



# An approach for response prediction of a single pile subjected to tension load considering modulus degradation of soil

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Received: 3 February 2019 / Accepted: 9 October 2019 / Published online: 11 October 2019  
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**Abstract** Determination on forms of load-transfer curve is a common concern in the response analysis of a single pile subjected to tension load using ‘t–z’ method in practice. Based on ‘t–z’ method, an approach for nonlinear analysis of the load–displacement response of a single pile subjected to tension load is proposed in the present paper. In this paper, based on shear displacement theory, a theoretical load-transfer model is established considering the influence of soil stress on nonlinear deformation of soil, modulus degradation characteristics and shear modulus of soil. The validity of the proposed model is checked using existing theoretical solutions. According to the pile–soil interaction mechanism in different soil types,

empirical correlations for limiting unit skin friction of an uplift pile embedded in cohesive and non-cohesive soils are established. To analyze the load–displacement response of a single pile subjected to tension load, a highly effective iterative computer program is developed based on the Runge–Kutta method. Comparisons of the present computed values, the reported centrifuge and field test results are made to verify the reliability of the proposed method.

**Keywords** Single uplift pile · Modulus degradation · Pile–soil interaction · Runge–Kutta method

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## 1 Introduction

As a structure that can sustain the uplift loading, uplift piles have been widely used in a large amount of projects such as foundation of transmission lines, mooring system of offshore platform and underground substations. At present, scholars have conducted theoretical research on the limiting bearing capacity of the single uplift pile (He 2001; Huang et al. 2007; Zhang et al. 2015a, 2018), however, it is crucial to study the response prediction of piles subjected to uplift loading, especially for the engineering constructions with deformation control.

Load Transfer Method was originally proposed by Seed and Reese (1957) to describe the linear elastic load-settlement relationship of a single pile.

Meanwhile, as a simplified calculation method that can reflect the pile–soil nonlinear deformation properties, response prediction of pile groups can also be accurately and quickly acquired by using the load transfer method (Zhang et al. 2016, 2019a, b). In order to consider the nonlinear pile–soil behavior, many scholars have developed various forms of load transfer models using the field test data and applied them to the deformation analysis of uplift pile (Sun and Yang 2008; Zhang and Zhang 2012, 2015a; Lashkari 2017; Cheng et al. 2018). Sun and Yang (2008) proposed a modified method of deformation compatibility to analyze the response prediction of uplift piles using a hyperbolic relationship between unit skin friction and pile–soil relative displacement. Based on the full-scale destructive loading filed tests results, Zhang et al. (2015b) proposed a softening model of skin friction and analyzed the single uplift pile response using the load transfer method. Lashkari (2017) proposed a simple critical state compatible semi-hyperbolic interface model by leading a state variable in the traditional hyperbolic model to reflect the volume change of the lateral earth, and he analyzed the load–displacement relationship of the single uplift pile using the segment analysis among the piles. Cheng et al. (2018) proposed a simple analytical approach to clarify the nonlinear response of a single pile subjected to tension load using a hyperbolic model of skin friction and the Taylor series expansion.

The key of the load transfer method is to establish a transfer function that can truly reflect the skin friction and the corresponding shear displacement of the pile–soil interface. However, a function that is based on experiences is usually used to describe the nonlinear behavior of the pile–soil interface in the traditional load transfer method. In actual engineering, the form of the load transfer function is unknown and needs to be determined based on the results of field or model test. Therefore, there is a need to establish the load-transfer function by analyzing the nonlinear stress–strain relationship of the pile–soil interface. In this paper, a theoretical load-transfer model is established considering the influence of soil stress on the nonlinear deformation of soil, modulus degradation characteristics and shear modulus of soil, and a calculation method is proposed for the analysis on the limiting unit skin friction of an uplift pile embedded in cohesive and non-cohesive soils. A highly effective iterative computer program is then developed based on the Runge–

Kutta methods to capture the response of a single pile subjected to tension load.

