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基于上下限解的膨胀土边坡首次滑动区域分析

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摘要: 受膨胀土胀缩性、裂隙性及超固结性的影响, 膨胀土滑坡灾害易发、频发, 而传统的计算分析方法难以反映出膨胀土边坡“渐进性”失稳破坏的特点, 并有效估算其首次滑动的区域。为揭示膨胀土边坡的成灾特性, 量化膨胀土边坡的阶段性成灾规模, 基于膨胀土吸水膨胀变形而引起部分土条底切向力方向可能改变的特性, 根据 Morgenstern-Price 静力平衡分析法, 严格地导出了膨胀土边坡滑动趋势临界点的位置, 分析了边坡首次滑动区域的上限解与下限解, 并以某工程为例进行了计算说明, 同时分析了抗剪强度对上下限解的影响。结果表明: 受膨胀作用影响, 按经典分析方法判定为稳定的边坡仍有可能发生局部失稳渐进破坏, 其首次滑动区域的上限解受有效抗剪强度的影响较小, 而残余抗剪强度对下限解的影响尤为显著; 上限解及其变化速率随有效黏聚力与有效内摩擦角的增大而减小, 下限解及其变化速率则随残余内摩擦角与残余黏聚力的增加而增加; 同时, 可基于上下限解的大小关系, 判定膨胀土边坡的稳定性。研究成果可为膨胀土滑坡的成灾规模评价及治理工程设计提供参考。

关键词: 膨胀土边坡; 首次滑动区域; 边坡稳定性; 上下限解; Morgenstern-Price 法

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First sliding area analysis of expansive soil slope based on the upper and lower bound solutions

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Abstract: Expansive soil landslides are prone to frequent disasters due to the expansion, cracking, and over-consolidation characteristics of expansive soil. However, traditional analysis methods cannot adequately reflect the progressive failure characteristic of expansive soil slopes, and effectively estimate the first sliding area. To better understand the disaster characteristics of expansive soil slopes and quantify the phased disaster scale, based on the change of the tangential force direction at the bottom of some soil strips caused by water swelling of expansive soil, the upper and lower bound solutions of the first sliding area of slope are analyzed. The sliding trend critical

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point of expansive soil slope is strictly derived using the Morgenstern-Price static equilibrium analysis method. A case study is used to illustrate the calculation and analyze the influence of shear strength on the upper solution and lower solutions. The results show that partial and progressive failure may still occur on expansive soil slope under the influence of expansion although they are determined to be stable by classical analysis methods. The upper bound solution of the first slip area is less affected by the effective shear strength, while the influence of residual shear strength on the lower bound solution is particularly significant. The upper bound solution and its change rate decrease with the increase of effective cohesion and effective internal friction angle, while the lower bound solution and its change rate increase with the increase of residual internal friction angle and residual cohesion. In addition, the stability of expansive soil slope can be determined based on the relationship between the upper and lower limit solutions. This study provides the basic information for evaluating disaster scales and designing prevention and control measures for expansive soil landslides.

Keywords: expansive soil slope; first sliding area; stability of slope; upper and lower bound solution; Morgenstern-Price method

膨胀土作为我国及世界范围内广泛分布的典型特殊性黏土^[1], 具有显著的往复吸水膨胀和失水收缩变形特性^[2-3], 严重影响着相关地区的工程建设和维护^[4-5]。特别是二十世纪末以来, 随着一批重点项目的谋划及实施^[6], 膨胀土工程问题日益突出, 俨然已成为岩土工程领域研究的重点课题、热点问题, 同时也是技术难题, 尤其是膨胀土边坡的稳定性问题^[7]。

