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地面超载下摩擦单桩负摩阻力的连续介质力学解

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摘要:针对地面超载引起的摩擦单桩负摩阻力,提出了一种基于桩土相互作用和连续介质力学的竖向位移模型来模拟桩土系统位移,模型中包含了桩土轴线位移函数和沿径向的衰减函数;采用变分法得到了桩土系统的总势能,从而得到了位移模型中桩土位移和衰减函数的耦合控制方程;利用迭代法求解了这两个耦合函数,得到了桩身轴力、中性面、负摩阻力、下拉力和土弹簧等效刚度等关键参数的数学表达式;通过和原位试验及已有理论结果的对比,验证了所提方法的正确性。结果表明:桩土模量比对桩的负摩阻力和下拉力有显著影响,中性面位置与长细比密切相关,而与桩土模量比关系不大;随着桩土模量比增大和桩长细比减小,桩身同一位置处轴力增长明显,但增长速率变缓;地面超载作用下桩周土体位移最大沉降出现在地表处,且地表以下0.3倍桩长范围内为位移强烈影响区;桩长细比和桩土模量比的增加均会使桩周土体位移场影响范围增大。

关键词:地面超载;摩擦桩;变分法;中性面;下拉力

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Continuum mechanical solution for negative frictional resistance of single floating pile under ground overload

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Abstract: A vertical displacement model based on pile-soil interaction and continuum mechanics was proposed to simulate the displacement of the pile-soil system in response to the negative friction force of a single pile caused by ground overload. The model included the displacement function of the pile-soil axis and the attenuation function along the radial direction. The total potential energy of the pile-soil system was obtained by using the variational calculus, and the coupled control equations for the pile-soil displacement and attenuation function in the displacement model were obtained. The iterative method was used to solve these two coupling

functions, and mathematical expressions for key parameters such as pile axial force, neutral plane, negative frictional resistance, down-drag force, and equivalent stiffness of soil spring were obtained. The correctness of the proposed method were verified through comparison with in-situ experiments and existing theoretical results. The results show that the ratio of pile-soil modulus has a significant impact on the negative friction resistance and down-drag force of piles, and the position of the neutral plane is closely related to the slenderness ratio, while it is not closely related to the ratio of pile-soil modulus. As the pile-soil modulus ratio increases and the pile slenderness ratio decreases, the axial force at the same position of the pile increases significantly, but the growth rate slows down. The maximum settlement of soil displacement around piles under ground overload occurs at the surface, and the range of 0.3 times the pile length below the surface is a strongly affected area of displacement. The increase in pile slenderness ratio and pile-soil modulus ratio will both increase the range of influence on the displacement field of the soil around the pile.

Key words: ground overload; floating pile; variational calculus; neutral plane; down-drag force

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0 引 言

中国沿海地区存在大规模深厚软土层,这些地区大面积填土导致邻近桩基出现负摩阻力,从而影响桩基的竖向承载力。其原因在于大面积堆载导致土体固结沉降,桩相对于土体向上运动,桩周土对桩身产生附加下拉荷载,增大桩身轴力,从而可能导致桩基础和上部结构严重破坏^[1-2]。桩侧负摩阻力的大小取决于桩-土相对位移量 and 中性点的位置,而现有规范^[3]规定中性点深度由桩周软弱土层的下限深度和经验系数确定,设计人员往往需要凭地区经验选取。因此,桩侧负摩阻力的确定既是一个重要工程问题,也具有明显理论研究意义。

目前,桩侧负摩阻力的确定方法主要包括有效应力法、弹性理论法、荷载传递法和数值分析法。在有效应力法^[4-7]中,一般通过试验数据^[8-11]或数值方法^[12],根据经验确定中性面和下拉力。显然,土的沉降对中性面和负摩阻力的影响被忽略,所求解的负摩阻力为极限值,不能反映桩土相对位移与负摩阻力关系。剪切传递法能充分考虑桩土相对位移,因而在计算中应用较多^[13-18]。超载下桩的荷载传递法实质是将周围土体模拟成一系列弹簧,然而这类方法难以选用合适的摩阻力-桩土相对位移模型,也不能考虑土的连续性。基于连续介质理论的分析方法能充分反映土层性质、桩土模量比、桩长细比、桩土相对位移量及桩端土层厚度等因素的影响,可以克服以上方法的局限性。Poulos 等^[19-20]基于连续介质理论的解,计算了端承单桩的负摩阻力变化。

