Computation of Seismic Bearing Capacity of Shallow Foundations with Prandtl Failure Mechanism by Using Pseudo-Dynamic Method

Ali Ramazan Borujerdí¹*, Morteza Jiryaei Sharahi²

1- M.Sc. Student, Department of Civil Engineering, Qom University of Technology, Iran.
2- Assistant Professor, Department of Civil Engineering, Qom University of Technology, Qom, Iran.

Received: 25 October 2017
Revised: 1 June 2018
Accepted: 12 July 2018
Printed: 10 September 2018

ABSTRACT: The appropriate evaluation of bearing capacity of shallow strip footing under seismic forces is an important area of research in earthquake geotechnical engineering. This paper presents a Pseudo-dynamic Method to compute the seismic bearing capacity of shallow strip footing resting on C-Φ soil using limit equilibrium analysis considering Prandtl failure mechanism. The bearing capacity factor ($N_{re}$) presents here for the concurrent resistance of unit weight, surcharge and cohesion, which is more practical to simulate the failure mechanism. The effect of soil friction angle (Φ), soil cohesion (C), shear wave and primary wave velocity ($V_s, V_p$) and horizontal and vertical seismic accelerations ($K_h,K_v$) are taken into account to evaluate the seismic bearing capacity of foundation. The results obtained from the present analysis are presented in both tabular and graphical non-dimensional form. Results are thoroughly compared with the existing values in the literature and the significance of the present methodology for designing the shallow strip footing is discussed.

Keywords: Prandtl Failure Mechanism, Limit Equilibrium Method, Pseudo-Dynamic Analysis, Shallow Foundations, Seismic Bearing Capacity.

1. Introduction

Determination of the seismic bearing capacity of shallow foundations against seismic loads has always been a great component of geotechnical engineering in seismic regions. The pioneering works in computing the bearing capacity in static condition were done by Prandtl, 1921; Terzaghi, 1943; Meyerhoff, 1957, 1963; Vesic, 1973; Saran and Agarwal, 1991 and many others. For foundation of structures built in seismically active areas, the demands to the sustain load and deformation during an earthquake will probably be the most severe in their design life. Thus the design of foundation in seismic areas needs special considerations compared to the static case. A number of researcher had analysed the seismic bearing capacity of shallow strip footings using pseudo-static approach with the help of different solution techniques such as method of slices, limit equilibrium,
method of stress characteristics an upper bound limit analysis. Budhu and Al-karni, 1993; Soubra, 1993, 1997, 1999; Richards et al. 1993; Choudhury and Subha Rao, 2005; Kumar and Ghosh, 2006; and many more had considered the effect of earthquake on the bearing capacity of a surface to a shallow strip footing under pseudo-static method using different approaches. However, in the pseudo-static method, the dynamic nature of earthquake loading is considered in a very approximate way without taking any effect of time and phase difference. To overcome this drawback, Steedman and Zeng, 1990; and Choudhury and Nimbalkar, 2005; developed the pseudo-dynamic solutions where the effects of both shear and primary waves as well as the amplification of excitation were considered during the earthquake along with the duration of earthquake and the period of lateral shaking to predict the seismic earth pressure behind the vertical retaining wall.

Recently, Ghosh, 2008; gives a solution of pseudo-dynamic bearing capacity of shallow strip footing resting on cohesionless soil using limit analysis method considering the Coulomb failure mechanism. But the solutions which were given for foundation resting on C-Φ soil, three different bearing capacity coefficients are suggested for three different failure mechanism. Here in this analysis, an attempt is made to solve this problem of pseudo dynamic bearing capacity of shallow strip footing resting on C-Φ soil considering the composite failure mechanism. A composite failure surface involving planar and log spiral surface is considered in the present analysis. To evaluate the bearing capacity under seismic loading condition, the simultaneous resistance of unit weight, surcharge and cohesion is taken into account. Results are presented in both tabular and graphical non-dimensional form including comparison with other available methods. Effects of wide range of variation of parameters like soil friction angle (Φ), cohesion factor (2C/γb), depth factor (Df/b) and horizontal and vertical seismic accelerations (Kh,Kv) along with primary wave and shear wave velocity on the pseudo-dynamic bearing capacity coefficient (Nγe) have been studied.

