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Seismic ultimate bearing capacity of strip footings on slope

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Abstract: The influence of earthquake forces on ultimate bearing capacity of foundations on sloping ground was studied. A solution to seismic ultimate bearing capacity of strip footings on slope was obtained by utilizing pseudo-static analysis method and taking the effect of intermediate principal stress into consideration. Based on limit equilibrium theory, the formulae for computing static bearing capacity factors, N_q, N_c, N_γ, and dynamic bearing capacity factors, N_{qd}, N_{cd}, $N_{\gamma d}$, which are associated with surcharge, cohesion and self-weight of soils respectively, were presented. A great number of analysis calculations were carried out to obtain the relationship curves of the static and dynamic bearing capacity factors versus various calculation parameters. The curves can serve as the practical engineering design. The calculation results also show that when the values of horizontal and vertical seismic coefficients are 0.2, the dynamic bearing capacity factors N_{qd}, N_{cd} and $N_{\gamma d}$, in which the effects of intermediate principal stress are taken into consideration, increase by 4%−42%, 3%−27% and 34%−57%, respectively.

Key words: slope foundation; bearing capacity; unified strength theory; limit equilibrium; earthquake effect

1 Introduction

The study on bearing capacity has been the hotspot of the geotechnical circle for ages. Among the limited literatures available on the seismic bearing capacity, MEYERHOF's[1−2] are perhaps the earliest, where the seismic forces were applied on the structure only as inclined pseudo-static loads. Effect of the seismic forces on the inertia of the supporting soil was not considered in these analyses. SARMA et al[3] and RICHARDS et al[4] considered the seismic forces both on the structure and on the supporting soil mass. Apart from these studies by the limit equilibrium method, DORMIEUX et al[5], SOUBRA[6−7] and CHOUDHURY et al[8] used the upper bound limit method for analyzing the seismic bearing capacity of shallow strip footings. YU[9] and YANG[10] utilized numerical simulating method and energy dissipation method respectively for determination of the static bearing capacity of foundation on slope. However, for the seismic bearing capacity of strip footings founded on the slope in mountainous area, little investigations have been conducted. And the existing calculating formulae have not considered the effect of intermediate principal stress on ultimate bearing capacity. In addition, the results obtained by the existing calculating formulae are usually smaller than real values and the potential strength of soil is not fully mobilized. So in this paper, based on the unified strength theory that takes the effect of intermediate principal stress into consideration, by supposing composite curved failure surface and employing pseudo-static approach and the method of limit equilibrium analysis, a new formula of seismic bearing capacity for slope foundation in mountainous areas was obtained.

2 Unified strength theory

Unified strength theory for geo-materials can be expressed as follows[11−12]:

$$F=\alpha\sigma_1-\frac{b\sigma_2+\sigma_3}{1+b}=\frac{2c_0\cos\varphi_0}{1+\sin\varphi_0},\quad \sigma_2\leqslant\frac{1}{2}(\sigma_1+\sigma_3)-\frac{\sin\varphi_0}{2}(\sigma_1-\sigma_3) \tag{1a}$$

$$F=\frac{\alpha}{1+b}(\sigma_1+b\sigma_2)-\sigma_3=\frac{2c_0\cos\varphi_0}{1+\sin\varphi_0},\quad \sigma_2\geqslant\frac{1}{2}(\sigma_1+\sigma_3)-\frac{\sin\varphi_0}{2}(\sigma_1-\sigma_3) \tag{1b}$$

where $\alpha=(1-\sin\varphi_0)/(1+\sin\varphi_0)$; σ_1 is the major principal stress; σ_3 is the minor principal stress; b is a coefficient that can reflect the influence of intermediate principal stress and normal stress on the failure degree of material; φ_0 is the original angle of internal friction and c_0 is the original cohesion of geo-material.

The normal stress σ_ω and shear stress τ_ω acting on any inclined plane oriented ω clockwise from the major principal plane can be determined:

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$$\sigma_\omega = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos(2\omega) \tag{2}$$

$$\tau_\omega = \frac{\sigma_1-\sigma_3}{2}\sin(2\omega) \tag{3}$$

FAN et al[13−14] recommended the following equation for ω in limit state:

$$2\omega = \pi - \arccos\frac{1+b-\alpha(kb+1)}{1+b+\alpha} \tag{4}$$

By solving Eqns.(2) and (3), we obtain

$$\sigma_1 = \sigma_\omega + \frac{1-\cos(2\omega)}{\sin(2\omega)}\tau_\omega \tag{5}$$

$$\sigma_3 = \sigma_\omega - \frac{1+(\cos 2\omega)}{(\sin 2\omega)}\tau_\omega \tag{6}$$

According to LEE et al[15], the intermediate principal stress can be expressed as follows:

$$\sigma_2 = k(\sigma_1+\sigma_3)/2 \tag{7}$$

where k is the coefficient of intermediate principal stress, $2\mu \leqslant k < 1$ (μ is Poisson ratio), and $k \to 1$ in plastic zone.

