Seismic bearing Capacity of strip footing embedded in slope situated below water table

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Abstract. An attempt is made to give a formulation of the pseudo-static bearing capacity coefficient of a shallow strip footing embedded in slope in c-φ nature of the soil in terms of a single coefficient ($N_{\gamma e}$) using limit equilibrium method which is subjected to groundwater flow. Failure surface is assumed linearly varying with cohesion, surcharge and unit weight of the soil. Iteration technique has been applied to optimize the solution. The ultimate bearing capacity equation was derived as a function of different properties of soils and footing i.e; width of footing, depth of footing, the cohesion of soil, unit weight of soil, depth of water table. A various parametric study has been studied to show the variation of bearing capacity coefficient with different parameters. Design chart of table has been shown for various range of parameters.

Keywords: Bearing capacity, Pseudo-Static, Water Table, Limit Equilibrium, c-φ soil

1 Introduction

The very first researches were done based on the static method of analysis in which bearing capacity coefficients were calculated on the basis of static loads on the footings and the weight of the soil mass in both active and passive conditions. The critical bearing capacity theories started from Rankine (1885), Prandtl (1921), Terzaghi (1943), Saran, et al (1989) and many others who have extensively studied the bearing capacity of shallow footings for static loading case. Terzaghi bearing capacity Theory (1943) was the first general theory for the bearing capacity of soils under a strip footing. The seismicity effect was not considered in the analysis and hence the name static. Meyerhof and Adams (1968), Rowe and Devis (1982 a,b), Subha Rao and Kumar (1994) worked on the ultimate uplift capacity of foundations under static condition by using different methods of analysis (limit equilibrium, non-linear finite element method, method of characteristics).The effects of dynamic loadings like seismic forces were not considered in the static analysis. Severe earthquakes such as the Kobe earthquake (1995), Santa Barbara (1925), Nigata (1964), Loma Prieta (1989) have earmarked the necessity of seismic design of structures. The dynamic loading of the earthquake has caused catastrophic effects due to foundation failure as well as anchor failure, creating an urgent need for seismic design of foundations and anchors under different conditions it can ve subjected to. Since a dynamic load is repetitive in nature, there is a need to determine the displacement of the foundation due to earthquakes and their damage potential.

In Pseudo-Static analysis, the seismic loadings are considered to be as equivalent inertia forces i.e., the weight of the wedge is multiplied with the acceleration coefficient (horizontal and vertical) and the inertia forces are found out on the basis of the Static equilibrium considerations.

Mononobe and Okabe (M-O) (1929) was the pioneer in the inclusion of “seismicity” in the design of structures (in this case retaining walls). IS 1893:1984 has also adopted the M-O method for the determination of seismic active and passive earth pressure behind the retaining walls. Among the limited literature available on the seismic bearing capacity, the earliest is the Meyerhof’s (1963) method, where the seismic forces were considered as inclined pseudo-static loads applied at the structure. Then, Sarma and lossifelis (1990), Richards et al. (1993), Buddhu and Al-Karni (1993) and Kumar and Kumar (2003) considered the seismic forces both on the structure and on the supporting soil mass which was not considered by Meyerhof. Researchers like Sarma and lossifelis (1990), Buddhu and Al-Karni (1993), Richards et al. (1993), Dormieux and Pecker (1995), Paolucci and Pecker (1997), Soubra (1997, 1999) , Kumar and Rao (2002) , Kumar (2003), and Choudhury and Subha Rao (2005) had studied the seismic bearing capacity of shallow footings for horizontal ground. But the study for the sloping ground is very limited. Sawada et al. (1994), Sarma (1999) and Askari and Farzaneh (2003) had given the solution for seismic bearing capacity of shallow foundations near the sloping ground, again some works for surface footing on the sloping ground was carried out by Zhu (2000), Kumar and Kumar (2003) , Kumar and Rao (2003), by limit equilibrium analysis, method of characteristic etc. but for foundations embedded in sloping ground research is still limited. Choudhury and Rao (2006), Chakraborty and Kuamr (2014) determined the seismic bearing capacity of a shallow foundation embedded in a sloping ground surface by using the theorems of limit equilibrium method and limit analysis in conjunction with finite elements and non-linear operations respectively. Larkin (2006) presented a method of assessing the probability of failure of shallow foundations in saturated fine-grained soil under multi directional seismic loading. Then Massiah and Soubra (2008) presented a reliability-based approach for the analysis and design of a shallow strip footing subjected to a vertical load with or without pseudo-static seismic loading. The computation of bearing capacity of foundations in the presence of groundwater flow is not straightforward. Because footings are generally impervious, the ground flow patterns beneath and around the foundation base may experience a change attributable to the construction of the footing. The effect of groundwater flow has been given less consideration in the literature. Vary recently, the bearing capacity of foundations subject to groundwater flow has been presented by Veiskaramiand Kumar (2012) and Kumar and Chakraborty (2013) by which the ultimate bearing capacity of strip foundations were calculated subjected to horizontal groundwater flow with the help of the stress characteristics method and lower bound finite element method respectively.