目前,为研究土介质的非线性和桩-土界面的滑移效应,摩擦桩的计算已成为研究重点^[21-29]。

虽然超载作用下桩的负摩阻力计算方法已有较大进展,但单桩负摩阻力计算仍未得到很好的解决^[30-35]。本文拟提出地面超载下摩擦单桩负摩阻力计算方法。首先,构建一种新的桩土位移模型,并根据 Vallabhan 等^[31]的思想,得到位移和衰减函数耦合控制方程;其次,采用迭代法求解耦合方程;然后,结合工程案例^[36]对该方法正确性进行了验证;最后,通过参数分析给出桩长细比、桩土模量比对桩身负摩阻力的影响。

1 计算模型

地面超载作用下摩擦单桩-土体系如图 1 所示,其中地面超载 p 为均布荷载, L 为桩长, R 为半径, D 为直径($D=2R$), E_p 为弹性模量, ν_p 为泊松比。假定桩周土为均匀各向同性线弹性介质,弹性模量为 E_s ,泊松比为 ν_s 。假定桩-土界面处无滑移或分离,且忽略横向位移。桩-土体系覆盖在深度为 H 的刚性基岩上。

图 1 模型中柱坐标体系 $r\theta z$ 下,桩-土系统位移场为 $\{\bar{u}, \bar{v}, \bar{w}\}$ 。无桩时,即地面超载作用下土体自由场记为 $w_f(z)$,其和竖向坐标 z 相关,而与 r 和 θ 无关。当有桩时,由于桩土相互影响,桩-土系统位移与自由场不同,环向位移 \bar{u} 和径向位移 \bar{v} 可忽略不计。由于本问题为轴对称问题,其位移与 θ 无关。现假设桩-土系统竖向位移 $\bar{w}(r, z)$ 为 r 和 z 的函数,即

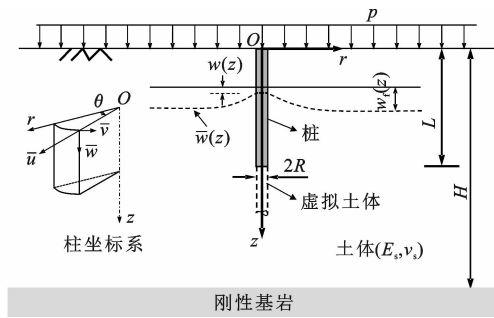


图 1 地面超载作用下桩-土系统

Fig. 1 Pile-soil system under ground overload

$$\bar{w}(r, z) = w_f(z) - [w_f(z) - w(z)]\phi(r) \quad (1)$$

$$\bar{u}(r, z) = \bar{v}(r, z) = 0 \quad (2)$$

式中: $w(z)$ 为沿桩轴线竖向位移; $\phi(r)$ 为沿 r 向量纲一的衰减函数, 满足 $\phi(R) = 1$ 和 $\phi(\infty) = 0$ 。

由式(1)可知: 当 $z \leq L$ 时, $w(z)$ 为桩位移函数 $w_p(z)$; 当 $L \leq z \leq H$, $w(z)$ 为桩下方土柱位移函数 $w_s(z)$ 。若 $w_f(z)$ 为 0, 则位移模型将简化为 Vallabhan 等^[31] 的位移模型。

2 自由场位移

图 1 中自由场竖向应变 ϵ_z 为

$$\epsilon_z = \frac{\partial w_f(z)}{\partial z} \quad (3)$$

非零径向、环向和竖向应力 σ_r 、 σ_θ 、 σ_z 分别为

$$\sigma_z = \sigma_r = \sigma_\theta = E_s^* \frac{\partial w_f(z)}{\partial z} \quad (4)$$

式中: $E_s^* = E_s(1 - v_s) / [(1 + v_s)(1 - 2v_s)]$ 。

自由场总势能 Π_f 为

$$\Pi_f = \int_0^{2\pi} \int_0^\infty p w_f(0) r dr d\theta + \frac{1}{2} \int_0^{2\pi} \int_0^\infty \int_0^H \epsilon_z \sigma_z r dr d\theta \quad (5)$$

