Comparison Study on Expansive Soil and Red Clay Creep Model

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ABSTRACT: A series of compression creep and unloading tests have been conducted for unsaturated Nanning expansive soils and red clay. The creep and rebounding curves of different soil samples are obtained and compared. According to the characteristics of the curves, the multi-component visco-elastic-plastic creep model for expansive soils and red clay are proposed, respectively. The relevant parameters in both models are measured from the tests. It is showed that the calculated curves fit well with the measured curves, which proves the correctness of the proposed models and provides reliable theoretical basis for analyzing the deformation and strength behaviors of expansive soils.

INTRODUCTION

The key of rheological properties study of soil is to establish a suitable rheological model with a constitutive equation which can reflect the different nature of the soil. Though many good results for the rheological properties of soil have been reported in recent years, the rheological properties study for expansive soil, especially the study for expansive soil rheological models, is still little. Engineering problems caused by expansive soil rheology are widespreadly concerned. Therefore, the study on rheological models of expansive soil shows important theoretical and engineering value.

The study is based on the existing results (Zheng et al. 2008; Xiao et al. 2007). A series of contrast tests (compression creep tests and unloading tests) between expansive soil and red clay are carried out. According to the properties of the curves, multi-component visco-elastic-plastic creep models of the two kinds of soil under the normal stresses condition are proposed (Zhou 1995).

PHYSICAL PROPERTIES OF SOIL

The expansive soil sample from ring road works of Nanning city, Gunagxi province, is alluvial gray expansive soil. The red clay sample is from Zhuzhou. Their physical properties indexes are shown in Table.1.

Table 1. Physical properties of soils.

Soil sample	Liquid Limit (%)	Plastic Limit (%)	Optimum Moisture Content (%)	Free Swelling Rate (%)	Maximum Dry Density (g/cm^3)
Expansive soil	61.4	22.8	15.8	62.5	1.89
Red clay	40.7	21.9	19.7	37	1.78

DETERMINATION OF MODEL AND PARAMETERS

Comparing with empirical models, theoretical models are easier to understand the various components of deformation (Yuan et al. 2001). What's more, components models enable to reveal the complex nature of the soil in an intuitive way. It is a facilitating numerical analysis and good choice in one-dimension rheological study.

Analysis supposes are as follow:

1) Soil mass is homogeneous continuum;

2) Soil mass satisfies linear superposition principle;

3) Systematic error of instrument, temperature and humidity are neglected (Вялов 1987).

According to the compression creep curves and unloading curves for the expansive soil and the red clay, as shown in Figs. 1 through 3 and Figs. 4 through 6, respectively. Relevant components and the way of combination in the components models can be inferred: 1) Both soil samples presented transient strains in the loading and unloading moment, which means the components model should have elastic component. 2) During the loading process and unloading process, the both soil samples show a relationship between time and deformation, so the components models should include viscosity components and elastic components in parallel. 3) The soil sample has irrecoverable deformations, which means the constitutive model should contain plastic components, and meanwhile there is no break in stress-strain curves, which illustrates the plastic component cannot stand alone or in parallel with elastic components. Expansive soil has an accelerating creep stage, while the red clay has a steady creep stage when strain rates are small, which implies that components models exists viscosity components and plastic components in parallel. 4) The creep curves of red clay are relatively flat. In the lightly loading conditions, the creep curves are similar to those of expansive soil. But in the highly loading conditions, the red clay sample does not have an obvious accelerated creep stage. Therefore, the red clay model may be different from the model of the expansive soil.

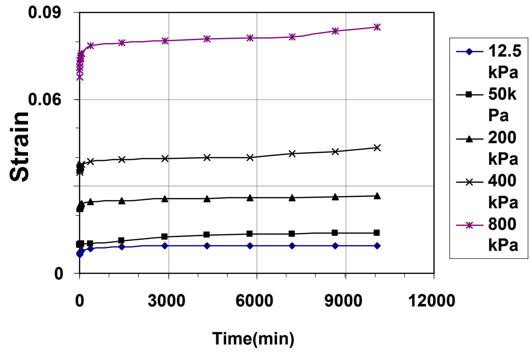

FIG. 1. Stress-time curves of expansive soil.