2. Proposed method

Here an attempt is made to give a formulation of pseudo-dynamic bearing capacity of a shallow strip footing resting on C-Φ soil using limit equilibrium method. The homogeneous soil of effective unit weight γ has Mohr-Coulomb characteristic C-Φ and can be considered as a rigid plastic body. Let us consider a shallow strip footing of width (b) resting below the ground surface at a depth of (Df) over which a load (PL) of column acts. For shallow foundation (Df ≤ b), the overburden pressure is idealized as a surcharge (q = γDf) which acts along the length of AF. The classical two-dimensional slip-line field obtained by Prandtl, 1921; is the traditional failure mechanism which has three regions such as active zone, passive zone and logarithmic radial-fan transition zone. In this composite failure mechanism, half of failure is assumed to occur along the surface ANDE, which is composed of a triangular elastic zone ABN, triangular passive Rankin zone ADE and in between them a log spiral radial shear zone ANDE shown in Figure 1. It is a log-sandwich mechanism that is defined by the angular parameters α1, α2. Figure 1 shows the detail free body diagram of elastic zone ABN and composite passive Rankin zone and log spiral shear zone ANDE respectively. The soil on the right side of the failure
plane AN gets partially mobilized and this is characterized by a mobilization factor m. The shear strength of the soil is expressed as
\[\tau = mc + \sigma \tan \phi_m\]  
(1)
\[\phi_m = \tan^{-1}(m \tan \phi)\]  
(2)

The radial log-spiral shearing zone AND is bounded by a log-spiral curve ND, where the equation for the curve in polar coordinates \((r, \theta)\) is \(r = r_0 e^{\theta \tan \phi}\). The Centre of this log-spiral ND is at point A and the radius \(r_0\) is the length of the line AN where:
\[r_0 = \frac{b \sin \alpha_2}{\sin(\alpha_1 + \alpha_2)}\]  
(3)

Figure 1. The failure mechanism used in the present analysis

In pseudo-dynamic analysis, a finite shear and primary wave velocity can be developed by assuming that the shear modulus \(G\) is constant with depth and the phase, not the magnitude of acceleration varies. In the present study, both shear wave velocity \(V_s\) and primary wave velocity \(V_p\) of the earthquake waves through soil medium are assumed to act within the soil mass during earthquake as shown in Figure 1. The analysis includes a period of lateral shaking, \(T\). Here, \(T = \frac{2\pi}{\omega}\) and
\[V_s = \sqrt{\frac{G}{\rho}}\]  
(4)
\[V_p = \sqrt{2G(1-v)/\rho(1-2v)}\]  
(5)

Where, \(\rho\), \(\mu\) and \(\omega\) are density, Poisson’s ratio and angular frequency respectively.

3. Seismic forces acting on the elastic and passive Rankin zone

The bearing capacity expression is developed by considering the equilibrium of elastic wedge ABN. The forces acting on the wedge include uniformly distributed column load.
$P_L$ on AB, horizontal and vertical inertia forces $Q_h$ and $Q_v$, earth pressure $P_p$ and $P_m$ and cohesion $C_a$ and $C_{am}$ acting on the sides BN and AN respectively. The mass of the thin elemental slice of thickness $dz$ at a depth of $z$ from the ground surface in elastic wedge ABN may be obtained as:

$$m_{ABN}(z) = \frac{\gamma}{g} \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right) (r_0 \sin \alpha_1 - z) dz$$

(6)

The total horizontal and vertical inertia force acting within the elastic zone can be expressed as follows,

$$Q_{h, ABN} = \gamma K_h \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right) \int_0^{r_0 \sin \alpha_1} (r_0 \sin \alpha_1 - z) \sin \omega \left( t - \frac{r_0 \sin \alpha_1 - z}{v_p} \right) dz$$

(7)

$$Q_{v, ABN} = \gamma K_v \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right) \int_0^{r_0 \sin \alpha_1} (r_0 \sin \alpha_1 - z) \sin \omega \left( t - \frac{r_0 \sin \alpha_1 - z}{v_p} \right) dz$$

(8)

Considering the vertical force equilibrium on the elastic wedge ABN,

$$P_L = \frac{P_p \cos (\alpha_1 - \Phi) + P_{pm} \cos (\alpha_2 - \Phi_m) - W + Q_v + c \sin \alpha_1 \sin \alpha_2 (1 + m)}{b} \sin(\alpha_1 + \alpha_2)$$

