

竖向入射地震 P 波激励下桩-土运动 相互作用理论研究

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摘要:考虑土体三维波动效应和桩-土运动相互作用,研究了竖向入射地震 P 波激励作用下端承单桩的地震响应问题。将竖向入射地震 P 波模拟为基岩处简谐纵向位移,同时考虑桩周土体纵向和径向位移,建立土体三维波动控制方程。将桩周土位移假设为自由场和散射场位移的叠加,推导得到地震作用下桩周土对桩的反力表达式。将桩假定为一维欧拉杆,然后将桩周土体反力表达式代入桩身控制方程,考虑桩-土连续条件及桩顶和桩底边界条件,求得了竖向入射地震 P 波激励下端承单桩地震响应解析解。将文中所得解与已有文献结果进行对比,验证了文中解的合理性。基于所得理论解,研究了桩-土主要参数对桩顶地震放大系数和运动响应因子、桩身位移和桩周土体地震反力以及桩周土体动力 Winkler 参数的影响。研究表明:地震 P 波作用下,桩-土体系中存在共振现象,且在土层 1 阶共振频率处共振最明显;桩长径比和桩-土模量比对桩顶地震放大系数和运动响应因子影响显著;桩身位移随着桩长径比减小或桩-土模量比增大而显著减小;与长桩和柔性桩相比,在地震激励作用下短桩和刚性桩受到的桩周土体地震反力更大;桩周土体动力 Winkler 参数受桩长径比的影响尤为突出,且随着桩长径比的增大而显著减小,而其受桩土模量比影响较小。研究成果可为桩基抗震分析和设计提供理论依据。

关键词:桩-土运动相互作用;三维波动效应;地震响应;地震 P 波;端承桩

中图分类号:TU473.1

文献标志码:A

Theoretical study on the kinematic soil-pile interaction under the excitation of vertically incident seismic P-wave excitation

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Abstract: Considering the 3D wave effect of soil and the kinematic soil-pile interaction, the seismic response of end-bearing single piles to vertically incident P-wave excitation is studied. The vertically incident P waves are modelled as the time-harmonic longitudinal displacement of the bedrock, and the governing equations of soil are established by considering both the longitudinal and radial displacements of the surrounding soil. The displacements of the surrounding soil are assumed as the summation of the free-field and scattered displacements, and subsequently the expression of the soil frictional force acting on the pile due to its motion is obtained. The pile is assumed to be a one-dimensional Euler bar. By substituting the soil frictional force into the governing equation of pile, the analytical solution of the seismic response of the pile under the action of vertically incident elastic P waves is obtained by considering the continuity conditions at the pile-soil interface and the boundary conditions at the top

收稿日期:2024-04-21; 修回日期:2024-07-08

基金项目:国家自然科学基金项目(52178318, 52178312);福建省自然科学基金项目(2021J011056)

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and bottom of the pile. The solution obtained is compared with existing studies to verify its validity. Finally, based on the obtained solutions, the effects of the main pile-soil parameters on the seismic amplification factor of the pile top, the kinematic response factor, the displacement of the pile, the frictional force of the soil and the kinematic Winkler parameters of the surrounding soil are investigated. The results show that: the resonance behavior of the pile-soil system occurs under the vertically incident P-waves, and the resonance behavior is particularly obvious at the first-order resonance frequency of soil layer. The pile slenderness ratio and pile-soil modulus ratio have significant effects on the seismic amplification factor of the pile top and kinematic response factor. The displacement of pile decreases significantly with the decrease of pile slenderness ratio or the increase of pile-soil modulus ratio. Compared with longer and softer piles, shorter and stiffer piles are subjected to a higher friction force of surrounding soil under seismic excitation. The influence of the pile slenderness ratio on the kinematic Winkler parameters is particularly prominent, and the kinematic Winkler parameters decrease significantly with the increase of the pile slenderness ratio, while the influence of the pile-soil modulus ratio is relatively small. The study can provide theoretical support for the seismic analysis and design of piles.

Key words: kinematic soil-pile interaction; 3D wave effect; seismic response; seismic P waves; end-bearing piles

0 引言

桩基广泛应用于地震活跃地区的高层建筑、大型桥梁、高速公路、高速铁路和风力发电机组等基础工程中^[1-7]。地震发生时,地震波在土体传递过程中遇到桩基会产生散射波,该散射波沿桩身产生附加的力和力矩,从而使桩基与周围土体的位移不一致。这种地震作用下桩-土运动相互作用对桩基抗震分析和设计具有重要影响^[8-13]。目前,针对地震波作用下桩-土耦合振动响应问题的研究大多集中于地震 S 波入射引起的桩-土相互作用问题^[14-17],然而,Kobe 地震和 Northridge 地震资料表明^[18],地震波中的 P 波分量同样可能会对桩基造成严重破坏。因此,对地震 P 波激励下桩基响应的研究同样具有重要意义。

