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Research Article

Study on Calculation Method of Foundation Nonlinear Settlement Based on In Situ Loading Test

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In this paper, a nonlinear deformation modulus method is proposed for foundation settlement calculation. In the proposed method, the nonlinear deformation modulus under different stress levels is obtained from the load-settlement curve of in situ loading test, which are then applied to the layerwise summation method for calculating foundation settlement. On this basis and referring to the Duncan-Chang model, a variable modulus constitutive model suitable for numerical calculation of foundation settlement is further proposed. The required parameters of this model are the same as those of the nonlinear deformation modulus method and can be determined by the in situ loading test. The validity of the proposed calculation methods of foundation settlement is verified by the in situ loading tests under different plate sizes. The results illustrate that both the nonlinear deformation modulus method and the variable modulus constitutive model compare quite well with the test results, and the deduced results can better reflect the nonlinearity of foundation settlement.

1. Introduction

The settlement calculation of foundation is a primary and significant content in the field of soil mechanics. The commonly used settlement calculation methods are mainly focused on soil sampling and laboratory compression testing, and the deduced compression curve is used for settlement calculation. Even though this method is relatively simple and practical, there exists two major unneglectable theoretical deficiencies: (i) soil sample disturbance can occur during sampling; (ii) the stress and deformation conditions of the laboratory compression test are quite different from the field conditions, which makes the calculation error estimation become difficult. Since the laboratory test cannot reflect the actual deformation characteristics of the in situ soil very well, it is difficult to deduce accurate foundation settlement results even using modern advanced numerical calculation methods. Thus, the better way is to improve the acquisition of calculation parameters, namely, calculating the foundation settlement based on parameters obtained from in situ tests. This can improve the accuracy of calculated foundation settlement, which can more accurately reflect deformation characteristics of foundations. Currently, some methods for calculating foundation settlement based on in situ test results are reported [1–5]. For example, Li [4] proposed a calculation method for foundation settlement based on cone penetration test considering different stratum conditions. Zhang et al. [5] proposed a nonlinear settlement calculation method based on the calculation parameters obtained from the pressuremeter tests. However, most of these methods are mainly provided by experiences.

The in situ loading test is considered to be one of the most reliable methods for determining the ultimate bearing capacity of foundation. The simultaneous deduced load-settlement curve can realistically reflect the deformation characteristics of the foundation during the loading process. Moreover, the deformation modulus of the in situ soil also can be determined, which can be effectively applied to the settlement calculation of foundations [6–10]. In addition, the actual settlement process of foundation is nonlinear, and the use of a single constant deformation modulus cannot truly reflect the nonlinear deformation characteristics of the foundation affected by loadings. Thus, in this study, the nonlinear deformation modulus under different stress levels is obtained from the load-settlement curve of in situ loading test, which are then applied to the layerwise summation method and numerical constitutive model for calculating the settlement of foundations. Based on these treatments, a calculation method of foundation nonlinear settlement based on in situ loading test is established and proposed. The validity of the proposed calculation method is verified by the in situ loading tests under different plate sizes.

2. Nonlinear Deformation Modulus Method for Foundation Settlement Calculation

2.1. Nonlinear Deformation Modulus Method. Assuming a soil layer with a thickness Δh_j located at a depth h_j is subjected to a small incremental load Δp_i, the deformation of this soil layer can be approximately viewed as linear process and given as follows.

$$\Delta s_{ij} = \frac{\Delta p_i \alpha \cdot \Delta h_j}{E_{ij}}, \tag{1}$$

where E_{ij} is the deformation modulus of the soil layer at a depth h_j under an external loading p_i; α is the stress distribution coefficient, and $\Delta p_i \alpha$ defines the stress increment generated by the incremental load Δp_i.

Then, according to the layerwise summation method, the total settlement of all layers under incremental load Δp_i can be summarized as

$$\Delta s_i = \sum_{j=1}^{n} \Delta s_{ij}, \tag{2}$$

where n refers to the total numbers of soil layers.