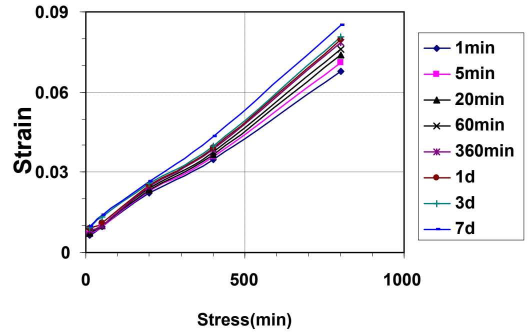

FIG. 2. Stress-strain curves of expansive soil.

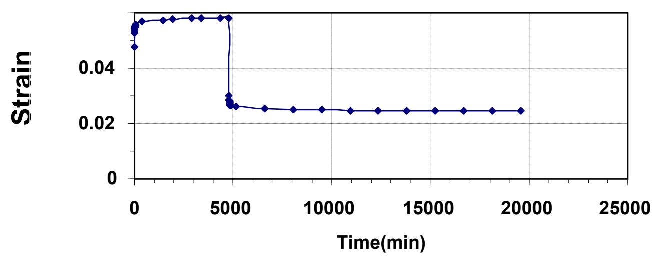

FIG. 3. Rebound curves of expansive soil.

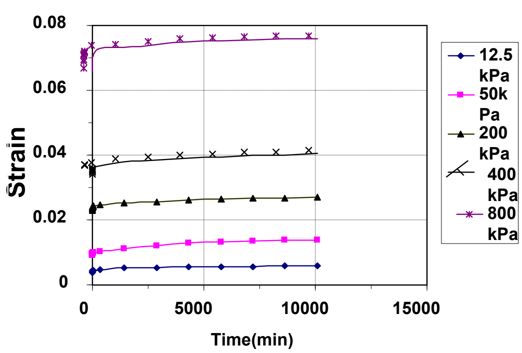

FIG. 4. Strain-time curves of red clay.

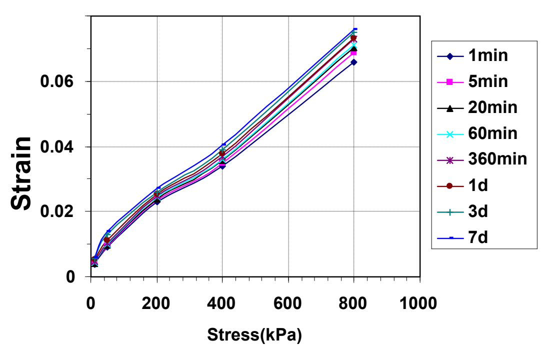

FIG. 5. Stress-strain curves of red clay.

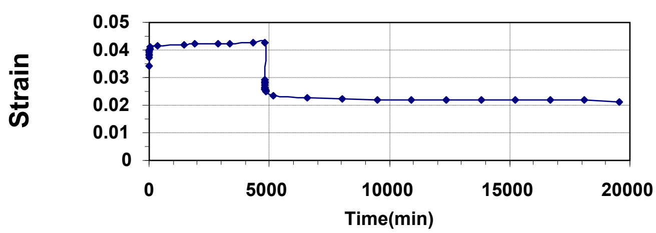

FIG. 6. Rebound curves of red clay.

Our former analyses described the expansive soil's creep by using Burger (Bu) model (Xiao et al. 2008). However, Bu model can hardly analyses the plastic deformation of expansive soil. Instead of that, this paper use Nishihara Masao model (Xiao et al. 2008), whose component species and their way of combined is suitable to analyze the expansive soil's properties described above. The structure of the Nishihara Masao model is shown in Figure 7. Nishihara Masao model =H-K-B, where H=Hooke model, K=Kelvin model and B=Bingham model.

However, Nishihara Masao model does not fit the measured values very well according regression curve and the measured values. Because single Kelvin model can hardly analyses the phenomenon that strain rate is gradually reduced with time in lowly loading conditions. According to generalized model concept, this paper adds another Kelvin model to Nishihara Masao model in order to develop a more accurate model. The improved model is shown in Figure 8: where σ = stress, E = elastic modulus, β = viscous coefficient, σ_s = yield limit.

