Ocean Engineering 149 (2018) 194–204

Contents lists available at ScienceDirect

Ocean Engineering

journal homepage: www.elsevier.com/locate/oceaneng

A novel model for the cumulative plastic strain of soft marine clay under long-term low cyclic loads

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ARTICLE INFO

Keywords:
Empirical model
Cumulative plastic strain
Long-term low cyclic loads
Soft soil

ABSTRACT

Long-term cyclic loads with a stress level lower than the critical cyclic stress applied on soft soil can lead to soil deformation but not to failure. The Monismith model is known for its simplicity and capacity to describe the cumulative plastic strain of soil under cyclic loads. However, it is unsuccessfully applied in these cases because the plastic strain in this model would increase endlessly till to failure with increasing number of load cycles. To solve this problem, a novel empirical model with three parameters is proposed based on analogy analysis of the Hardin-Drnevich model and Monismith model. The proposed model is verified by experimental data from existing literature and is shown to have better capability and performance than the Monismith model in predicting the cumulative plastic strain of soft soil subjected to long-term low cyclic loads. The value of parameter b is recommended to be 0.5, and relationships of parameters a and c with the cyclic stress ratio are also proposed for soft clay. Applications of the proposed model are elaborated in detail, and in situ test results for settlement of soft subgrade are used to evaluate the performance of this model. The prediction results are consistent with the test results. The research results present a promising method for investigating the development of deformation and the settlement of soft foundation in the near-shore and off-shore areas caused by wave and/or traffic loads.

1. Introduction

Soft marine clay has a high void ratio, a high water content, low permeability, high compressibility and high sensitivity and is susceptible to outside disturbances. It is also widely distributed in the east coast area of China. In this marine environment, the foundation design of offshore installations (suction anchors, wind turbines, gravity platforms, etc.) or near-shore structures (seawalls, harbors, dockyards, lighthouse, etc.) is governed by the bearing capacity and the serviceability under cyclic loads (Andersen, 2009; Hu and Ding, 2010; Wichtmann et al., 2013). These cyclic loads are caused by waves, vehicle traffic and the operation of machinery, resulting in a reduction of strength (Rao and Panda, 1999; Moses and Rao, 2003; Li et al., 2011) and bearing capacity (Wichtmann et al., 2013; Tang et al., 2011), unexpected settlement (Ren et al., 2012; see Ng et al., 2013 for an example of settlement; Lei et al., 2016), and other geotechnical engineering problems (Mayoral et al., 2016).

A model for cumulative deformation is a theoretical basis for solving the problems of dynamic stability and settlement of subgrade under long-term cyclic loads. At present, the cumulative plastic deformation models under cyclic loading are mainly classified into two categories: theoretical models and empirical models. In the scientific literature, theoretical models mainly include the modified Cambridge model (Carter et al., 1982), nested yield surface model (Mroz, 1967; Prevost, 1977, 1978), bounding surface model (Dafalias, 1986a, 1986b; Hu and Liu, 2015), etc. These theoretical models are capable of calculating the plastic strain generated by each cyclic load, generally being high in accuracy but complex in calculation. However, to meet the accuracy requirements, most of these models need to adopt sufficiently small calculation steps to simulate the process of each cycle of loading and unloading. This often leads to model failure due to excessive calculation, especially for a large number of cyclic loads. Empirical models are usually established by using the results of laboratory tests and field tests, which avoids excessive calculation. As they meet the requirements of engineering practice, empirical models have been widely used in practical engineering despite being less accurate than theoretical models.

A number of empirical models have been developed for predicting cumulative plastic deformation in soil under cyclic loading. However, the most commonly used is the following power model (Monismith et al., 1975):

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https://doi.org/10.1016/j.oceaneng.2017.12.028
Received 14 December 2016; Received in revised form 28 September 2017; Accepted 10 December 2017

$$\varepsilon_p = A \times N^b \tag{1}$$

where ε_p is the cumulative plastic strain(%); N is the number of cyclic load applications; and A and b are two parameters that depend on the soil type, soil properties and stress state. Afterward, many researchers improved the power model (1). Li and Selig (1996) took into account the influence of the cyclic deviator stress (σ_d) and soil statistic strength (σ_s) on coefficient A in (1) as follows:

$$A = a \times (\sigma_d/\sigma_s)^m \tag{2a}$$

$$\varepsilon_p = a \times (\sigma_d/\sigma_s)^m \times N^b \tag{2b}$$

where a and m are material parameters, m has a range of 1.0–4.2, and a has a range of 0.3–3.5 for 22 soils studied. Chai and Miura (2002) considered the effect of the initial deviator stress (σ_{id}) on the cumulative plastic strain and proposed the following equation:

$$\varepsilon_p = a \times \left(\frac{\sigma_d}{\sigma_s}\right)^m \left(1 + \frac{\sigma_{id}}{\sigma_s}\right)^n N^b \tag{3}$$

where n is a constant coefficient with a recommended value of 1.0. Various similar models have been developed (Parr, 1972; Moses et al., 2003; Abdelkrim et al., 2003; Huang et al., 2006; Shahin et al., 2011; Wang et al., 2013; Guo et al., 2013; Wang and Li, 2015; etc.) for different engineering backgrounds and practical applications. In general, these empirical models represent a relationship between the cumulative plastic strain and the number of repeated load applications. Some models also considered stresses, the strain rate, the soil type and soil properties.