From Eqn.(7), we can calculate the ultimate bearing capacity of foundation by using the following condition:

$$\sigma_2 \geqslant \frac{\sigma_1+\sigma_3}{2} - \frac{\sigma_1-\sigma_3}{2}\sin(2\varphi_0) \tag{8}$$

We can see that σ_2 satisfies Eqn.(1b). So substitute Eqns.(2)−(8) into Eqn.(1b), we get

$$A\tau_\omega + B\sigma_\omega - Dc_0 = 0 \tag{9a}$$

where

$$A = [(\frac{\alpha kb}{2(1+b)} + \frac{\alpha}{1+b})(1-\cos(2\omega)) - (\frac{\alpha kb}{2(1+b)} - 1)(1+\cos(2\omega))]/\sin(2\omega),$$

$$B = \frac{\alpha kb}{1+b} + \frac{\alpha}{1+b} - 1,\ D = \frac{2\cos\varphi_0}{1+\sin\varphi_0}.$$

Eqn.(9a) may be written in the form:

$$\tau_\omega = \frac{Dc_0}{A} - \frac{B}{A}\sigma_\omega \tag{9b}$$

Comparing Eqn.(9b) with Mohr-Coulomb criterion, we can obtain the cohesion and angle of internal friction (c and φ) based on unified strength theory:

$$\begin{cases} c = Dc_0/A \\ \varphi = \dfrac{\pi}{2} - \arccos\dfrac{1+b-\alpha(kb+1)}{1+b+\alpha} \end{cases} \tag{10}$$

3 Seismic bearing capacity of strip footing on slope

3.1 Failure mode of slope ground

Failure mode of slope ground is shown in Fig.1. It consists of two slip surfaces DEK and EFG. DEK is a realistic failure plane and EFG is a virtual failure one. Failure plane EFG does not exist. This is just for force-analysis of the soil mass behind the footings.

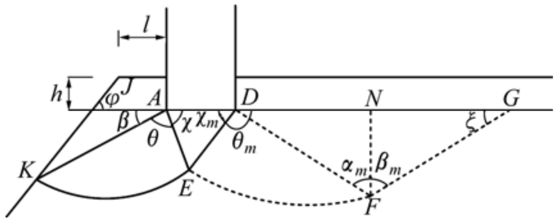

Fig.1 Computational model of slope ground

3.1.1 Three wedges in part DEK

1) Wedge DAE. Wedge DAE is an asymmetrical triangular wedge in which soil is in elastic state. Assuming the strip footing is rough enough, let χ and χ_{m} denote the base angles of the wedge ADE, respectively, and

$$\chi + \chi_{\mathrm{m}} = \frac{\pi}{2} + \varphi \tag{11}$$

2) Wedge AEK. Wedge AEK is a transitional zone from active earth pressure under the footing through passive earth pressure in the surrounding soil. Assuming EK to be a logarithmic spiral curve, point A is the center of this spiral curve, so the equation of this curve can be expressed as follows:

$$r = r_0 \exp(\theta_0 \tan\varphi) \tag{12}$$

where r_0 is the length of line AE and it can be calculated by the following formula:

$$r_0 = \frac{B\sin\chi_m}{\sin(\chi+\chi_m)} \tag{13}$$

Hence, the length of AK can be written as follows:

$$r_{AK} = r_0 \exp(\theta\tan\varphi) \tag{14}$$

3) Wedge AKJ. The wedge AKJ is a Rankine passive earth pressure zone.

3.1.2 Virtual failure plane EFG

Part $DEFG$ involves a transitional zone EDF and a Rankine passive earth pressure zone DFE.