The key aspect of the above calculation method is the reasonable determination of the deformation modulus E_{ij}. It can be seen from previous analysis that the load-settlement curve obtained by the in situ loading test can more accurately reflect the deformation characteristics of foundations during the loading process, and the deformation modulus of the foundation soil can be determined accordingly. Previous researches [11–14] indicate that the load-settlement (p-s) curve obtained by the in situ loading test is generally expressed by a hyperbolic function.

$$p = \frac{s}{a + \mathrm{bs}}, \tag{3}$$

where a and b are unknown coefficients that need to be determined. According to the characteristics of hyperbolic function and the theory of elastic mechanics [11]

$$\left.\begin{aligned} a &= \frac{D\left(1-\mu^2\right)\omega}{E_0} \\ b &= \frac{1}{p_u} \end{aligned}\right\}, \tag{4}$$

where D is the side length or diameter of the in situ loading test plate; μ is the Poisson's ratio of the soil; ω is the coefficient that reflects the shape and stiffness of the plate, for rigid square and circular plates, and the values of ω respective are equal to 0.88 and 0.79; E_0 is the initial deformation modulus of the soil; while p_u is the ultimate load.

In addition, the foundation settlement caused by the in situ loading test can be calculated approximately by the Boussinesq solution of elastic mechanics:

$$\Delta s = \frac{D \cdot \Delta p \cdot \left(1-\mu^2\right)}{E} \cdot \omega, \tag{5}$$

where E is the deformation modulus of soil at the bottom of the plate corresponding to the applied loading p.

Transformation from the Equation (5) can be found that

$$E = \frac{\Delta p}{\Delta s} \cdot D\left(1-\mu^2\right) \cdot \omega. \tag{6}$$

From the Equation (3), the tangent derivative of p-s curve, deduced from the in situ loading test, at any point can be expressed as

$$\frac{dp}{ds} = \frac{(1-bp)^2}{a}. \tag{7}$$

Assuming that $\Delta p/\Delta s = dp/ds$ and substituting Equations (7) and (4) into Equation (6), the deformation modulus of the soil at the bottom of the loading plate is

$$E = \left(1 - \frac{p}{p_u}\right)^2 \cdot E_0. \quad (8)$$

In the above equation, p/p_u is the ratio of the load p to the ultimate load p_u of foundation soil at the bottom of the loading plate, which reflects the influence of stress level on the deformation modulus E of the soil. For soils at different depths, the additional load p after diffusion decreases, and the corresponding ultimate load p_u increases with an increase in the depth, which leads to an increase in the deformation modulus E according to Equation (8). It conforms to the nonlinear deformation characteristics of foundation. Then, applying the deformation modulus of soil at different depths to the layerwise summation method, namely, substituting Equation (8) into Equations (1) and (2), the settlement calculation of the foundation is performed, which is called as nonlinear deformation modulus method in this paper.

2.2. Parameter Determinations Based on In Situ Loading Tests. From above section, it can be concluded from Equation (8) that the nonlinear deformation modulus method needs to determine the ultimate load p_u of the soil and the initial deformation modulus E_0, wherein p_u can be calculated according to the cohesion c and internal friction angle φ of soil and the plate size. Therefore, three soil parameters of soil (i.e., c, φ, and E_0) are required to be determined for the nonlinear deformation modulus method. In order to overcome the shortcomings of the parameters determination in the laboratory test, in this study, these three parameters are obtained based on the in situ loading tests.

Note that the load-settlement (p-s) curve can be readily obtained from the in situ loading tests. Using the hyperbolic function as an input to fit the p-s curve, the coefficients a and b can be easily deduced, which can be directly applied to inverse calculation of parameters c, φ, and E_0 for the nonlinear deformation modulus method.

The c and φ values can be inversely calculated from Vesic's ultimate bearing capacity equation.

$$p_u = cN_cS_c + qN_qS_q + \frac{1}{2}\gamma DN_\gamma S_\gamma, \quad (9)$$

where N_c, N_q, and N_γ are the bearing capacity coefficients and can be obtained according to the φ value; S_c, S_q, and S_γ are the plate shape factors of the in situ loading test; q is the surcharge loading applied on both sides of the plate; γ is the unit weight of the soil, which equals to effective weight when below the groundwater level.

It should be pointed out that, in Equation (9), there are only two unknown parameters, namely, c and φ. For sand, $c = 0$ and φ can be inversely calculated from the ultimate bearing capacity (p_u) of foundation soils obtained from the in situ loading tests. For clay, then, one of these two parameters is assumed empirically, and the value of the other parameter can be inversely calculated from the ultimate bearing capacity (p_u).