近年来,国内外学者逐渐开始对 P 波作用下桩基地震响应进行研究,其研究方法主要有振动台模型试验法、数值分析法以及解析理论法。通过振动台模型试验,王建华等^[19]研究了地震作用下饱和砂土中孔压对桩基动力响应的影响,说明了饱和砂土中孔压增长会降低桩身内力;杨耀辉等^[20-21]研究了排水刚性单桩、群桩在地震作用下地基土体的超孔压响应、加速度响应以及桩顶的侧向永久位移等动力响应特性;何静斌等^[22]研究了强震区桩-土-断层耦合作用下桩基动力响应特性,分析了地震作用下桩身加速度、桩顶相对位移及桩身弯矩的动力响应规律;吴琪等^[23]、汪刚等^[24]分别研究了珊瑚礁砂、中硬黏土地基中桩-土-结构在地震作用下的运动相互作用特征;田兆阳等^[25]研究了强震作用下软土地基中桩基负摩阻力的产生机理及分布特性,证明了竖向地震作用下全桩均会产生负摩阻力;张聪等^[26]研究了地震作用下软弱土层差异厚度对单桩动力响应的影响,证明了桩周土体的约束作用受软弱土层厚度的影响显著。然而,振动台模型试验成本较高,并且模型比例无法做到 1:1,对于涉及动荷载、力学参数、土体与工程结构的相互作用这类复杂实际工况,很难满足模型与研究对象复杂的物理力学相似关系,可能对试验结果的准确性产生一定的影响。数值分析方法主要有有限元法和边界元法。基于有限元法,肖晓春等^[27]建立了地震作用下桩-土-结构相互作用的计算模型并总结了其分析方法;沈婷等^[28]研究了地震作用下超深桩基础的动力响应;罗川等^[29]研究了地震激励下桩-土体系的水平和竖向动力响应;朱升冬等^[30]考虑土体非线性研究了核岛结构桩基础三维地震反应特性;JIA 等^[31]研究了液化土和非液化土中桩基地震响应的差异及其对数值模型参数的影响,证明了可液化土和不可液化土的桩-土体系地震响应存在显著差异。MAMOON 等^[32]、JI 等^[33]基于边界元法,研究了 P 波作用下均质土中桩的动力响应。然而,由于地震作用下三维动态数值模型建模复杂且计算耗时长,模型某些相关参数取值具有不确定性,难以对实际工况进行完美的模拟,使其在实际工程中较少被采用。

对于解析法,目前应用最广泛的解析模型是基于动力 Winkler (BDWF) 模型^[34-37],其通过弹簧和阻尼来模拟地震作用下桩周土体对桩身的作用。动力 Winkler 模型的缺陷在于其不能考虑土体波动效应,无法真正考虑地震激励下桩-土耦合振动,并且其模型参数取值往往需要通过经验公式确定,在理论上存在一定程度的简化。LIU 等^[38]基于改进 Vlasov 模型,研究了均质土中 P 波作用下单桩的地震响应。KE 等^[39]基于

LIU 等^[38]的研究,研究了 P 波作用下多层土中单桩的动力响应。然而,基于改进 Vlasov 模型的求解过程中往往涉及迭代计算,使求解过程复杂繁琐。连续介质模型将土体视为三维连续介质,严格考虑地震作用下桩-土的耦合振动,理论上更加严密。ANOYATIS 等^[40]基于连续介质理论方法,忽略地震作用下土体径向变形,研究了桩基竖向地震响应。

本文将桩周土假设为均匀黏弹性体,基于三维连续介质理论,同时考虑桩周土纵向和径向位移建立土体三维波动方程,得到了桩底基岩处竖向入射地震 P 波激励作用下端承单桩动响应解析解。将本文所得解与已有文献简化解和有限元结果进行对比,验证了本文解的正确性,并对所得解进行参数化分析,得到了桩-土体系主要参数对地震 P 波作用下端承单桩地震响应的影响。

1 计算模型及基本假定

本文计算模型见图 1,桩长为 H 、半径为 r_0 的端承单桩埋入下卧刚性基岩的均质黏弹性土层中。基岩处施加时谐纵向位移 $u_g e^{i\omega t}$ 以模拟垂直入射的地震 P 波^[40], $i = \sqrt{-1}$, u_g 为基岩在地震波作用下产生的纵向位移幅值, ω 为地震波圆频率, t 为时间。将桩视为线弹性一维欧拉杆,假定桩-土接触面处位移和应力连续^[35,40]。为简便起见,在下文推导中省略时间分量 $e^{i\omega t}$ 项。

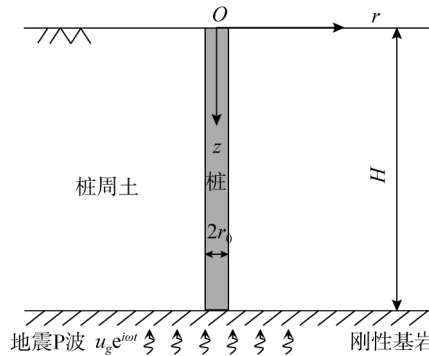


图 1 计算模型示意图

Fig. 1 Schematic diagram of the computational model

1.1 控制方程的建立

根据弹性动力学理论,建立轴对称柱坐标系下的土体三维波动控制方程:

$$G_s^* (\nabla^2 - \frac{1}{r^2}) u_r + (\lambda_s^* + G_s^*) \frac{\partial e}{\partial r} = -\rho_s \omega^2 u_r \tag{1}$$

$$G_s^* \nabla^2 u_z + (\lambda_s^* + G_s^*) \frac{\partial e}{\partial z} = -\rho_s \omega^2 u_z \tag{2}$$

式中: $\nabla^2 = \frac{\partial^2}{\partial r^2} + \frac{1}{r} \frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2}$; $u_r(r, z)$ 、 $u_z(r, z)$ 为土体径向、纵向位移; $e = \frac{\partial u_r}{\partial r} + \frac{1}{r} u_r + \frac{\partial u_z}{\partial z}$ 为土体应变; λ_s^* 和 G_s^* 为土体复拉梅常数, $\lambda_s^* = \lambda_s (1 + 2i\beta_s)$, $G_s^* = G_s (1 + 2i\beta_s)$; β_s 为土体滞回阻尼比; ρ_s 为土体密度; z 为土体沿深度方向距离桩顶的距离。