(9)

Similarly, as elastic zone the total horizontal and vertical inertia force acting within this zone can be expressed as follows,
\[
Q_{h2} = \frac{\gamma K_h}{\tan X_m} \int_0^{FD} (FD - z) \sin \omega \left( t - \frac{FD - z}{V_s} \right) dz
\]

(10)

\[
Q_{v2} = \frac{\gamma K_v}{\tan X_m} \int_0^{FD} (FD - z) \sin \omega \left( t - \frac{FD - z}{V_p} \right) dz
\]

(11)

---

**Figure 3. The forces acting on passive Rankin zone**

The mass of a thin element of the passive Rankin zone at depth \( z \) is as follow.

\[
m_2(z) = \frac{\gamma (FD - z)}{g \tan X_m} d
\]

(12)

And the horizontal and vertical inertia force acting within this \( i^{th} \) slice can be expressed as follows:

\[
(Q_{h1})_i = \frac{2\pi d_\theta_m \gamma K_h}{360^\circ \sin^2(\alpha + (i - 0.5)d_\theta_m)} \int_0^{H_i} z_i \sin \omega \left( t - \frac{H_i - z_i}{V_s} \right) (dz)_i
\]

(13)

\[
(Q_{v1})_i = \frac{2\pi d_\theta_m \gamma K_v}{360^\circ \sin^2(\alpha + (i - 0.5)d_\theta_m)} \int_0^{H_i} z_i \sin \omega \left( t - \frac{H_i - z_i}{V_p} \right) (dz)_i
\]

(14)

The Mass of strip on the \( i^{th} \) slice of the log-spiral zone ADN,

\[
m_1(z)_i = \frac{\gamma}{g} \frac{2\pi d_\theta_m}{360^\circ \sin^2(\alpha + (i - 0.5)d_\theta_m)} \int_0^{H_i} z_i (dz)_i
\]

(15)
4. Computation of passive earth pressures $P_{pm_y}, P_{pm_q}$ and $P_{pm_c}$

The passive earth pressure $P_{pm_y}$, $P_{pm_q}$ and $P_{pm_c}$ are determined by taking the moments of all the forces about the center, O, of the logarithmic spiral. For the determination of the passive pressures $P_{pm_y}$, $P_{pm_q}$ and $P_{pm_c}$ forces involved in the equilibrium of soil mass ANDE are given in (16)-(33), as shown in Table 1.