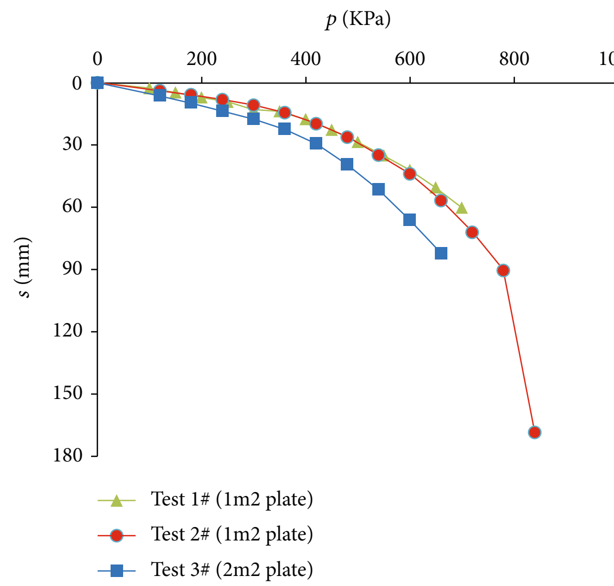


FIGURE 1: The p - s curves of in situ plate loading test.

The initial deformation modulus E_0 of the soil is determined by the Boussinesq solution of the elastic mechanics. When the hyperbolic function coefficient a is fitted according to the p - s curve of the in situ loading tests, the expression for E_0 value is

$$E_0 = \frac{D\left(1 - \mu^2\right)\omega}{a}. \quad (10)$$

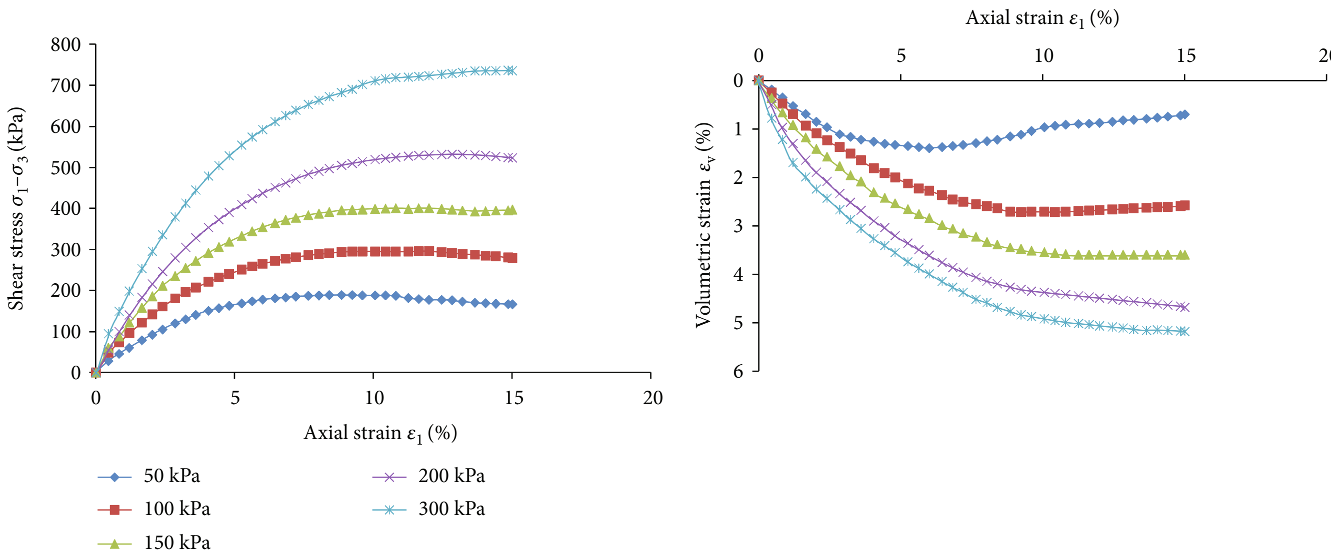
3. Variable Modulus Constitutive Model for Foundation Settlement Calculation

Currently, the determinations of constitutive model parameters of soils are mainly focused on laboratory tests. However, for strong structural soils, the results from laboratory tests may be greatly influenced due to the stress release and disturbance caused by sampling, which leads to a quite different results compared with the actual ones. Thus, it is difficult to ensure the accuracy of calculated results according to laboratory test parameters. Meanwhile, as the constitutive properties of soils are quite complicate, it is almost impossible to establish a constitutive model that combines all characteristics of the soils. Under such specific case, establishing a practical constitutive model that reflects the main characteristics and parameters that are easy to measure is more effective and of great significance.