桩的纵向位移 $u_p(z, t)$ 可表示为

$$E_p A_p \frac{d^2 u_p}{dz^2} + f(z) = -\rho_p A_p \omega^2 u_p \tag{3}$$

式中: E_p 、 A_p 、 ρ_p 分别为桩身弹性模量、横截面面积和密度; $f(z)$ 为地震作用下单位长度桩周土对桩作用的竖向反力。需要指出的是,地震作用下桩周土对桩的作用力与桩顶受激振荷载情况是相反的。地震激励作用下,桩和土均受到底部地震激励,由于桩的刚度大于土,桩周土对桩的作用力与桩的运动方向相同,是一种拖拽力;而桩顶受激振荷载时,动荷载由桩身向桩周土传递,桩周土对桩的作用力与桩的运动方向相反,是一种摩阻力。

1.2 自由场位移求解

土体自由场位移 $u_f(z)$ 可表示为

$$(\lambda_s^* + 2G_s^*) \frac{d^2 u_f}{dz^2} = -\rho_s \omega^2 u_f \quad (4)$$

竖向入射地震 P 波可用刚性基岩的简谐运动表示为^[40]

$$u_f(H) = u_g \quad (5)$$

结合边界条件(5)和土体表面自由边界 $\left. \frac{\partial u_f}{\partial z} \right|_{z=0} = 0$, 代入方程(4), 可得自由场土体位移为

$$u_f(z) = u_g \frac{\cos(\chi_s z)}{\cos(\chi_s H)} \quad (6)$$

式中: $\chi_s = \frac{\rho_s \omega^2}{\lambda_s^* + 2G_s^*}$.

1.3 土体位移求解

竖向入射的地震 P 波由于桩的存在会在土体中产生散射波, 因此, 土体运动可表示为自由场运动和土体散射运动的叠加^[14,40], 如式(7)所示:

$$u_r = u_r^s, \quad u_z = u_z^s + u_f(z) \quad (7)$$

引入势函数对土体控制方程进行解耦:

$$u_r^s(r, z) = \frac{\partial \varphi(r, z)}{\partial r} + \frac{\partial^2 \psi(r, z)}{\partial z \partial r} \quad (8)$$

$$u_z^s(r, z) = \frac{\partial \varphi(r, z)}{\partial z} - \frac{\partial^2 \psi(r, z)}{\partial r^2} - \frac{1}{r} \frac{\partial \psi(r, z)}{\partial r} \quad (9)$$

将势函数代入到方程(1)和方程(2), 可得:

$$\nabla^2 \varphi + \frac{\rho_s \omega^2}{\lambda_s^* + 2G_s^*} \varphi = 0 \quad (10)$$

$$\nabla^2 \psi + \frac{\rho_s \omega^2}{G_s^*} \psi = 0 \quad (11)$$

式中, φ 和 ψ 分别为代表土体中 P 波和 S 波传播的势函数。方程(10)和方程(11)的通解为

$$\varphi = [A_1 K_0(qr) + B_1 I_0(qr)] [C_1 \sin(gz) + D_1 \cos(gz)] \quad (12)$$

$$\psi = [A_2 K_0(pr) + B_2 I_0(pr)] [C_2 \sin(gz) + D_2 \cos(gz)] \quad (13)$$

式中: $q = \sqrt{g^2 - \frac{\rho_s \omega^2}{\lambda_s^* + 2G_s^*}}$, $p = \sqrt{g^2 - \frac{\rho_s \omega^2}{G_s^*}}$, q 和 p 分别为土体中 P 波和 S 波沿径向传播的复波数, g 为可以由边界条件求得的变量, $I_0(\cdot)$ 和 $K_0(\cdot)$ 分别表示第一类和第二类零阶修正贝塞尔函数, $A_1, B_1, C_1, D_1, A_2, B_2, C_2, D_2$ 为待定系数。

桩周土体的边界条件如式(14)~式(17)所示:

$$\sigma_z \Big|_{z=0} = 0 \quad (14)$$

$$u_z \Big|_{z=H} = u_g \quad (15)$$

$$u_r \Big|_{r \rightarrow \infty} = 0, \quad u_z \Big|_{r \rightarrow \infty} = u_f \quad (16)$$

$$u_r \Big|_{r=r_0} = 0 \quad (17)$$

将势函数式(12)和式(13)代入桩周土体的边界条件式(14)~式(17), 求解可得 $B_1 = D_1 = 0, B_2 = C_2 = 0$, 故势函数最终表达式可表示为

$$\varphi = \sum_{n=1}^{\infty} A_n K_0(q_n r) \sin(g_n z) \quad (18)$$

$$\psi = \sum_{n=1}^{\infty} B_n K_0(p_n r) \cos(g_n z) \quad (19)$$

式中: $g_n = \frac{(2n-1)\pi}{2H}$ ($n=1, 2, 3, \dots$); $q_n = \sqrt{g_n^2 - \frac{\rho_s \omega^2}{\lambda_s^* + 2G_s^*}}$; $p_n = \sqrt{g_n^2 - \frac{\rho_s \omega^2}{G_s^*}}$; A_n, B_n 为待定系数。

因此,桩周土体位移可表示为

$$u_r = \sum_{n=1}^{\infty} [B_n g_n p_n K_1(p_n r) - A_n q_n K_1(q_n r)] \sin(g_n z) \tag{20}$$

$$u_z = \sum_{n=1}^{\infty} [A_n g_n K_0(q_n r) - B_n p_n^2 K_0(p_n r)] \cos(g_n z) + u_f(z) \tag{21}$$

将式(20)代入边界条件式(17),可得:

$$A_n = \frac{g_n p_n K_1(p_n r_0)}{q_n K_1(q_n r_0)} B_n \tag{22}$$

桩-土交界面处土体纵向位移可表示为

$$u_z |_{r=r_0} = \sum_{n=1}^{\infty} B_n V_n \cos(g_n z) + u_f(z) \tag{23}$$

式中: $V_n = \frac{p_n g_n^2 K_1(p_n r_0) K_0(q_n r_0)}{q_n K_1(q_n r_0)} - p_n^2 K_0(p_n r_0)$ 。