Since the plane EFG does not reach its ultimate limit state and it is difficult to determinate its stress distribution, thereby we introduce a coefficient m to indicate the mobilization degree of shear on plane EFG.

$$m = \tau/\tau_\omega \tag{15}$$

Substitute Eqn.(13) into Eqn.(9b), then

$$m\tau_\omega = m\frac{D}{A}(\varphi)c_0 - m\frac{B}{A}(\varphi)\sigma_\omega = \frac{D}{A}(\varphi_m)c_m - \frac{B}{A}(\varphi_m)\sigma_\omega p \tag{16}$$

where τ_ω is the ultimate shear stress, c_m and φ_m are the cohesion and the angle of friction that are related to m, respectively.

The values of c_m and φ_m, varying with the values of b and m can be obtained by computation (see Table 1). When b=0, Eqn.(9b) becomes Mohr-Coulomb criterion ($\tau=c+\sigma\tan\varphi$), so Mohr-Coulomb criterion is just a special case of this method.

Similar to EK, the equation of EF can be given as follows:

$$r_{m\theta}=r_{m0}\exp(\theta_{m0}\tan\varphi_m) \tag{17}$$

where r_{m0} is the length of DE, calculated by

$$r_{m0}=\frac{B\sin\chi}{\sin(\chi+\chi_m)} \tag{18}$$

Hence, the length of DF can be written as follows:

$$r_{DF}=r_m=r_0\exp(\theta_m\tan\varphi_m) \tag{19a}$$

Similar to AJK, DFG is assumed as a Rankine passive states zone, and

$$\angle DFG=\pi/2+\varphi_m \tag{19b}$$

Table 1 Values of c_m, φ_m varying with b and m under k=1.0, φ_0=20°, c_0=20.0 kPa

b	m=1.0		m=0.9		m=0.8		m=0.7	
	c_m/kPa	φ_m/(°)	c_m/kPa	φ_m/(°)	c_m/kPa	φ_m/(°)	c_m/kPa	φ_m/(°)
1.0	21.5	24.1	19.3	21.8	17.0	19.5	14.8	17.1
0.8	21.3	23.5	19.1	21.3	16.9	19.0	14.7	16.7
0.6	21.1	22.9	18.9	20.8	16.7	18.5	14.6	16.3
0.4	20.8	22.2	18.7	20.1	16.6	17.9	14.4	15.8
0.2	20.5	21.2	18.4	19.2	16.3	17.2	14.2	15.1
0.0	20.0	20.0	18.0	18.1	16.0	16.2	14.0	14.3

3.2 Limit equilibrium analysis

The free-body diagram of wedge ADE is shown in Fig.2. The forces acting on AE are $p_{\mathrm{p}\gamma}$, p_{pq}, p_{pc} and C_{a}, where $p_{\mathrm{p}\gamma}$ is produced by the unit weight (the weight per unit volume), p_{pq} by the surcharge loading, and p_{pc} by the soil cohesion. Both p_{pc} and p_{pq} act at the midpoint of AE, and $p_{\mathrm{p}\gamma}$ is assumed to act at two-third position of AE. The directions of all these forces make the same angle of φ to the normal of AE. In addition, the adhesion $C_{\mathrm{a}}=c\cdot r_{\mathrm{DE}}$, where c is the unit cohesion.

In a similar manner, the forces acting on DE are shown in Fig.2. The subscript m of these forces indicates the mobilization degree of shear on virtual failure plane EF. $p_{\mathrm{p}m\gamma}$, $p_{\mathrm{p}m\mathrm{q}}$ and $p_{\mathrm{p}m\mathrm{c}}$ are produced by the unit weight, surcharge and soil cohesion, respectively. The directions and positions of these forces are similar to those of the forces acting on AE. $C_{\mathrm{a}m}=c_m\cdot r_{DE}$, where c_m is the unit cohesion.

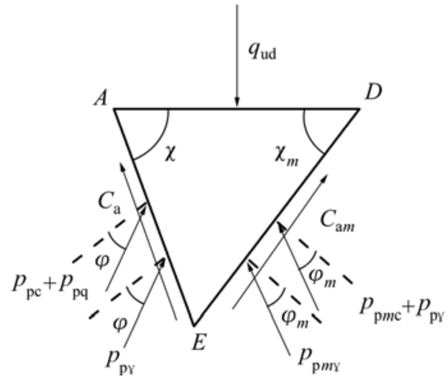

Fig.2 Free-body diagram of wedge ADE

The vertical seismic passive resistance can be computed as

$$p_{\mathrm{p}}=(p_{\mathrm{p}\gamma}+p_{\mathrm{pc}}+p_{\mathrm{pq}})\cos(\chi-\varphi) \tag{20}$$

$$p_{\mathrm{p}m}=(p_{\mathrm{p}m\gamma}+p_{\mathrm{p}m\mathrm{c}}+p_{\mathrm{p}m\mathrm{q}})\cos(\chi_m-\varphi_m) \tag{21}$$

From the vertical equilibrium of all the forces, we can obtain:

$$q_{\mathrm{ud}}=p_{\mathrm{p}}+p_{\mathrm{p}m}+C_{\mathrm{a}}\sin\chi+C_{\mathrm{a}m}\sin\chi_m \tag{22}$$

Determination of the values of all the passive resistances is as follows.