Table 1. Force and Moment Equations

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Force</th>
<th>Moment of the forces about center of log spiral A (Fig. 3)</th>
<th>Eq. No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$W_1$</td>
<td>$M_{W_1} = \frac{\gamma}{2} b^2 r_0 \frac{2 \sin^2 \alpha_1}{3 \sin^2 (\alpha_1 + \alpha_2)} \frac{1}{(1 + 9 \tan^2 \Phi_m)} \left[ e^{3 \Theta_m \tan \Phi_m} (3 \tan \Phi_m \sin \theta_m - \cos \theta_m + 1) \sin \alpha_2 \right]$</td>
<td>(16)</td>
</tr>
<tr>
<td>2</td>
<td>$W_2$</td>
<td>$M_{W_2} = \frac{\gamma}{2} b^2 r_0 \frac{\sin^2 \alpha_1}{3 \sin^2 (\alpha_1 + \alpha_2)} \left[ 2 \sin^2 \gamma_m \cos^2 \frac{\gamma}{2} \frac{\sin \gamma_m e^{3 \Theta_m \tan \Phi_m}}{\sin \alpha_2} \right]$</td>
<td>(17)</td>
</tr>
<tr>
<td>3</td>
<td>$Q_{v1}$</td>
<td>$M_{Q_{v1}} = \frac{\gamma}{2} b^2 r_0 \frac{\sin^2 \alpha_1}{\sin^2 (\alpha_1 + \alpha_2)} \frac{2 \eta d \theta_m}{3} \frac{\sin \phi_m}{(1 + 9 \tan^2 \Phi_m)} (e^{2 \Theta_m \tan \Phi_m - 1}) \left[ \frac{\eta}{2 \pi H_i} \left( \sin 2 \pi \frac{t}{T} - \sin 2 \left( \frac{t}{T} - \frac{H}{\lambda} \right) \right) \right.$</td>
<td>(18)</td>
</tr>
<tr>
<td>4</td>
<td>$Q_{h1}$</td>
<td>$M_{Q_{h1}} = \frac{\gamma}{2} b^2 r_0 \frac{\sin^2 \alpha_1}{\sin^2 (\alpha_1 + \alpha_2)} \frac{2 \lambda d \theta_m}{3} \frac{\sin \phi_m}{(1 + 9 \tan^2 \Phi_m)} (e^{2 \Theta_m \tan \Phi_m - 1}) \left[ \frac{\lambda}{2 \pi H_i} \left( \sin 2 \pi \frac{t}{T} - \sin 2 \left( \frac{t}{T} - \frac{H}{\lambda} \right) \right) \right.$</td>
<td>(19)</td>
</tr>
<tr>
<td>5</td>
<td>$Q_{v2}$</td>
<td>$M_{Q_{v2}} = \frac{\gamma}{2} b^2 r_0 \frac{\sin^2 \alpha_1}{\sin^2 (\alpha_1 + \alpha_2)} \frac{2 K_{m} e^{3 \Theta_m \tan \Phi_m} \eta}{3 \pi F_D} \frac{\sin^2 H_m}{(1 + 9 \tan^2 \Phi_m)} (e^{2 \Theta_m \tan \Phi_m - 1}) \left[ \cos 2 \pi \left( \frac{t}{T} - \frac{F_D}{\eta} \right) \right.$</td>
<td>(20)</td>
</tr>
<tr>
<td>6</td>
<td>$Q_{h2}$</td>
<td>$M_{Q_{h2}} = \frac{\gamma}{2} b^2 r_0 \frac{\sin^2 \alpha_1}{\sin^2 (\alpha_1 + \alpha_2)} \frac{K_{m} e^{3 \Theta_m \tan \Phi_m} \lambda}{3 \pi F_D} \frac{\sin^2 H_m}{(1 + 9 \tan^2 \Phi_m)} (e^{2 \Theta_m \tan \Phi_m - 1}) \left[ \cos 2 \pi \left( \frac{t}{T} - \frac{F_D}{\lambda} \right) \right.$</td>
<td>(21)</td>
</tr>
<tr>
<td>7</td>
<td>$C$</td>
<td>$M_c = \frac{\gamma}{2} b^2 r_0 \frac{2 \alpha \sin \alpha_1 (e^{2 \Theta_m \tan \Phi_m - 1})}{y b \sin (\alpha_1 + \alpha_2) \tan \Phi_m}$</td>
<td>(22)</td>
</tr>
<tr>
<td>8</td>
<td>$C_{am}$</td>
<td>Zero</td>
<td>(23)</td>
</tr>
<tr>
<td>9</td>
<td>$F_m$</td>
<td>Zero</td>
<td>(24)</td>
</tr>
</tbody>
</table>
Computation of Seismic Bearing Capacity of Shallow Foundations with Prandtl …