桩-土交界面处桩周土体对桩的竖向地震反力可表示为

$$f(z) = \pi d \tau_{rz} |_{r=r_0} = - \sum_{n=1}^{\infty} B_n S_n \cos(g_n z) \tag{24}$$

式中: $S_n = \pi d G_s^* p_n (p_n^2 - g_n^2) K_1(p_n r_0)$, $\tau_{rz} = G_s^* \left(\frac{\partial u_r}{\partial z} + \frac{\partial u_z}{\partial r} \right)$ 为桩周土体剪应力。

1.4 单桩地震响应求解

将桩周土竖向地震反力表达式代入桩的控制方程可得:

$$E_p A_p \frac{\partial^2 u_p}{\partial z^2} + \rho_p A_p \omega^2 u_p = \sum_{n=1}^{\infty} B_n S_n \cos(g_n z) \tag{25}$$

方程(25)的解为

$$u_p = N_1 \sin(\chi_p z) + N_2 \cos(\chi_p z) + \sum_{n=1}^{\infty} U_n B_n \cos(g_n z) \tag{26}$$

式中: $\chi_p = \sqrt{\frac{\rho_p \omega^2}{E_p}}$, $U_n = \frac{S_n}{E_p A_p (\chi_p^2 - g_n^2)}$ 。

桩-土界面处的位移连续:

$$u_z |_{r=r_0} = u_p \tag{27}$$

将式(23)和式(26)代入边界条件式(27)可得:

$$N_1 \sin(\chi_p z) + N_2 \cos(\chi_p z) + \sum_{n=1}^{\infty} U_n B_n \cos(g_n z) = \sum_{n=1}^{\infty} B_n V_n \cos(g_n z) + u_f(z) \tag{28}$$

将方程(28)两边同时乘以 $\cos(g_n z)$,然后在区间 $[0, H]$ 上积分可得:

$$B_n = \kappa_{1n} N_1 + \kappa_{2n} N_2 - \kappa_{3n} \tag{29}$$

式中:

$$\kappa_{1n} = \frac{2 \int_0^H \sin(\chi_p z) \cos(g_n z) dz}{H [V_n - U_n]} \tag{30}$$

$$\kappa_{2n} = \frac{2 \int_0^H \cos(\chi_p z) \cos(g_n z) dz}{H [V_n - U_n]} \tag{31}$$

$$\kappa_{3n} = \frac{2 \int_0^H u_f(z) \cos(g_n z) dz}{H [V_n - U_n]} \tag{32}$$

则式(26)中桩身竖向位移可写为如式(33)最终表达式:

$$u_p = N_1 \left[\sin(\chi_p z) + \sum_{n=1}^{\infty} \kappa_{1n} U_n \cos(g_n z) \right] + N_2 \left[\cos(\chi_p z) + \sum_{n=1}^{\infty} \kappa_{2n} U_n \cos(g_n z) \right] - \sum_{n=1}^{\infty} \kappa_{3n} U_n \cos(g_n z) \tag{33}$$

考虑桩顶和桩底边界条件如式(34)、式(35)所示:

$$\frac{\partial u_p}{\partial z} \Big|_{z=0} = 0 \quad (34)$$

$$u_p \Big|_{z=H} = u_g \quad (35)$$

将式(33)代入边界条件式(34)和式(35),可得:

$$N_1 = 0 \quad (36)$$

$$N_2 = \frac{u_g + \sum_{n=1}^{\infty} \kappa_{3n} U_n \cos(g_n H)}{\cos(\chi_p H) + \sum_{n=1}^{\infty} \kappa_{2n} U_n \cos(g_n H)} \quad (37)$$