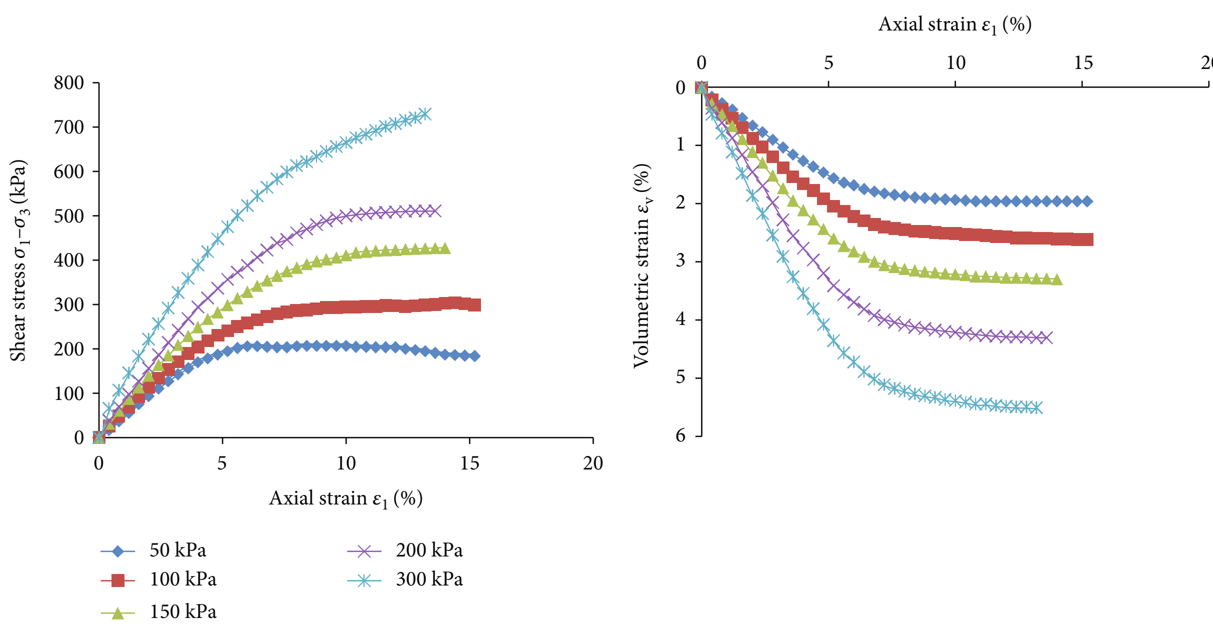
It should be mentioned that the main feature of foundation settlement problem is that the deformation of soils is nonlinear [15–17]. The Duncan-Chang model based on the generalized Hooke's law is widely used in practical

Table 1: Physical property test results of the laboratory tests.

Depths (m)	Moisture content (w) (%)	Relative density (Gs)	Wet density (ρ) (g/cm^3)	Dry density (ρ_d) (g/cm^3)	Void ratio (e)	Saturation (Sr) (%)
1.0	28.60	2.70	1.76	1.37	0.97	79.70
2.5	29.80	2.70	1.77	1.37	0.98	82.40



(a) Soil samples at a depth of 1 m



(b) Soil samples at a depth of 2.5 m

Figure 2: Stress-strain curves of the conventional triaxial compression tests (different confining pressures).

engineering because it is easy to determine the required parameters and can further reflect the most prominent deformation characteristics (i.e., nonlinearity) of soils. However, as these require parameters are usually determined by the conventional triaxial compression test in the laboratory, it also has certain limitations especially for some strong structural soils. In order to combine the advantages and make up for the limitations in parameter determinations of Duncan-Chang model, a simplified Duncan-Chang model, called variable modulus constitutive model in this paper, is proposed on the basis of the nonlinear deformation modulus method and Duncan-Chang model. Hence, the constitutive model is

$$\begin{Bmatrix} d\sigma_x \\ d\sigma_y \\ d\sigma_z \\ d\tau_{xy} \\ d\tau_{yz} \\ d\tau_{zx} \end{Bmatrix} = [D] \begin{Bmatrix} d\varepsilon_x \\ d\varepsilon_y \\ d\varepsilon_z \\ d\gamma_{xy} \\ d\gamma_{yz} \\ d\gamma_{zx} \end{Bmatrix}, \tag{11}$$