2 Method of Analysis

A shallow strip footing of width (B) resting below the ground surface at a depth of (D_f) over which a load (P_L) acts. For shallow foundation ($D_f \leq B$), the overburden pressure is idealized as a triangular surcharge load over the slope line which acts about a length EY at a slope angle of i. The failure surface has two main regions – the active and the passive wedge and thereby is assumed to be a simple Coulomb failure mechanism as shown in (Fig. 1). The detailed free body diagram of active zone AMC and passive zone MCF respectively is shown in (Fig.2). The water table is taken at a depth of $D_w$from the ground level (1st case) see (Fig.1). And the water table is taken at a depth of $D_w$from the base of the footing (2nd case) see (Fig.3, Fig.4).H is the depth of the failure wedge from the base of the footing, h is the depth from the level the level of the water table up to the end of the failure zone and H is the total depth of the failure mechanism from the ground level to the end of the Failure zone.

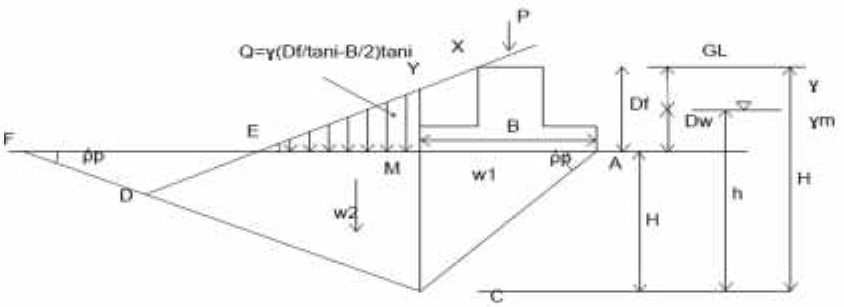

Fig. 1. 1st condition (Water table above the base of footing)

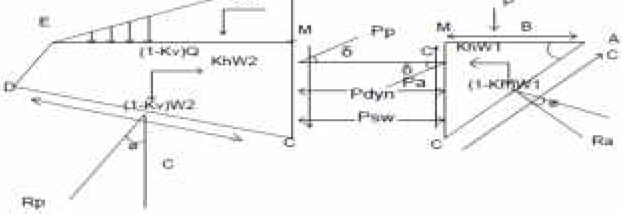

Fig. 2. Free body diagram of forces under Pseudo-static approach

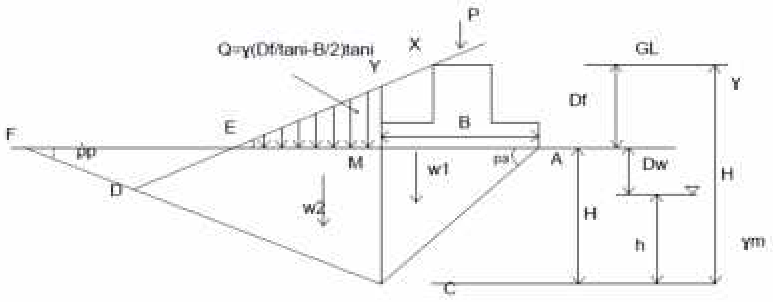

Figure 3. 2nd condition (Water table below the base of the footing)