将系数 N_1 和 N_2 代入方程(33),即可得到端承单桩竖向位移的最终解析表达式。

将桩顶地震放大系数 A 定义为桩顶竖向位移与基岩位移幅值之比,即

$$A = \frac{u_p(0)}{u_g} \quad (38)$$

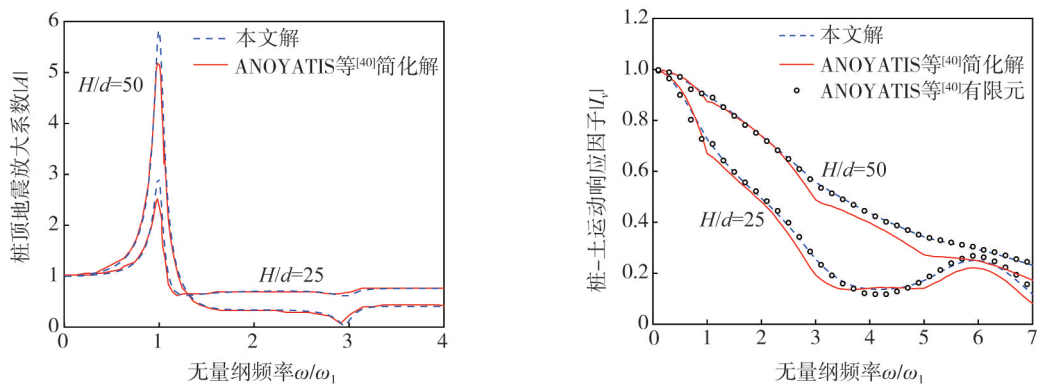
将桩-土运动响应因子 I_v 定义为桩顶竖向位移与自由场表面位移之比,即

$$I_v = \frac{u_p(0)}{u_f(0)} \quad (39)$$

2 对比验证与参数分析

2.1 与已有文献简化解和有限元结果对比

将本文解与 ANOYATIS 等^[40] 简化解和有限元结果进行对比,以验证本文解的合理性,对比结果见图2。对比中采用无量纲参数: $E_p/E_s = 100$ (E_s 为土体弹性模量), $\rho_s/\rho_p = 0.7$, 土体泊松比 $\nu_s = 0.4$, $\beta_s = 0.05$, $H/d = 25$ 或 50 (d 为端承桩直径), 无量纲频率 ω/ω_1 , ω_1 为土层 1 阶共振圆频率,具体分析见下文。由图2可知,相比 ANOYATIS 等^[40] 简化解,本文解与有限元结果更为吻合,这是因为 ANOYATIS 等^[40] 忽略了桩周土体径向位移而本文解严格考虑了土体三维波动效应。图2 本文解与 ANOYATIS 等^[40] 简化解的对比也说明忽略土体径向位移会低估桩基地震响应,导致桩基抗震设计偏于不安全。



(a) 本文解与 ANOYATIS 等^[40] 简化解对比曲线

(b) 本文解与 ANOYATIS 等^[40] 有限元结果对比曲线

图2 本文解与 ANOYATIS 等^[40] 简化解和有限元结果对比验证

Fig. 2 Comparison and verification among the solution in this paper, the simplified solution by ANOYATIS et al.^[40] and the finite element results

2.2 桩顶地震放大系数和运动响应因子

对于桩-土体系,本节主要分析主要桩-土参数:桩-土模量比和桩长径比对端承单桩地震响应的影响。计算中如无其他说明采用参数是: $H/d = 20$, $E_p/E_s = 1000$, $\rho_s/\rho_p = 0.7$, $\nu_s = 0.3$, $\beta_s = 0.05$ ^[16,40]。

不同桩-土模量比和桩长径比的桩顶地震放大系数 A 和运动响应因子 I_v 变化规律见图3、图4。由图可知,地震放大系数和运动响应因子在土层共振频率处存在共振现象。根据式(21)中给出的土体纵向位移可

得到,当 $q_n = 0$ 时修正贝塞尔函数 $K_0(q_n r)$ 趋近于无穷大,此时土层发生共振,而根据 $q_n = 0$ 可得到土层发生共振时的圆频率 ω_n 为

$$\omega_n = V_l^* g_n = V_l^* \frac{(2n - 1)\pi}{2H} \tag{40}$$

式中: $V_l^* = \sqrt{\frac{\lambda_s^* + 2G_s^*}{\rho_s}}$ 为地震 P 波在土体中传播的复波速。其中共振最显著的 1 阶共振圆频率 ω_1 为

$$\omega_1 = \frac{\pi V_l^*}{2H} \tag{41}$$

桩顶地震放大系数和运动响应因子在初始频率处均等于 1,随着地震波频率增大,桩顶地震放大系数逐渐增大,并在桩-土体系 1 阶共振频率处激烈震荡达到峰值,在高阶共振频率处同样会产生震荡,但共振幅值远小于 1 阶共振频率处。地震激励频率较低时,运动响应因子小于 1,这说明由于桩-土运动相互作用使端承单桩地震响应小于自由场响应,运动响应因子随着地震波频率增大也会激烈震荡,并且在桩-土体系共振频率处达到其极小值。在进行桩基抗震设计时,应注意桩-土体系 1 阶共振频率,否则容易导致桩-土体系在地震作用下产生共振,影响桩基及上部结构的抗震安全。

不同桩-土模量比时端承单桩桩顶地震放大系数和运动响应因子随无量纲频率的变化规律见图 3。对于桩顶地震放大系数,桩-土模量比对其共振幅值有显著影响,桩-土模量比越大,桩顶地震放大系数的共振幅值越小。这说明实际工程中采用刚性桩可以削弱地震发生时基岩运动的放大效应,有效地提高结构体系的抗震性能。此外,桩-土模量比对运动响应因子也有显著影响,地震波频率小于 1 阶共振频率时($\omega/\omega_1 < 1$),运动响应因子随桩-土模量比的增大而减小,这说明地震 P 波作用下,刚性桩的桩-土运动相互作用比柔性桩更显著,可以更有效地滤除到达桩顶的地震波能量。地震波频率大于 1 阶共振频率时,运动响应因子随桩-土模量比的增大而大幅增大,其震荡现象也更加剧烈。这说明在小于 1 阶共振频率时,刚性桩传递到上部结构的地震能量比柔性桩小,此时刚性桩对抗震有利;而在大于 1 阶共振频率时,柔性桩传递到上部结构的地震能量比刚性桩小,此时柔性桩对抗震有利。

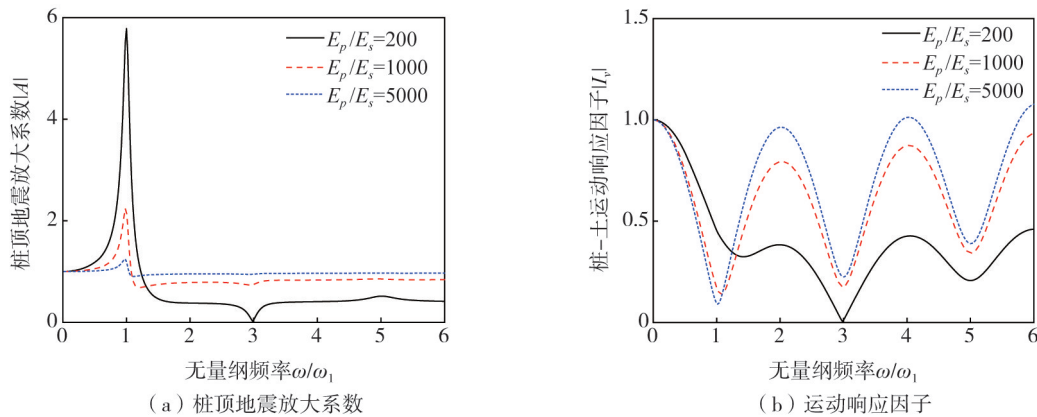


图 3 桩-土模量比对桩顶地震放大系数和运动响应因子的影响

Fig. 3 Influence of the pile-soil modulus ratio on the seismic amplification factor of the pile top and the kinematic response factor