将 Π_f 对 $w_f(z)$ 取变分, 可得

$$\frac{\partial w_f(z)}{\partial z^2} = 0 \quad (6)$$

且有边界条件

$$p + E_s^* \frac{\partial w_f(z)}{\partial z} = 0 \quad z = 0 \quad (7)$$

$$w_f(z) = 0 \quad z = H \quad (8)$$

由式(6)~(8)可得自由场位移 $w_f(z)$ 为

$$w_f(z) = \frac{p}{E_s^*} (H - z) \quad (9)$$

可见自由场位移 $w_f(z)$ 随深度 z 线性衰减, 且 $w_f(H) = 0$, $w_f(0) = pH/E_s^*$ 。

3 求解方法

3.1 桩-土系统总势能

由式(1)、(2)描述的桩-土位移模式, 桩-土系统

中非零应变 ϵ_z 和 ϵ_{rz} 为

$$\epsilon_z = \frac{\partial \bar{w}}{\partial z}; \quad \epsilon_{rz} = \frac{\partial \bar{w}}{\partial r} \quad (10)$$

非零径向、环向和竖向正应力 σ_r 、 σ_θ 、 σ_z 和剪应力 σ_{rz} 分别为

$$\sigma_r = \sigma_z = \sigma_\theta = E_s^* \frac{\partial \bar{w}}{\partial z}; \quad \sigma_{rz} = \mu_s \frac{\partial \bar{w}}{\partial r} \quad (11)$$

式中: $\mu_s = E_s / [2(1 + v_s)]$ 。

桩-土系统总势能 Π 为

$$\begin{aligned} \Pi = & \frac{1}{2} \int_0^L E_p A_p \left(\frac{\partial w_p}{\partial z} \right)^2 + \frac{1}{2} \int_L^H E_s A_p \left(\frac{\partial w_s}{\partial z} \right)^2 + \\ & \frac{1}{2} \int_0^{2\pi} \int_R^\infty \int_0^H (\epsilon_r \sigma_r + \epsilon_z \sigma_z + \epsilon_\theta \sigma_\theta + \epsilon_{rz} \sigma_{rz}) \cdot \\ & r dz dr d\theta - 2\pi \int_0^\infty p \bar{w}(r, 0) r dr \end{aligned} \quad (12)$$

式中: A_p 为桩横截面积, $A_p = \pi R^2$ 。

对总势能 Π 取变分, 有

$$\delta \Pi = 0 \quad (13)$$

3.2 位移函数

收集式(13)的 δw_s 项, 则土体位移函数 $w_s(z)$ 的控制方程可写为

$$\frac{\partial^2 w_s}{\partial z^2} = \beta^2 (w_s - w_f) \quad (14)$$

式中: $\beta^2 = k / (E_s A_p + 2t)$, k 为等效土弹簧刚度, $2t$ 为联系土弹簧的膜影响^[35]。

k 和 $2t$ 分别表示为

$$k = 2\pi \mu_s \int_R^\infty r \left(\frac{\partial \phi}{\partial r} \right)^2 dr \quad (15)$$

$$2t = 2\pi E_s^* \int_R^\infty r \phi^2 dr \quad (16)$$

考虑桩-土交界面处、桩底和土柱顶部位移连续, 土柱底部位移为 0, 于是有

$$w_s(L) = w_p(L) \quad (17)$$

$$w_s(H) = 0 \quad (18)$$

利用式(17)、(18), 式(14)的解可写为

$$w_s(z) = \Gamma(z) [w_p(L) - w_f(L)] + w_f(z) \quad (19)$$

式中: $\Gamma(z) = \frac{e^{\beta(z-H)} - e^{-\beta(z-H)}}{e^{\beta(L-H)} - e^{-\beta(L-H)}}$ 。

应指出的是, 在式(19)中 $w_p(L)$ 未知。

同样, 收集式(13)中的 δw_p 项, 可得桩位移函数 $w_p(z)$ ($0 \leq z \leq L$) 控制方程为

$$\frac{\partial^2 w_p}{\partial z^2} = \alpha^2 (w_p - w_f) \quad (20)$$

式中: $\alpha^2 = k / (E_p A_p + 2t)$ 。

考虑桩-土交界面应力连续, 收集 $\delta w_p|_{z=0}$ 和 $\delta w_p|_{z=L}$ 项, 桩顶和桩端边界条件为

$$(E_p A_p + 2t) \frac{\partial w_p}{\partial z} = 2t \frac{\partial w_f}{\partial z} - p A_p \quad z=0 \quad (21)$$

$$(E_p A_p + 2t) \frac{\partial w_p}{\partial z} = (E_s A_p + 2t) \frac{\partial w_s}{\partial z} \quad z=L \quad (22)$$