Continue of Table 1. Force and Moment Equations

<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Force</th>
<th>Moment of the forces about center of log spiral A (Fig. 3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>( q )</td>
<td>( M_q = \frac{\gamma}{2} b^2 r_0 \frac{D_f}{b} \frac{\sin \alpha_1}{\sin (\alpha_1 + \alpha_2)} (e^{2 \theta_m \tan \phi_m}) \cos^2 X_m ) (25)</td>
</tr>
<tr>
<td>11</td>
<td>( P_{pmc} )</td>
<td>( M_{ppmc} = P_{pmc} \frac{1}{2} r_0 \cos \Phi_m ) (26)</td>
</tr>
<tr>
<td>12</td>
<td>( P_{pmy} )</td>
<td>( M_{pmy} = P_{pmy} \frac{2}{3} r_0 \cos \Phi_m ) (27)</td>
</tr>
<tr>
<td>13</td>
<td>( P_{pq} )</td>
<td>( M_{pq} = P_{pq} \frac{1}{2} r_0 \cos \Phi_m ) (28)</td>
</tr>
<tr>
<td>14</td>
<td>( P_{pmy \text{ Static}} )</td>
<td>( M_{pmy \text{ Static}} = \frac{\gamma}{2} b^2 r_0 \frac{2}{3} \sin^2 (\alpha_1 + \alpha_2) (e^{2 \theta_m \tan \phi_m}) \sin^2 X_m \cos X_m ) (29)</td>
</tr>
<tr>
<td>15</td>
<td>( P_{pmy \text{ Decrement}} )</td>
<td>( M_{pmy \text{ Decrement}} = \frac{\gamma}{2} b^2 r_0 \frac{1}{3} \sin^2 (\alpha_1 + \alpha_2) (e^{2 \theta_m \tan \phi_m}) \sin^2 X_m \cos X_m (K_p) \text{Decrement} ) (30)</td>
</tr>
<tr>
<td>16</td>
<td>( P_{pq \text{ Static}} )</td>
<td>( M_{pq \text{ Static}} = \frac{\gamma}{2} b^2 r_0 \frac{1}{2} \sin (\alpha_1 + \alpha_2) (e^{2 \theta_m \tan \phi_m}) \sin 2 X_m ) (31)</td>
</tr>
<tr>
<td>17</td>
<td>( P_{pq \text{ Decrement}} )</td>
<td>( M_{pq \text{ Decrement}} = \frac{\gamma}{2} b^2 r_0 \frac{1}{3} \sin (\alpha_1 + \alpha_2) (e^{2 \theta_m \tan \phi_m}) \sin 2 X_m (K_p) \text{Decrement} ) (32)</td>
</tr>
<tr>
<td>18</td>
<td>( P_{pmy \text{ Static}} )</td>
<td>( M_{pmy \text{ Static}} = \frac{\gamma}{2} b^2 r_0 \frac{2}{3} \sin^2 (\alpha_1 + \alpha_2) (e^{2 \theta_m \tan \phi_m}) \sin^2 X_m ) (33)</td>
</tr>
</tbody>
</table>

Now, taking moment equilibrium due to weight, surcharge and cohesion about the center of the logarithmic spiral we get \( P_{pmy}, P_{pq} \) and \( P_{pmc} \) respectively. Total moment due to weight equal to zero:

\[
M_{pmy} = M_{w_1} + M_{w_2} + M_{pmy \text{ Static}} - M_{q_1} - M_{q_2} - M_{qv_1} - M_{qv_2} - M_{pmy \text{ Decrement}}
\] (34)

Total moment due to surcharge equal to zero

\[
M_{pq} = M_q + M_{pq \text{ Static}} - M_{pq \text{ Decrement}}
\] (35)

Total moment due to cohesion equal to zero

\[
M_{pmc} = M_c + M_{pmc \text{ Static}}
\] (36)

The values of passive earth pressure \( P_{py}, P_{pq} \) and \( P_{pc} \) can be obtained by substituting the angle \( \Phi_m \) by \( \Phi \) and changing the wedge angle \( \alpha_2 \) to \( \alpha_1 \) and \( \alpha_1 \) to \( \alpha_2 \) in the equation of \( P_{pmy}, P_{pq} \) and \( P_{pmc} \):

\[
P_p = P_{py} + P_{pq} + P_{pc}
\] (37)

\[
P_pm = P_{pmy} + P_{pq} + P_{pmc}
\] (38)

Putting all the values of \( W, Q_v, P_p \) and \( P_pm \) in equation (9) we get ultimate vertical failure load \( P_L \)

\[ P_L = \frac{1}{2} \gamma b N_{ye} \]  \hspace{1cm} (39)

Where, \( N_{ye} \) = single bearing capacity coefficient.

5. Results and Discussions

The values of the bearing capacity coefficients are calculated by optimizing \( N_y \) with respect to \( \theta_m, \theta \) and \( t/T \) using the simplex optimization method with \( H_l/\lambda, FD/\lambda, r_0 \sin \alpha_1/\lambda = H/\lambda = 0.3 \) and \( H_l/\eta, FD/\eta, r_0 \sin \alpha_1/\eta = H/\eta = 0.16 \).

\[
\begin{array}{cccccc}
\Phi & 2c/\gamma b & 0.25 & 0.5 & 0.75 & 1 \\
\hline
20^0 & 0 & 3.759 & 7.548 & 11.764 & 14.369 \\
 & 0.25 & 5.943 & 9.361 & 12.658 & 15.478 \\
 & 0.5 & 7.586 & 11.367 & 13.764 & 16.758 \\
30^0 & 0 & 25.657 & 34.567 & 42.974 & 50.258 \\
 & 0.25 & 29.067 & 34.625 & 42.258 & 50.587 \\
 & 0.5 & 32.196 & 40.294 & 47.687 & 55.794 \\
40^0 & 0 & 135.453 & 160.864 & 186.349 & 211.864 \\
 & 0.25 & 140.063 & 166.438 & 190.594 & 215.374 \\
 & 0.5 & 145.648 & 171.289 & 196.467 & 219.564 \\
\end{array}
\]