不同桩长径比时端承单桩桩顶地震放大系数和运动响应因子随无量纲频率的变化规律见图 4。由图可知,桩顶地震放大系数的 1 阶共振频率处共振幅值随桩长径比的增大而显著增大。显然,桩长径比越大,土层越厚,从而导致在 1 阶共振频率处土层对基岩运动的放大效应变强。地震波频率小于 1 阶共振频率时,运动响应因子随桩长径比增大而增大,这是由于对于端承桩而言,桩长越长,桩体越柔,使长细桩对桩-土运动响应因子的影响表现出与柔性桩类似的变化规律。地震波频率大于 1 阶共振频率时,桩-土运动相互作用更加复杂,运动响应因子随桩长径比的增大而大幅减小,其震荡现象也趋于缓和。类似地,在小于 1 阶共振频率时,短桩传递到上部结构的地震能量比长桩小,此时短桩对抗震有利;而在大于 1 阶共振频率时,长桩传递到上部结构的地震能量比短桩小,此时长桩对抗震有利。

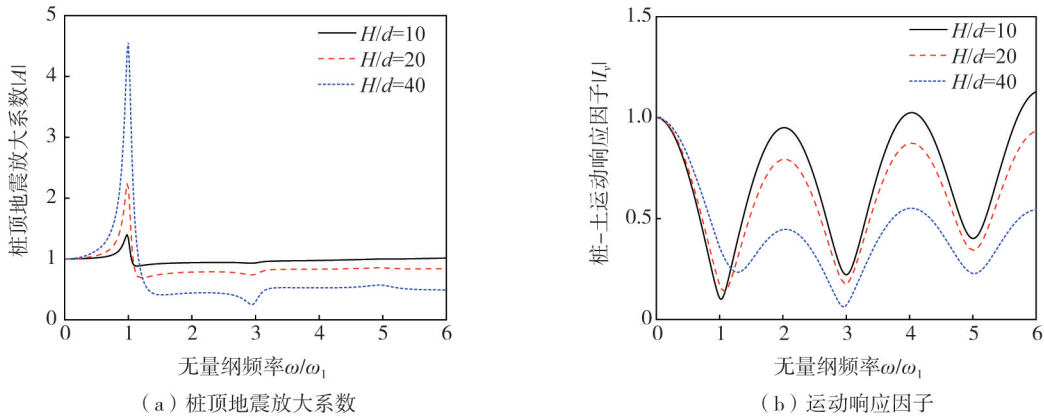


图4 桩长径比对桩顶地震放大系数和运动响应因子的影响
 Fig. 4 Influence of the pile slenderness ratio on the seismic amplification factor of the pile top and the kinematic response factor

2.3 桩身位移和桩周土体地震反力

桩周土体地震反力是反映地震波激励下桩-土运动相互作用的重要指标,本节研究了桩-土体系共振最显著的土层一阶共振频率处(无量纲频率 $\omega/\omega_1 = 1$)不同桩-土模量比和桩长径比下桩身位移和桩周土体沿深度的变化情况。

地震波无量纲频率 ω/ω_1 为1时桩-土模量比对桩身位移和桩周土体地震反力的影响见图5。由图可知,桩身位移随桩-土模量比增大而减小,桩周土体地震反力随桩-土模量比增大而增大。与柔性桩相比,刚性桩位移与自由场差别更大(图3),其桩-土运动相互作用更加剧烈,因此虽然刚性桩桩身位移比柔性桩小,其桩周土体地震反力却更大。

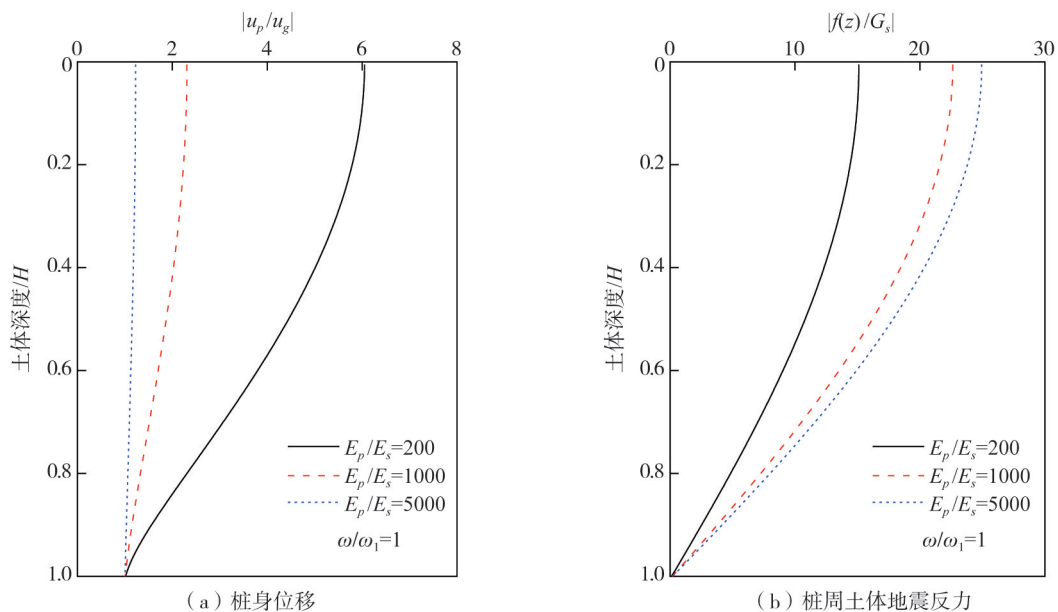


图5 桩-土模量比对桩身位移和桩周土体地震反力的影响($\omega/\omega_1 = 1$)