## 2 Nonlinear behavior of soil

At low loading level, soils are commonly shown as linear elastic behavior, and appear to be an nonlinear behavior with increasing loading level. As suggested by Zhu and Chang (2002), the load-transfer behavior along a bored pile will be significantly affected by the nonlinear decreasing of the soil modulus. To consider the nonlinear behavior of soil, a modulus degradation curve of soil is commonly adopted in practice. Kondner (1963) was the first to describe the nonlinear behavior of soils using a hyperbolic curve. However, the hyperbolic relationship between modulus and strain only remains at the early loading period, which is not accurate enough to describe the whole degradation process of modulus. Based on the hyperbolic model, an S-type degradation curve with better feasibility was proposed by Fahey and Carter (1993), as shown in Eq. (1).

$$G_{\text{sec}} = G_0 \left( 1 - f \cdot \left( \frac{\tau}{\tau_f} \right)^g \right) \quad (1)$$

A modified equation from Lee and Salgado (1999) is shown in Eq. (2), in terms of the effect of confining pressure on shear modulus during the loading process.

$$G_{\text{sec}} = G_0 \left( 1 - f \cdot \left( \frac{\tau}{\tau_f} \right)^g \right) \left( \frac{p'}{p'_0} \right)^n \quad (2)$$

where  $p'$  is the current mean effective stress;  $p'_0$  is the initial mean effective stress; and  $n$  is the exponent of confining pressure on shear modulus. Additionally, for the non-cohesive soils,  $n = 0.5$ , while for the cohesive soils,  $n = 1$ .

The initial shear modulus or small-strain shear modulus  $G_0$  can be directly obtained through the shear wave speed  $v_s$  derived from seismic cone penetration test, and it can also be calculated using Eq. (3) suggested by Hardin and Drnevich (1972).

$$G_0 = C_g p_a \frac{(e_g - e_0)^2}{1 + e_0} \sqrt{\frac{p'}{p_a}} \quad (3)$$

where  $C_g$ , and  $e_g$  are dimensionless parameters depending on soil properties;  $e_0$  is the initial porosity;  $p'$  is the mean effective stress; and  $p_a$  is the reference pressure, whose value can be taken as 100 kPa.

### 3 Establishment of the theoretical load-transfer model

As suggested by Randolph and Wroth (1978), the vertical displacement of the soil can be calculated by integrating the shear strain  $\gamma$  from pile radius  $r_0$  to a limited distance  $r_m$ , as shown in Eq. (4).

$$w_s = \int_{r_0}^{r_m} \gamma dr = \int_{r_0}^{r_m} \frac{\tau_{soil}}{G_{sec}} dr \tag{4}$$

where  $\tau_{soil}$  is the shear stress around the pile at a radial distance  $r$  from the pile axis;  $r_0$  is the pile radius;  $G_{sec}$  is the shear modulus of soil; and  $r_m$  is the radius of influence of piles, and can be computed following the suggestion of Zhang et al. (2014).

The radial attenuation of the soil shear stress can be defined using Eq. (5).

$$\tau_{soil} = \tau_s \frac{r_0}{r} \tag{5}$$

where  $\tau_s$  is the shear stress of the pile–soil interface.

Substituting Eqs. (2) and (5) into Eq. (4), the theoretical load-transfer model of modulus degradation of soil considering the influence of confining pressure can be computed by:

$$w_s = \frac{\tau_s r_0}{G_0 g \left(\frac{p'}{p'_0}\right)^n} \ln \left[ \frac{\left(\frac{r_m}{r_0}\right)^g - f \left(\frac{\tau_s}{\tau_f}\right)^g}{1 - f \left(\frac{\tau_s}{\tau_f}\right)^g} \right] \tag{6}$$

If the confining pressure  $p'/p'_0$  is not under the consideration, and  $f = R_f$ ,  $g = 1$ , a load-transfer function based on the hyperbolic stress–strain relation is degenerated according to Kraft et al. (1981), as shown in Eq. (7).

$$w_s = \frac{\tau_s r_0}{G_0} \ln \left[ \frac{\left(\frac{r_m}{r_0}\right) - R_f \left(\frac{\tau_s}{\tau_f}\right)}{1 - R_f \left(\frac{\tau_s}{\tau_f}\right)} \right] \tag{7}$$

When  $f = R_f = 0$ , Eq. (6) is further degenerated to the linear load-transfer model suggested by Randolph and Wroth (1978).

$$w_s = \frac{\tau_s r_0}{G_0} \ln \left( \frac{r_m}{r_0} \right) \tag{8}$$

### 4 Determination of the limiting shaft resistance of uplift piles

A reasonable  $\tau_f$  value is the key to accurately evaluate the response prediction of a single pile, because the value of the limiting shaft resistance of the uplift pile plays a vital role in the calculation result, whether in the bearing capacity calculation or in the deformation analysis. Shear dilation and shear shrinkage are often shown in the pile–soil interface of the single pile under non-cohesive soils, while under the cohesive soils, the main mechanism between pile and soil interface is the undrained shear characteristic. As a result, it is necessary to discuss about the limiting shaft resistance in the two above-mentioned situations as.