对此, 相关学者立足于膨胀土的基本特性, 开展了系列模型试验与数值模拟研究, 并基于所获现象, 分析了不同因素对边坡稳定性的影响, 建立了相应的计算方法^[8-9]。在理论分析方面, 不少学者以作用于土体单元的膨胀力反映膨胀性的不利影响, 包括仅考虑侧面或底面的膨胀作用, 以及同时考虑侧面与底面膨胀作用等情况^[10]。其中, 计算时对膨胀力的分布形式也存在不同假设, 常见的有三角形分布^[11]、矩形分布^[12]、抛物线型分布^[13]及折线型分布^[14]等。此外, 连继峰等^[15]通过概化膨胀土边坡破坏形态为“顺坡曲面”失稳模式, 采用低应力下强度非线性幂函数模型, 分析了膨胀土路堤边坡浅层稳定性。另一方面, 由于理论分析的复杂性及其假设的简化性, 采用数值模拟方式分析膨胀土边坡的稳定性更为活跃^[16-17]。其中, 采用湿度应力场理论模拟膨胀作用较为普遍, 即通过吸湿膨胀将膨胀应变以附加应变的形式加入到总应变中^[18-20]。其次, 考虑渗流软化作用, 以抗剪强度参数与含水率的关系为输入, 动态反映因土体含水率变化而引起的强度衰减^[21-22]。

可见, 无论是理论分析还是数值模拟膨胀土边坡的稳定性时, 考虑膨胀土的两大主要特性是十分必要的, 一是裂隙引起的强度衰减; 二是膨胀力的作用。

其中, 裂隙引起的强度衰减可通过试验有效获取^[23-25], 膨胀力也可在给定条件下通过室内试验获得^[26-29], 但其在边坡中作用的机制复杂, 且作用模式尚未达成统一。因此, 本文基于膨胀土边坡“渐进性”破坏的特点, 导出了仅考虑膨胀力宏观影响的边坡首次滑动区域上下限解, 以快速估测膨胀土边坡首次滑动的规模, 为其危害评估及工程防治提供参考。同时, 该方法亦可用于判定膨胀土边坡的稳定性, 并界定边坡稳定分析中所施加的膨胀力的上限。

1 土条底的切向力方向分析

1.1 经典边坡计算方法

诸如 Fellenious 法、Bishop 法、Janbu 法等经典边坡稳定分析方法, 其重要特点是假设自然作用下所有土条均趋于向坡脚滑动, 因而土条底的切向力方向均延续, 且与滑动方向相反。

1.2 膨胀土边坡

膨胀土在吸水膨胀时, 将沿无外界约束或约束作用较弱的方向产生膨胀变形。以置于水平面上的单位宽度均质膨胀土块为例, 其长、高在膨胀初始时分别为 $x(t_0)$ 、 $y(t_0)$, 膨胀终止时分别为 $x(t_f)$ 、 $y(t_f)$, 假设在土块底边存在点 s , 在膨胀过程中始终处于“静止”状态, 且以此点为界, 被分割的土块底边长分别为 $x_1(t)$ 、 $x_2(t)$, 如图 1 所示。

假设土块底部与其接触面的黏聚力为 $c(t)$ 、内摩擦角为 $\varphi(t)$, 土体重度为 $\gamma(t)$, 则在膨胀过程中有:

$$x_1(t)[c(t) + y(t)\gamma \tan \varphi(t)] = x_2(t)[c(t) + y(t)\gamma \tan \varphi(t)] \quad (1)$$

由式(1)可知, $x_1(t) = x_2(t)$, 说明 s 点为土块底部中

$$\frac{dG_1}{dx} - \tan \delta_1 \frac{d\beta}{dx} G_1 = P_1(x) \sec \delta_1 \quad (3)$$

式中:

$$P_1(x) = (c_1 \cos \varphi_1 - u \sin \varphi_1) \sec \alpha + \sin(\varphi_1 - \alpha) dW/dx - \cos(\varphi_1 - \alpha) dQ/dx, \delta_1 = \varphi_1 - \alpha + \beta。$$

同时, 对土条底中点取矩, 建立力矩平衡方程, 并简化后有:

$$G_1 \sin \beta + y_h \frac{d}{dx} (G_1 \cos \beta) = \frac{d}{dx} (y_{t1} G_1 \cos \beta) + \frac{dQ}{dx} h_e \quad (4)$$