Case 1: $c=c_m=0$, $q_2=q_m=0$, $\gamma\neq 0$

Considering the forces acting on the wedge AKE, as shown in Fig.3, then from the moment equilibrium of all the forces about the focus A we have

$$p_{\mathrm{p}\gamma}=\frac{3}{2r_0\cos\varphi}(M_1-M_2-M_3-M_4) \tag{23}$$

where

$$M_1=\frac{1-k_v}{6}\gamma r_{AJ}\cdot r_{AK}\sin\beta(r_{AJ}+r_{AK}\cos\beta);$$

$$M_2=\frac{k_h}{6}\gamma r_{AJ}\cdot(r_{AK}\sin\beta)(r_{AK}\sin\beta);$$

$$M_3=\frac{\gamma r_0^3(1-k_v)}{3}\int_0^\theta \exp(3\theta_0\tan\varphi)\cos(\theta_0+\chi)\mathrm{d}\theta_0;$$

$$M_4=\frac{\gamma r_0^3 k_h}{3}\int_0^\theta \exp(3\theta_0\tan\varphi)\sin(\theta_0+\chi)\mathrm{d}\theta_0.$$

Angles θ and β can be derived from the known conditions. The force F_1 must act through the point A, so its moment for A is zero.

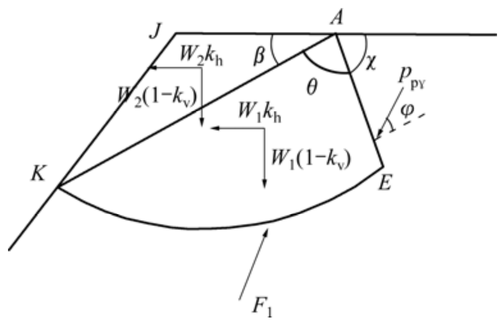

Fig.3 Diagram of $AEKJ$ only weight considered

Considering the forces acting on the wedge *DEFN*, as shown in Fig.4, then from the moment equilibrium of all the forces about the focus *D*, we have

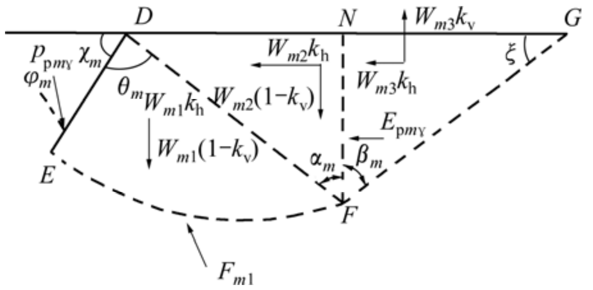

Fig.4 Diagram of *DEFN* only weight considered

$$P_{pm\gamma}=\frac{3}{2r_{m0}\cos\varphi_n}(-M_{\gamma m1}+M_{\gamma m2}+M_{\gamma m3}+M_{\gamma m4}+ M_{\gamma m5}-M_{\gamma m6}+M_{\gamma m7}) \tag{24}$$

where

$$M_{m1}=\frac{\gamma r_{m0}^3(1-k_v)}{3}\int_0^{\theta_m}\exp(3\theta_{m0}\tan\varphi_m)\cos(\theta_{m0}+\chi_m)\mathrm{d}\theta_{m0};$$

$$M_{m2}=\frac{\gamma r_{m0}^3k_h}{3}\int_0^{\theta_m}\exp(3\theta_{m0}\tan\varphi_m)\mathrm{d}\theta_{m0};$$

$$M_{m3}=\frac{1-k_v}{3}\gamma r_m^2H_m\sin^2\alpha_m;$$

$$M_{m4}=\frac{k_h}{6}\gamma r_m^2H_m\sin\alpha_m\cos\alpha_m;$$

$$M_{m5}=\frac{2}{3}H_mE_{pm\gamma};$$

$$M_{m6}=\frac{k_v}{2}\gamma H_m^2(r_m\sin\alpha_m+\frac{H_m\tan\beta_m}{3})\tan\beta_m;$$