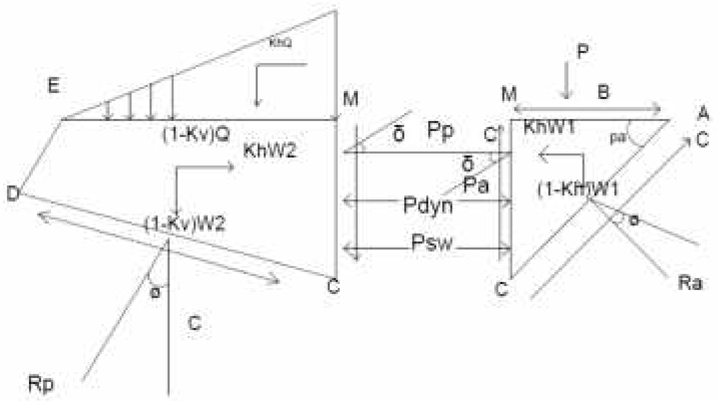

Figure 4. Free body diagram of forces under Pseudo-static approach

The hydrostatic force due to the presence of the water is given by :

$$P_{stat} = \frac{\gamma_w}{2} h^2 \tag{1}$$

Which acts at a height of h/3 from the base of the system.
As given by Ebeling and Morrison (1992), is replaced by such as:

$$¥_{we} = ¥_w + \left(\overline{¥} - ¥_w\right) r_u \qquad (2)$$

The hydrostatic force i.e. under seismic condition is calculated by using Westergaard's approach(1993):

$$P_{dyn} = \frac{7}{12} k_h ¥_w h^2 \qquad (3)$$

Which acts at a height of 0.4h from the base of the system.

Pseudo-Static analysis:-

1st condition (water table above the base of the footing)

Active region:

Let width of the footing AM = B.

The angle of inclination of the active wedge with the horizontal is $\check{}_a$ and therefore the height of the wall MC = B tan(ρ_a)which is considered as H say.

Applying the limit equilibrium conditions in the triangular region AMC ie,summation of horizontal forces and vertical forces equal to zero, we get :

$$\sum H = 0 \Rightarrow P_A \cos ¢ = R_A \sin\left(°_a - \Phi\right) - \frac{cB}{\cot °_a} \cot °_a + \left(W_1 + P\right) k_h - P_{hyd} - P_{sw} \qquad (4)$$

$$\sum V = 0 \Rightarrow P_A \sin ¢ = -R_A \cos\left(°_a - \Phi\right) + \left(W_1 + P\right)\left(1 - k_v\right) - 2cB \tan °_a \qquad (5)$$

$$P_A = f\left(k_h, k_v, °_a, ¤, ¢, D_w\right) \qquad (6)$$

Passive region :

Applying the limit equilibrium condition in the polygonal region MCDE i.e., summation of horizontal forces and vertical forces equal to zero we get:

$$\sum H = 0 \Rightarrow P_P \cos ¢ = R_P \sin\left(°_p + \Phi\right) + \frac{c \cos °_p}{\sin °_p} \times B \times \left[\tan °_a - \left(\frac{\tan °_a}{\tan °_p} - \frac{n}{\tan i} + \frac{1}{2}\right) \times \left(\frac{\sin °_p \sin i}{\sin(i+p)}\right)\right] + \left(W_2 + Q\right) k_h - P_{hyd} - P_{sw} \qquad (7)$$

