Long-Term Settlement Prediction for Over-Consolidated Soft Clay under Low Embankment

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ABSTRACT

The long-term post-construction settlement of an embankment laid on a deep, soft soil foundation can give rise to a series of safety concerns and significant structural damage. The settlement is primarily attributed to the creep deformation of the soft soil following the removal of the surcharge load and the impact of traffic loads on the soft soil. In this study, a plan strain triaxial test was conducted to investigate the deformation of an undisturbed soft clay specimen subjected to static and cyclic loading. The results demonstrate that the volume creep and vertical creep are associated with the overconsolidation state of the soft soil. The Over-Consolidated Ratio (OCR_q) of shear stress, can be used as a parameter to describe the state of overconsolidation of soft soil under spherical stress. Based on the vertical creep coefficient and considering the influence of stress history on the stress state of soft soil, a two-dimensional long-term settlement model under cyclic loading has been proposed.

Keywords-soft soil; long-term settlement; cyclic loading; static loading; creep coefficient

I. INTRODUCTION

The deformation and strength of soft soil are dependent on time, and this type of soil is widely distributed in many places. As a result, the deformation effects of soft soil affect urban development. There is a growing concentration of infrastructural constructions in densely populated urban areas, which induce long-term settlement, long-term strength, and creep deformation. A combination of quantitative and qualitative studies of soil deformation were conducted, and solutions were proposed [1]. The time-dependent deformation of cohesive soils can be attributed to two primary factors: hydrodynamic lag (consolidation) and the viscous behavior of the soil skeleton (creep). The creep behavior of soil is contingent upon a number of primary factors, including time, temperature, soil type, soil structure, stress history, stress state, and drainage conditions [2]. The creep deformation of overconsolidated soft soil represents a principal component of post-construction settlement of soft soil foundations, particularly in instances of repeated loading. It is generally accepted that the creep settlements of soft clay after primary consolidation are smaller than the primary consolidation settlements. However, creep settlements of soft clay may affect the safe operation of certain civil engineering structures and are a significant design issue, particularly when the structure is sensitive to ground movements at its foundation or is situated on a thick layer of soft clay [3].

The deformation effect under examination is found to bear a relationship with the normalized shear strength of over-consolidated clay and the normalized shear strength of normally consolidated clay [4]. The ratio of the two quantities is a simple power function of an OCR value [5]. Additionally, assessments of lateral earth pressure at rest were conducted by authors in [6] and the study indicates that for normally consolidated soils, the volumetric strain of creep in plane strain tests varies in a manner analogous to the secondary consolidation observed under one-dimensional conditions. It was therefore reasonable to determine the volumetric creep coefficient and the axial creep coefficient directly by using the OCR_p [7]. Other studies have demonstrated that the OCR_q, which is defined by generalized shear stress, is a function of the principal stress ratio and the pre-consolidation pressure. This ratio has been shown to more accurately describe the behavior of over-consolidated soft clay than the over-consolidation spherical stress ratio. Additionally, it was determined that in plane strain tests, the creep coefficients decrease with increasing OCR_q. Furthermore, for a given OCR_q, over-consolidated soils with a small principal stress ratio exhibit larger creep coefficients [8].

In the field of empirical modeling, two significant deficiencies have been persistently overlooked for an extended period. Firstly, it is imperative that clayey deposits undergo preloading using a surcharge, vacuum load, or a combination of both, prior to the construction of the upper pavement. The preloading process serves to reduce post-construction settlement in soft foundations. However, upon the removal of the load, the clay transitions into an Over-Consolidated (OC) state. Despite having undergone substantial past effective stress, which leads to a high OCR, the soft foundation may still experience significant deformation [9-13]. The second issue pertains to the fact that soft foundations situated beneath high embankments typically exist in a plane strain condition. It is unfortunate that current empirical models are unable to account for the combined effects of both OCR and plane strain conditions directly. In this study, both the OCR_p and the OCR_q are employed to elucidate the creep behavior of over-consolidated soft clay subjected to static and cyclic loading under plane strain conditions. Furthermore, an empirical formula is proposed which reflects the relationship between the OCR and creep deformation coefficients of soft clay, with the aim of providing a more accurate description of the creep behavior of soft clay under different loading conditions in a plane strain state. Finally, the proposed equation is applied to case histories from Guangdong province, China, and its efficacy is demonstrated.

II. LABORATORY STUDIES

A. Sample and Specimen

Undisturbed soil samples were collected from depths ranging from 4.0 m to 9.0 m in a soft clay deposit beneath the Shanfen Highway embankment in China. The plane strain creep tests were conducted on cuboid-shaped specimens with dimensions of 10 cm in height, 5 cm in width, and 10 cm in length. The specimens were confined within a chamber constructed from rigid metal plates, and loads were applied in both the vertical and lateral directions (along the *z* and *x* axes, respectively). Concurrently, the strain was constrained in the *y* direction, as shown in Figure 1. The soil properties were as follows: density ρ = 1.52~ 1.56 g/cm^3, natural water content w_n = 81.4~ 86.6%, plasticity index I_P = 32~ 35%, liquidity index I_P = 1.1~ 1.5%, initial void ratio e = 1.65~ 1.83, compression factor $\alpha_{0.1\text{-}0.2}$ = 2.61~ 2.84MPa^{-1}, compression modulus E_s = 1.11~ 1.22 MPa, coefficient of consolidation C_{v100} = 0.448×10^{-3}~ 0.704×10^{-3}cm^2/s and C_{v200} = 0.645x10^{-3}~ 0.667×10^{-3}cm^2/s.