若桩端嵌固,则其边界条件为

$$w_p(L) = 0 \quad (23)$$

由上述边界条件求解式(20),则桩的位移为

$$w_p(z) = C_1 e^{\alpha z} + C_2 e^{-\alpha z} + w_f(z) \quad (24)$$

$$C_1 = \frac{\xi - \eta_2 \kappa}{\eta_1 - \eta_2}, C_2 = \frac{\xi - \eta_1 \kappa}{\eta_1 - \eta_2} \quad (25)$$

$$\eta_1 = e^{\alpha L} \left[1 - \frac{\alpha}{\beta} \frac{e^{\beta(L-H)} + e^{-\beta(L-H)}}{e^{\beta(L-H)} - e^{-\beta(L-H)}} \right] \quad (26)$$

$$\eta_2 = e^{\alpha L} \left[1 + \frac{\alpha}{\beta} \frac{e^{\beta(L-H)} + e^{-\beta(L-H)}}{e^{\beta(L-H)} - e^{-\beta(L-H)}} \right] \quad (27)$$

$$\xi = \frac{w_f'(L)}{\alpha} (\frac{\alpha^2}{\beta^2} - 1) \quad (28)$$

$$\kappa = \frac{\alpha}{k} [2t w_f'(0) - p A_p] - \frac{w_f'(0)}{\alpha} \quad (29)$$

对于桩端嵌固情况,系数 C_1 和 C_2 分别为

$$C_1 = \frac{\kappa e^{-\alpha L}}{e^{\alpha L} + e^{-\alpha L}}, C_2 = \frac{\kappa e^{\alpha L}}{e^{\alpha L} + e^{-\alpha L}} \quad (30)$$

3.3 衰减函数求解

收集式(13)中 $\delta\phi$ 项,考虑式(26)、(27),可得衰减方程 $\phi(r)$ 的控制方程为

$$\frac{\partial^2 \phi}{\partial r^2} + \frac{1}{r} \frac{\partial \phi}{\partial r} - (\frac{\gamma}{R})^2 \phi = 0 \quad (31)$$

由边界条件 $\phi(R) = 1$ 和 $\phi(\infty) = 0$,式(31)中的未知变量 γ 可定义为

$$(\frac{\gamma}{R})^2 = \frac{n_1 - 12n_2 + n_3}{m_1 - 2m_2 + m_3} \quad (32)$$

$$n_1 = 2\pi E_s^* \int_0^H (\frac{\partial w_f}{\partial z})^2 dz \quad (33)$$

$$n_2 = 2\pi E_s^* (\int_0^L \frac{\partial w_f}{\partial z} \frac{\partial w_p}{\partial z} dz + \int_L^H \frac{\partial w_f}{\partial z} \frac{\partial w_s}{\partial z} dz) \quad (34)$$

$$n_3 = 2\pi E_s^* [\int_0^L (\frac{\partial w_p}{\partial z})^2 dz + \int_L^H (\frac{\partial w_s}{\partial z})^2 dz] \quad (35)$$

$$m_1 = 2\pi \mu_s \int_0^H w_f^2 dz \quad (36)$$

$$m_2 = 2\pi \mu_s (\int_0^L w_f w_p dz + \int_L^H w_f w_s dz) \quad (37)$$

$$m_3 = 2\pi \mu_s (\int_0^L w_p^2 dz + \int_L^H w_s^2 dz) \quad (38)$$

显然,系数 n_1 和 n_2 满足

$$n_1 = 2\pi p w_f(0), n_2 = 2\pi p w_p(0) \quad (39)$$

式(31)为 Bessel 方程,其解为

$$\phi(r) = \frac{K_0(\gamma r/R)}{K_0(\gamma)} \quad (40)$$

式中: $K_0(\cdot)$ 为第零阶第二类变形 Bessel 函数。

4 迭代算法

由式(24)、(40)可见,位移函数 $w_p(z)$ 和 $w_s(z)$ 与衰减函数 $\phi(r)$ 完全耦合,无法直接求解。为得到未知函数 $w_p(z)$ 、 $w_s(z)$ 和 $\phi(r)$ 的解,文献[34]提出采用迭代方法计算。本文采用类似算法,具体步骤如下:

(1) 设定 γ 初始值,如 $\gamma = 1$ 。

(2) 由式(40)求解衰减函数 $\phi(r)$ 。

(3) 计算参数 k 和 $2t$,通过式(19)、(24)求解位移函数 $w_p(z)$ 和 $w_s(z)$,由式(9)求解 $w_f(z)$ 。

(4) 通过式(33)~(38)得到 $n_i (i=1,2,3)$ 和 $m_i (i=1,2,3)$,由式(32)计算新的 γ 值。

(5) 以第(4)步新 γ 值替换旧的 γ 值。

(6) 重复步骤(1)~(5),直到新旧 γ 值相差很小。

5 设计计算参数

5.1 桩身轴力

桩身轴力 $Q(z)$ 可由桩-土系统变分方法得到,即

$$Q(z) = -E_p A_p \frac{\partial w_p}{\partial z} - 2t \frac{\partial}{\partial z} (w_p - w_f) \quad (41)$$

5.2 中性面

中性面为桩与自由场位移之间的相对位移 $\Delta w(z)$ 趋近于 0 的位置,以 L_{np} 表示。 $\Delta w(z)$ 定义为 $\Delta w(z) = w_p(z) - w_f(z) = C_1 e^{\alpha z} + C_2 e^{-\alpha z}$ (42) L_{np} 可通过 $\Delta w(z) = 0$ 计算得到。

5.3 摩阻力

根据控制方程式(20),桩身摩阻力 $\tau(z)$ 可写为

$$\tau(z) = k [w_p(z) - w_f(z)] \quad (43)$$

$$k = 2\pi \mu_s \int_R^\infty \phi^2(r) dr \quad (44)$$

当相对位移 $\Delta w(z)$ 为负 ($z < L_{np}$) 时,产生负摩阻力。当 $z > L_{np}$ 时,出现方向向上的正摩阻力。

5.4 下拉力

下拉力 $P_n(z)$ 为桩身表面摩阻力之和,即

$$P_n(z) = \int_0^z \tau(z) dz \quad (45)$$

显然,下拉力最大值为 $P_n = P_n(L_{np})$ 。

5.5 土弹簧等效刚度

在工程中,常用 Winkler 弹簧来模拟桩与土之间相互作用。应用连续介质力学所得的桩身等效刚度 k_s 和桩顶等效刚度 k_t ,可转化得到 Winkler 模型中对应参数。在荷载传递法中,这两个刚度基于 Randolph 等^[17] 的成果得到。在本文中,桩身等效

刚度 k_s 可直接由式(46)得到。

$$k_s=k\frac{E_pA_p}{E_pA_p+2t}$$

(46)

由式(46)可知,桩侧等效弹簧刚度不仅和土相关,还和桩有关。应指出的是, k 和 $2t$ 不是常数,而是和土体参数及衰减函数 $\phi(z)$ 相关。然而,由于位移函数中包含 $w_f(z)$ 项,故很难推导出 Vallabhan 等^[31] 的桩顶刚度显式形式。

6 模型验证与工程案例

6.1 模型验证

图 2 为本文结果与已有方法计算的量纲一的桩身下拉力对比,其中 $P^*=P_n/[E_sLw_f(0)]$ 。图 3 为不同桩土模量比的量纲一的桩顶沉降量对比,其中 $w^*=w_p(0)E_p/(2\rho R)$ 。图 2 和图 3 表明,本文结果与已有方法计算结果吻合良好,这证明了本文方法的正确性。