where $d\sigma_x$, $d\sigma_y$, $d\sigma_z$, $d\tau_{xy}$, $d\tau_{yz}$, and $d\tau_{zx}$ are the incremental stress components; $d\varepsilon_x$, $d\varepsilon_y$, $d\varepsilon_z$, $d\gamma_{xy}$, $d\gamma_{yz}$, and $d\gamma_{zx}$ are the incremental strain components; $[D]$ is the elastic matrix and given by

$$[D] = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)} \begin{bmatrix} 1 & \frac{\mu}{1-\mu} & \frac{\mu}{1-\mu} & 0 & 0 & 0 \\ \frac{\mu}{1-\mu} & 1 & \frac{\mu}{1-\mu} & 0 & 0 & 0 \\ \frac{\mu}{1-\mu} & \frac{\mu}{1-\mu} & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1-2\mu}{2(1-\mu)} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1-2\mu}{2(1-\mu)} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1-2\mu}{2(1-\mu)} \end{bmatrix}, \tag{12}$$

where E and μ define the deformation modulus and Poisson's ratio of the soil, respectively. In the variable modulus constitutive model, the simplified expressions of these two parameters are as follows:

$$E = \left(1 - \frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_f}\right)^2 E_0,$$
$$\mu = \mu_0 + \left(\mu_f - \mu_0\right) \frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_f}. \tag{13}$$

In the above two equations, E_0 and μ_0, respectively, refer to the initial deformation modulus and Poisson's ratio of the soil. μ_f is the Poisson's ratio of the soil under ultimate failure state and generally equal to 0.49. $(\sigma_1 - \sigma_3)_f$ is the ultimate strength of soils based on the Mohr-Coulomb failure criterion and expressed as

$$(\sigma_1 - \sigma_3)_f = \frac{2c \cos\varphi + 2\sigma_3 \sin\varphi}{1 - \sin\varphi}. \tag{14}$$

From above analysis, it can be concluded that the basic parameters of the variable modulus constitutive model include cohesion c, internal friction angle φ, initial deformation modulus E_0, and initial Poisson's ratio μ_0. In general, μ_0 is approximately set to 0.3, and other three soil parameters of soil (i.e., c, φ, and E_0) can be determined by the in situ loading tests. Therefore, a variable modulus constitutive model based on in situ test parameters can readily be established as a simple but practical constitutive model for foundation settlement calculations.

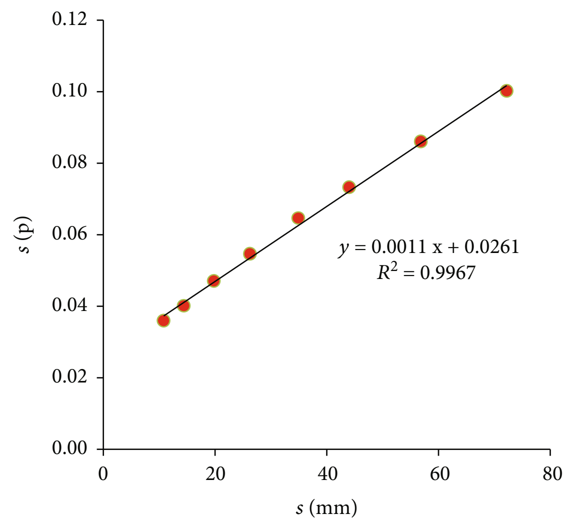


Figure 3: The fitting relationship between s/p and s based on the results of in situ loading test 2#.

Table 2: Calculated parameters of the nonlinear deformation modulus method.