$$M_{m7}=\frac{k_h}{6}\gamma H_m^3\tan\alpha_m.$$

ξ is the included angle between *GF* and ground given by Choudhury and Subba Rao[8]:

$$\zeta=\frac{\pi}{4}-\frac{\varphi}{2}+\frac{1}{2}\arctan^{-1}(\frac{k_h}{1\text{-}k_v})-\frac{1}{2}\arcsin^{-1}\frac{\sin[\arctan^{-1}(\frac{k_h}{1-k_v})]}{\sin\varphi}.$$

where k_h is horizontal acceleration under seismic action; k_v is vertical acceleration under seismic action; $\beta_m=\pi/2-\xi$; $\alpha_m=\varphi_m+\xi$; $\angle DFG=\pi/2+\varphi_m$.

The passive pressure $E_{pm\gamma}$ that acts on *NF* is horizontal and can be obtained as

$$E_{pm\gamma}=\frac{1}{2}H_m^2\gamma K_m'$$

where $K_m'=1+\sin\varphi_m/(1-\sin\varphi_m)$; $H_m=r_m\cos\alpha_m$.

The force F_{m1} must act through the point *D*, so its moment for *D* is zero.

Case 2: $q=q_m=0$, $\gamma=0$, c, $c_m\neq0$

Considering the forces acting on the wedge *AKE*, as shown in Fig.5, then from the moment equilibrium of all the forces about the focus *A*, we have

$$p_{pc}=\frac{2M_{c1}}{r_0\cos\varphi} \tag{25}$$

where $M_{c1}=\frac{cr_0^2}{2\tan\varphi}[\exp(2\theta)-1]$.

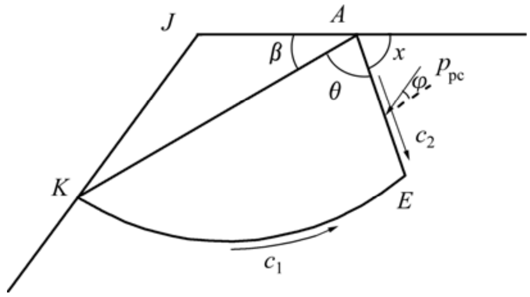

Fig.5 Diagram of *AEJK* applied by cohesion

Then considering the forces acting on the area *DEFN*, as shown in Fig.6, and the moment equilibrium of all the forces about the focus *D*, we can get

$$p_{pmc}=\frac{M_{cm1}+\frac{1}{2}H_mE_{pmc}}{\frac{1}{2}r_{m0}\cos\varphi_m} \tag{26}$$

where

$$M_{cm1}=\frac{c_mr_{mo}^2}{2\tan\varphi_m}[\exp(2\theta_m\tan\varphi_m)-1],$$

$$E_{pmc}=c_mH_m\frac{\cos\varphi_m}{\sin(\beta_m-\varphi_m)\cos\beta_m}.$$

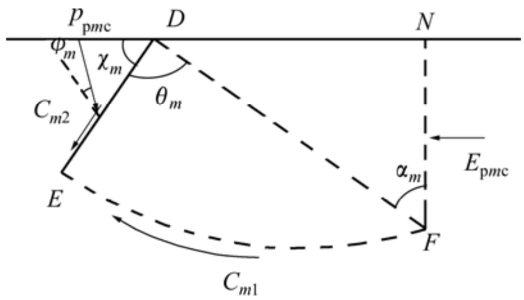

Fig.6 Diagram of *DEFN* applied by cohesion

Case 3: $\gamma=0$, $c=c_m=0$, q, $q_m\neq0$,

Considering the forces acting on the wedge *AKE*, as shown in Fig.7, and the moment equilibrium of all the forces about the focus *A*, we get

$$P_{\mathrm{pq}}=\frac{\frac{1}{2}qr_{AJ}^{2}(1-k_{\mathrm{v}})+\frac{1}{2}k_{\mathrm{h}}qhr_{AJ}}{\frac{1}{2}r_{0}\cos\varphi} \tag{27}$$