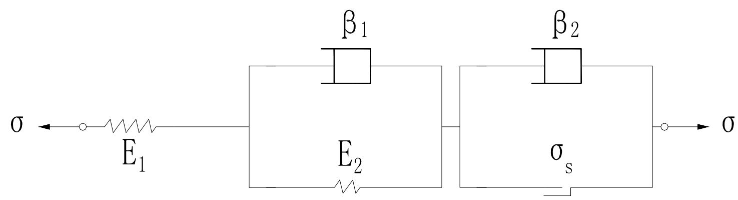

FIG. 7. Nishihara Masao model.

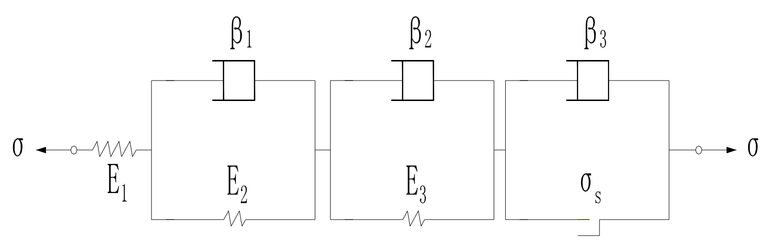

FIG. 8. Improved model.

The constitutive equation of the model is:

When $\sigma_o < \sigma_s$,

$$\varepsilon(t) = \frac{\sigma_0}{E_0} + \frac{\sigma_0}{E_1}(1 - e^{-E_1 t/\beta_1}) + \frac{\sigma_0}{E_2}(1 - e^{-E_2 t/\beta_2})$$

When $\sigma_o \geq \sigma_s$,

$$\varepsilon(t) = \frac{\sigma_0}{E_0} + \frac{\sigma_0}{E_1}(1 - e^{-E_1 t/\beta_1}) + \frac{\sigma_0}{E_2}(1 - e^{-E_2 t/\beta_2}) + \frac{\sigma_0 - \sigma_s}{\beta_3} t$$

The comparison result is shown in Figure 9. According the test and the regression curves shown in Figure 9, the improved model fits the measured values very well. Therefore, the improved model is reliable to analyses expansive soil's creep. This paper uses a new model to analyses red clay's creep, the new model= H-K-(H-st.V) || N, and its model structure is shown in Figure 10.

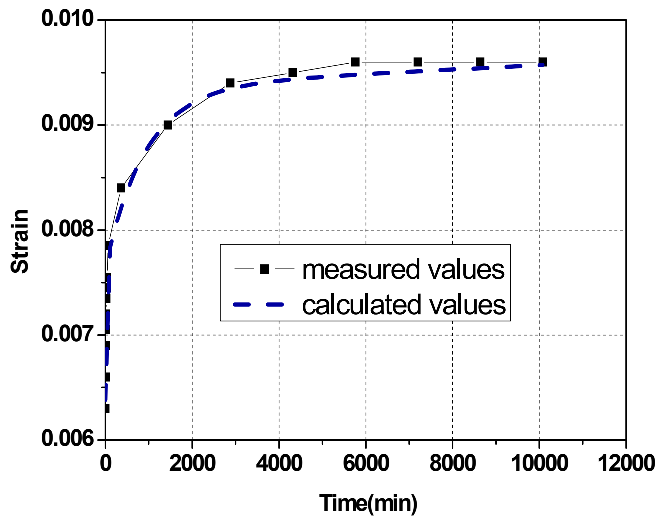

FIG. 9. Creep test and regression curve of improved model when vertical stress is 12.5 kPa.

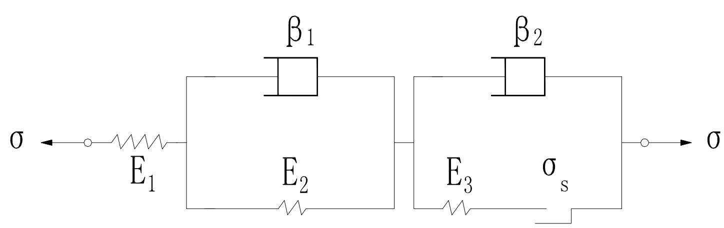

FIG. 10. Red clay model.

The constitutive equation of the model is:

When $\sigma_o < \sigma_s$, $$\varepsilon(t)=\frac{\sigma_0}{E_0}+\frac{\sigma_0}{E_1}(1-e^{-E_1 t/\beta_1})+\frac{\sigma_0}{E_2}(1-e^{-E_2 t/\beta_2})$$

When $\sigma_o \geq \sigma_s$, $$\varepsilon(t)=\frac{\sigma_0}{E_0}+\frac{\sigma_0}{E_1}(1-e^{-E_1 t/\beta_1})+\frac{\sigma_0-\sigma_s}{\beta_2}t$$

The comparison results are shown in Figure 11.