B. Test Procedure

A test setup for creep under plain strain conditions, as proposed by authors in [14-16], was developed and is presented in Figure 2. The test was developed based on the principles of conventional triaxial testing, combining both the loading system of the former and the air pressure controlling system of the latter. To ascertain the variation in strain, lateral pressure, and the magnitude of the cyclic load, the transducers are positioned in accordance with the aforementioned specifications and linked to the channels, which are in turn connected to the display unit that provides the electronic readout. A data acquisition system is linked to the computer.

C. Loading Scheme

A series of creep tests were conducted using the plane strain creep apparatus on both normally and over-consolidated clay, in which the principal stress ratio equals the K_0 value of 0.5. The tests were performed under the stress control method for each loading step with vertical consolidation stresses (σ_z) of 100 kPa, 200 kPa, 300 kPa, and 400 kPa for the static tests and 75 kPa, 100 kPa, and 150 kPa for the cyclic loading tests, as shown in Tables I and II.

III. THE OCR OF SATURATED SOIL IN PLANE STRAIN STATE

The soft foundation may exhibit considerable deformation even when subjected to a significant maximum past effective stress, that is, when the OCR is large.

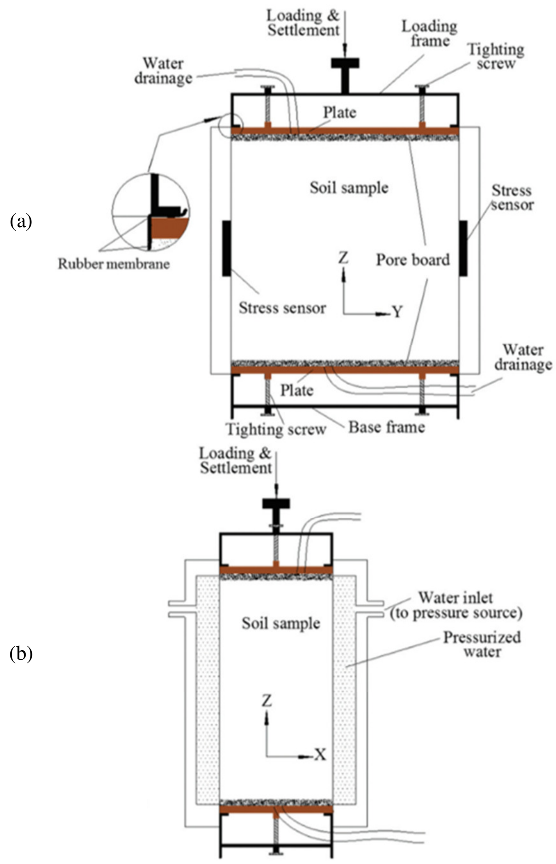

Fig. 1. Soil specimen and pressure chamber in test: (a) from *x* direction, (b) from *y* direction.

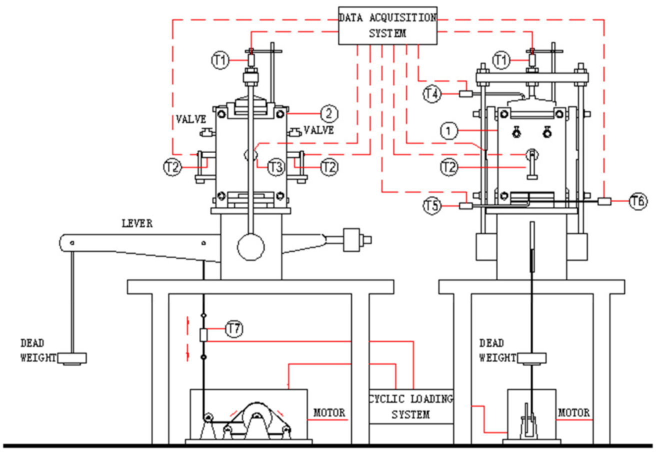

Fig. 2. Sketch of the apparatus:1,2. Rigid Plates; T1. Axial Strain Transducer; T2. Lateral Strain Transducer; T3. Lateral Pressure Transducer; T4. Water Drainage Transducer; T5. PWP Transducer at Base; T6. PWP Transducer at Center; T7. Cyclic Load Transducer.