$$\sum V = 0 \Rightarrow P_P \sin ¢ = R_P \cos\left(°_p + \Phi\right) - cB \tan °_a - cB \times \left[\tan °_a - \left(\frac{\tan °_a}{\tan °_p} - \frac{n}{\tan i} + \frac{1}{2}\right) \times \left(\frac{\sin °_p \sin i}{\sin(i+p)}\right)\right] + \left(W_2 + Q\right)\left(1 - k_v\right) \qquad (8)$$

$$P_P = f\left(k_h, k_v, °_a, ¤, ¢, D_w\right)$$

From the equilibrium of two wedges, the active pressure and passive pressure will be equal. Thus we can find out maximum load (P_L) acting on the foundation from the equilibrium of the two wedges.

So,

$$P_A = P_P \tag{9}$$

$$P_L = \frac{1}{2}\text{¥} BN_{\text{¥}e} \tag{10}$$

2nd Condition (water table below the base of the footing)

Active region

Similarly, as in 1st condition

Applying the limit equilibrium condition in the triangular region AMC i.e.,

summation of the horizontal forces and the vertical forces equal to zero we get:

$$\sum H = 0 \Rightarrow P_A \cos\text{¢} = R_A \sin\left(\circ_a - \Phi\right) - \frac{cB}{\cot \circ_a}\cot \circ_a + \left(W_1 + P\right)k_h - P_{hyd} - P_{sw} \tag{11}$$

$$\sum V = 0 \Rightarrow P_A \sin\text{¢} = -R_A \cos\left(\circ_a - \Phi\right) + \left(W_1 + P\right)\left(1 - k_v\right) - 2cB\tan\circ_a \tag{12}$$

$$P_A = f\left(k_h, k_v, \circ_a, \text{¤}, \text{¢}, D_w\right) \tag{13}$$

Passive region

Applying the limit equilibrium condition in the polygonal region MCDE i.e.

summation of the horizontal forces and the vertical forces equal to zero the authors get:

$$\sum H = 0 \Rightarrow P_P \cos\text{¢} = R_P \sin\left(\circ_p + \Phi\right) + \frac{c\cos\circ_p}{\sin\circ_p} \times B \times \left[\tan\circ_a - \left(\frac{\tan\circ_a}{\tan\circ_p} - \frac{n}{\tan i} + \frac{1}{2}\right) \times \left(\frac{\sin\circ_p \sin i}{\sin\left(i + p\right)}\right)\right] + \left(W_2 + Q\right)k_h - P_{hyd} - P_{sw} \tag{14}$$

$$\sum V = 0 \Rightarrow P_P \sin\text{¢} = R_P \cos\left(\circ_p + \Phi\right) - cB\tan\circ_a - cB \times \left[\tan\circ_a - \left(\frac{\tan\circ_a}{\tan\circ_p} - \frac{n}{\tan i} + \frac{1}{2}\right) \times \left(\frac{\sin\circ_p \sin i}{\sin\left(i + p\right)}\right)\right] + \left(W_2 + Q\right)\left(1 - k_v\right) \tag{15}$$

$$P_P = f\left(k_h, k_v, \circ_a, \text{¤}, \text{¢}, D_w\right) \tag{16}$$

From the equilibrium of two wedges, the active pressure and passive pressure will be equal. Thus we can find out maximum load (P_L) acting on the foundation from the equilibrium of the two wedges.

So,

$$P_A = P_P \tag{17}$$

$$P_L = \frac{1}{2} ¥ B N_{¥e} \tag{18}$$

3 Results and discussion

The bearing capacity coefficient $N_{¥e}$ is optimized with respect to $°_A, °_p$ by iterative technique. From the global concave curve, the minimum value is taken as optimum value. Tables and parametric studied have been determined for pseudo-static values of strip footing which has been subjected to water condition under seismic criteria. Design charts shows in Tabular form in Table 1.