The red clay model has fewer components than the expansive soil model mentioned above. According the test values and the regression curves shown as Figure 9, the red clay model fit the measured values very well. Therefore, the red clay model is reliable to analyses red clay's creep. Moreover, it is relatively simple that red clay model is one parameter less than expansive soil model.

Stress-strain curves of expansive soil and red clay have obvious turning point. Both soil samples have irrecoverable deformation, which implies the plastic components should be taken into account in the components models. Therefore, the turning point can be considered as the yield stress σ_s of the plastic component. According the stress-strain relationship when t = 0, E_i can be obtained. The parameters can be derived by regression analysis method. The parameters are listed in Table 2 and Table 3.

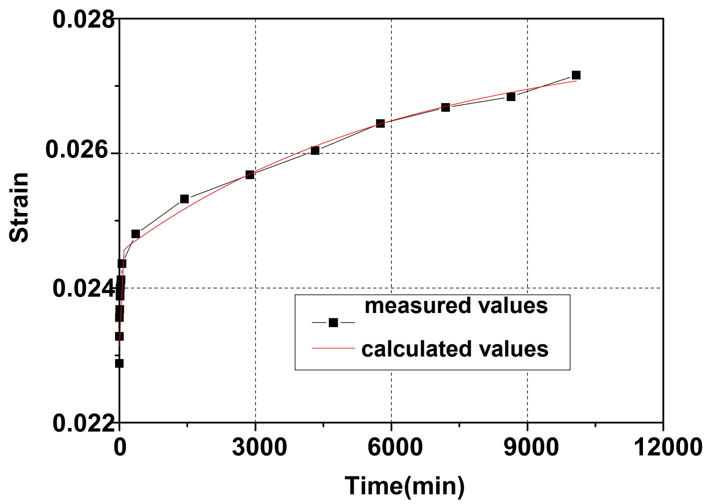

FIG. 11. Creep test and regression curve of red clay model when vertical stress is 200 kPa.

Table 2. Parameters of expansive soil model.

σ (kPa)	E_o (MPa)	E_1 (MPa)	β_1 (MPa·min)	E_2 (MPa)	β_2 (GPa·min)	β_3 (MPa ·min)	σ_s (kPa)	R^2
12.5	2.05	8.45	0.014	6.28	6.04E3	--		0.99
50	6.84	19.91	0.099	11.65	3.49E5	--		0.99
200	9.01	96.62	0.059	76.05	4.41E5	--	400	0.98
400	11.51	127.79	0.061	19.31	7.38E5	--		0.97
800	12.09	136.52	0.021	129.03	7.53E3	5.86E-7		0.99

According the test value and the regression curves shown in Figures 12 and 13 the expansive soil model and red clay model fit the measured values very well. Theoretically, the model's parameters should be explosive. However, the data is to be found that a set of different parameters for each class loading. The reason is that the deformation of soil is non-linear, while the [H] components and [N] components in model are linear components (Sun 1999). The foundation of the work is usually under 200 kPa. When σ = 200 kPa, the constitutive equation of expansive soil is

$$\varepsilon(t) = 0.0222 + 0.00207\,(1 - e^{-0.0353\,t}) + 0.00263\,(1 - e^{-1.72e-4t})$$

When σ = 200 kPa, the constitutive equation of red clay is

$$\varepsilon(t) = 0.0228 + 0.00189\,(1 - e^{-0.05276\,t}) + 0.006\,(1 - e^{-5.81e-5t})$$

Table 3. Parameters of red clay model.