where

$$q=\gamma\frac{lh\tan\varphi+\frac{1}{2}h^{2}}{l\tan\varphi+h}.$$

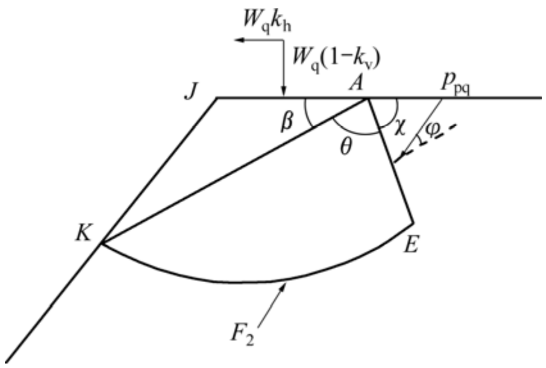

Fig.7 Diagram of *AJEK* applied with surcharge

The force F_2 must act through the point A, so its moment for A is zero.

Then considering the forces acting on the wedge *DEFN*, as shown in Fig.8, and the moment equilibrium of all the forces about the focus D, we get

$$P_{\mathrm{pmq}}=\frac{M_{\mathrm{q}m1}-M_{\mathrm{q}m2}+M_{\mathrm{q}m3}-M_{\mathrm{q}m4}-M_{\mathrm{q}m5}}{\frac{1}{2}r_{m0}\cos\varphi_{m}} \tag{28}$$

where

$$M_{\mathrm{q}m1}=\frac{1}{2}(1-k_{\mathrm{v}})\gamma hH_{m}^{2}\tan^{2}\alpha_{m},$$

$$M_{\mathrm{q}m2}=\frac{1}{2}k_{\mathrm{h}}\gamma h^{2}H_{m}\tan\alpha_{m},$$

$$M_{\mathrm{q}m3}=\frac{1}{2}E_{\mathrm{pmq}}H_{m},$$

$$M_{\mathrm{q}m4}=k_{\mathrm{v}}\gamma hH_{m}^{2}(\tan\alpha_{m}+\frac{1}{3}\tan\beta_{m})\tan\beta_{m},$$

$$M_{\mathrm{q}m5}=\frac{1}{2}k_{\mathrm{h}}\gamma h^{2}H_{m}^{2}\tan^{2}\beta_{m},$$

$$q_{m}=\gamma h,\ E_{\mathrm{pmq}}=\gamma H_{m}hK_{m}'.$$

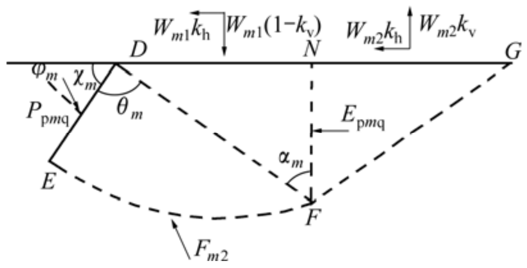

Fig.8 Diagram of *DEFN* applied with surcharge

The force F_{m2} must act through the point D, so its moment for D is zero.

3.3 Seismic bearing capacity

Substituting Eqn.(23)−(28) into the Eqn.(20), the ultimate seismic bearing capacity based on the unified strength theory can be expressed in the following form:

$$q_{\mathrm{ud}}=cN_{\mathrm{cd}}+qN_{\mathrm{qd}}+\frac{1}{2}\gamma BN_{\gamma\mathrm{d}} \tag{29}$$

where $N_{\mathrm{cd}}=\dfrac{P_{\mathrm{pc}}+P_{\mathrm{p}m\mathrm{c}}}{cB}+\dfrac{(c+c_{m})\sin\varphi\sin\varphi_{m}}{c\sin(\varphi+\varphi_{m})}$;

$N_{\mathrm{qd}}=\dfrac{P_{\mathrm{pq}}+P_{\mathrm{pmq}}}{\gamma hB}$; $N_{\gamma\mathrm{d}}=\dfrac{2P_{\mathrm{p}\lambda}+2P_{\mathrm{p}m\gamma}}{\gamma B^{2}}$.

4 Examples and comments

The angle χ should be assumed for the analysis of limit equilibrium. The trial and error method was applied to obtain the minimal coefficients of seismic bearing capacity.

1) Comparison of bearing capacity based on unified strength theory with that based on Mohr-Coulomb Theory. For a sloping ground, given $l/B=h/B=0.5$, $\varphi=45^\circ$, $\varphi_0=20^\circ$, $c_0=20$ kPa, $k=1.0$. Analysis shows that if the intermediate principal stress is considered, the coefficients of seismic and static bearing capacity (given $k_h=k_v=0.2$) are larger, for N_{cd} about 3%−27%, for N_{qd} about 4%−42%(see Fig.9(a)), and for $N_{\gamma d}$ about 34%−57%. The static bearing capacity increases by 17%−43% and the seismic one increases by 16%−40%(see Fig.9(b)).