Table 2. Single bearing capacity coefficient \( (N_{ye}) \) for static case

\[
\begin{array}{cccccc}
\Phi & 2c/\gamma b & 0.25 & 0.5 & 0.75 & 1 \\
\hline
20^0 & 0 & 2.364 & 6.175 & 9.431 & 12.308 \\
 & 0.25 & 4.891 & 8.311 & 11.601 & 14.178 \\
 & 0.5 & 6.159 & 9.609 & 12.891 & 15.441 \\
30^0 & 0 & 22.086 & 29.924 & 37.003 & 43.871 \\
 & 0.25 & 25.274 & 32.693 & 39.694 & 46.521 \\
 & 0.5 & 28.212 & 35.435 & 42.376 & 49.168 \\
40^0 & 0 & 116.93 & 140.29 & 163.17 & 185.72 \\
 & 0.25 & 122.52 & 145.75 & 168.55 & 191.05 \\
 & 0.5 & 128.64 & 151.21 & 173.92 & 196.37 \\
\end{array}
\]

Table 3. Pseudo-dynamic bearing capacity coefficient \( (N_{ye}) \) for \( K_h = 0.1 \)

\[
\begin{array}{cccccc}
\Phi & 2c/\gamma b & 0.25 & 0.5 & 0.75 & 1 \\
\hline
20^0 & 0 & 1.964 & 5.287 & 8.673 & 11.634 \\
 & 0.25 & 4.492 & 7.786 & 11.002 & 13.614 \\
 & 0.5 & 5.856 & 9.172 & 12.446 & 14.929 \\
30^0 & 0 & 20.562 & 28.425 & 35.256 & 41.841 \\
 & 0.25 & 24.022 & 31.229 & 37.954 & 44.492 \\
 & 0.5 & 27.024 & 33.984 & 40.632 & 47.123 \\
40^0 & 0 & 20.562 & 28.425 & 35.256 & 41.841 \\
 & 0.25 & 24.022 & 31.229 & 37.954 & 44.492 \\
 & 0.5 & 27.024 & 33.984 & 40.632 & 47.123 \\
\end{array}
\]

The optimum values of \( N_y \) are represented as \( N_{ye} \). These pseudo-dynamic passive resistance coefficients \( (N_{ye}) \) are presented in Tables 2-4.

The results are presented in the form of tables and graphs. Variations of parameters considered in the present analyses are as follows.
Table 4. Pseudo-dynamic bearing capacity coefficient ($N_{ye}$) for $K_h = 0.2$

<table>
<thead>
<tr>
<th>$\phi$</th>
<th>$2c/\gamma b$</th>
<th>0.25</th>
<th>0.5</th>
<th>0.75</th>
<th>1</th>
<th>0.25</th>
<th>0.5</th>
<th>0.75</th>
<th>1</th>
<th>0.25</th>
<th>0.5</th>
<th>0.75</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>20°</td>
<td>0</td>
<td>0.854</td>
<td>3.555</td>
<td>6.252</td>
<td>8.932</td>
<td>-</td>
<td>2.305</td>
<td>4.651</td>
<td>6.996</td>
<td>0.942</td>
<td>2.887</td>
<td>4.829</td>
<td>6.761</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>2.875</td>
<td>5.577</td>
<td>8.269</td>
<td>10.842</td>
<td>1.948</td>
<td>4.297</td>
<td>6.642</td>
<td>8.989</td>
<td>2.871</td>
<td>4.806</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>4.897</td>
<td>7.598</td>
<td>10.279</td>
<td>12.337</td>
<td>3.935</td>
<td>6.283</td>
<td>8.629</td>
<td>-</td>
<td>8.358</td>
<td>16.266</td>
<td>22.878</td>
<td>27.644</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>20.244</td>
<td>27.057</td>
<td>33.161</td>
<td>39.075</td>
<td>16.733</td>
<td>23.917</td>
<td>29.447</td>
<td>34.724</td>
<td>16.933</td>
<td>23.394</td>
<td>28.171</td>
<td>32.744</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>23.364</td>
<td>29.728</td>
<td>35.749</td>
<td>41.626</td>
<td>20.643</td>
<td>26.635</td>
<td>32.033</td>
<td>37.266</td>
<td>17.008</td>
<td>23.937</td>
<td>28.831</td>
<td>32.744</td>
</tr>
<tr>
<td>40°</td>
<td>0</td>
<td>97.776</td>
<td>118.91</td>
<td>139.47</td>
<td>159.04</td>
<td>84.672</td>
<td>103.34</td>
<td>121.36</td>
<td>139.04</td>
<td>71.008</td>
<td>87.337</td>
<td>102.83</td>
<td>117.91</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>103.27</td>
<td>124.24</td>
<td>144.75</td>
<td>164.87</td>
<td>90.243</td>
<td>108.74</td>
<td>126.62</td>
<td>144.21</td>
<td>76.779</td>
<td>92.755</td>
<td>108.16</td>
<td>123.09</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>108.72</td>
<td>129.56</td>
<td>149.84</td>
<td>170.06</td>
<td>95.742</td>
<td>114.02</td>
<td>131.84</td>
<td>149.38</td>
<td>82.392</td>
<td>98.117</td>
<td>113.33</td>
<td>128.25</td>
</tr>
</tbody>
</table>