σ (kPa)	E_o (MPa)	E_1 (MPa)	β_1 (GPa·min)	E_2 (MPa)	β_2 (GPa·min)	σ_s (kPa)	R^2
12.5	3.38	13.59	0.29	9.33	35.04		0.98
50	5.50	63.71	0.96	11.39	45.88		0.99
200	8.77	110.50	2.09	33.33	57.37	400	0.98
400	11.76	179.37	2.15	90.50	35.73		0.99
800	12.12	128.21	2.15	--	91.06		0.94

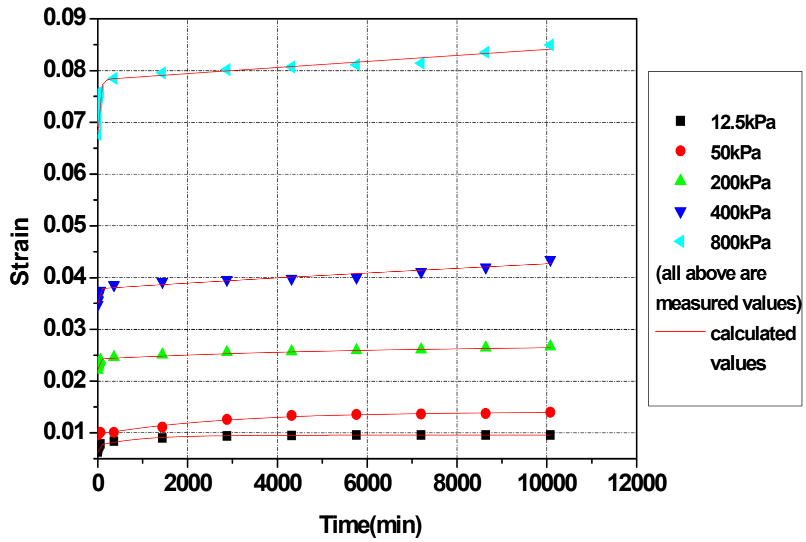

FIG. 12. Creep test and regression curve of expansive soil.

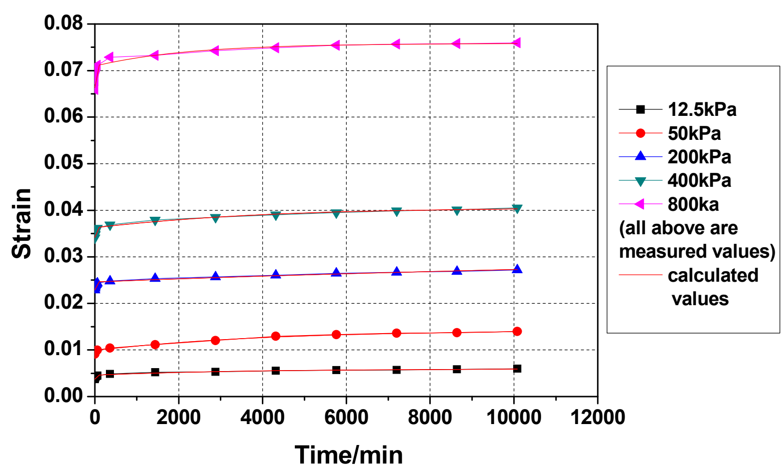

FIG. 13. Creep test and regression curve of red clay.

CONCLUSIONS

According to test results, it is sure that both expansive soil and red clay are visco-elastic-plastic body. So it is necessary to use visco-elastic-plastic to analyze their creep properties. Through the analysis and comparison of expansive soil and red clay's creep curves, expansive soil's creep compression properties is more complex than that of the red clay. Compression creep properties of expansive soil and the red clay can be analyzed by the proposed component models in this paper. The measured results and fitting ones seems agreeable to each other. Furthermore, only small number of easily obtained parameters from the general Lab tests makes our model to be a good option in analyzing the rheological behaviors for expansive soil.

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Lateral Resistance of Single Pile Located Near Carpet Shred Reinforced Slope

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ABSTRACT: Carpet shreds and carpet shred-soil mixtures can be used as alternative backfill material in many geotechnical applications. In this study an experimental testing program was undertaken using small scale model tests with goal of evaluating the lateral resistance of a single pile located near a reinforcing sandy slope. A broad series of conditions including unreinforced cases, was tested by varying parameters such as: carpet shred content, aspect ratios of carpet shreds, pile distance from the slope crest, relative density of sand, angle of the slope, embedded length, skin friction and cross section of pile. The carpet shred content and aspect ratio were found to influence the lateral resistance of single pile located near reinforcing sandy slope. The optimum carpet shred content and aspect ratio of carpet shred were 1.5 and 3 respectively, and with these values a good improvement in lateral resistance of piles was reached.