式中: h_e ——水平地震力作用点距土条底的距离/m。

以反向的 ΔT_2 取代 ΔT_1 , 建立 II 区土条的力学计算模型, 如图 5 所示。

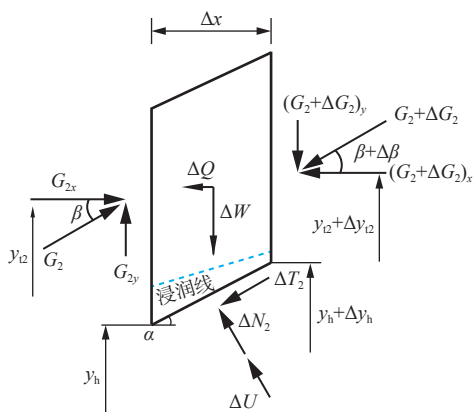


图 5 II 区土条受力分析示意图

Fig. 5 Schematic diagram of force analysis for soil stripes in area II

注: 图中变量符号含义参照图 4, 下标 2 表示 II 区土条。

同理, 对 II 区土条建立力的平衡方程, 并整理后有:

$$\frac{dG_2}{dx} + \tan \delta_2 \frac{d\beta}{dx} G_2 = P_2(x) \sec \delta_2 \quad (5)$$

式中:

$$P_2(x) = (u \sin \varphi_1 - c_1 \cos \varphi_1) \sec \alpha - \sin(\varphi_1 + \alpha) dW/dx - \cos(\varphi_1 + \alpha) dQ/dx, \delta_2 = \varphi_1 + \alpha - \beta。$$

对 II 区土条建立力矩平衡方程, 并简化后有:

$$G_2 \sin \beta + y_h \frac{d}{dx} (G_2 \cos \beta) = \frac{d}{dx} (y_{t2} G_2 \cos \beta) + \frac{dQ}{dx} h_e \quad (6)$$

2.3 滑动趋势临界点的确定

式(3)与式(5)分别为关于 G_1 、 G_2 的一阶非齐次线性微分方程, 对其求解, 并结合边界条件 $G_1(o) = 0$ 、 $G_2(a) = 0$, 有:

$$G_1(x) = \sec \delta_1 S_1^{-1}(x) \int_{x_o}^x P_1(l) S_1(l) dl \quad (7)$$

$$G_2(x) = \sec \delta_2 S_2^{-1}(x) \int_{x_a}^x P_2(l) S_2(l) dl \quad (8)$$

其中,

$$S_1(x) = \sec \delta_1 \exp \left(- \int_{x_o}^x \tan \delta_1 \frac{d\beta}{dl} dl \right)$$

$$S_2(x) = \sec \delta_2 \exp \left(\int_{x_a}^x \tan \delta_2 \frac{d\beta}{dl} dl \right)$$

式中: x_o 、 x_a ——滑面曲线与坡面曲线交点 o 、 a 两点的横坐标/m。

对式(4)、式(6)分别积分, 可得:

$$y_{t1}(x) = \frac{\int_{x_o}^x G_1 (\sin \beta - \cos \beta \tan \alpha) dx - \int_{x_o}^x \eta \frac{dW}{dx} h_e dx}{G_1 \cos \beta} + y_h \quad (9)$$

$$y_{t2}(x) = \frac{\int_{x_a}^x G_2 (\sin \beta - \cos \beta \tan \alpha) dx - \int_{x_a}^x \eta \frac{dW}{dx} h_e dx}{G_2 \cos \beta} + y_h \quad (10)$$

上述 β 为一未知变量, 假定其符合某一形状分布^[31]:

$$\tan \beta = f_0(x) + \lambda f(x) \quad (11)$$

式中: $f_0(x)$ 、 $f(x)$ ——假定的函数分布;