TABLE I. LOADING SCHEME OF STATIC TESTS

Nº	State	Static Loading (σ_x, σ_z) (kPa)	Duration for loading step /days	
I	Normal consolidation	(25, 25)	Initial	1
		(50, 100)- (100, 200)- (150, 300)- (200, 400)	Creep	7
II	Over consolidation	(25, 25)	Initial	1
		(50, 100)- (100, 200)- (150, 300)	Consolidation	1
		(25, 25)	Rebound	1
		(50, 100)- (100, 200)- (150, 300)	Creep	7
III	Over consolidation	(25, 25)	Initial	1
		(50, 100)- (100, 200)- (150, 300)- (200, 400)	Consolidation	1
		(25, 25)	Rebound	1
		(50, 100)- (100, 200)- (150, 300)- (200-400)	Creep	7

TABLE II. LOADING SCHEME OF CYCLIC TESTS

Nº	State	Static Loading		Cyclic Loading		Duration for loading step /days	
		σ_x (kPa)	σ_z (kPa)	σ_d (kPa)	f (Hz)		
IV-1	Normal consoli-dation	25	25	-	-	Initial	1
		37.5	75	20	0.05	Creep	7
		50	100	20	0.05	Creep	7
		75	150	20	0.05	Creep	7
		25	25	-	-	Rebound	1
IV-2	Over consoli-dation	37.5	75	20	0.05	Creep	7
		50	100	20	0.05	Creep	7
		75	150	20	0.05	Creep	7

The following definition of OCR index is derived from [17-19]:

$$\frac{\varepsilon_{oc}}{\varepsilon_{nc}} = OCR \tag{1}$$

where ε_{nc}, ε_{oc} are axial strain of normally and over-consolidated clay under a constant stress intensity after an elapsed time. Authors in [20] studied the creep behavior of normally and over-consolidated soils through plane strain tests. The study showed that, the volumetric and axial creep coefficient can be determined directly by using a ratio termed the over consolidation spherical stress ratio, OCR_p defined as:

$$OCR_P = \frac{p_m^c}{p_m^0} \tag{2}$$

where $p^c{}_m$, $p^0{}_m$ are the generalized spherical stresses corresponding to pre-consolidation and current loading respectively, expressed as:

$$\begin{cases} p_m^c = \frac{\sigma_{xc}+\sigma_{yc}+\sigma_{zc}}{3} \\ p_m^0 = \frac{\sigma_x+\sigma_y+\sigma_z}{3} \end{cases} \tag{3}$$

where (σ_{xc}, σ_{yc}, σ_{zc}) and (σ_x, σ_y, σ_z) are the principal stresses in x, y, z directions of the specimen corresponding to the highest values of load at pre-consolidation and current pressures. Combining (2) and (3) gives the OCR_p index:

$$OCR_p = \frac{\sigma_{xc}+\sigma_{yc}+\sigma_{zc}}{\sigma_x+\sigma_y+\sigma_z} \tag{4}$$

For a linear isotropic material subjected only to compressive forces, its deformation along one axis will produce deformation along the other axis in three dimensions. Thus, it is possible to generalize Hooke's Law along the y axis as:

$$\varepsilon_y = \frac{1}{E}\left[\sigma_y - \vartheta(\sigma_z + \sigma_x)\right] \tag{5}$$

where ε_y is strain along the y axis; σ_z, σ_x are stresses along the z and x axis respectively, E is Young's modulus and ν is Poisson's ratio. Under plane strain condition, the deformation along the y axis is zero (i.e., $\varepsilon_y = 0$), thus:

$$\begin{cases} \sigma_y = \vartheta(\sigma_z + \sigma_x) \\ \sigma_{yc} = \vartheta_0(\sigma_{zc} + \sigma_{xc}) \end{cases} \tag{6}$$

The principal stress ratio in this study equals the constant ratio of K, which is defined as the ratio of horizontal effective stress to vertical effective stress measured:

$$K = \frac{\sigma_x}{\sigma_z} = \frac{\sigma_{xc}}{\sigma_{zc}} \tag{7}$$

Computing σ_y and σ_{yc} from (6) and combining with (7) and substituting into (3) results in:

$$OCR_p = \frac{\sigma_{zc}(\vartheta_0+1)}{\sigma_z(\vartheta+1)} \tag{8}$$

Authors in [21] studied the effect of principal stress ratio on creep behavior of over-consolidated clay under plane strain and defined the over-consolidation shear stress ratio, OCR_q as:

$$OCR_q = \frac{q^c}{q^0} \tag{9}$$

where q_c, q_0 is generalized shear stresses corresponding to pre-consolidation and current loading respectively, expressed as:

$$\begin{cases} q^c = \frac{1}{\sqrt{2}}\sqrt{\left(\sigma_{xc}-\sigma_{yc}\right)^2 + \left(\sigma_{yc}-\sigma_{zc}\right)^2(\sigma_{zc}-\sigma_{xc})^2} \\ q^0 = \frac{1}{\sqrt{2}}\sqrt{\left(\sigma_x-\sigma_y\right)^2 + \left(\sigma_y-\sigma_z\right)^2(\sigma_z-\sigma_x)^2} \end{cases} \tag{10}$$

where (σ_{xc}, σ_{yc}, σ_{zc}) and (σ_x, σ_y, σ_z) are the principal stresses in x, y, z directions of specimen corresponding to the highest values of load at pre-consolidation and current pressures. Computing σ_y and σ_{yc} from (6) and combining with (7) and substituting into (10) results in:

$$OCR_q = \frac{\sigma_{zc}}{\sigma_z}\sqrt{\frac{\vartheta_0(\vartheta_0-1)+\frac{K^2-K+1}{(K+1)^2}}{\vartheta(\vartheta-1)+\frac{K^2-K+1}{(K+1)^2}}} \tag{11}$$

where ν_0 and ν is Poisson's ratio corresponding to over-consolidated and normally consolidated states respectively, and K is the principal stress ratio. This study presents a summary of the values of OCR_p and OCR_q, calculated based on the results of the plane strain test under static and cyclic loading for a period of seven days, as presented in Tables III and IV, respectively. Figure 3 shows the correlation between OCR_p and OCR_q under both static and cyclic loading scenarios and data are best fitted with a linear relation. As indicated by authors in [21], the OCRs under plane strain conditions are dependent upon the principal stress ratio and the pre-consolidation pressure. Furthermore, their relation with the OCR of soil samples in oedometer creep tests is:

$$OCR_p < OCR < OCR_q \tag{12}$$

TABLE III. SUMMARY OF OVER-CONSOLIDATION RATIO IN PLANE STRAIN STATE UNDER STATIC LOADING

States	Curent stress/kPa			Pre-consolidation stress/kPa			OCR_p	OCR_q
	σ_x	σ_y	σ_z	σ_{xc}	σ_{yc}	σ_{zc}		
Normally	50	100	14.5	50	100	14.5	1.0	1.0
	100	200	42	100	200	42		
	150	300	90	150	300	90		
	200	400	137	200	400	137		
Over cosonlidation	50	100	46	150	300	100	2.81	3.46
	100	200	79				1.45	1.61
	150	300	109				0.98	1.04
	50	100	59	200	400	158	3.63	4.85
	100	200	91				1.94	2.14
	150	300	133				1.30	1.41
	200	400	175				0.99	1.05

TABLE IV. SUMMARY OF OVER-CONSOLIDATION RATIO IN PLANE STRAIN STATE UNDER CYCLIC LOADING

States	Curent stress (kPa)			Pre-consolidation stress (kPa)			OCR_p	OCR_q
	σ_x	σ_y	σ_z	σ_{xc}	σ_{yc}	σ_{zc}		
Normally	37.5	75	27.2	37.5	75	27.2	1.0	1.0
	50	100	36.1	50	100	36.1		
	75	150	52.8	75	150	52.8		
	200	400	137	200	400	137		
Over	37.5	75	47.5	75	150	52.8	1.65	2.03
	50	100	54.4				1.33	1.59
	75	150	69.7				0.95	1.10

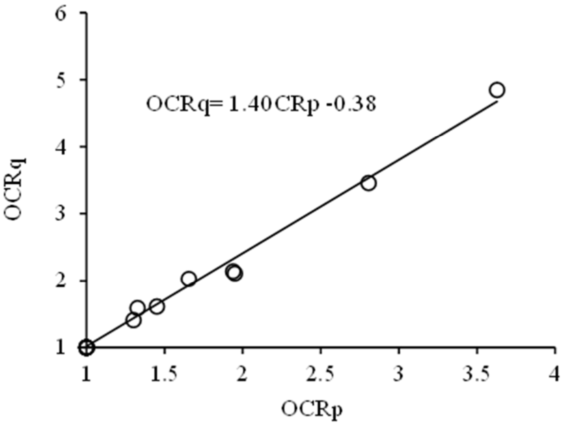

Fig. 3. Relationship between OCR_p and OCR_q.

IV. EFFECT OF THE OVER CONSOLIDATION RATIO ON THE CREEP BEHAVIOR OF CLAY

Figure 4 shows the variation of the axial creep strain, ε_z, and the logarithm of the elapsed time for the overconsolidated soil specimens with different values of OCRs subjected to static and cyclic loading. The curves show that ε_z increases with decreasing OCRp and OCRq for both static and cyclic loading cases. The slope of the tangent shown by the dashed lines indicates that the variation of the axial strain during creep depends on the values of the OCRs index. The higher the value of OCRs, the slower is the rate of variation of the axial creep strain. The axial creep strain coefficient in plane strain tests is defined as the ratio of the axial strain increment during creep and the corresponding time increment, given by:

$$\beta = \frac{\Delta\varepsilon_z}{\Delta \log t} \tag{13}$$

where β, and $\Delta\varepsilon_z$ are the axial creep strain coefficients and axial creep strain increment, respectively, in plane strain state. In this study, the values of β determined from the plane strain test results are summarized in Table V, which shows that for over-consolidated soil specimen, the rate of variation of axial strain increases with increasing external load level. In addition, the change in creep strain coefficients when the soil is subjected to cyclic loading is much smaller than when the soil is subjected to static loading.

