# ULTIMATE SEISMIC BEARING CAPACITY OF STRIP FOOTING USING MODIFIED KREY'S METHODS (FRICTION CIRCLE METHOD)

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## ABSTRACT

In geotechnical investigation, determination oftheseismic bearing capacity of foundation soil constitutes an important task. The bearing capacity of soil under static loading has been extensively studied since the early work of Prandtl (1921).Design of foundation in seismic areas needs special considerations compared to the static case. The inadequate performance of structure during recent earthquake has motivated researches to revise existing methods and to develop new method for seismic resistant design. For foundation of structure built in seismic areas the demands to sustain load and deformation during earthquake will probably be the severe in their design life. Due to seismic loading foundation may experience decreases in bearing capacity and increases in settlement. Two source of loading must be taken into consideration inertial loading caused by lateral forces imposed on the superstructure, kinematic loading caused by the ground movement developed during earthquake.

Many techniques used for studying the effect of seismic forces on the soil bearing capacity such as, limit equilibrium method, kinematic approach of yield theory, a variational approach, and unified theory of stress, which the shape of failure surface has been assumed. The seismic forces are considered as pseudo-static forces acting both on the footing and on the soil under the footing. However, finite element and stress characteristics methods shape of the failure is not required to be assumed.

In the present paper, a theoretical analysis has been performed on the basis of Krey's method (friction circle method) with radius of friction circle equal to $= r\sin\left(\emptyset - \tan^{-1}\frac{k_h}{1-k_v}\right)$where r is the radius of the circle slip surface to determine the influence of the earthquake acceleration coefficients on the seismic bearing capacity of foundation with assisted by a computer program. The present study is compared with the various theoretical solutions. The comparison of that the present study predicted values of ultimate seismic bearing capacity of soil are less than others theories of ultimate seismic bearing capacity. In order facilitate the calculation of seismic bearing capacity, using the proposed equations. It is a function of (B, $R_{f,}$ , $\tan\emptyset$ , $k_h$ and c)

***Keywords:***Seismic Ultimate bearing capacity, strip footing, mechanism of failure, Centre location of slip failure, shape of slip failure, Krey's method

## 1. Introduction

In geotechnical investigation, determination oftheseismic bearing capacity of foundation soil constitutes an important task. The bearing capacity of soil under static loading has been extensively studies since the early work of Prandtl[17]. Design of foundation in seismic areas needs special considerations compared to the static case. The inadequate performance of structure during recent earthquake has motivated researches to revise existing methods and to develop new method for seismic resistant design. For foundation

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of structure built in seismic areas the demands to sustain load and deformation during earthquake will probably be the severe in their design life.

Due to seismic loading foundation may experience decreases in bearing capacity and increases in settlement. Two source of loading must be taken into consideration inertial loading caused by lateral forces imposed on the superstructure and kinematic loading caused by the ground movement developed during earthquake.

Many researchers have studied the seismic bearing capacity of shallow foundations [1-24].The analytical solutions consist of limit equilibrium method as [3, 7, 8, 18, 19, 21], kinematic approach of yield design theory [20], a variational approach [11, 22], unified strength theory [4], stress characteristics [13], and finite element method [12].DeepankarandSubba[8]used the limit equilibrium method for obtaining the seismic bearing capacity factors of footing considering a composite failure surface with new methodology.Vesic et al [22]have been used a pseudo–static method base on a variational approach for evaluating the seismic bearing capacity of strip footing. In this method, the inertia force is treated as an equivalent concentrated force (pseudo-static force) applied at the center gravity of the structure.Castelli and Motto [10]used Bishop's of slices method with a limit equilibrium method which the failure slip surface as circular from foundation propagates until the ground surface is reached. Chen et al [4] utilized pseudo-static analysis and taking the effect of intermediate stress into consideration based on the unified strength theory. He concluded that the reduction of bearing capacity is mainly due to the inclination effects resulting from cyclic earthquake shear and normal loads because of structural inertia. The ratio of seismic to static bearing capacity factors depend on the accelerations coefficients of seismic. Roberto and Pecker [20] used the kinematic approach of yield design theory for evaluating the seismic effects on the ultimate bearing capacity of shallow foundation on Mohr- Coulomb soil. The development of a new kinematic mechanism taking into account the possible uplift of the foundation under strong load eccentricities has permitted the investigation of the general case where the foundation is subjected to combined action of inclined and eccentric load as well as soli inertia. Kumar and Mohon[13] used the method of stress characteristics for determining the ultimate seismic bearing capacity factors.In this method, the shape of the failure surface is not required to be assumed, which the solution is obtained by satisfying simultaneously the equilibrium and failure conditions everywhere within the plastic domain.