Fig. 5 Influence of the pile-soil modulus ratio on the displacement of pile and the frictional force of soil at $\omega/\omega_1 = 1$

地震波无量纲频率 ω/ω_1 为1时不同桩长径比对桩身位移和桩周土体地震反力的影响见图6。由图可知,桩身位移随桩长径比增大而显著增大,而桩周土体地震反力随桩长径比增大反而显著减小。如前所述,土层增大会加剧基岩运动的放大效应,因此长径比越大桩身位移越大;短桩位移相比长桩虽然绝对值更小,其与自由场差别却更大(图4),其桩-土运动相互作用更加剧烈,因此其桩周土体地震反力更大。

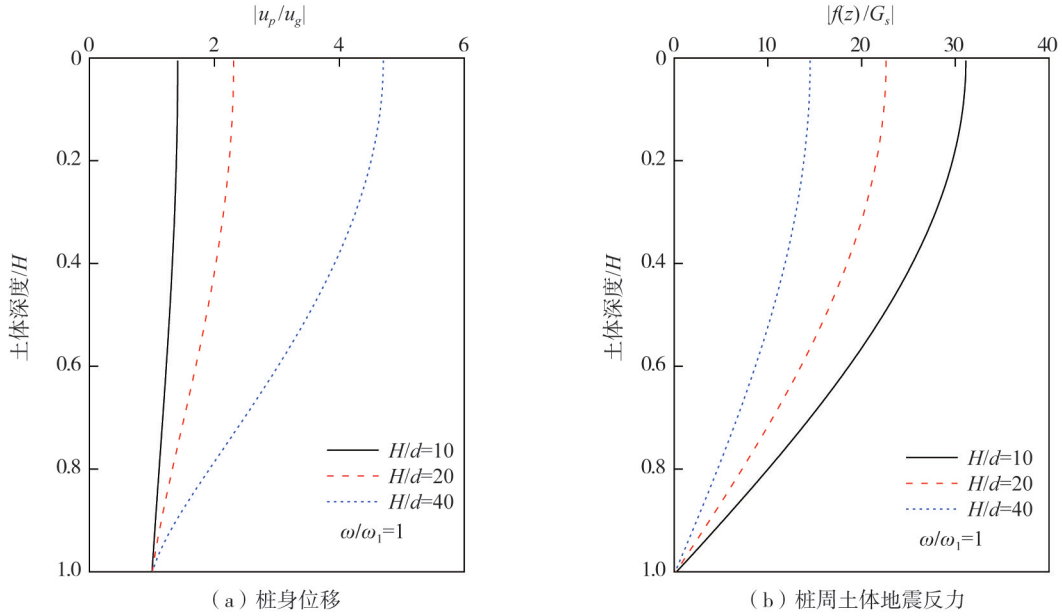


图 6 桩长径比对桩身位移和桩周土体地震反力的影响 ($\omega/\omega_1 = 1$)

Fig. 6 Influence of the pile slenderness ratio on the displacement of pile and the frictional force of soil at $\omega/\omega_1 = 1$

2.4 桩周土体动力 Winkler 参数

根据本文推导得到的竖向入射地震 P 波激励下端承单桩地震响应解析解,求得桩周土体动力 Winkler 参数闭合表达式,其定义为桩-土交界面处土体对桩的地震反力与桩土相对位移(桩身位移和自由场位移的差)的比值^[40],如式(42)所示:

$$K(z, \omega) = \frac{f(z)}{u_p(z) - u_f(z)} = - \frac{\sum_{n=1}^{\infty} B_n S_n \cos(g_n z)}{\sum_{n=1}^{\infty} B_n V_n \cos(g_n z)} \tag{42}$$

式中,桩周土体动力 Winkler 参数还可以用复值形式来表示 $K(z, \omega) = k(1 + 2ic)$, $K(z, \omega)$ 的实部 k 和虚部 c 分别表示 Winkler 模型的弹簧刚度和粘壶阻尼系数^[35]。

地震波无量纲频率 ω/ω_1 为 1 时,桩-土模量比对桩周土体动力 Winkler 参数的影响,可以发现,桩-土模量比对动力 Winkler 模型的弹簧刚度和粘壶阻尼系数几乎没有影响,见图 7。