2) Coefficients of the seismic bearing capacity and coefficients of static bearing capacity. Figs.10−12 show the comparison between coefficients of seismic and that of static bearing capacity. Given parameters of a slope as: $l/B=h/B=0.5$, $\varphi=45^\circ$, $\varphi_0=20^\circ$, 30°, 40°, $c_0=20.0$ kPa, $k=1.0$, $b=1.0$, considering seismic effect ($k_h=0.2$, $k_v=0.2$; $k_h=0.2$, $k_v=0.4$), through the variation of $m(=\tau/\tau_m)$ and initial friction angle φ, we can see that under the seismic loading, N_{cd} changes very little, while both N_{qd} and $N_{\gamma d}$ reduce to some extent. Meanwhile, the seismic bearing capacity is 10%−20% less than the static bearing capacity.

3) Effect of b on intermediate principal stress. Along with the increase of b, this reflects intermediate principal stress, the coefficients of seismic bearing capacity increases gradually. As far as the range of increase is concerned, $N_{\gamma d}$ is the most obvious, N_{qd} is in the second place, and comparatively, N_{cd} is the most insensitive (see Fig.13).

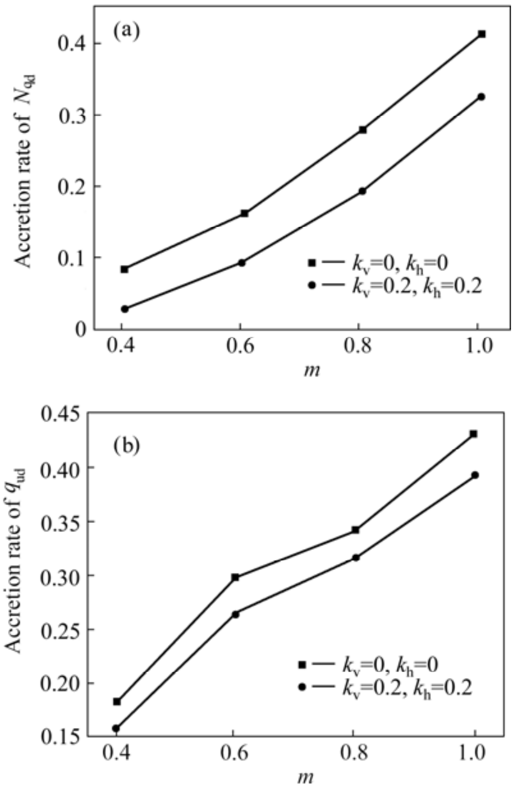

Fig.9 Comparison between bearing capacity by unified theory and bearing capacity by non-unified theory
Accretion rate: (a) N_{qd}; (b) q_{ud}

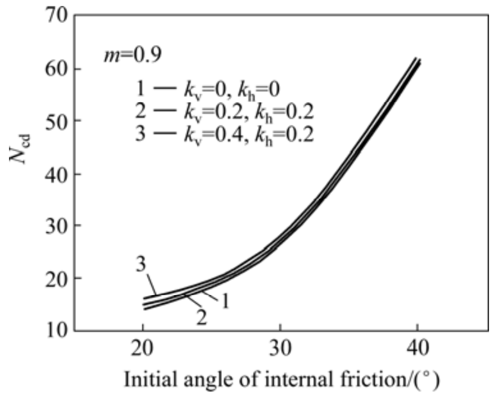

Fig.10 Variation of seismic bearing capacity N_{cd} with k_h and k_v

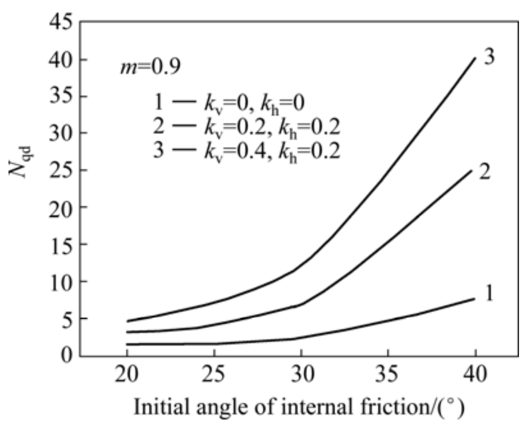

Fig.11 Variation of surcharge seismic bearing capacity coefficients N_{qd} with k_h and k_v