λ ——待求参数。

根据连续性条件, 在滑动趋势临界点 s 处有:

$$G_1(s) = G_2(s) \quad (12)$$

$$y_{t1}(s) = y_{t2}(s) \quad (13)$$

$f_0(x)$ 、 $f(x)$ 一旦确定, 即可联立式(7)——(13), 求解 s 及 λ 。

3 膨胀土边坡首次滑动区域的上下限解

3.1 膨胀土边坡的滑面形态

基于某干渠 22 处膨胀土滑坡的资料收集与野外调研, 其滑面形态及采用二次函数曲线拟合的结果如图 6 所示。

由图 6 分析可知, 18 处滑面(占比为 81.82%)采用二次函数曲线拟合的效果较好, 其余 4 处(SS-2#、SS-3#、XF-4#、AT-3#)滑面的拟合效果则一般。进一步地, 采用三次函数曲线对该 4 处滑面拟合, 可获得较为理想的结果。由此可见, 采用二次函数曲线可有效拟合绝大部分膨胀土滑坡的滑面, 而其余部分则可采用三次函数曲线进行拟合。

在此, 为便于说明, 以其滑面及坡面均可拟合为二次函数曲线为例, 分析边坡首次滑动区域的上下限解。

3.2 首次滑动区域的上限解

假定滑面曲线与坡面曲线分别为:

$$y_h(x) = q_1 x^2 + p_1 x + q_1 \quad (x_o \leq x \leq x_a) \quad (14)$$

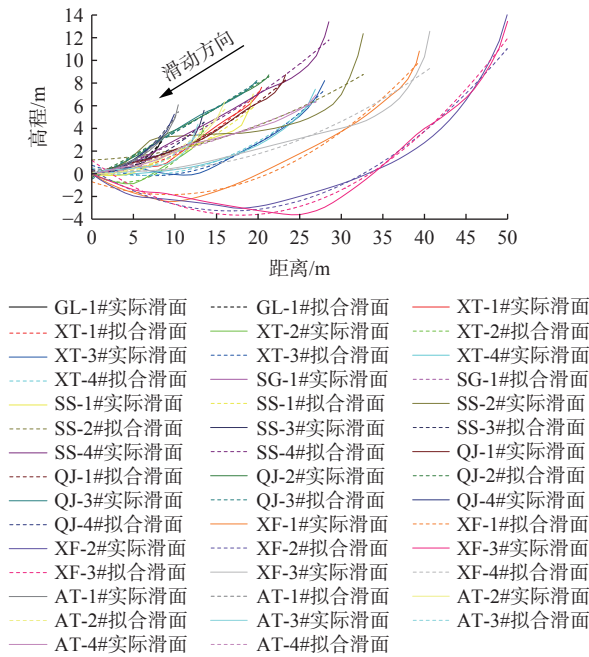


图 6 膨胀土滑坡滑面形态及其拟合结果

Fig. 6 Sliding surface shape of expansive soil landslides and its fitting results

$$y_p(x) = o_2 x^2 + p_2 x + q_2 \quad (x_o \leq x \leq x_a) \quad (15)$$

式中: o_i 、 p_i 、 q_i ($i = 1, 2$)——拟合参数。

假定 $\beta(x)$ 为常数^[31], 联立式(7)(14)(15)有:

$$G_1 = M_{11}x^3 + M_{12}x^2 + M_{13}x - M_{14} \ln\left(1 + \frac{E_1 x}{H_1}\right) \quad (16)$$

式中:

$$M_{11} = A_1 / (3E_1)$$

$$M_{12} = B_1 / (2E_1) - A_1 H_1 / (2E_1^2)$$

$$M_{13} = C_1 / E_1 - B_1 H_1 / E_1^2 + A_1 H_1^2 / E_1^3$$

$$M_{14} = (-D_1 E_1^3 + C_1 H_1 E_1^2 - B_1 E_1 H_1^2 + A_1 H_1^3) / E_1^4$$

$$A_1 = -2\gamma o_1 (o_2 - o_1) (\cos \varphi_t + \eta \sin \varphi_t)$$

$$B_1 = 4o_1^2 c_t \cos \varphi_t - 2\gamma o_1 (p_2 - p_1) (\cos \varphi_t + \eta \sin \varphi_t) + \gamma (o_2 - o_1) (\sin \varphi_t - \eta \cos \varphi_t - p_1 \cos \varphi_t - p_1 \eta \sin \varphi_t)$$