c/kPa	φ /°	E_0/MPa
25	25	30

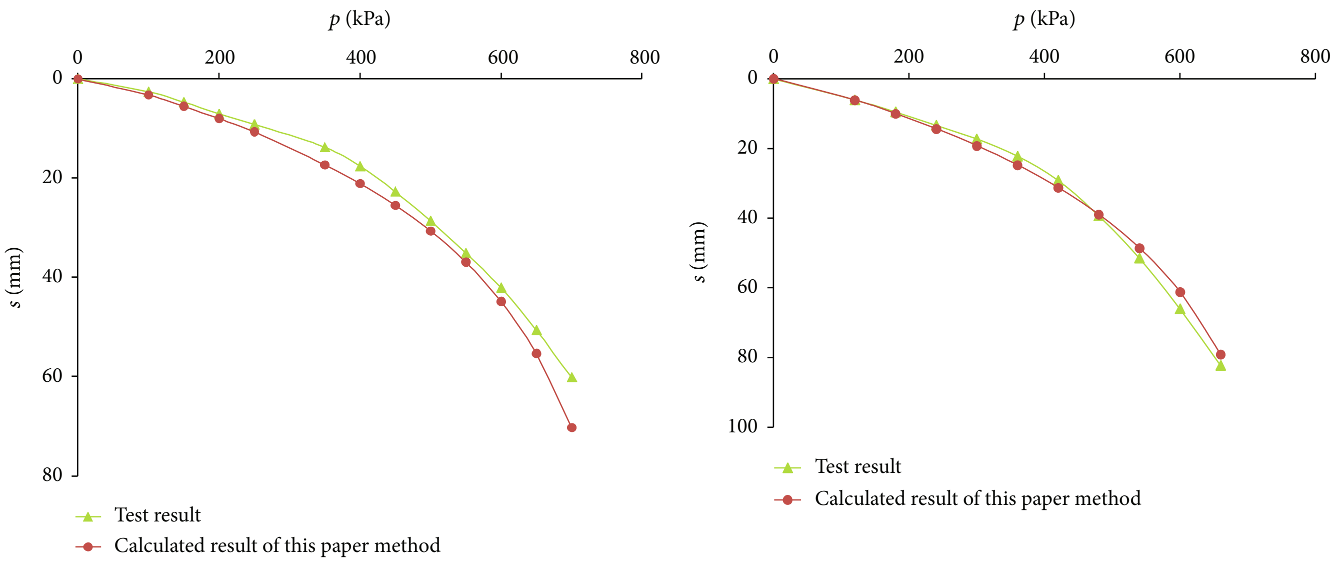
4. In Situ Tests and Verifications

4.1. In Situ Tests. In order to verify both the nonlinear deformation modulus method and variable modulus constitutive model for foundation settlement calculation, the representative strong structural granite residual soil in Guangzhou (China) was selected, and several series of in situ loading tests with the size of square plates equal to $1\,\mathrm{m}^2$ ($1\,\mathrm{m} \times 1\,\mathrm{m}$) and $2\,\mathrm{m}^2$ ($1.4\,\mathrm{m} \times 1.4\,\mathrm{m}$) were carried out. The obtained p - s curves of in situ plate loading tests are shown in Figure 1. It can be seen that test 1# was loaded by 50 kPa each stage, and tests 2# and 3# were loaded by 60 kPa each stage, and the p - s curves of the two in situ load tests 1# and 2# with identical plate size ($1\,\mathrm{m}^2$) are basically the same. In addition, a rapid increase in the settlement indicates that the loading applied to the test 2# almost reaches the ultimate bearing capacity of the granite residual soil.

For comparison, laboratory tests, including soil physical property tests and conventional triaxial compression tests, also carried out soil samples. The sampling depths are 1.0 m and 2.5 m, respectively. The deduced test results are shown in Table 1 and Figure 2.

4.2. Verifications

4.2.1. Verification of the Nonlinear Deformation Modulus Method. According to previous analysis, the nonlinear deformation modulus method requires three soil parameters: c, φ, and E_0, and these three parameters can be readily



(a) In situ plate loading test 1#

(b) In situ plate loading test 3#

Figure 4: Comparison of calculated results of the nonlinear deformation modulus method and in situ test results.

obtained from the in situ loading test results. In present study, the parameters c, φ, and E_0 of granitic residual soil are determined based on the results of in situ loading test 2# (as shown in Figure 1). The nonlinear deformation modulus method is then adopted to calculate the p - s curves, which are compared with the results of in situ loading tests 1# and 3# and to validate the rationality of the nonlinear deformation modulus method.

From the results of the in situ loading test 2# (as shown in Figure 1), a rapid increase in the settlement was observed when the applied loading increases from 780 kPa to 840 kPa, which indicates that this applied loading almost reaches the ultimate bearing capacity of the granite residual soil. Taking the average value of the two-stage loadings as the ultimate bearing capacity, thus the value of p_u equals to 815 kPa, and the parameters c and φ can be inversely calculated after substituting the p_u values into Equation (9). According to the empirical value of the soil parameters in Guangzhou, it is assumed that the frictional angle of the granite residual soil (φ) is equals to 25°; then, the calculated cohesion of soils (c) is 25 kPa.