Table 1. Pseudo-static bearing capacity coefficients ($N_{¥e}$) for $k_h = 0.1, D_w / B = 0.25$.

ϕ	δ	2c/γB	K_v=0				K_v=k_h/2			
			D_f/B				D_f/B			
20	0	0	7.6	8.7	10.4	12.4	8.0	8.9	10.5	12.5
		0.25	10.16	11.39	13.21	15.33	10.45	11.66	13.47	15.60
		0.5	12.57	14.03	16.05	18.31	12.97	14.42	16.42	18.72
	ϕ/2	0	8.57	9.90	12.08	14.69	8.75	10.04	12.211	14.832
		0.25	15.52	13.20	15.61	18.44	11.83	13.49	15.91	18.74
		0.5	14.32	16.33	19.01	22.06	14.76	16.75	19.45	22.53
	ϕ	0	9.462	11.29	13.97	17.30	9.65	11.35	14.11	17.45
		0.25	13.01	15.23	19.36	22.07	13.35	15.55	18.59	22.53
		0.5	16.3	19.09	22.51	26.49	16.80	19.49	23.01	27.03
30	0	0	15.68	17.86	20.98	24.57	15.95	18.07	21.12	24.72
		0.25	19.32	21.80	21.32	25.077	19.25	22.18	25.08	29.29
		0.5	22.84	25.92	29.22	31.24	23.42	25.27	29.75	29.99
	ϕ/2	0	20.02	23.03	28.01	34.31	20.30	23.84	28.71	36.71
		0.25	24.84	28.85	34.92	40.41	20.19	25.7	34.71	40.89
		0.5	29.51	34.15	39.89	46.41	30.2	34.79	40.55	47.08
	ϕ	0	26.09	32.04	39.93	49.11	26.37	32.26	40.05	49.21
		0.25	33.07	39.93	48.59	58.48	33.62	40.44	49.10	58.97
		0.5	39.82	47.58	57.01	67.60	40.63	48.40	57.85	68.48

Parametric study

A detailed parametric study have been conducted using the results that are obtained from the optimising spread sheet which gives the optimum Reduced Seismic Bearing capacity coefficient $N_{¥e}$ and discussions are made for the different variation of parameters for all the two different mechanisms adopted in the work i.e., limit equilibrium principle : pseudo-static method.

From the graph it can be seen that the bearing capacity coefficient $\left(N_{¥e}\right)$ increases with the increase in the soil friction angle (Φ) . Increase in Φ increases the strength of the soil (or the internal resistance of the soil) against the shearing resistance.

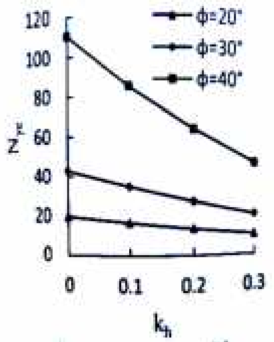

Fig. 5(a). 1st condition (Pseudo-Static Analysis)

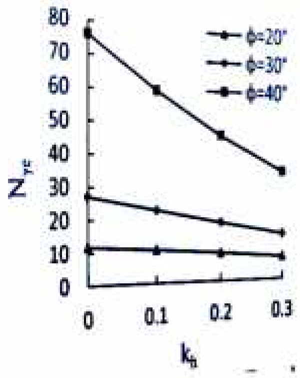

Fig. 5(b). 2nd condition

Fig. 5. (a), (b) : Variation of Bearing Capacity Coefficient with respect seismic acceleration $\left(k_h\right)$ for different soil friction angle $\left(\varphi = 20^\circ, 30^\circ, 40^\circ\right)$ at

$$\delta = \frac{\varphi}{2},\ k_v = \frac{k_h}{2},\ r_u = 0.2,\ \frac{D_w}{B} = 0.25,\ \frac{D_f}{B} = 0.5,\ \frac{2c}{¥ B} = 0.5,\ i = 15^\circ$$

Fig.2 (a), (b) has done for a particular case, $k_h = 0.2$, when φ increases from $30^\circ\ to\ 40^\circ$, $N_{¥e}$ increases by Pseudo-static analysis: 1st condition – 133%, 2nd condition – 147% respectively which may be due to the fact that increase in φ , increases the strength of the soil against shearing resistance.