Many experimental work have been done to determine the seismic ultimate bearing capacity and mechanism of failure for soil under footing [1, 5, 16, 23, 24]

With the latest advances in computer speed, linear and nonlinear analyses have found more applications in soil mechanics including the seismic bearing capacity problem have been used. However, finite element solutions are approximations to the exact solution. Finite element method used to determine seismicultimate bearing capacity of soil [12]. They have been used the finite element method with pseudo-static approach to estimate the seismic bearing capacity of strip footing which satisfies Mohr-coulomb strength criterion for wide range of $\emptyset$ and seismic coefficient using Plaxis 2D. They concluded that soil inertia plays a negligible role compared to the structural seismic load.

In the present work, a numerical study is carried out for the strip footing to investigate the effect of the footing width and depth to width ratio rest on the humongous soil (c,φ ) on the ultimate seismic bearing capacity using modified Krey's method[14] assisted by a computer , MATLAB, program,

## 2. Modified Krey'sMethod for Seismic Analysis

In fact, the surface failure of the soil due to footing load is continuous surface not broken lines. Krey (1936) suggested a graphical method to determine the soil bearing capacity under strip footing. The surface being assumed to consist of a circular arc underthe footing, terminating in a tangent at

$$\beta = \frac{\pi}{4} - \frac{\emptyset}{2} + 0.5\tan^{-1}\left(\frac{k_h}{1-k_v}\right) - 0.5\sin^{-1}\left(\frac{\sin\left(\frac{k_h}{1-k_v}\right)}{\sin\emptyset}\right) \quad \text{(Choudhury and SubbaRao[7]}$$

degrees to the ground. Krey's method is the same friction circle method of the stability of slop. The radius of the friction circle equal to$r\sin\left(\emptyset - \tan^{-1}\left(\frac{k_h}{1-k_v}\right)\right)$.Krey statedthat the center of the most dangerous circle would lieon the same level as the underside of the footing andvarious trial centers are taken at this level.

Krey'smodels contain active zone ABDJK and passive zone DGJ. Failure occurs when passive zone sliding up on the plane DG by the effect of rotating mass of the active zone about center of arc BD see Fig.(1).