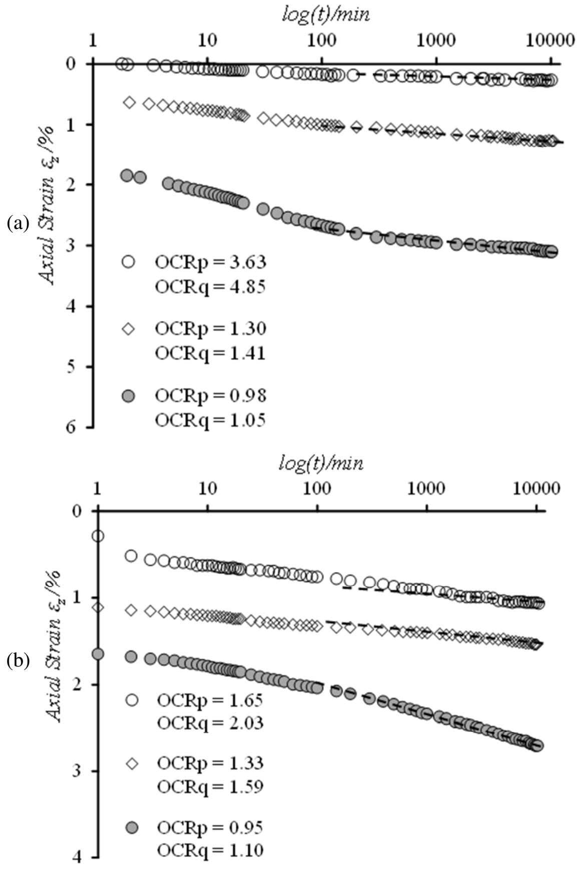

Fig. 4. Axial of volumetric creep strain for different OCR_p and OCR_q: (a) Static loading, (b) Cyclic loading.

V. PROPOSED FORMULA TO CALCULATE THE LONG- TERM SETTLEMENT

Figure 5 shows the relationship between the ratio of the creep coefficient of axial strain for over-consolidated soil to the creep coefficient of axial strain for normally consolidated soil (β_O/β_N) and the different values of OPRp and OPRq for both static and cyclic loading conditions. Test results and theoretical formula show that β decreases with increasing OCRs, and the creep strain coefficient of soft clay that is required at OCR is more than 1, the trend of the curves is quite identical. Based on the test results, (2) shows the effect of overconsolidation on the axial creep strains of soils subjected to static and cyclic loading, applicable to the principal stress ratio, K, of 0.5.

$$\frac{\beta^0}{\beta^N} = \begin{cases} 0.04 + 0.96e^{[5.81(1-OCR_p)]} \\ 0.08 + 0.92e^{[2.42(1-OCR_p)]} \end{cases} \tag{14}$$

$$\frac{\beta^0}{\beta^N} = \begin{cases} 0.07 + 0.93e^{[6.85(1-OCR_q)]} \\ 0.26 + 0.74e^{[3.19(1-OCR_q)]} \end{cases} \tag{15}$$

TABLE V. SUMMARY OF CREEP STRAIN COEFFICIENTS IN PLANE STRAIN STATE UNDER DIFFERENT LOADING CONDITION

Load	States	Curent stress (kPa)			Pre-consolidation stress (kPa)			β/10^{-3}
		σ_x	σ_y	σ_z	σ_{xc}	σ_{yc}	σ_{zc}	
Static loading	Normally	50	100	14.5	50	100	14.5	0.466
		100	200	42	100	200	42	0.598
		150	300	90	150	300	90	0.933
		200	400	137	200	400	137	0.902
	Over cosonlidation	50	100	46	150	300	100	0.352
		100	200	79				0.845
		150	300	109				2.159
		50	100	59	200	400	158	0.255
		100	200	91				0.644
		150	300	133				0.958
		200	400	175				1.501
Cyclic loading	Normally	37.5	75	27.2	37.5	75	27.2	1.136
		50	100	36.1	50	100	36.1	1.667
		75	150	52.8	75	150	52.8	1.734
		200	400	137	200	400	137	0.436
	Over	37.5	75	47.5	75	150	52.8	0.658
		50	100	54.4				1.352
		75	150	69.7				0.466

Figure 6 displays the relationship between the creep coefficients for normally-consolidated soil β_N and the different values of principal stress ratio, K, for cyclic loading conditions. The curves shows that β_N increases rapidly with increasing of $1/K$ for cyclic loading case. The relation is best fitted with the linear equation:

$$\beta^N = 0.449\frac{1}{K}\text{-}0.656 \tag{16}$$

VI. FORMULA DERIVATION AND ANALYSIS

In the case of uniform loading, the settlement of the i^{th} soil layer with hi being the average thickness, is given by:

$$S_i = \varepsilon_{zi} h_i \tag{17}$$

where, ε_{zi} is the axial creep strain of the i^{th} soil layer, and ε_{zi} is calculated from (13) with assumption the value of axial creep coefficient, β_i, is constant during secondary consolidation under over-consolidated state:

$$\varepsilon_{zi} = \beta_i^0 \log \Delta t \tag{18}$$

Based on the test results, the proposed formulas of (15) and (16) which have been developed to reflect the effect of over-consolidation on the axial creep strains of soils subjected to cyclic loading:

$$\frac{\beta_i^0}{\beta_i^N} = a + (1-a)e^{[b(1-OCR_{qi})]} \tag{19}$$

$$\beta_i^N = \frac{c}{K} + d \tag{20}$$

where, β_i^O, β_i^N are the creep coefficient of axial strain for over-consolidated and normally-consolidated soil, respectively, OCR_{qi} is the over-consolidation shear stress ratio is defined by (8), K is the principal stress ratio is calculated by $K=\sigma_{xc}/\sigma_{zc}$, and parameters of a, b, c and d are constant, depended on the soil properties, are obtained from the plane strain tests analysis.

