228+750	2,72E-07	2,72E-07	1,63E-07
	PR 228+850	2,72E-07	2,72E-07	1,58E-07
	PR 228+920	2,72E-07	2,72E-07	1,14E-07
R3058	PR305+680	1,00E-07	1,00E-07	1,06E-07
	PR305+760	1,00E-07	1,00E-07	1,39E-07
	PR305+900	1,00E-07	1,00E-07	6,59E-08
R3119	PR 310+500	1,00E-06	1,00E-06	1,37E-06
	PR 310+700	1,00E-06	1,00E-06	6,28E-07
	PR 310+900	1,00E-06	1,00E-06	7,09E-07
	PR 311+067	1,00E-06	1,00E-06	
	PR 311+300	1,00E-06	1,00E-06	6,76E-07
	PR 311+700	1,00E-06	1,00E-06	4,55E-07

4. CONCLUSION

This work aimed to study settlements and consolidation rates in High Speed Railway project in Morocco. By comparing measured and calculated settlements, it was showed that the elastic method using the pressuremeter modulus, yields more realistic results. The oedometric method is considered less accurate. Afterwards, the method suggested to estimate the coefficients of vertical and radial consolidation in four embankments of the HSR project, consists on the following steps:

1) The use of elastic method with the pressuremeter modulus for settlement calculations;

2) The estimation of Cv by the US-Navy correlation between Cv and Wl;

3)The assumption of isotropic behavior of soil.

It is shown that the suggested method has led to relevant results. Therefore, it would be recommanded to be used, especially in the pre-design phase of geotechnical works.

5. REFERENCES

1. Barron, R.A., "Consolidation of fine-grained soils by drain wells", (1948).
2. Hansbo, S., "Consolidation of fine-grained soils by prefabricated drains", ***Proc. 10th ICSMFE,*** Vol. 3, (1981), 677-682.
3. Casagrande, A., "Notes on soil mechanics:-first semester, Graduate School of Engineering, Harvard University, (1938).
4. Taylor, D.W., "Fundamentals of soil mechanics, LWW, Vol. 66, (1948).
5. Navy, U., "Soil mechanics, foundations, and earth structures", ***NAVFAC Design Manual DM-7. US Navy, Waghington, DC***, (1971).
6. Sridharan, A. and Nagaraj, H., "Coefficient of consolidation and its correlation with index properties of remolded soils", (2004).
7. Solanki, C., Desai, M. and Desai, J., "Quick settlement analysis of cohesive alluvial deposits using new empirical correlations", ***Journal of Civil Engineering Research and Practice***, Vol. 7, No. 2, (2010), 49-58.
8. Asma, Y., "Al-tae's, abbas f. Al-ameri."Estimation of relationship between coefficient of consolidation and liquid limit of middle and south iraqi soils""", ***Journal of Engineering***, Vol. 17, No. 3, (2011), 430-440.
9. Devi, S.P., Devi, K.R., Prasad, D. and Raju, G.P., "Study on consolidation and correlation with index properties of different soils in manipur valley", (2015).
10. Bjerrum, L., "Embankments on soft ground", in Proc. Specialty Conference on Performance of Earth-Supported Structures. Vol. 2, (1972), 1-54.
11. Jaky, J., "The coefficient of earth pressure at rest", ***Journal of the Society of Hungarian Architects and Engineers***, Vol. 78, No. 22, (1944), 355-358.
12. Leroueil, S., Magnan, J.-P. and Tavenas, F., "Remblais sur argiles molles, (1985).
13. Cassan, M., "Les essais in situ en mecanique des sols-1) realisation et interpretation; 2) applications et methodes de calcul", (1978).
14. Baguelin, F. and LE Mehaute, A., "Essais cycliques au pressiometre autoforeur", ***Bull Liaison Lab Ponts Chauss***, No. SPEC XI F, (1981).
15. Skempton, A. and Henkel, D., "The post-glacial clays of the thames estuary at tilbury and shellhaven", in proceedings of the 3rd international conference on soil mechanics and foundation engineering. Vol. 1, (1953), 302-308.
16. Terzaghi, K., "Erdbaumechanik auf bodenphysikalischer grundlage", (1925).
17. Absi, E., "Generalisation de la theorie de consolidation de terzaghi au cas d'un multicouche", in Annales de l" ITBTP., (1965).
18. Mariotti, M., "Le gonflement des sols argileux surconsolides (aspects du phenomene, influence sur les structures, precautions a envisager)", ***Mines et geologie***, Vol. 39, (1976), 13-28.
19. Asaoka, A., "Observational procedure of settlement prediction", ***Soils and Foundations***, Vol. 18, No. 4, (1978), 87-101.

Settlements and Consolidation Rates under Embankments in a Soft Soil with Vertical Drains

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چکیده

در این مقاله، مطالعه ای برای برآورد میزان شهرک سازی و میزان تلفیق در زیر خاکی های ساخته شده در خاک نرم در مراکش انجام شده است. اندازه گیری های توافقی در چندین دیواره خاکی در پروژه راه آهن سرعت بالا بین دو شهر مراکش، تانگیر وAssilah ، تجزیه و تحلیل شد. هدف از این مطالعه ارزیابی مقادیر سکونت و میزان توافق با دقت کافی در خاک های نرم تحت بارگذاری های خاکی است. مشخص شد که روش الاستیسیته با استفاده از مدول فشار سنج نتایج دقیق تری نسبت به روش اودومتری ایجاد می کند. علاوه بر این، میزان توافق می تواند با استفاده از همبستگی بین ضریب عمودی تثبیت و حد مایع، و همچنین با توجه به رفتار ایزوتروپیک خاک، با دقت درست و عادلانه تعیین گردید.

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