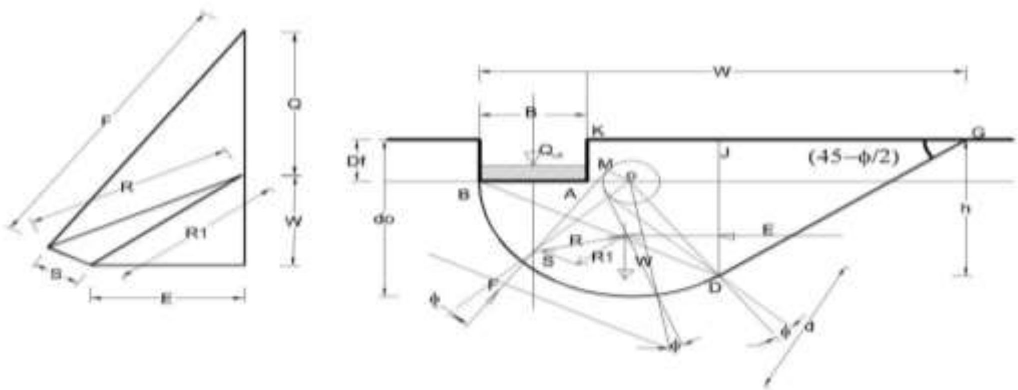


**Fig. 1.** Failure mechanism, according to Krey's method (after [5])

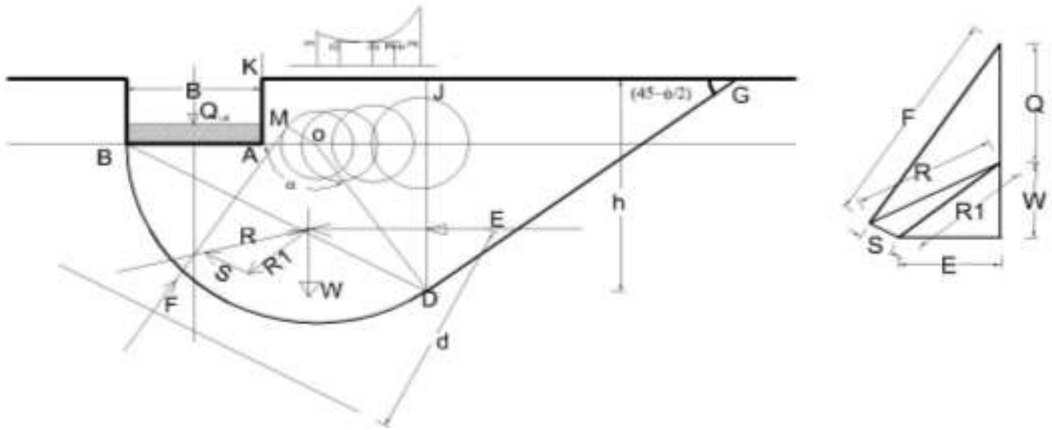


**Fig.2.** Determine seismic ultimate bearing capacity force ($Q_{ultd}$) using modified Krey's method (friction circle method)

### *2.1.The graphical procedure is as follows*

1- Let the centre of the slip surface on the same level as the underside of the footing (Fig. (2).

2- Measure DJ and calculate E for JDG to get

$P_{pe}=0.5 * k_{pe}\, \gamma\, (DJ)^2 + 2xcxDJx\sqrt{K_{pe}}$

Where

$$k_{pe} = \frac{cos^2\,(\emptyset-\theta)}{cos^2(\theta)}\left[\frac{1}{1-\sqrt{\frac{\sin\emptyset\sin(\emptyset-\theta)}{\cos\theta}}}\right]^2$$ (Egyptian code)(202/6)

where$\theta = \tan^{-1}\frac{k_h}{1-k_v}$

$$k_{ps} = \tan^2\left(45+\frac{\emptyset}{2}\right)$$

For determining the resulting position of the seismic passive earth pressures do the following:

- Determine the static passive earth pressure by using $k_{ps}$
- Determine the seismic passive earth pressure by using $k_{pe}$
- The seismic force equal $P_{ps}$-$P_{pe}$
- The position of the resultant of reduction due to seismic at $\frac{2}{3}$ h above the point D

3- Measure the area ABDJK and calculate W= area(ABDJK)*$\gamma$

4- The resultant of the horizontal force E = $P_{pe} - W(1-k_h)$

5- Determine the resultant E and W to give $R_1$

6- Determine the cohesive force along the slip surface

$S=c*L_{arc} = c*r*\alpha$

with distance from centre of slip $d = \frac{L_{arc}}{L_{ch}} = \frac{\alpha r}{2\sin(\alpha/2)}$

7- Find the resultant of W, E and S to give R

8- Now there are three forces at only one point R (resultant of (W, E and S), F (soil resultant reaction on slip surface, known direction and application point tangent of friction circle from left side but undetermined value) and $Q_{ultd}$(ultimate load can carry by footing, know direction and application point.

9- Draw a tangent to the friction circle through M (intersected R, $Q_{ultd}$) to obtain the direction of the F, the force triangle can be completed and $Q_{ultd}$can be obtained.

10- This procedure is repeated for several trial circles and the minimum value of the$Q_{ultd}$can be obtained.

To avoid the phenomenon of shear fluidization (i.e., the plastic flow of the material at finite effective for the certain combination of $k_h$ and $k_v$ (Richards et al (1990) and from the stability criteria (Sarma (1990) the $\frac{k_h}{1-k_v}$ consider in the analysis are to satisfy the relationship given by

$$\frac{k_h}{1-k_v} \leq \frac{c}{q_o} + \tan \emptyset$$

In the present study all trials which were and shown in Fig.2 produced automatically by the program and $Q_{ultd}$ (seismic ultimate bearing load) can be easily obtained.

## 3. Main Aim of the Present Work

The main aim of the present work is to transfer the shown case of the seismic bearing capacity of soil, using the modified Krey's method into group of equations can be solved easily by computer with accuracy. Many trials are used to find the minimum soil seismic bearing capacity which center of the slip arc locates on line pass on base of footing.