Fig. 6. shows that it can be seen that $N_{¥e}$ decreases with the increase in depth of water level (D_w) for water table both above and below the base of the footing as the soil looses its strength with increase in the content of water. From Fig.3: for a particular

case, $k_h = 0.2$, when $D_w$increases from 0.5m to 1.0m, $N_{\gamma e}$ decreases by Pseudo-static analysis: 1st condition – 17%, 2nd condition – 2% respectively.

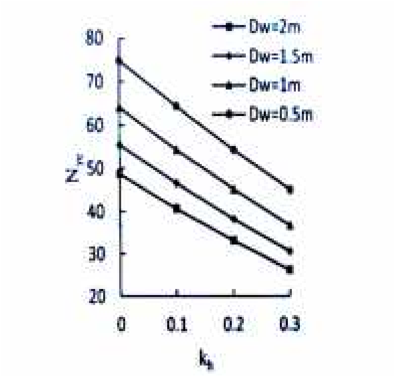

Fig. 6(a). 1st condition

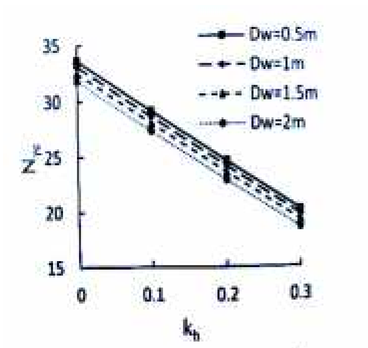

Fig.6(b). 2nd condition

Fig.6(a), (b): Variation of bearing capacity coefficient with respect to seismic acceleration (k_h) at different depths of water table (D_w in m) at $\delta = \frac{\varphi}{2},\ k_v = \frac{k_h}{2},\ r_u = 0.2,\ \frac{D_w}{B} = 0.25,\ \frac{D_f}{B} = 0.5,\ \frac{2c}{\gamma B} = 0.5,\ i = 15^\circ$

From fig.7. it can be seen that $N_{\gamma e}$ decreases with the increase in the width of the footing (B) with increasing seismic acceleration for the water table lying both above and below the base of the footing. This is due to the stress effect i.e., the stress dependency – smaller footing will have smaller stress whereas the larger footing will have higher stress and hence smaller bearing capacity coefficient. For a particular case, $k_h = 0.2$ when B increases from 2 m to 2.5 m $N_{\gamma e}$ decreases by Pseudo-static analysis: 1st condition – 17%, 2nd condition – 23% respectively.