$$C_1 = 4o_1 p_1 c_t \cos \varphi_t - 2\gamma o_1 (\cos \varphi_t + \eta \sin \varphi_t) (q_2 - q_1) + \gamma (\sin \varphi_t - \eta \cos \varphi_t - p_1 \cos \varphi_t - p_1 \eta \sin \varphi_t) (p_2 - p_1)$$

$$D_1 = \gamma (q_2 - q_1) (\sin \varphi_t - \eta \cos \varphi_t - p_1 \cos \varphi_t - p_1 \eta \sin \varphi_t) + c_t \cos \varphi_t (p_1^2 + 1)$$

$$E_1 = 2o_1 \sin(\varphi_t + \beta)$$

$$H_1 = \cos(\varphi_t + \beta) + p_1 \sin(\varphi_t + \beta)$$

同理, 可求得:

$$G_2 = M_{21}(x^3 - a^3) + M_{22}(x^2 - a^2) + M_{23}(x - a) + M_{24} \ln\left(\frac{H_2 + E_2 a}{H_2 + E_2 x}\right) \quad (17)$$

式中:

$$M_{21} = A_2 / (3E_2)$$

$$M_{22} = B_2 / (2E_2) - A_2 H_2 / (2E_2^2)$$

$$M_{23} = C_2 / E_2 - B_2 H_2 / E_2^2 + A_2 H_2^2 / E_2^3$$

$$M_{24} = (-D_2 E_2^3 + C_2 H_2 E_2^2 - B_2 E_2 H_2^2 + A_2 H_2^3) / E_2^4$$

$$A_2 = -2\gamma o_1 (o_2 - o_1) (\cos \varphi_t - \eta \sin \varphi_t)$$

$$B_2 = -4o_1^2 c_t \cos \varphi_t - 2\gamma o_1 (p_2 - p_1) (\cos \varphi_t - \eta \sin \varphi_t) - \gamma (o_2 - o_1) (\sin \varphi_t + \eta \cos \varphi_t + p_1 \cos \varphi_t - p_1 \eta \sin \varphi_t)$$

$$C_2 = -4o_1 p_1 c_t \cos \varphi_t - 2\gamma o_1 (\cos \varphi_t - \eta \sin \varphi_t) (q_2 - q_1) - \gamma (\sin \varphi_t + \eta \cos \varphi_t + p_1 \cos \varphi_t - p_1 \eta \sin \varphi_t) (p_2 - p_1)$$

$$D_2 = -\gamma (q_2 - q_1) (\sin \varphi_t + \eta \cos \varphi_t + p_1 \cos \varphi_t - p_1 \eta \sin \varphi_t) - c_t \cos \varphi_t (p_1^2 + 1)$$

$$E_2 = -2o_1 \sin(\varphi_t - \beta)$$

$$H_2 = \cos(\varphi_t - \beta) - p_1 \sin(\varphi_t - \beta)$$

联立式(9)、式(14)——(16)有:

$$y_{t1}(x) = \frac{J_{G1}(\sin \beta - p_1 \cos \beta) - 2o_1 \cos \beta J_{xG1} - \eta \gamma J_{w1} / 2}{G_1 \cos \beta} + \frac{o_1 x^2 + p_1 x + q_1}{G_1 \cos \beta} \quad (18)$$

式中:

$$J_{G1} = \frac{M_{11}}{4} x^4 + \frac{M_{12}}{3} x^3 + \frac{M_{13}}{2} x^2 - M_{14} \left[\left(\frac{H_1}{E_1} + x \right) \ln \left(1 + \frac{E_1 x}{H_1} \right) - x \right]$$

$$J_{xG1} = \frac{M_{11}}{5} x^5 + \frac{M_{12}}{4} x^4 + \frac{M_{13}}{3} x^3 - \frac{M_{14}}{2} \left[\left(x^2 - \frac{H_1^2}{E_1^2} \right) \ln \left(1 + \frac{E_1 x}{H_1} \right) - \frac{x^2}{2} + \frac{H_1}{E_1} x \right]$$

$$J_{w1} = \frac{(o_2 - o_1)^2}{5} x^5 + \frac{(o_2 - o_1)(p_2 - p_1)}{2} x^4 + \frac{2(o_2 - o_1)(q_2 - q_1) + (p_2 - p_1)^2}{3} x^3 + (p_2 - p_1)(q_2 - q_1) x^2 + (q_2 - q_1)^2 x$$