## 4. Parameters Used In the Program

### *4.1. Footing characteristics*

Footing width B= 1, 2, 3 and 4 m

Ratio of footing depth to footing width, ($R_f = \frac{D_f}{B}$) =0.0, 0.5, 1, 1.5 and 2

### *4.2. Soil properties*

Cohesion of soil c = 0, 2, 4, 6 and8 t/m$^2$

Angle of internal friction of soil φ = 5, 10, 15, 20, 25, 30, 35, 40 and 45 degree.

### *4.3. Ground accelerations coefficient*

$K_h$ = 0.0, 0.2, 0.4 and 0.6 and $k_v$=0.0

## 5. Procedure of Calculations

1. For a constant value of B=1(width of footing) and $k_h$ = 0.0 henceφ is changed nine time φ = 5, 10, 15, 20, 25, 30, 35, 40 and 45 and corresponding $Q_{ultd}$(seismic ultimate load) was obtained. Theultimate seismic bearing capacity can be determined by ($Q_{ultd}$/B), maximum extent of failure surface, w, where

$$\frac{w}{B}=\frac{r+\left(D_f+r\cos\beta\right)\cot\beta+r\sin\beta}{B}=\frac{\left(\frac{D_f}{\tan\beta}+\frac{r(1+\sin\beta)}{\sin\beta}\right)}{B}$$

, $$\beta=\frac{\pi}{4}-\frac{\emptyset}{2}+0.5\tan^{-1}\left(\frac{k_h}{1-k_v}\right)-0.5\sin^{-1}\left(\frac{\sin\frac{k_h}{1-k_v}}{\sin\emptyset}\right)$$

and maximum depth of failure surface , $\frac{d_0}{B}=\frac{r+D_f}{B}$

2. The value B is changed four times = 1, 2, 3 and 4and step No. 1 is repeated.
3. The value $k_h$is changed four times = 0.0, 0.2, 0.4 and 0.6 and step No. 1 and 2 is repeated.
4. For $R_f$ (Depth to width ratio) =0, 0.5, 1.0, 1.50 and 2 steps 1,2 and 3 are repeated.
5. For c= 0, 2, 4, 6, and 10 t/m$^2$steps 1, 2, 3 and 4 are repeated.
6. Results for steps 1, 2, 3, 4 and 5 are shown in figures (3-10)
7.