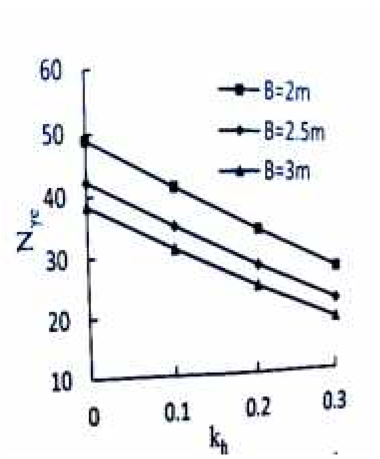

Fig. 7(a). 1st condition

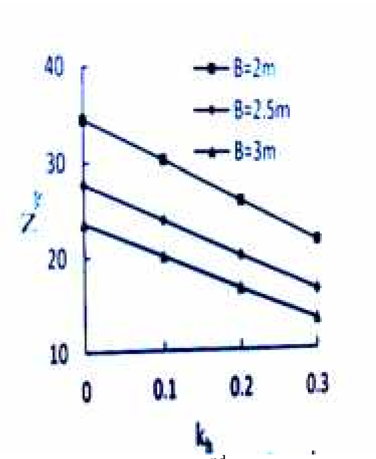

Fig. 7(b). 2nd condition

Fig.7(a),(b): Variation of Bearing Capacity Coefficient with respect to seismic acceleration (k_h) for different width of footing at $\varphi = 30^\circ at\ \delta = \frac{\varphi}{2},\ k_v = \frac{k_h}{2},\ r_u = 0.2,\ \frac{D_w}{B} = 0.25,\ \frac{D_f}{B} = 0.5,\ \frac{2c}{¥ B} = 0.5,\ i = 15^\circ$

Fig. 8. shows that $N_{¥e}$ increases with the increase in the depth of foundation (D_f) for the case of both water table lies below and above the base of the footing. The effect of increase in depth on increase in bearing capacity coefficient is predominant due to increase in surcharge weight, thus causing maximum passive resistance which governs the safe bearing capacity of the soil. For a particular case, k_h=0.2 when D_f increases from 0.5 m to 1.0 m $N_{¥e}$ increases by Pseudo-static analysis: 1st condition – 19%, 2nd condition – 39% respectively.

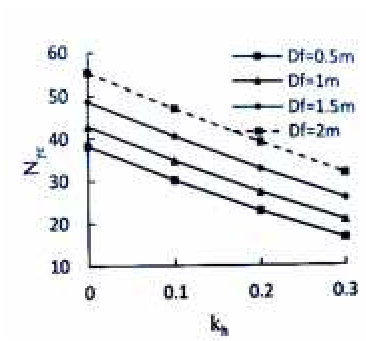

Fig. 8(a). 1st condition

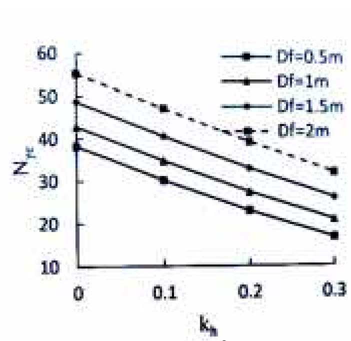

Fig. 8(b). 2nd condition

Fig. 8(a).(b). Variation of bearing Capacity Coefficient with respect to seismic acceleration (k_h) for different depth of footing (D_f in m) at $\varphi = 30^\circ at\ \delta = \frac{\varphi}{2},\ k_v = \frac{k_h}{2},\ r_u = 0.2,\ \frac{D_w}{B} = 0.25,\ \frac{D_f}{B} = 0.5,\ \frac{2c}{\gamma B} = 0.5,\ i = 15^\circ$

Fig. 9. Shows the variation of $N_{\gamma e}$ under seismic condition i.e., k_h at different values of cohesion (c= 0 kN/m^2,5 kN/m^2,10 kN/m^2). Seismic Bearing Capacity coefficient increases as cohesion increase, which causes increase in intermolecular attraction among the soil particle thus offering more bearing capacity for the conditions where table lies both below and above the base of the footing. For a particular case k_h= 0.2, when c increases from 0 kN/m^2 to 5 kN/m^2 $N_{\gamma e}$ increases by Pseudo-static analysis: 1st condition – 29%, 2nd condition – 53% respectively.