For the single pile in non-cohesive soils, the limiting shaft resistance  $\tau_f$  is usually calculated using the Mohar–Coulomb failure criterion, as shown in Eq. (9).

$$\tau_f = \sigma'_{hf} \tan \delta = K \sigma'_{v0} \tan \delta \tag{9}$$

where  $K$  is the coefficient of lateral earth pressure;  $\sigma'_{hf}$  is the horizontal effective stress of the soil in a destroyed state;  $\sigma'_{v0}$  is the vertical effective stress of soil; and  $\delta$  is the friction angle of the pile–soil interface.

There are many factors affecting the coefficient of lateral earth pressure  $K$ , such as the coefficient of earth pressure at rest  $K_0$ , the initial sand density, and the stress state around the pile. Loukidis and Salgado (2008) proposed an empirical formula of the coefficient of lateral pressure of bearing piles  $K$  using a two-surface-plasticity constitutive model, as shown in Eq. (10).

$$\frac{K}{K_0} = \frac{C \exp \left\{ \frac{D_r}{100\%} \left[ 1.3 - 0.2 \ln \left( \frac{\sigma'_{s0}}{p_a} \right) \right] \right\}}{F(K_0)} \tag{10}$$

where  $D_r$  is the relative density;  $K_0 = 1 - \sin \phi'_{cv}$ ,  $\phi'_{cv}$  is the constant-volume (or critical-state) friction angle;  $F(K_0)$  is a function considering the effect of  $K_0$  on the value of  $K$ , and can be taken as  $\exp(0.2\sqrt{K_0} - 0.4)$ ; and  $C$  can be adopted as 0.7 following the suggestion of Loukidis and Salgado (2008).

The friction angle of the pile–soil interface in sandy soil  $\delta$  is mainly affected by the roughness of pile shaft and the average diameter of the soil  $D_{50}$  (Kishida and Uesugi 1987), and the value is commonly taken as 0.75 to 1 times friction angle of soil  $\varphi'$ . As to the large pile–soil roughness, e.g., bored pile and rough steel pile, the value of  $\delta$  can be assumed to be identical to the  $\varphi'$  value (Loukidis and Salgado 2008; Lehane 2005).

For the single pile in cohesive soil, an empirical coefficient  $\alpha$  is introduced with the undrained shear strength of soil  $s_u$  to calculate the limiting shaft resistance  $\tau_f$ , as shown in Eq. (11).

$$\tau_f = \alpha s_u \quad (11)$$

where the undrained shear strength of soil  $s_u$  can be obtained through the in situ testing or the laboratory undrained shear test; and the empirical coefficient  $\alpha$  can be obtained through static load test results or empirical equations.

Based on a large amount of field measured data, Fleming et al. (2009) proposed an empirical relationship between  $\alpha$  and strength ratio  $s_u/\sigma'_{v0}$ , as shown in Eq. (12).

$$\alpha = \begin{cases} \frac{0.5}{\left(\frac{s_u}{\sigma'_{v0}}\right)^{0.5}}, & \frac{s_u}{\sigma'_{v0}} < 1 \\ \frac{0.5}{\left(\frac{s_u}{\sigma'_{v0}}\right)^{0.25}}, & \frac{s_u}{\sigma'_{v0}} \geq 1 \end{cases} \quad (12)$$

Note that the method mentioned above is only applicable to the piles subjected to vertical tension load. Previous researches demonstrated that when the pile shaft was subjected to the pull-out resistance, the radial stress was weakened due to the opposite loading path, and under the same site, the limiting shaft resistance value of the uplift pile was smaller than that of the axially loaded vertical pile. According to the field test results, Zhang et al. (2015b) proposed that the limiting shaft resistance value of uplift piles could be adopted as 0.7 times the value estimated from the limiting shaft resistance value of axially loaded vertical piles. Therefore, the limiting shaft resistance value of uplift piles used in this paper is adopted as 0.7 times the value of Eq. (10) or Eq. (12).