After rewriting the Equation (3), the simplified expression of the form is

$$y = \frac{s}{p} = a + bs. \tag{15}$$

Using Equation (15) to fit the p - s curve of the in situ loading test 2#, the fitted relationship is

$$y = \frac{s}{p} = 0.0011s + 0.0261. \tag{16}$$

Figure 3 displays the fitted relationship of results of in situ loading test 2#. As can be seen from Figure 3, these results can be extremely well expressed as hyperbolic functions, and the fitted hyperbolic function coefficient α is 0.0261. Thus, the initial deformation modulus of the granite residual soil can be obtained from Equation (10) and given as follows.

Table 3: Calculated parameters of the variable modulus constitutive model.

c/kPa	φ /°	E_0/MPa
25	25	30

$$E_0 = \frac{D\left(1 - \upsilon^2\right)}{a}\omega = \frac{1 \times \left(1 - 0.3^2\right)}{0.0261} \times 0.88 \approx 30\text{MPa}. \tag{17}$$

Table 2 presents the calculation parameters c, φ, and E_0 of the nonlinear deformation modulus method that are obtained according to the results of the in situ loading test 2#. The calculated results of the p - s curves with identical loading conditions as in situ tests 1# and 3# by the nonlinear deformation modulus method are shown in Figure 4. The associated in situ test results are also included. It can be seen that the p - s curves calculated by the nonlinear deformation modulus method compare very well with the in situ test results, which primarily verifies the rationality of the nonlinear deformation modulus method.

4.2.2. Verification of the Variable Modulus Constitutive Model. For the variable modulus constitutive model, the three parameters c, φ, and E_0 are also required, and they also can be obtained according to the in situ loading test results. Similarly, the values of c, φ, and E_0 of the granite residual soil are determined based on the results of in situ loading test 2# and presented in Table 3. In this section, the rationality of the variable modulus constitutive model is verified by comparisons of numerical results using the FLAC^{3D} and the results of in situ loading tests 1# and 3#. The calculation mesh is shown in Figure 5, and the associated comparisons

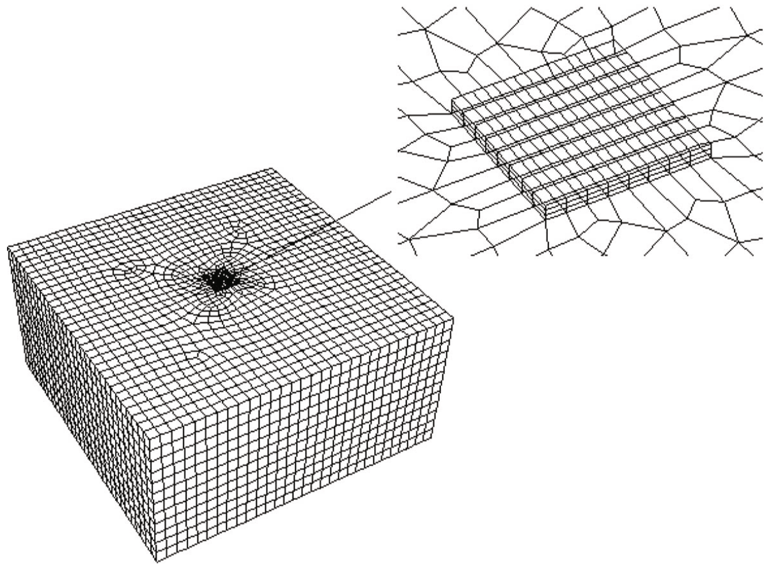
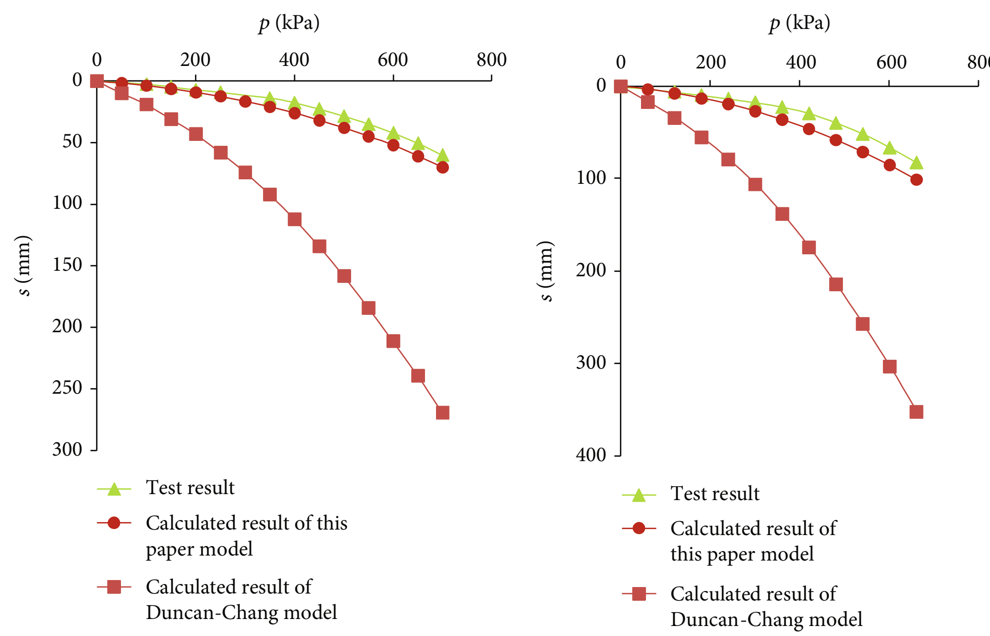