- $\Phi = 20, 30, \text{ and } 40°$
- $K_h = 0.1, 0.2, \text{ and } 0.3$
- $K_v = 0, 0.5K_h, \text{ and } K_h$
- $2c/\gamma b = 0, 0.25, \text{ and } 0.5$
- $D_f/b = 0.25, 0.5, 0.75$

6. Effect of soil friction angle ($\Phi$)

Effect of soil friction angle ($\Phi$) on the normalized seismic bearing capacity factor ($N_{ye}$) is shown in Fig. 4. It is observed that the seismic bearing capacity factor increases with increase in soil friction angle.

![Figure 4. Effect of soil friction angle ($\Phi$) on the seismic bearing capacity factor $N_{ye}$ for different values of $K_h$ with ($\Phi = 20°, 30°, 40°$), $2c/\gamma b = 0.25$, $D_f = 0.5$ and $K_v = K_h/2$.](image-url)

Due to increase in $\Phi$, the internal resistance of the soil particles will be increased which resembles the fact that increase in seismic bearing capacity factor.

7. Effect of cohesion factor ($2c/\gamma b$)

Effect of cohesion factor ($2c/\gamma b$) on the normalized seismic bearing capacity factor ($N_{ve}$) is shown in Fig. 5. It is observed that the seismic bearing capacity factor increases with increase in cohesion factor.

Due to increase in cohesion, seismic bearing capacity factor will be increased as increase in cohesion causes increase in intermolecular attraction among the soil particle which offers more resistance against shearing failure of foundation.

8. Effect of depth of foundation ($D_f/b$)

Effect of depth of foundation ($D_f/b$) on the normalized seismic bearing capacity factor ($N_{ve}$) is shown in Figure 6. It is observed that the seismic bearing capacity factor increases with increase in depth factor.

Due to increase in depth factor ($D_f/b$) surcharge weight increases which increases the passive resistance and hence increase in seismic bearing capacity factor.
9. Effect of vertical and horizontal seismic accelerations coefficient \((K_v, K_h)\)

Effect of seismic accelerations \((K_v, K_h)\) on the normalized seismic bearing capacity factor \(N_{ye}\) is shown in Figure 7. It is observed that the seismic bearing capacity factor decreases with increase in vertical and horizontal seismic accelerations.
Due to increase in seismic acceleration, the disturbance in the soil particles increases, which allows more soil mass to participate in the vibration and hence decrease its resistance against bearing capacity.

10. Comparison of Result

Table 5 shows the comparison of seismic bearing capacity coefficient ($N_{yre}$) for different values of $K_v$ and $K_h$ with available seismic analysis. It has been seen that the results obtained from the present analysis is between the pseudo-static analysis (Budhu and Al-Karni, 1993; Choudhury and Subba Rao, 2005) and pseudo-dynamic limit analysis (Ghosh, 2008). It has been seen that the results obtained from the present analysis is in lesser than pseudo-static analysis such as Budhu and Al-karni, 1993; Choudhury and Subba Rao (2005); Soubra (1997) ($M_1$ and $M_2$). In this plot, one pseudo-dynamic limit analysis, Ghosh (2008) also compared which shows the present analysis gives the lesser values of bearing capacity coefficients in comparison to Ghosh’s method.