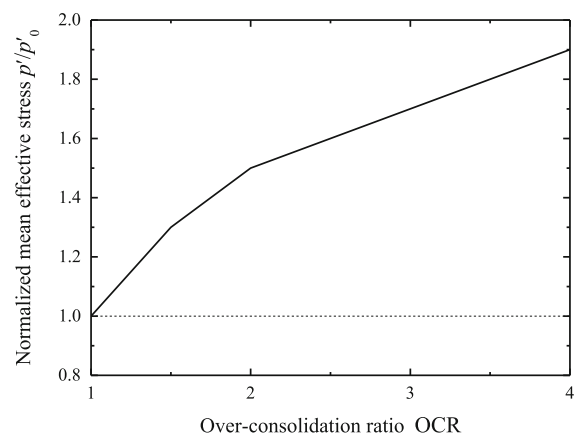
## 5 Determination of the mean effective stress of soil around piles

Because of the construction disturbance and the interaction between the pile–soil interface at working load, the stress state of soil around pile under the normal condition of single pile is greatly different from that under natural state. Combining the definition of mean effective stress ( $p' = (1 + 2K)\sigma'_{v0}/3$ ) and the coefficient of earth pressure under non-cohesive soils mentioned in Sect. 4, the value of mean effective stress under non-cohesive soils can be computed by:

$$\frac{p'}{p'_0} = \frac{1 + 2K}{1 + 2K_0} \quad (13)$$

Sheil et al. (2015) analyzed the mean effective stress of soil around pile after installation and subsequent consolidation using the advanced MIT-S1 constitutive model and a modified version of cavity expansion method, and established a relationship associate the over-consolidation ratio with normalized mean effective stresses in the zone of soil surrounding the pile, providing a good method for engineering practice.

Figure 1 shows the relationship between normalized mean effective stress and over-consolidation ratio after installation and consolidation. Following the analysis of Sheil et al. (2015), soil type has little effect on the relationship shown in Fig. 1, and the relationship shown in Fig. 1 can be suitable for applications in cohesive soils.



**Fig. 1** Relationship between normalized mean effective stress and over-consolidation ratio after installation and consolidation

## 6 Analysis on response prediction of the single uplift pile

### 6.1 Load-transfer relationship of pile shaft

Ignoring the pile self-weight and considering the equilibrium conditions of pile elements, the relationship between axial force  $P(z)$  and unit shaft resistance  $\tau(z)$  can be computed as:

$$dP(z) = -U\tau(z)dz \tag{14}$$

where  $U$  is the pile perimeter.

The shaft displacement at depth  $z$  can be calculated by:

$$w(z) = w_t - \int_0^z \frac{P(z)}{E_p A_p} dz \tag{15}$$

where  $w_t$  is the pile head displacement.

The relationship between shaft displacement at depth  $z$   $w(z)$  and axial force  $P(z)$  can be expressed as:

$$dw(z) = -\frac{P(z)}{E_p A_p} dz \tag{16}$$

Substituting Eq. (14) into Eq. (16), the load-transfer differential equation of pile shaft between unit shaft resistance  $\tau(z)$  and shaft displacement at depth  $z$   $w(z)$  can be expressed by:

$$\frac{d^2 w(z)}{dz^2} = f(z) = \frac{U\tau(z)}{E_p A_p} \tag{17}$$

where  $E_p$  is the elastic modulus of the pile; and  $A_p$  is the cross-sectional area of the pile.

Equation (17) is a divalent non-linear differential equation and it is hard to get the analytical solution. Therefore, numerical differentiation is adopted in this paper.