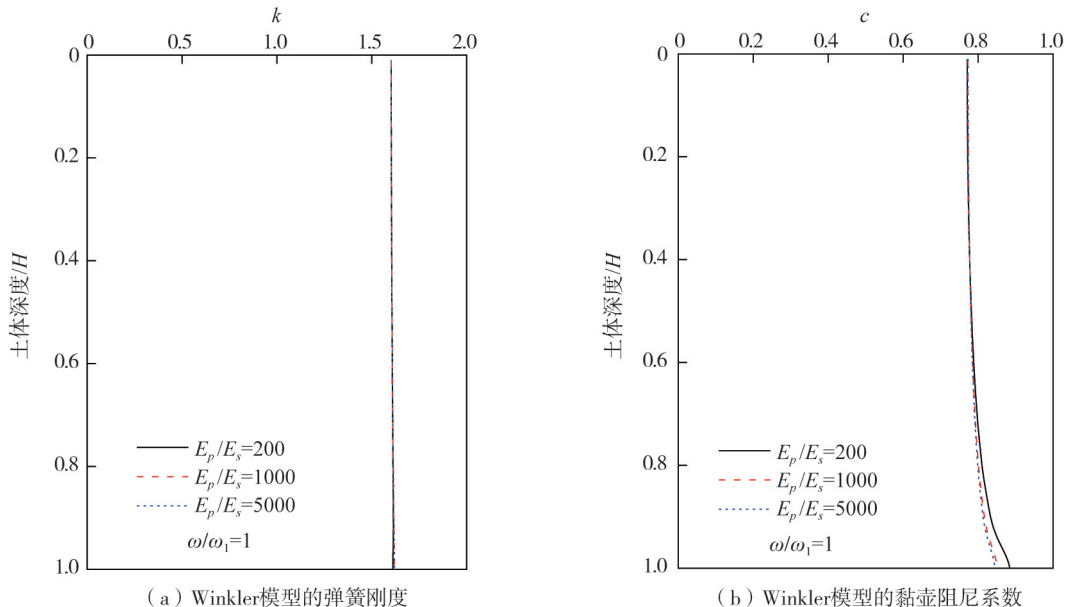


图 7 桩-土模量比对 Winkler 模型的弹簧刚度和黏壶阻尼系数的影响 ($\omega/\omega_1 = 1$)

Fig. 7 Influence of the pile-soil modulus ratio on the Winkler model of spring stiffness and damping coefficient of the dashpot at $\omega/\omega_1 = 1$

地震波无量纲频率 ω/ω_1 为 1 时, 桩长径比对桩周土体动力 Winkler 参数的影响见图 8。由图可知, 桩长径比对桩周土体动力 Winkler 参数有显著影响, 且动力 Winkler 模型的弹簧刚度和黏壶阻尼系数均随桩长径比增大而显著减小。

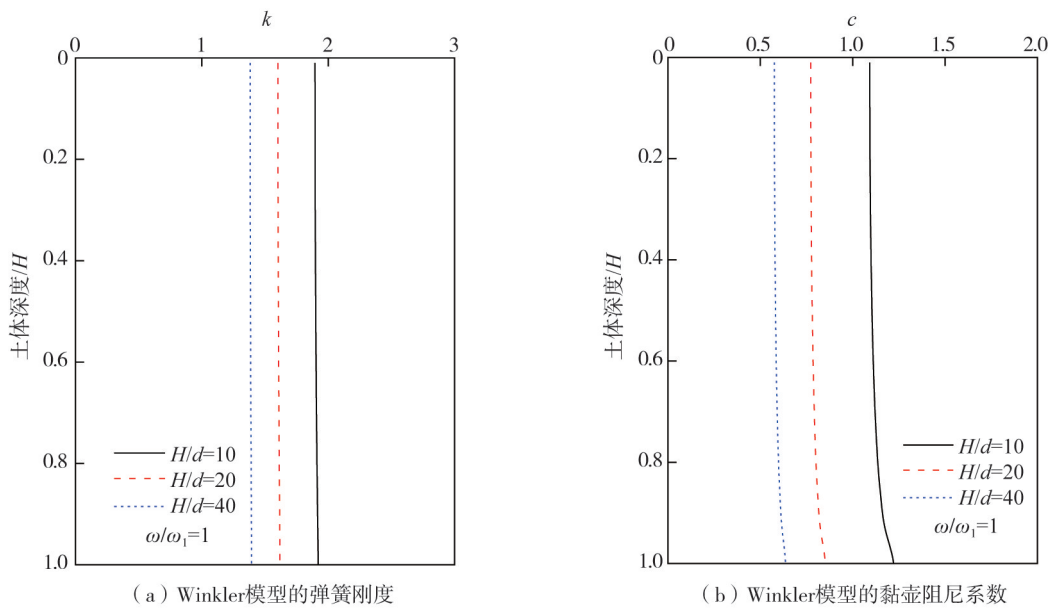


图 8 桩长径对 Winkler 模型的弹簧刚度和黏壶阻尼系数的影响 ($\omega/\omega_1=1$)

Fig. 8 Influence of the pile slenderness ratio on the Winkler model of spring stiffness and damping coefficient of the dashpot at $\omega/\omega_1=1$

3 结论

本文在考虑土体三维波动效应和桩-土运动相互作用的基础上, 推导了竖向入射地震 P 波激励作用下端承单桩地震响应解析解, 通过与已有文献简化解及有限元结果对比验证了本文解的正确性。基于所得解进行端承单桩地震响应及桩-土运动相互作用机理进行分析, 得出如下结论:

1) 桩底基岩处地震 P 波激励作用下桩-土体系在土层共振频率处存在共振现象, 并且在土层 1 阶共振频率处共振最明显, 需要在抗震设计中考虑这一现象。

2) 桩长径比和桩-土模量比对桩顶地震放大系数的共振幅值和运动响应因子影响显著, 且桩顶地震放大系数共振幅值随桩长径比增大或桩-土模量比减小而增大, 地震波频率小于 1 阶共振频率时, 运动响应因子随桩长径比增大或桩-土模量比减小而增大, 而地震波频率大于 1 阶共振频率时, 运动响应因子随桩长径比增大或桩土模量比减小反而大幅减小, 其震荡现象也趋于缓和。

3) 桩身位移随着桩长径比减小或桩-土模量比增大而显著减小, 桩周土体地震反力随着桩长径比减小或桩-土模量比增大而增大。与长桩和柔性桩相比, 地震激励作用下短桩和刚性桩的桩-土运动相互作用更加剧烈, 其桩周土体地震反力更大。

4) 桩周土体动力 Winkler 模型的弹簧刚度和黏壶阻尼系数受桩长径比的影响尤为突出, 且均随桩长径比的增大而显著减小, 而其受桩-土模量比的影响较小。

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