INTRODUCTION

Pile supported structures are often subject to both axial and lateral loads. Sometimes it is possible for a structure resting on vertical piles to be placed near natural or man-made slopes. The behavior of pile foundations which are located near slopes is different from their behavior while they are located on ground level as piles not only may induce failure in the slope (specially at shallow depth) but also the lateral bearing capacity of the piles themselves may decrease to a great extent due to adjacent to a slope. Poulos (1976), reported that the deflection of a pile in a slope could be 1.6 times that of same pile in level ground. Schmidt (1977), carried out a series of physical model test to study the behavior of piles located at the crest of sandy slopes. Mezazigh and Levacher (1998) performed centrifuge tests to study the responses of piles adjacent to slope. El-Sawwaf (2006, 2008), and Begum and Muthukkumaran (2008) reported the results of numerical or experimental study of reducing lateral load resistant of pile located near slope. However, among the mentioned studies, except El-Sawwaf (2006,2008) the effect of using slope reinforcement techniques on the behavior of vertical piles subjected to lateral loads near reinforced slopes has not yet been investigated. Nevertheless the effect of techniques such as reinforcing using yarn, fiber and carpet shreds has never been studied yet. In this case we've succeeded to accomplish two important goals which are reusing the waste materials in large aspects and modification of different engineering properties and mechanical behaviors of soil. Thus the main goal in this study is to investigate the effect of using a kind of polymer-yarn wastes (carpet shreds) as a reinforcing element on lateral behavior of a single vertical pile near a sandy slope. The focus of this study is on randomly distributed inclusions the effect of which is expected to led to improved mechanical behavior of composite material.

MODEL PILES AND BOX

The fabricated steel pipe with outer and inner diameters of 21.3 and 16.1 mm respectively was used as circular cross section pile where the fabricated steel box with outer and inner dimensions of 20 and 17 mm respectively was used as square cross section pile. The lateral load was transferred from the load hanger to the pile head through a cable connected at the point load application as shown in Fig. 1.

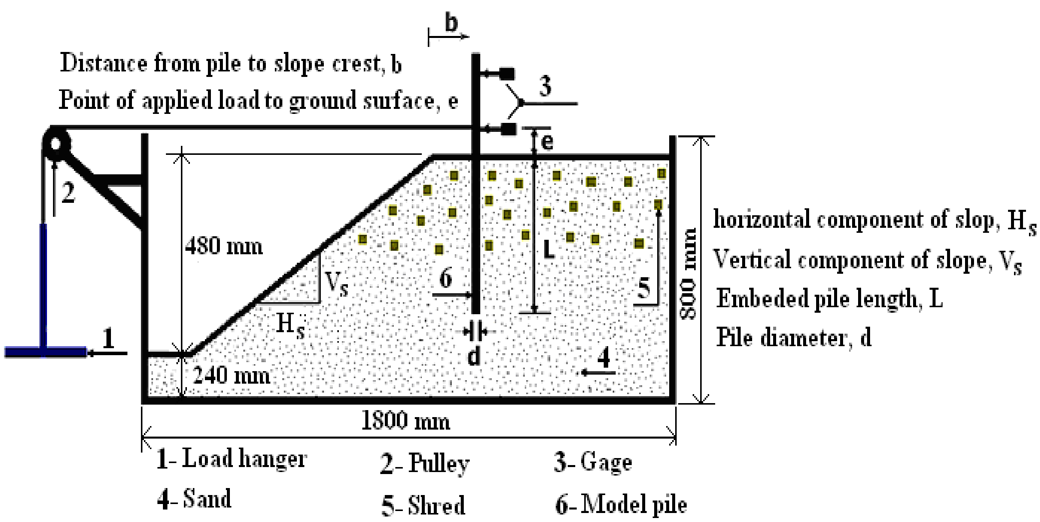

FIG. 1. Schematic view of the test setup.

One of the dominant factors that affect the lateral resistance of a pile is the pile stiffness factor (T). In cohesionless soils, this factor is calculated using the following equation:

$$T = \sqrt[5]{\frac{E_P I_P}{n_h}} \tag{1}$$

Where E_p=modulus of elasticity of pile material (20 ××10^7 kN/m^2), I_p=moment of inertia of the pile cross section (6.81 ××$10^{-9}m^4$), and n_h= constant of subgrade reaction at pile tip. Suggested values of n_h for dense, medium-dense and loose sands are 13500, 6000 and 1900 kN/m^3 respectively (Terzaghi, 1955). Broms (1964) suggested that for a free head pile, the embedded length of the pile to be considered as a short rigid pile,