For the single pile subjected to tension load, the load mobilized at the pile base is zero during the whole loading level. The boundary conditions of the uplift piles can be described as:

$$\begin{cases} w(L) = w_b \\ dw(z)/dz|_{z=L} = 0 \end{cases} \tag{18}$$

The variable  $T(z)$  is introduced in the differential relationship  $dw(z)/dz$ , Eq. (18) can be rewritten as:

$$\begin{cases} dT(z)/dz = f(z) \\ w(L) = w_b, T(L) = 0 \end{cases} \tag{19}$$

Fourth-order Runge–Kutta methods is a commonly used algorithm in traditional numerical analysis with the characteristics of simple structure and high efficiency, and it is used in this paper. The following four sets of additional variables can be written as:

$$\begin{cases} k_1 = w^k(z), l_1 = f(w^k(z)) \\ k_2 = w^k(z) + \frac{h}{2}l_1, l_2 = f\left(w^k(z) + \frac{h}{2}k_1\right) \\ k_3 = w^k(z) + \frac{h}{2}l_2, l_3 = f\left(w^k(z) + \frac{h}{2}k_2\right) \\ k_4 = w^k(z) + hl_3, l_4 = f(w^k(z) + hk_3) \end{cases} \tag{20}$$

where  $k$  represents the  $k$ th calculating node;  $h$  is the calculating step; and depth  $z$  is adopted as  $kh$ .

The state-variable value of next calculating step can be solved by using the fourth-order Runge–Kutta methods. One obtains:

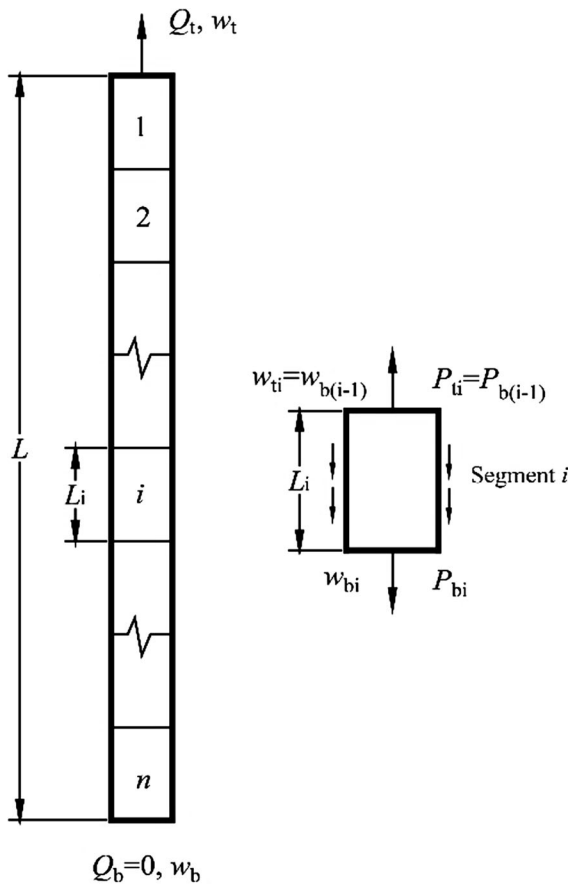
$$\begin{cases} w^{k+1}(z) = w^k(z) + hT^k(z) + \frac{h^2}{6}(l_1 + l_2 + l_3) \\ T^{k+1}(z) = T^k(z) + \frac{h}{6}(l_1 + l_2 + l_3 + l_4) \end{cases} \tag{21}$$

Therefore, the shaft displacement and axial force at depth  $z$  of the pile can be calculated using pile end displacement  $w_b$  and Eq. (21).

### 6.2 Algorithm for the iterative calculation

Considering the changes of soil stiffness along with the changes of stress level and depth, it is necessary to divide the pile into multiple segments analysis (see Fig. 2). The pile shaft can be divided into a serials of segments according to the distribution of the soil layer. The iterative method for a tension pile embedded in multi-layered soils can be analyzed with the following procedure:

- (1) Assume a single pile is divided into  $n$  segments.
- (2) Assume a small pile end displacement  $w_b$ .
- (3) Substitute  $w_b$  into Eq. (19).
- (4) Calculate the pile head displacement  $w_{ti}$  of segment  $i$  and pile head load  $P_{ti}$  using Eq. (21).
- (5) Using  $w_{ti} = w_{b(i-1)}$  and  $P_{ti} = P_{b(i-1)}$  as the initial value condition of the calculating, repeat steps 3 and 4 until the load–displacement relationship at the uplift pile head is obtained.



**Fig. 2** Load-transfer analysis of a single uplift pile

- (6) Using a different assumed pile end displacement, repeat step 2 to 5 until a series of load–displacement values are obtained.