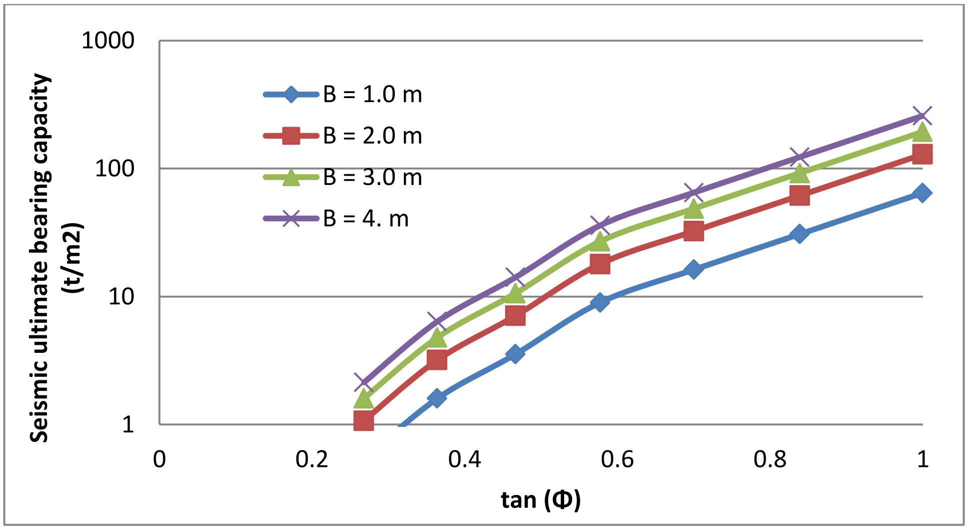


**Fig. 3.**Ultimate seismic bearing capacity versus $\tan\emptyset$ at $R_f$ = 0.0, $k_h$ =0.2 for different values of B

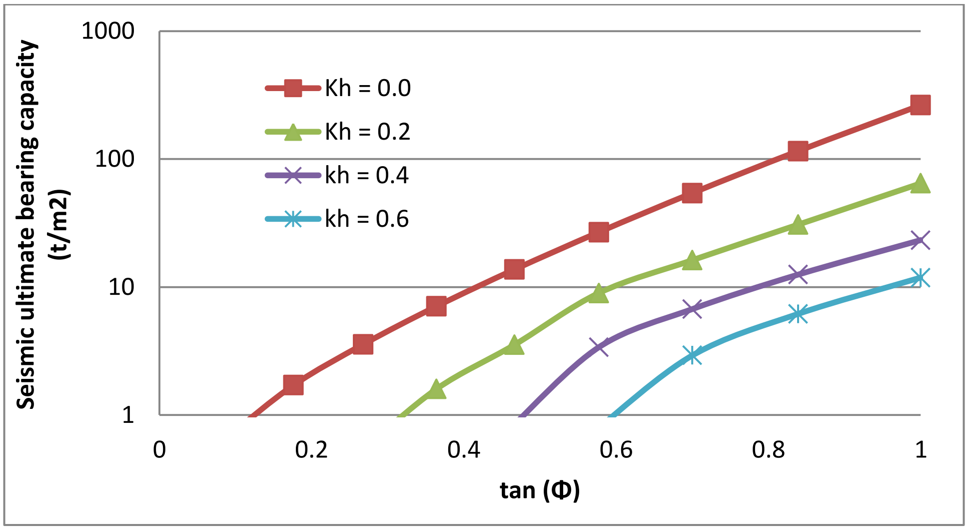


**Fig. 4.**Ultimate seismic bearing capacity versus $\tan\emptyset$ at $R_f$ = 0.0 for different values of$k_h$

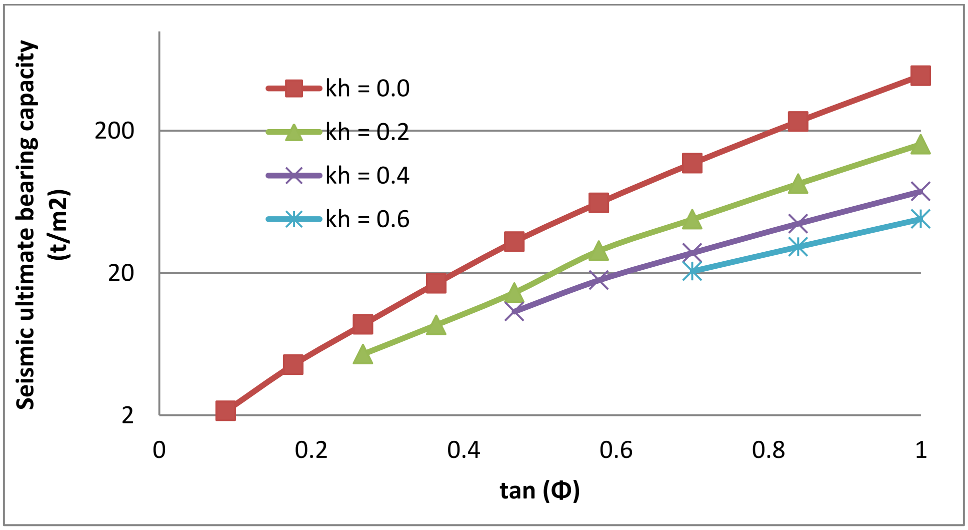