同理, 可求得:

$$y_{t2}(x) = \frac{J_{G2}(\sin \beta - p_1 \cos \beta) - 2o_1 \cos \beta J_{xG2} - \eta \gamma J_{w2} / 2}{G_2 \cos \beta} + \frac{o_1 x^2 + p_1 x + q_1}{G_2 \cos \beta} \quad (19)$$

式中:

$$J_{G2} = \left[\begin{array}{l} \frac{M_{21}}{4}x^4 + \frac{M_{22}}{3}x^3 + \frac{M_{23}}{2}x^2 - \\ M_{24} \left(\left(\frac{H_2}{E_2} + x \right) \ln(H_2 + E_2x) - x \right) - \\ (M_{21}a^3 + M_{22}a^2 + M_{23}a - M_{24} \ln(H_2 + E_2a))x \end{array} \right]_{x_a}^x$$

$$J_{xG2} = \left[\begin{array}{l} \frac{M_{21}}{5}x^5 + \frac{M_{22}}{4}x^4 + \frac{M_{23}}{3}x^3 - \\ M_{24} \left(\left(\frac{x^2}{2} - \frac{F_2^2}{2E_2^2} \right) \ln(H_2 + E_2x) - \frac{x^2}{4} + \frac{H_2}{2E_2}x \right) - \\ \frac{M_{21}a^3 + M_{22}a^2 + M_{23}a - M_{24} \ln(H_2 + E_2a)}{2}x^2 \end{array} \right]_{x_a}^x$$

$$J_{w2} = \left[\begin{array}{l} \frac{(o_2 - o_1)^2}{5}x^5 + \frac{(o_2 - o_1)(p_2 - p_1)}{2}x^4 + \\ \frac{2(o_2 - o_1)(q_2 - q_1) + (p_2 - p_1)^2}{3}x^3 + \\ (p_2 - p_1)(q_2 - q_1)x^2 + (q_2 - q_1)^2x \end{array} \right]_{x_a}^x$$

联立式(12)(13)、式(16)一(19)即可求得首次滑动区域的上限解, 对应于该处的条间作用力亦可用于界定该处膨胀力的上限。

需要特别说明的是, 按经典边坡计算方法求得的边坡整体安全系数小于 1 的情况, 不在该分析方法所包含的范围内。

3.3 首次滑动区域的下限解

受膨胀作用影响, 被推动部分土体的抗剪强度将转变为残余强度, 故首次滑动区域的下限解(d)基于残余强度的平衡状态求取。仍可采用上述求取上限解的方法按式(16)计算求取, 所不同的是, 部分参数(c_i 变为残余黏聚力 c_r , φ_i 变为残余内摩擦角 φ_r)及边界条件的变化:

$$G_1(d) = 0 \quad (20)$$

$$y_{il}(d) = y_h(d) \quad (21)$$

具体计算过程在此不再赘述。

4 算例分析

某支渠膨胀土边坡如图 7 所示, 地层岩性为: ①层上更新统重粉质壤土(Qp_3^{al}): 夹粉质黏土, 灰黄、棕黄色, 硬可塑~硬塑, 局部坚硬, 湿, 夹铁锰质结核及灰白色高岭土, 中等压缩性; ②层下白垩统朱巷组全风化~强风化泥质粉砂岩(K_1z): 软质岩石, 泥质胶结, 紫红色, 破碎, 岩心呈土状, 少数块状, 揭露最大厚度 3.20 m。潜在滑体均处于①层, 其湿重度为 19.4 kN/m³, 场地地下水类型分为两类: 一类为上层滞水, 分布于