## 7 Case analysis

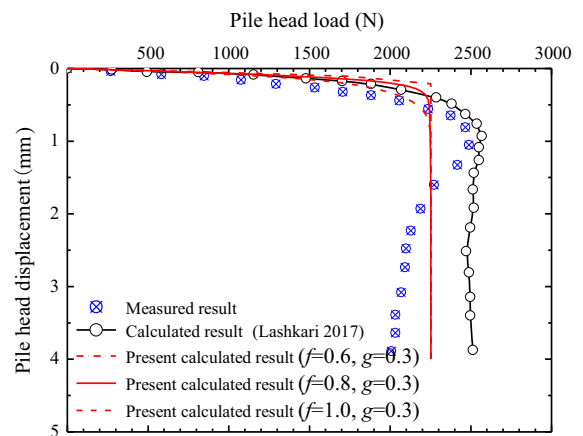
### 7.1 The centrifugal test

Guerra (2010) investigated the response prediction of uplift piles in the dry FF sand under  $N_g = 80$  centrifugal acceleration using the centrifugal test. The rough aluminum model pile with a length of 32 cm and a diameter of 3.2 cm was used in the test, and the elastic modulus of the pile shaft was 70 GPa. For the FF sand, the relative density  $D_r$  was 85%, the original porosity  $e_0$  was 0.804, and the unit weight  $\gamma$  was 14.87 kN/m<sup>3</sup>. According to the experimental data from FF sand-aluminum interfaces (Lashkari 2017),

the back-calculated constant friction  $\phi_{cv}$  was adopted as 39°. According to a large amount of back-analyzed results, the value of parameters  $f$  and  $g$  can be determined in the modulus degradation curve of sandy soil during the centrifugal tests ( $f = 0.8$ ,  $g = 0.3$ ). To analyze the influence of parameter  $f$  on the response prediction of single piles, the load–displacement relationships are calculated using different values of  $g$  and  $f$ , e.g.,  $g = 0.3$  and  $f = 0.6, 1$  in this paper. Following the suggestions of Lashkari (2017), Eq. (3) was used to calculate the initial shear modulus, and the values of  $C_g$  and  $e_g$  are determined as 250 and 2.97, respectively. Note that during the centrifugal tests on the response predictions of uplift piles, a scale  $N_g$  defined as the ratio of centrifugal acceleration to gravitational acceleration should be considered. Therefore, it is necessary to replace volumetric weight  $\gamma$  under the conventional gravity with  $\gamma_N = N_g \times \gamma$ . When Eq. (10) is used to determine the lateral earth pressure  $K$ , the value of  $K_0$  is too small to affect the value of  $K$ . Therefore,  $F(K_0)$  can be approximately taken as 1.

Load–displacement curves of the single uplift pile under different uplift loads can be obtained using the above-mentioned procedure. To verify the correctness of the present method, comparisons of the present load–displacement curve, the measured result of Guerra (2010) and the calculated result of Lashkari (2017) are shown in Fig. 3.

Figure 3 shows that the present load–displacement curve is generally consistent with the field measured result of Guerra (2010) and the calculated result of



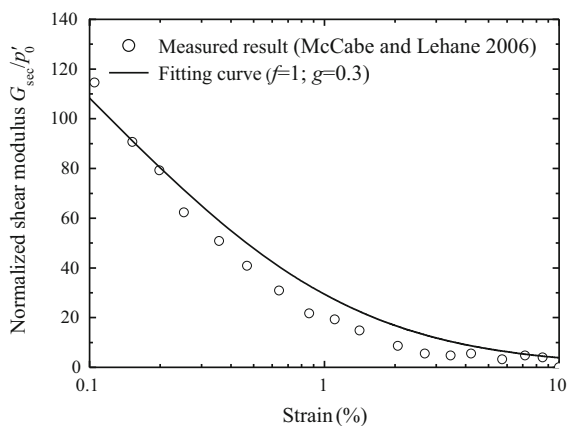
**Fig. 3** Measured and calculated load–displacement curve of a single uplift pile embedded in sand



Lashkari (2017). When the pile head load is larger than 2250 N, the calculated pile head displacement obviously increases, indicating a full mobilization of the shaft resistance of the whole pile, and the single uplift pile has been pulled out. The measured limit bearing capacity is taken as 2000 N, which is consistent with the present calculated result. Under the same load level, the pile head displacement increases with increasing value of parameter  $f$ . However, the influence on parameter  $f$  is limited, because the change in stiffness controlled by parameter  $f$  is not significant under the working load.