**Fig. 5.** Ultimate seismic bearing capacity versus $\tan\emptyset$at $R_f$ = 1.5 for different values of$k_h$

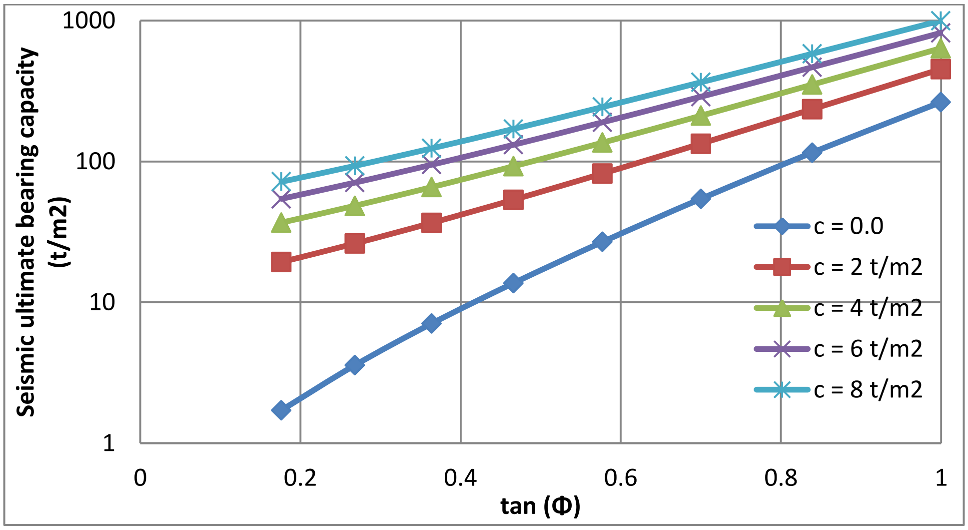


**Fig.6.**Ultimate seismic bearing capacity versus tan Øat $R_f$ = 0, $k_h$ = 0.0 for different values of c

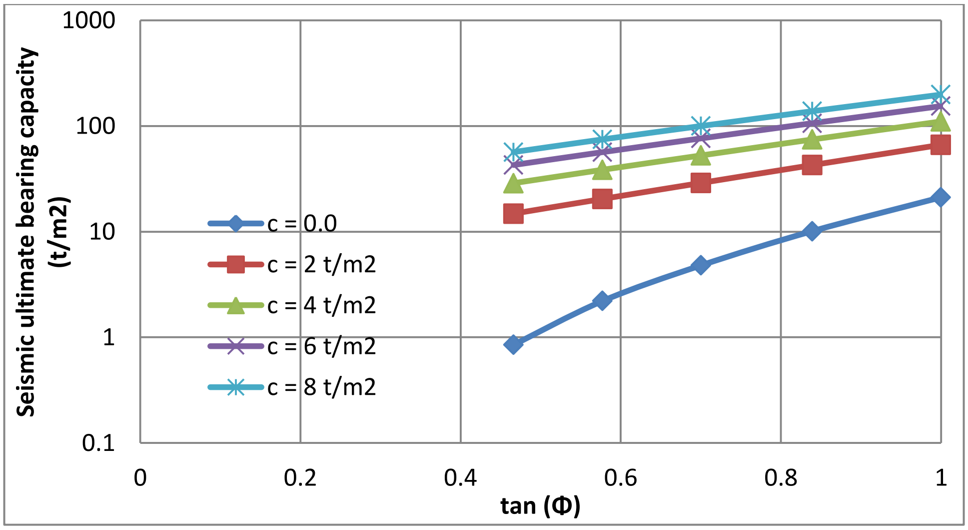


**Fig.7.**Ultimate seismic bearing capacity versus tan Øat $R_f$ = 0.0$k_h$ = 0.40 