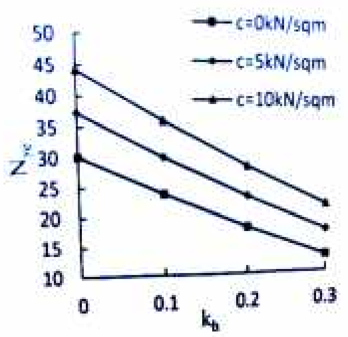

Fig. 9(a). 1st condition

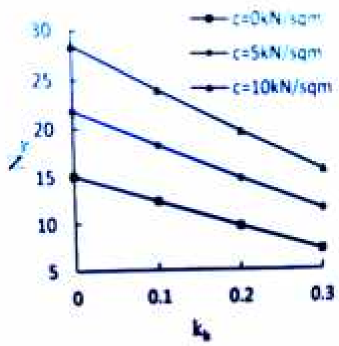

Fig. 9(b). 2nd condition

Fig.9(a),(b): Variation of bearing Capacity Coefficient with respect to seismic acceleration (k_h) for different values of cohesion for $\varphi = 30^\circ at\ \delta = \frac{\varphi}{2},\ k_v = \frac{k_h}{2},\ r_u = 0.2,\ \frac{D_w}{B} = 0.25,\ \frac{D_f}{B} = 0.5,\ \frac{2c}{\gamma B} = 0.5$

Fig.10. Variation of bearing Capacity Coefficient with respect to seismic acceleration (k_h) for different for different unit weight of soil (γ in kN/m^3) at

$$\varphi = 30^\circ \; at \; \delta = \frac{\varphi}{2},\; k_v = \frac{k_h}{2},\; r_u = 0.2,\; \frac{D_w}{B} = 0.25,\; \frac{D_f}{B} = 0.5,\; \frac{2c}{¥B} = 0.5,\; i = 15^\circ$$

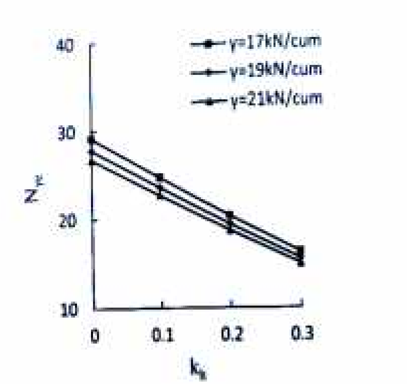

Fig.10(a). 1st condition

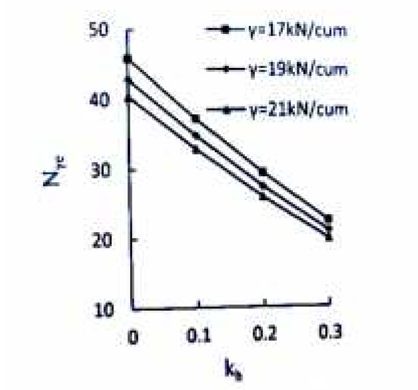

Fig.10(b). 2nd condition

Fig. 10(a), (b): Shows the variation of $N_{¥e}$ with respect to horizontal seismic acceleration (k_h) at three different values of unit weight of soil (γ), seismic bearing capacity factor will be decreased because the (2c/γB) portion decreases in the expression of $N_{¥e}$. For a particular case, $k_h = 0.2$, when γ increases from 17 kN/m$^{3 \text{ to } 19}$ kN/m^3, $N_{¥e}$ decreases by Pseudo-static analysis: 1st condition – 07%, 2nd condition – 53% respectively.

The change in bearing capacity coefficient due to increase in pore water pressure is found to be very minimal for both the cases of water table above and below the base of the footing and pseudo-static analysis.

4 Conclusions

An effort has been built to assess the Pseudo-static bearing capacity of shallow strip footing embedded in slope in c-φ nature of soil in terms of a single coefficient ($N_{\gamma e}$) using limit equilibrium method which is subjected to ground water flow. In the analysis, coincident resistance of unit weight, surcharge and cohesion is taken into account to calculate the pseudo-static bearing capacity coefficients in which linear failure surface is considered. From the parametric study it has been observed ultimate bearing capacity decreases by increasing depth of water table, Seismic coefficients and bearing capacity increases by increasing width of the footing, cohesion, unit weight of the soil. Design chart has been represented in table which can be applied in practical field.

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