①层重粉质壤土表层孔隙中, 受大气降水和地表水渗入补给, 无统一水位, 水量一般较小, 多随季节性降水变化而变化, 勘探期间, 水位位于滑坡鼓丘以下 5~7 m, 地下水受降水及干渠河水的补给。第二类为基岩裂隙水, 分布于下伏基岩泥质粉砂岩中, 微承压性, 水量小。采用固结快剪试验测得潜在滑面原状土体的总黏聚力 $c_t=10$ kPa, 总内摩擦角 $\varphi_t=19^\circ$; 残余黏聚力 $c_r=2.5$ kPa, 残余内摩擦角 $\varphi_r=12^\circ$ 。此外, 水平地震影响系数(η)取 0.05。

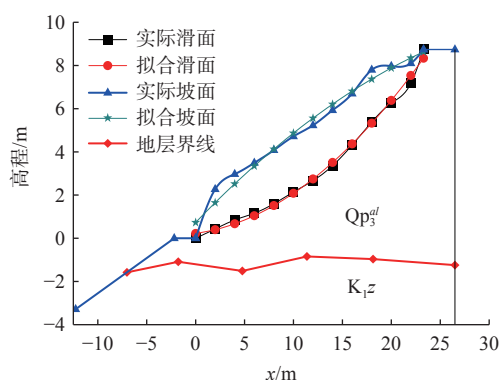


图 7 某支渠膨胀土边坡示意图

Fig. 7 Schematic diagram of expansive soil slope along a branch canal

拟合后的滑面曲线与坡面曲线(单位均为 m)分别为:

$$y_h(x) = 0.0122x^2 + 0.064x + 0.2224 \quad (0 \leq x \leq 23.3) \quad (22)$$

$$y_p(x) = -0.0056x^2 + 0.4696x + 0.7319 \quad (0 \leq x \leq 23.3) \quad (23)$$

经编程计算, 边坡整体稳定性系数为 1.62, 在本次研究范围内, 其首次滑动区域的上限解 $s=17.71$ m, 下限解 $d=11.47$ m, 由此可估算该膨胀土滑坡单位宽度首次滑动的体量(S)介于 23.56~39.67 m³, 如图 8 所示。

5 抗剪强度对上下限解的影响

基于上述算例, 分别分析有效抗剪强度对上限解的影响及残余抗剪强度对下限解的影响。

5.1 有效抗剪强度对上限解的影响

为保障边坡不发生整体失稳, 取有效黏聚力的变化范围为 7~13 kPa, 相应地有效内摩擦角的变化范围取 13°~25°。经计算整理, 得到上限解与有效黏聚力及有效内摩擦角的关系, 如图 9 所示。