for differentvalues of t c

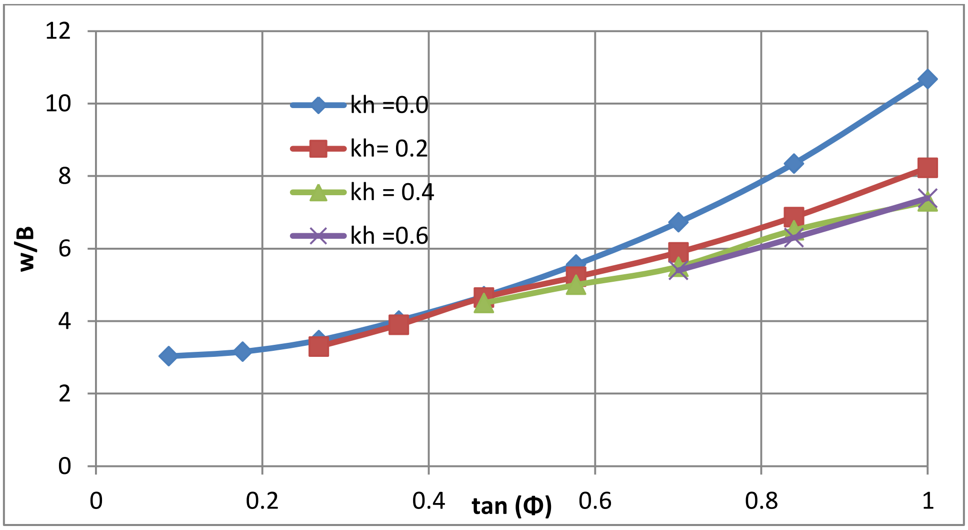


**Fig. 8.** Maximum extent of failure surface (w/B) versus tan (φ) at$R_f$ = 0.5 c = 0.0 for differentvalues of $k_h$

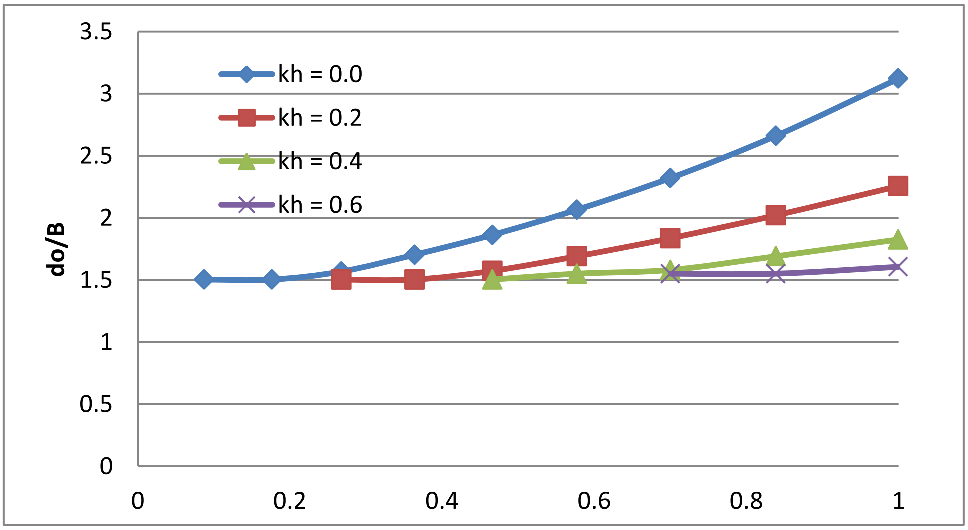


**Fig. 9.** Maximum depth of failure surface form ground surface ($d_o$/B) versus tan (φ) at $R_f$ = 0.5 c = 0.0 for different values of$k_h$