The reason behind it limit analysis method is used considering the linear failure surface in Ghosh’s method (2008) whereas, in the present analysis limit equilibrium method is used considering composite failure surface.

Table 5. Comparison of Seismic Bearing Capacity Factor ($N_{yre}$) for Different Values of $K_v$ and $K_h$

<table>
<thead>
<tr>
<th>$K_h$</th>
<th>Present study ($H/\lambda = 0.3$ and $H/\eta = 0.16$)</th>
<th>Ghosh (2008)</th>
<th>Budhu and Al-Karni (1993)</th>
<th>Choudhury and Subba Rao (2005)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_v = 0.5K_h$</td>
<td>$K_v = 1.0K_h$</td>
<td>$K_v = 0.5K_h$</td>
<td>$K_v = 1.0K_h$</td>
</tr>
<tr>
<td>0.1</td>
<td>6.91</td>
<td>6.34</td>
<td>20.39</td>
<td>20.04</td>
</tr>
<tr>
<td>0.2</td>
<td>2.55</td>
<td>1.87</td>
<td>9.98</td>
<td>8.82</td>
</tr>
<tr>
<td>0.3</td>
<td>0.71</td>
<td>0.17</td>
<td>3.85</td>
<td>2.35</td>
</tr>
<tr>
<td>0.4</td>
<td>0.07</td>
<td>-</td>
<td>0.82</td>
<td>-</td>
</tr>
</tbody>
</table>

11. Conclusions

By considering pseudo-dynamic approach, the effect of shear wave and primary wave velocities travelling through the soil layer and the time and phase difference along with the horizontal and vertical seismic acceleration are studied. A mathematical model is suggested and using limit equilibrium method for simultaneous resistance of unit weight, surcharge and cohesion is evaluated. A composite failure mechanism which includes both planer and log spiral zone is considered here to develop this mathematical model for the shallow strip footing resting on C-Φ soil. Optimization of single bearing capacity coefficient is done and results are presented in tabular non-dimensional form. The effect of various parameters such as soil friction angle (Φ), seismic accelerations (K_v, K_h), cohesion factor (2c/γb), depth factor (D_f/b) are studied here. It is seen that, the pseudo-dynamic bearing capacity coefficient (N_ye) increases with the increase in Φ, 2c/γb, D_f/b and m but it decreases with the increase in horizontal and vertical seismic acceleration (K_v, K_h). The values obtained from the present analysis are thoroughly compared with available pseudo-static analysis as well as pseudo-dynamic analysis and it is seen that the values obtained from the present study are in the lower side in comparison to the available analysis.

Notification

2c/γb = Cohesion factor
B = Width of the footing
C = Cohesion of soil
Ca = Adhesion of soil
D_f = Depth of footing below ground surface
D_f/b = Depth factor
$F = \text{Reaction force}$

$g = \text{Acceleration due to gravity, } G = \text{Shear modulus of soil}$

$K_v, K_h = \text{Horizontal and vertical seismic accelerations}$

$m = \text{mobilization factor}$

$N_{oe} = \text{Optimized single seismic Bearing capacity coefficient}$

$P_t = \text{uniformly distributed column load}$

$P_r = \text{Earth pressure}$

$q = \text{Surcharge loadings}$

$r_0, r = \text{Initial and Final radius of the log-spiral zone, (i.e. BE and BD) respectively.}$

$t = \text{Time of vibration, } T = \text{Period of lateral shaking}$

$V_p = \text{Primary wave velocity, } V_s = \text{Shear wave velocity}$

$\alpha_1, \alpha_2 = \text{Base angles of triangular elastic zone under the foundation}$

$\beta = \text{Angle that makes the log spiral part in Log Spiral mechanism}$

$\gamma = \text{Unit weight of soil medium}$

$\rho = \text{Mass density of the soil medium, } \nu = \text{Poisson’s ratio of the soil medium}$

$\Phi = \text{Angle of internal friction of the soil, } \omega = \text{Angular frequency}$

### References


Computation of Seismic Bearing Capacity of Shallow Foundations with Prandtl...