### 7.2 The field test

The example came from a load test of uplift piles in a soft clay-silt carried out by McCabe and Lehane (2006). All piles employed were 0.25 m square precast concrete sections with the equivalent diameter was 0.282 m to reach their final penetration depth of 6 m, the elastic modulus of pile shaft was 30 GPa. The pile test was performed at a site in Belfast lightly over consolidated silt, the in situ peak vane strength around piles  $s_u$  was adopted as 20 kPa derived from the in situ vane shear test (McCabe and Lehane 2006). The initial shear modulus  $G_0$  was adopted as 10 MPa based on the results of the cone penetration tests (CPT). Using the degradation curve of shear modulus shown in Fig. 4, the value of  $f$  and  $g$  can be adopted as 1.0 and 0.3, respectively. To analyze the influence of parameter  $g$  on the response prediction of single piles, the  $g$  value of 0.1 and 0.2 were used in the present calculation.



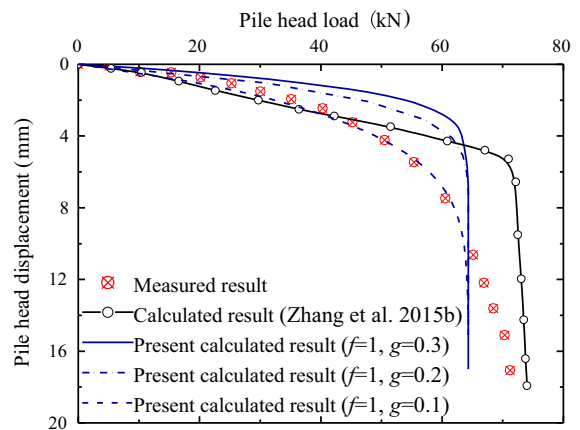
**Fig. 4** Calibration of shear modulus degradation parameters

As suggested by McCabe (2002), in this test site from soil depth of 2 m to 3–6 m of, the value of OCR decreased from 2 to 1.2. The ratio of in situ peak vane strength  $s_u/\sigma'_{v0}$  is computed as about 0.4, and the value of  $\alpha$  can be adopted as 0.55 according to Eq. (12).

Under different uplift loading levels, the single pile response can be obtained using the proposed method. To check the validity of the proposed method, comparisons of the present load–displacement curve, the measured result of McCabe (2002) and the calculated result of Zhang et al. (2015b) are made, as shown in Fig. 5.

Figure 5 shows that the present load–displacement curve of the single uplift pile is generally consistent with the measured result of McCabe (2002) and the calculated result of Zhang et al. (2015b). When the pile head load is larger than 64 kN, the shaft resistance of the uplift pile is fully mobilized. The measured limit load of the single pile is about 72 kN (McCabe 2002), which is in good agreement with the present calculated value. The predicted pile head displacement is less than the measured value at the same load up to the limit load. This discrepancy may be due to the mismatch between the in situ test data and the loading filed test data (McCabe 2002; Sheil and McCabe 2016).

From Fig. 5 it can also be concluded that the pile head displacement decreases with increasing value of  $g$  under the same loading level. The parameter  $g$  has a great influence on the load–displacement curve, because the rate of stiffness degradation is mainly controlled by the value of  $g$ . Furthermore, the



**Fig. 5** Measured and calculated load–displacement curve of a single uplift pile embedded in clay

calculated result is in a good agreement with the measured result when the value of  $g$  is adopted as 0.1.

## 8 Conclusions

This paper presents a new approach for the response prediction of a single pile based on the traditional load transfer method. Based on shear displacement theory, a theoretical load-transfer model is established considering the influence of soil stress on nonlinear deformation of soil, modulus degradation characteristics and shear modulus of soil. According to the pile–soil interaction mechanism in different soil types, empirical correlations for limiting unit skin friction of an uplift pile embedded in cohesive and non-cohesive soils are established. To analyze the load–displacement response of a single pile subjected to tension load, a highly effective iterative computer program is developed based on the Runge–Kutta method. Comparisons of the present computed values, the reported centrifuge and field test results are made to verify the reliability of the proposed method.

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