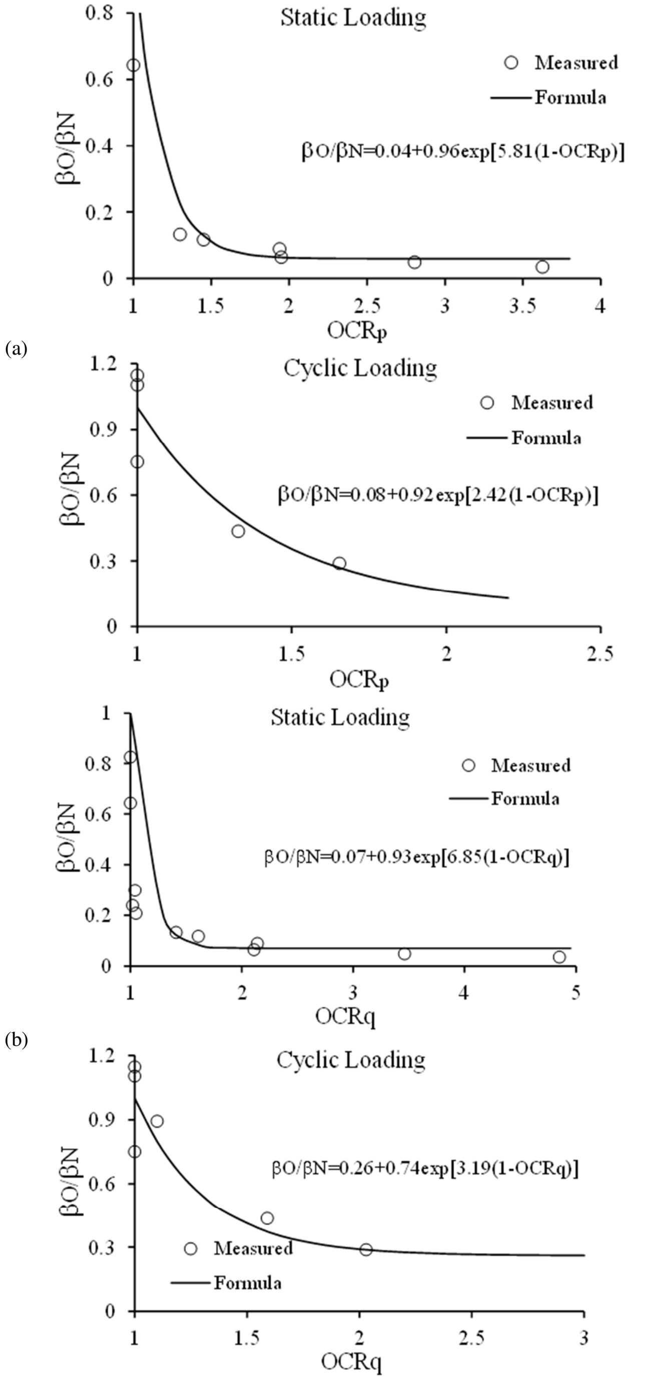

Fig. 5. Axial creep coefficient for different: (a) OCR_p and (b) OCR_q for both static and cyclic loading.

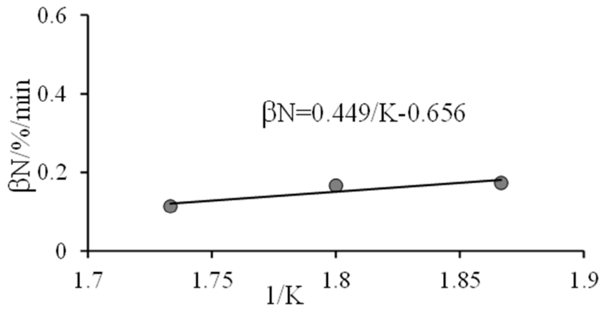

Fig. 6. Relationship between the creep coefficients and principal stress ratio, K, for cyclic loading conditions.

Computing β_i^N from (20) and combining with (19) and substituting into (18) results:

$$\varepsilon_{zi} = \left(c\frac{1}{K} + d\right)\left\{a + (1-a)e^{[b(1-OCR_{qi})]}\right\}\log(\Delta t) \quad (21)$$

Combining (17) and (21) gives the settlement of the i^{th} soil layer, S_i:

$$S_i = \left(c\frac{1}{K} + d\right)\left\{a + (1-a)e^{[b(1-OCR_{qi})]}\right\}\log\left(\frac{t}{t_0}\right)h_i \quad (22)$$

where, K is the principal stress ratio is calculated by $K=\sigma_{xc}/\sigma_{zc}$, h_i is the average thickness of i^{th} soil layer, OCR_{qi} is the over-consolidation shear stress ratio, t_0, is the starting time of the secondary consolidation, taken as the time for removal of surcharge (i.e., 50 days), t, is the finishing time of secondary consolidation, taken as the time settlement was measured after the road was opened to traffic. In the (22), the parameters of a, b, c, d are obtained from results of a series of plane strain creep tests which are carried out for soft remolded clay under static and cyclic loading. In this study, the a, b, c, d values, are 0.260, 3.190, 0.449 and -0.656, respectively.