FIGURE 5: Mesh divisions for numerical analysis using the variable modulus constitutive model.



(a) In situ plate loading test 1#

(b) In situ plate loading test 3#

FIGURE 6: Comparison of numerical results using the variable modulus constitutive model and in situ test results.

are shown in Figure 6. The p - s curves calculated by numerical analysis based on the variable modulus constitutive model match well with the in situ test results, which verify the rationality of the proposed variable modulus constitutive model.

It is known that the Duncan-Chang model can reflect the nonlinear deformation characteristics of foundation soils, but the required parameters are primarily obtained from conventional triaxial tests. In order to compare with the calculated results of the variable modulus constitutive model, the associated numerical results of the Duncan-Chang model are also included in Figure 6. The calculated parameters of the Duncan-Chang model are shown in Table 4.

TABLE 4: Calculated parameters of the Duncan-Chang model.

k	n	c/kPa	$\varphi/^{\circ}$	R_f	D	G	F
115.7	0.4	25.2	31.2	0.7	3.1	0.2	0.1

As can be seen from Figure 6, the numerical results of the Duncan-Chang model based on conventional triaxial test parameters are quite different from in situ loading test results and numerical results of the variable modulus constitutive model. The main reason may be that the granite residual soil has strong structural characteristics [18–21], and the effects of stress release and disturbance caused by sampling lead to large differences between laboratory test parameters and actual ones. Even though the variable modulus constitutive model is a simplified model based on the Duncan-Chang model, it can reflect the nonlinear deformation of the foundation soil, and its parameters are obtained from in situ test. Comparing the numerical results with the in situ loading test results, the variable modulus constitutive model based on the in situ test parameters for calculation of foundation nonlinear settlement has some advantage over the Duncan-Chang model based on the conventional triaxial test parameters.

5. Conclusions

In this study, a nonlinear deformation modulus method and a variable modulus constitutive model for foundation settlement calculation based on in situ loading test are proposed. The validity of these two calculation models are verified by the comparisons of calculated results with in situ loading test results under different plate sizes. The main conclusions can be summarized as follows:

(1) The calculation parameters, which can reflect the undisturbed nature and the deformation nonlinearity of the soil under the actual stress state, are the key factors that directly affect the calculation accuracy of foundation settlement. The nonlinear deformation modulus method and variable modulus constitutive model proposed based on in situ loading test in this paper can better take these two key factors into account

(2) The results calculated from both the nonlinear deformation modulus method and the variable modulus constitutive model proposed in this study compare very well with the in situ loading test results under different plate sizes, which can also better reflect the nonlinearity of foundation settlement

(3) The required parameters of both the nonlinear deformation modulus method and the variable modulus constitutive model are the cohesion c, the internal friction angle φ, and the initial deformation modulus E_0 of soils, which are common parameters for practical engineering and can be readily deduced

Data Availability

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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