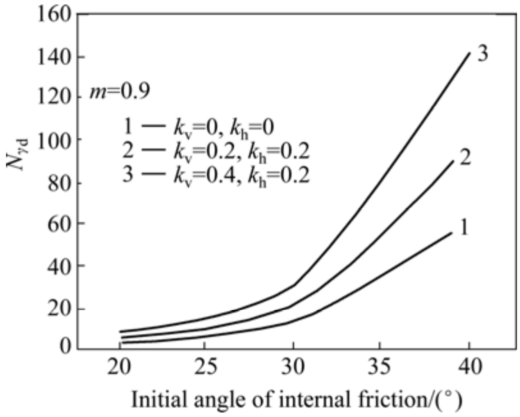

Fig.12 Variation of bearing capacity coefficients $N_{\gamma d}$ with k_h and k_v considering effect of self-weight

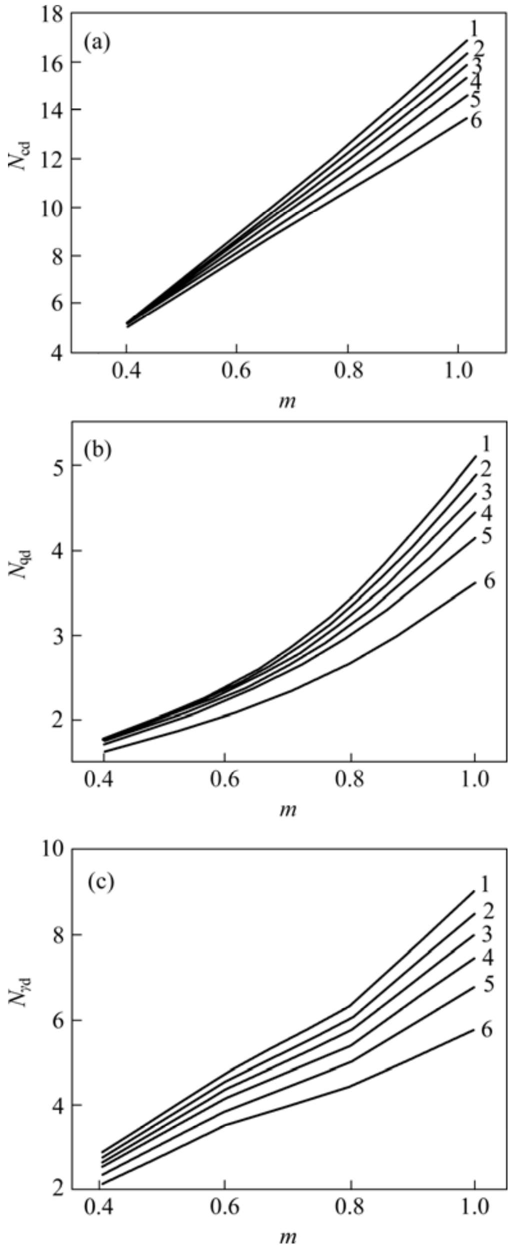

Fig.13 Seismic bearing capacity coefficients: (a) N_{cd}, (b) N_{qd} and $N_{\gamma d}$ affected by b at k_v=0.2 and k_h=0.2
1—b=1.0; 2—b=0.8; 3—b=0.6; 4—b=0.4; 5—b=0.2; 6—b=0

5 Conclusions

1) Based on the unified strength theory that takes the effect of intermediate principal stress into consideration, by supposing composite curved failure planes and employing pseudo-static approach and the method of limit equilibrium analysis, a new formula of seismic bearing capacity of slope foundation was obtained.

2) The static and seismic bearing capacity that takes the effect of intermediate stress into account is larger than that without considering the effect of intermediate stress; the static bearing capacity increases by 17%−43% and the seismic one increases by 16%−40%.

3) Under the seismic loading, N_{cd} changes very little, while both N_{qd} and $N_{\gamma d}$ reduce to some extent. Meanwhile, the seismic bearing capacity is 10%−20% less than the static bearing capacity.

4) Along with the increase of b that reflects intermediate principal stress, the coefficients of seismic bearing capacity increases gradually. As far as the range of increase lies concerned, $N_{\gamma d}$ is the most obvious, N_{qd} in the second place, and comparatively, N_{cd} is the most insensitive.

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