由图可见, 上限解随有效黏聚力及有效内摩擦角的增大而减小, 且其减小速率也均随有效黏聚力及有效内摩擦角的增大而减小。当有效黏聚力为 7 kPa, 有

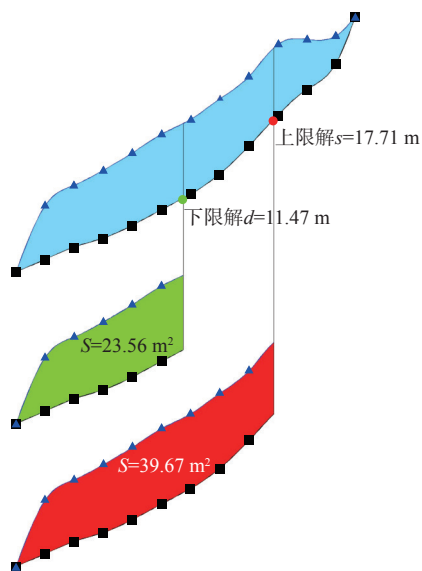


图 8 首次滑动区域的上下限解

Fig. 8 Upper and lower bound solutions in the first sliding area

注: 此图为滑动区域的平面示意, 故 S 采用面积单位 m^2 。

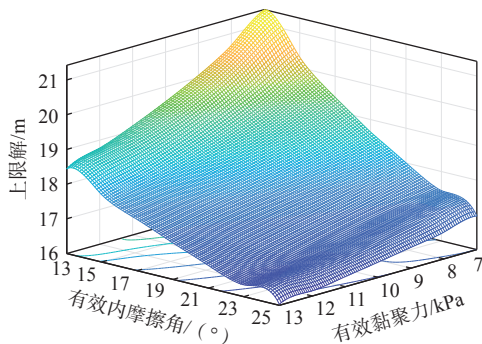


图 9 上限解与有效抗剪强度的关系曲面

Fig. 9 Relationship between upper bound solution and effective shear strength

效内摩擦角由 13° 增至 25° 时, 上限解减小 4.44 m ; 而当有效黏聚力为 13 kPa , 有效内摩擦角由 13° 增至 25° 时, 上限解减小 2.41 m 。同样地, 当有效内摩擦角为 13° , 有效黏聚力由 7 kPa 增至 13 kPa 时, 上限解减小 2.97 m ; 而当有效内摩擦角为 25° , 有效黏聚力由 7 kPa 增至 13 kPa 时, 上限解减小仅为 0.94 m 。

5.2 残余抗剪强度对下限解的影响

残余黏聚力取 $0.5 \sim 4.5 \text{ kPa}$, 相应地, 残余内摩擦角取 $8^\circ \sim 16^\circ$ 。经计算整理, 得到下限解与残余黏聚力及残余内摩擦角的关系, 如图 10 所示。

图中反映出, 下限解随残余内摩擦角及残余黏聚力的增加而增加, 且其增加速率也均随残余黏聚力及残余内摩擦角的增加而增加。当残余黏聚力为 0.5 kPa , 残余内摩擦角由 8° 增至 16° 时, 下限解增加 9.57 m ; 而当

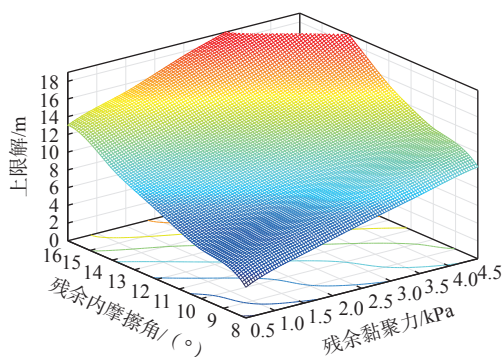


图 10 下限解与残余抗剪强度的关系曲面

Fig. 10 Relationship between lower bound solution and residual shear strength

残余内摩擦角为 8° , 残余黏聚力由 0.5 kPa 增至 4.5 kPa 时, 下限解增加 6.9 m 。

特别地, 在残余黏聚力及残余内摩擦角均较大的区域 (如: $\varphi_r = 16^\circ$, $c_r \geq 3.5 \text{ kPa}$), 无下限解, 表明该类情况下, 边坡不会发生失稳破坏。

综合分析, 膨胀土边坡首次滑动区域的上限解受有效抗剪强度的影响较小, 而残余抗剪强度对于下限解的影响尤为显著。同时, 可进一步基于上下限解的大小关系, 分析膨胀土边坡的稳定性:

- (1) 当下限解小于上限解时, 边坡可能发生局部失稳破坏, 其首次滑动区域可估算确定;
- (2) 当下限解等于上限解时, 边坡可能发生局部失稳破坏, 且其首次滑动区域可准确确定;
- (3) 当无下限解或下限解大于上限解时, 边坡处于稳定状态。

6 结论

(1) 采用二次函数可有效拟合绝大部分膨胀土滑坡的滑面, 而其余部分则可进一步地采用三次函数曲线进行拟合。

(2) 受膨胀作用影响, 按经典分析方法判定为稳定的膨胀土边坡仍有可能发生局部失稳渐进破坏, 其首次滑动区域的范围可通过上下限解确定。

(3) 膨胀土边坡首次滑动区域的上限解受有效抗剪强度的影响较小, 而残余抗剪强度对于下限解的影响尤为显著。上限解随有效黏聚力及有效内摩擦角的增大而减小, 且其减小速率也均随有效黏聚力及有效内摩擦角的增大而减小; 下限解随残余抗剪强度的变化则相反。

(4) 可基于上下限解的大小关系, 分析膨胀土边坡的稳定性。当下限解不大于上限解时, 边坡可能发

生局部失稳破坏; 当无下限解或下限解大于上限解时, 边坡处于稳定状态。

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