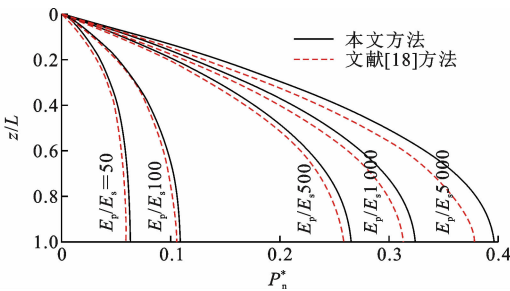


图 2 桩身下拉力对比

Fig. 2 Comparisons of down-drag force along pile shaft

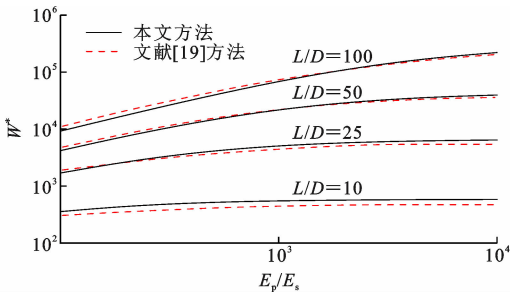


图 3 桩顶沉降对比

Fig. 3 Comparisons of settlement of pile head

6.2 工程应用

某桥梁钻孔灌注桩穿越沿海深厚软土地区^[36],桩长 83.68 m,桩径 1.5 m,堆载高度为 4 m,堆载为 84 kPa。地表以下有 24 m 淤泥质黏土,基桩穿越土层均为淤泥质黏土和粉质黏土,土泊松比为 $\nu_s=0.35$,土的物理力学参数如表 1 所示。

图 4 给出了按本文方法计算的桩身轴力结果和实测值对比。可见本文计算结果与实测值整体上吻

表 1 土的物理力学参数

Table 1 Physical and mechanical parameters of soil

土体	层底埋深 <i>d</i> /m	压缩模量 <i>E_s</i> /MPa	极限侧阻力标准值 <i>q_{sk}</i> /kPa
粉质黏土	3.36	6.50	30
淤泥质黏土	12.00	2.01	12
淤泥质黏土	26.00	2.28	12
黏土	34.80	3.07	20
粉质黏土	57.90	4.60	30
粉质黏土	70.20	4.75	35
粉质黏土	83.68	5.00	40

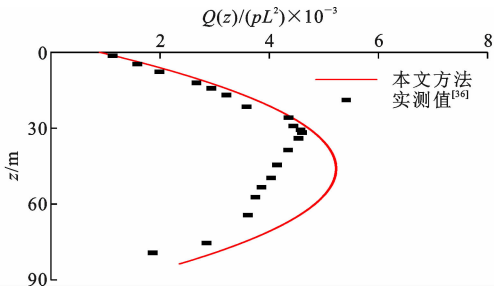


图 4 某工程桩基轴力对比

Fig. 4 Comparisons of axial forces of a practical pile

合较好,尤其是中性面以上部分,二者几乎一致。中性面以下本文计算结果与实测值有一定差异,这可能是由于本文方法是基于均匀地层的连续理论解,难以准确模拟下部非均匀土层情况。

7 参数分析

7.1 桩土模量比影响

不同桩土模量比 E_p/E_s 下,桩身轴力沿深度分布见图 5。可以看出,由于负摩擦阻力影响,桩身轴力随深度呈先增大后减小趋势,其最大值出现在中性面处。当桩土模量比较小时,桩身轴力沿深度变化不明显,这是由于此时桩土相对位移量很小,桩周土体对桩产生向下的拖曳力并不显著。随着桩土模量比增大,同一深度处桩身轴力明显增长,但增长速率变缓。值得一提的是,改变桩土模量比对中性面位置没有显著影响。

图 6 为不同桩土模量比作用下桩-土系统量纲一的位移 $\bar{w}/w_f(0)$ 等值云图。可以看出:当 $E_p/E_s=10$ 时,约 3 倍桩径范围内土体沉降变化最为显著,该范围外土体位移与自由场相当;当 $E_p/E_s=1\,000$ 时,受桩土相互作用影响的土体范围约为 5 倍桩径;地面以下 0.3 倍桩长范围内土体沉降最为显著,随埋深增加土的沉降值逐渐减小。

7.2 桩长细比影响

桩长细比 L/D 对桩侧摩阻力影响见图 7。可

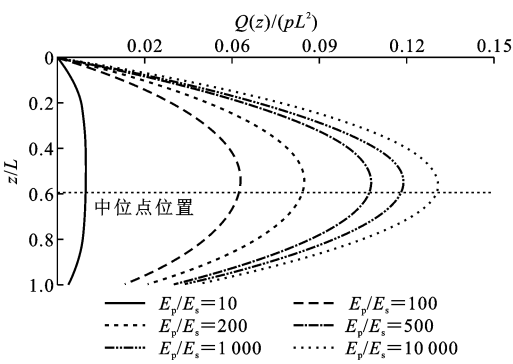


图 5 模量比不同时桩身轴力和深度的关系

Fig. 5 Relationship between axial force and depth of pile body under different modulus ratios