The aim of this paper is to develop a novel empirical model for cumulative plastic deformation of soft marine clay subjected to cyclic loads characterized by a large number of repeated load applications and low stress. This will provide an opportunity to discuss the settlement and strength decrease of foundation soils in the near-shore and off-shore areas under wave or traffic loads and to provide a broad, even if not complete, overview of deformation characteristics and their controlling factors, as well as the relationship with time. In the following, we discuss the major influencing factors on cumulative deformation in terms of the stress state, soil type and soil properties in Section 2. Section 3 presents a cumulative plastic deformation model, and the proposed model will be verified and evaluated by test results in Section 4. Relationships between three model parameters, the physical soil state and the stress state, applications of the proposed model, and model limitations are discussed in Section 5. Summaries and main conclusions are drawn in the last part of the paper.

2. Influencing factors

A good prediction model for cumulative plastic deformation should take into account the major influencing factors. The list of the most frequently quoted factors includes the physical properties of soil (water content, Atterberg limits, specific gravity, particle size, specific surface area, etc.), stress state (dynamic stress, confining pressure, overconsolidation ratio, loading frequency, etc.), drainage conditions, and stress path (loading waveform, loading method, test control mode, the rotation of principal stress axis, etc.) (Li and Selig, 1996; Huang et al., 2006; Ren et al., 2012; Cai et al., 2013; Lei et al., 2015; Gu et al., 2016; Elia and Rouainia, 2016; Hicher, 2016). Among these factors, for a given soil, the most dominant influencing factor of cumulative plastic deformation is the cyclic deviator stress σ_d, which was recognized through laboratory test results conducted by many researchers such as Seed et al. (1955), Monismith et al. (1975), Li and Selig (1996), Huang et al. (2006), Shahin et al. (2011), etc. The second important factor is the confining pressure. These two factors mainly determine the development model of strain and the dissipation of excess pore water.

The existence of a critical cyclic stress (or threshold stress) has long been recognized by many researchers (e.g., Larew and Leonards, 1962; Mitchell and King, 1977; Sangrey et al., 1978; Lefebvre et al., 1989; Tang et al., 2003; Shahin et al., 2011), and the critical cyclic stress for several different soft soils is summarized in Table 1. The critical cyclic stress is defined as the stress below which the soil will not suffer failure regardless of the number of repeated load applications and above which soil deformation will continually increase up to failure. In other words, when the applied cyclic stress is lower than the critical stress, the cumulative plastic deformation of soil will not continually increase with an increase in the number of repeated load applications but tend to a stable limited value. Based on the critical cyclic stress level, Cai and Cao (1996) classified the development of the permanent deformation into three types, attenuation type, critical type, and destructive type, and suggested applying different models for the different deformation types.

In many practical cases, the cyclic loads applied on the mucky soft soil consist of long-term repeated applications and a low stress level, which is often lower than the critical stress of the soil. Therefore, the plastic accumulative deformation induced by these loads will gradually increase at the beginning, but the deformation rate gradually decreases. After a certain time, the cumulative plastic deformation reaches the maximum and tends to be stable, no longer increasing over time. However, Monismith's model, the most widely used empirical model, is not capable of predicting the deformation caused by the stress that is below the critical cyclic stress because the plastic strain it describes will increase endlessly with increasing number of cyclic loading applications.

3. Cumulative plastic deformation model

Fig. 1 (a) shows the skeleton curve of stress-strain relationship of soils subjected to cyclic loading, which was described as a hyperbolic equation by Hardin and Drnevich (1972), i.e. the well-known classical Hardin-Drnevich model (Eq. (4)).

$$\sigma_d = \frac{\varepsilon_d}{A + B\varepsilon_d} \tag{4}$$

where A and B are model parameters.

Fig. 1 (b) shows the typical curve of cumulative deformation of soft soils under long-term low cyclic stress. It can be seen from Fig. 1 that the two curves look very similar. It was therefore concluded that a function similar to that of Equation (4) might be applicable to the cumulative plastic strain of soft soils under cyclic load conditions. Combining the Monismith model (Eq. (1)) and Hardin-Drnevich model, a new model is proposed as Equation (5) for the cumulative plastic deformation caused by cyclic stress with long-term application and a low stress level, especially for the stress lower than the critical stress of soil.

$$\varepsilon_p = \frac{N^b}{a + cN^b} \tag{5}$$

where a, b and c are parameters that depend on the stress path, stress state and physical properties of soil.

The impact of parameters a, b and c on the cumulative plastic strain is shown in Fig. 2 (b), (a) and (c) respectively. Fig. 2(a) shows that for given values of parameters a and c, the slope of the curve increases with increasing exponent parameter b, while the initial and final strain remains the same regardless of b. Fig. 2(b) shows that the slope will remain stable and the curve will parallelly move toward the left or right if parameter b is given a certain value. This observation suggests that the exponent parameter b characterizes the rate of accumulative plastic strain and does not affect the final amount of deformation of soil. Following this observation, it might be reasonable to conclude that parameter b depends on the physical properties of the soil itself, such as the water content, void ratio, particle size and particle specific surface area. Monismith et al. (1975) suggested that the rate of cumulative plastic strain is unrelated to the dynamic stress level, namely, dynamic stress does not affect exponent parameter b. In addition, Li and Selig