VII. APPLICATION OF PROPOSED FORMULA

The proposed formula is applicable to the evaluation of long-term settlement of surcharge-preloaded low embankments on soft ground subjected to cyclic loading. The proposed formula has been validated using total settlement data from the K34+850 section of the Jie Pu highway in Guangdong. It should be noted, however, that the total settlement comprises two distinct components: consolidation settlement due to embankment load and settlement induced by traffic loading. Therefore, the settlement prediction was conducted by summing the settlements under both embankment and traffic loading. The extent of settlement was calculated using a one-dimensional consolidation analysis, taking into account the effects of cyclic loading time. Construction of the Jie Pu highway, situated on a deposit of soft soil, commenced in December 2001 and was completed in December 2003, at which point it was opened to traffic. To investigate post-construction settlement, field measurements were performed along the K36+150 section. The soil properties at this section are presented in Table VI. The soil at this site comprises, from top to bottom, loose fill, silty sand, silty mud, sandy clay, muddy clay, and sand. Table VI shows that the alluvial deposit is comprised of two distinct layers: a silty mud layer and a peaty clay layer, with a marine silty sand layer and a sandy clay layer, respectively. These layers are situated at depths ranging from 2.7 m to 5.6 m and from 8.7 m to 17.0 m, respectively. A layer of coarse sand is present below a depth of 17.0 m. It can be posited that all of these layers may act as drainage layers. The overburden stresses on the ground are typically in a state of consolidation when compared with consolidation yield stresses along the depth direction.

TABLE VI. INDEX PROPERTIES OF SOIL AT SECTION K36+150

Soil layer	*h* (m)	*w* (%)	γ(kN/m^3)	*e*	α (MPa^{-1})	I_p
Silty sand	2.7	27.4	19.4	0.70	0.3	13
Silty mud	2.9	72.5	14.7	1.88	2.1	16
Sandy clay	3.1	35.0	19.1	0.65	0.3	12
Muddy clay	8.3	50.7	13.3	1.32	0.74	-
Coarse sand	29.3	21.3	18.6	0.75	0.37	10

h = thinkness of soil layer; w = moisture content; γ = unit weight; e = void ratio; α = compression factor; I_p = plasticity index.

The total post-construction settlement of the low embankment was obtained by summing the predicted settlements due to both embankment and traffic loading for the second layer. The settlement of the fourth layer was computed using only embankment loading and the self-weight of soil above this layer. The parameters were determined from the data presented in Table VII. Once more, the requisite stress components may be obtained by integrating the appropriate Boussinesq expressions over the rectangular area. For engineering design purposes, the most crucial component is the vertical and lateral direct stress, for which a number of solution approaches have been proposed. The distribution of stress caused by a vertical strip load (finite width and infinite length) is given by (23) and (24). These equations are derived from the Boussinesq solution of stresses produced at any point in a homogeneous, elastic, and isotropic medium as the result of a point load applied on the surface of an infinitely large half-space (Boussinesq, 1883):

$$\sigma_z = \int_0^b \frac{2p_n}{\pi}\frac{z^3 d\zeta}{[(x-\zeta)^2+z^2]^2} = K_s^z p_n \quad (23)$$

$$\begin{cases}\sigma_x = K_s^x p_n \\ \tau_{xz} = K_s^\tau p_n\end{cases} \quad (24)$$

TABLE VII. PARAMETERS FOR CALCULATION THE SETTLEMENT OF EMBANKMENT AT K36+150

Layer	*h* (m)	*b* (m)	*x* (m)	*z* (m)	*p* (kPa)	ΔP_1 (kPa)	ΔP_2 (kPa)	σ_d (kPa)
2nd	2.9	26	0	4.15	44.2	87.8	56.4	20
4th	8.3	26	0	12.85	92.9	77.1	48.4	20

h = thickness of soil layer, b = the total width of road pavement, x and z = normal coordinates of the settlement calculated point; P = the average stress due to self-weight of soil layer; ΔP_1, ΔP_2 = the average stress due to surcharge preloading before and after unloading, σ_d = the amplitude of traffic cyclic loading, respectively.