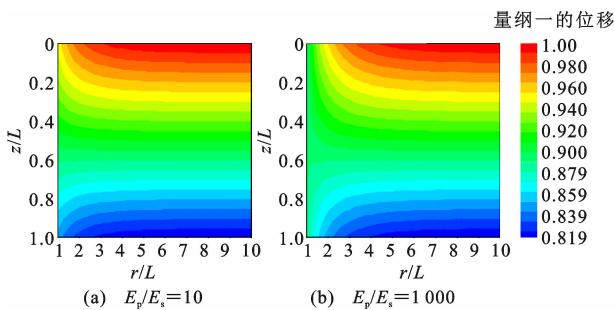


图 6 不同模量比时量纲一的位移等值云图

Fig. 6 Contour maps of nondimensional displacement with different modulus ratios

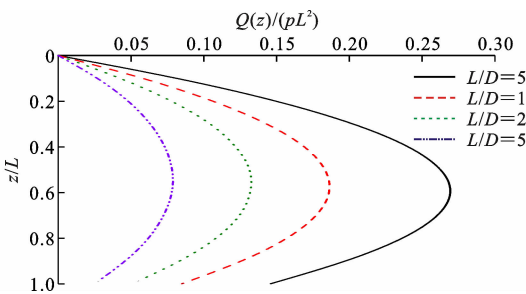


图 7 长细比不同时桩身轴力和深度的关系

Fig. 7 Relationship between axial force and depth of pile with different slenderness ratios

以看出,桩身轴力随长细比增加而减少。长细比小于 20 时,桩身轴力随长细比减小而急剧增长;当长细比大于 20 时,桩身轴力增长速率减缓。应指出,当长细比小于 20 时,中性面深度随着长细比的增大而急剧减小,如图 8 所示。

图 9 给出了不同长细比下桩-土系统量纲一的位移等值云图。从图 9(a)可以看出,地面超载情况下,桩周土体最大沉降出现在地表处,且地表以下 0.3 倍桩长范围内为位移强烈影响区。随着长细比增加,桩-土系统影响范围越大,这说明长细比对桩-土体系的位移场有显著影响,如图 9(b)所示。

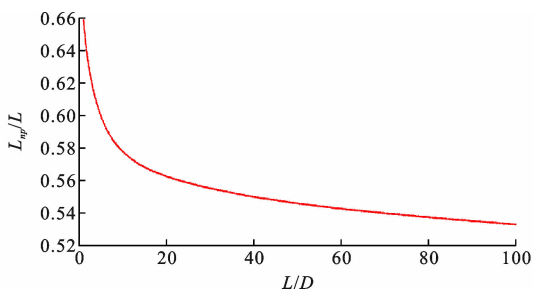


图 8 长细比和中性面深度的关系

Fig. 8 Relationship between neutral plane depth and slenderness ratio

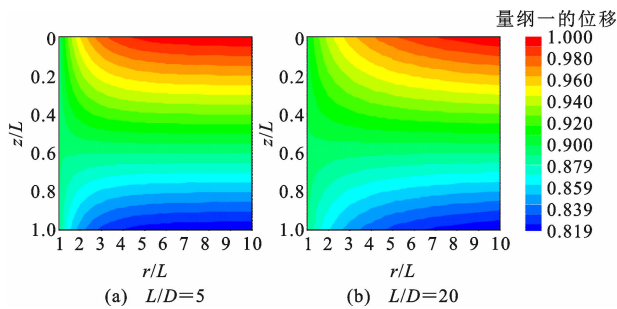


图 9 不同长细比时量纲一的位移等值云图

Fig. 9 Contours maps of nondimensional displacement with different slenderness ratios

8 结 语

- (1)当桩土模量比较小时,桩身轴力随桩土模量比增大而显著增加。
- (2)桩身轴力随桩长细比的增加而减少。当长细比小于 20 时,桩身轴力随长细比减小而急速增长。
- (3)中性面位置与长细比密切相关,而与桩土模量比关系不大。当长细比小于 20 时,中性面位置随长细比的增加而快速减小;当长细比大于 20 时,中性面位置随长细比的增加变化不明显。

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