Authors in [22] plotted sets of curves for values of K are a function of $m = x/b$ and $n = z/b$ as shown in Figure 7, giving the vertical stress at depth z under a corner of a uniformly-loaded area of normal coordinates x and breadth B. Alternatively, value of K_s^z, K_s^x, K_s^τ can be determined analytically and tubulated for value of $m = x/b$ and $n = z/b$. The settlement was calculated using two-dimensional consolidation analysis, with the parameters determined from the data shown in Table VIII, in which:

$$\sigma_y = \nu_0(\sigma_x + \sigma_z) \quad (25)$$

with:

$$\nu_0 = 0.181OCR_p + 0.064 \quad (26)$$

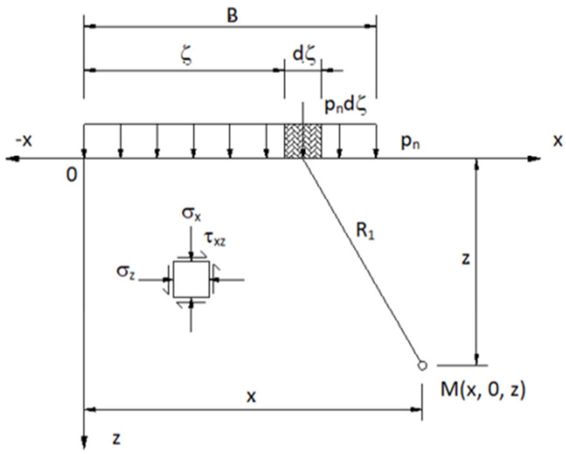

Fig. 7. Distribution of stress due to a uniformly strip load.

TABLE VIII. SUMMARY OF PRINCIPAL STRESSES IN X, Y, Z DIRECTIONS FOR CALCULATING THE SETTLEMENT OF EMBANKMENT AT K36+150 SECTION

Soil Layer	K_s^z	K_s^x	ν_o	OCR_{qi}	k 10^{-2}
2nd	0.498	0.393	0.26	1.10	78.9
4th	0.541	0.213	0.26	1.06	39.4

K_s^z, K_s^x, is the influence factors for stress under the center of a uniformly-loaded area at the depth z; ν_0 is Poisson's ratio for over-consolidated states; (σ_{xc}, σ_{yc}, σ_{zc}) and (σ_x, σ_y, σ_z) are the principal stresses in x, y, z directions corresponding to the highest values of load at pre-consolidation and current pressures, respectively.

Figure 8 presents the comparison of the calculated settlement for both the 1-D and 2-D consolidation analyses and the observed settlement history of the embankment at the K36+150 section. In fact, the settlement calculated according to the conditions of plane strain state (2-D) is greater than that in the analysis with 1-D consolidation. Furthermore, the difference (ΔS) has been observed to increase with elapsed time. The ΔS values of 2.65 mm (approximately 5.4%) and 6.92 mm (approximately 16.2%) were obtained for elapsed time values (Δt) of 1,480 days (approximately 4 years) for comparison of the 2-D and 1-D calculated with observed data, respectively. In the proposed formulas for both analysis cases, the value of the starting time of secondary consolidation, t_1, is taken as the time required for the removal of surcharge (i.e., 50 days). The finishing time of secondary consolidation, t_2, is taken as the time at which settlement was measured after the road was opened to traffic. It is important to note that the measurements were taken 50 days after construction, during which the calculated settlement according to the proposed formula assumes that settlement under traffic loading occurs under conditions of drained cyclic loading, in which the excess pore pressure generation was dissipated completely.

VIII. CONCLUSIONS

In this study, the Over-Consolidation spherical (OCRp) stress ratio and the Over-Consolidation shear (OCRq) stress ratio, are employed to elucidate the creep behavior of over-consolidated soft clay subjected to static and cyclic loading under plane strain conditions. The axial creep strain increases with a reduction in both the OCR_p and the OCR_q for both static and cyclic loading cases. Furthermore, the higher the value of OCRs, the slower the rate of variation of the axial creep strain.

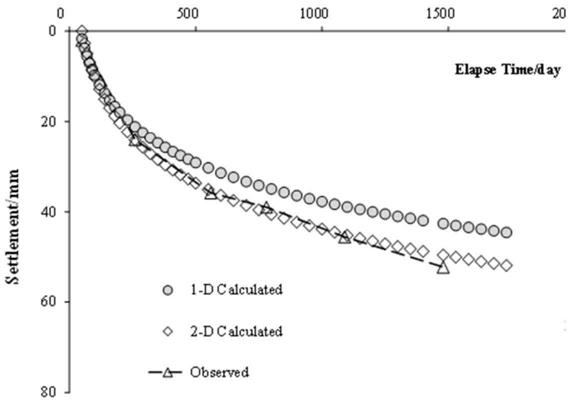

Fig. 8. Calculated and observed settlement histories.

In specimens of over-consolidated soil, the rate of variation of axial strain increases with the level of external loading. Furthermore, the alteration in creep strain coefficients when the soil is subjected to cyclic loading is considerably less significant in comparison to when the soil is subjected to static loading. Based on the test results, empirical formulas have been developed to reflect the effect of over-consolidation on the axial creep strains of soils subjected to static and cyclic loading, applicable to different values of the principal stress ratio, K. Discussions on the validity of the proposed method as well as the effect of the over-consolidation shear stress ratio, OCRq, and the axial creep strain coefficients, β, under plane strain state on the long-term settlement of low embankments subjected to cyclic loading, were presented.

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