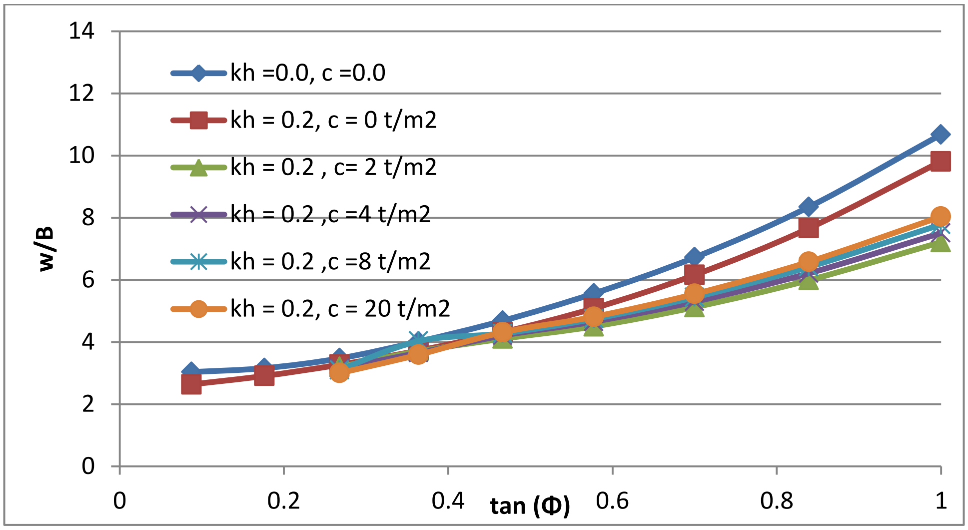


**Fig. 10.** Maximum extent of failure surface (w/B) versus tan (φ) at $R_f$ = 0.0 for different values of c and $k_h$

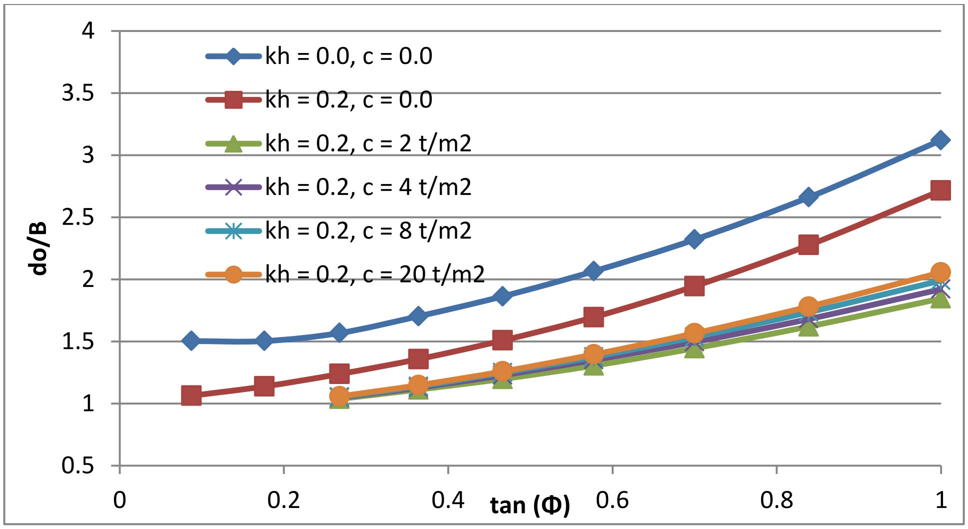


**Fig. 11.**Maximum depth of failure surface form ground surface ($d_o$/B) versus tan (φ) at $R_f$ = 0.5 for different $k_h$and c

## 6. Analysis and Discussion

The discussion illustrates the effect of foundation width, depth to width ratio, accelerations seismic coefficients ($k_h$and $k_v$) and soil properties (c,φ) on the following items:

Ultimate seismicbearing capacity of soil,$q_{ultd,}$

Maximum extent of failure surface (w/B),

Maximum depth of failure surface ($d_o$/B) and

The deduced formula for determining $q_{ultd}$,$N\gamma q$,$N_{cd}$,$r_c$, $r_q$ and $r_\gamma$

## *6.1. Seismic ultimate bearing capacity of soil ($q_{ultd}$)*

The relation between ultimate seismic bearing capacity of soil($q_{ultd}$) versus*tan*φ (φ is the angle of internal friction of soil) at different acceleration seismic coefficients ($k_h$and $k_v$)are plotted and shown in Figs. (3-7). It is clear that with increasing φ and B the $q_{ultd}$increases for a constant value of $R_f$. Figs.(3-4) have the same trend for the given values of $R_f$ = 0.0 , 0.5, 1,and 2). Figs (5-6) show the relation between $q_{ultd}$and *tan* (φ) for different value of cohesion of soil, c.It is clear that with increasing φ and c the $q_{ultd}$increasers for a constant value of $R_f$. Figs.(5-6) have the same trend for the given values of $R_f$ = 0.0 , 0.5, 1, 1.5 and 2. The ultimate seismic bearing capacity decreases considerably with increasing accelerations coefficients.

## *6.2.Maximum extent of failure surface (w/B)*

The relation between **(w/B)** versus*tan*(φ) are plotted and shown in Figs. (8and 10). It is clear that with increasing (φ) the w/B value increases.Fig (10) shows the relation between (w/B) and *tan* (φ) for different,$k_h$,andc. It is clear that by increasing (c)the (w/B) value slightly effectfor a constant value of $k_h$. Form Figs (9, 10) may be neglected the effect of the cohesion of soil on the (w/B) and take the effect of friction only and accelerations coefficients.

## *6.3.Maximum depth of failure surface from the ground surface (do/B)*

The relation between **(**($d_o$/B)**)** versus*tan (*φ)are plotted in Figs. (9 and 11).It is clear that with increasing φ the ($d_o$/B) increases and decreasing with $k_h$. Fig (11) shows slightly effect of c on the $d_o$/B while $d_o$/B decreases with increasing of kh.

## *6.4.The deduced formula for determining $q_{ultd}$,$Nq_d$,$N\gamma_d$and $Nc_d$*

### *6.4.1.Coshionless Soil c=0.0*

Based on the results, the relation between $\mathbf{q_{ultd}}$and*tan* φ is drawn for different values of B= 1, 2, 3 and 4, kh = 0.0, 0.2, 0.4, and 0.6 and $R_f$ =0.0, 0.5, 1, 1.5 and 2. As shown in Figs.(3-4) for $R_f$ =0.0 and 0.5. At all cases the ultimate seismic bearing pressure increases exponentially with increasing $\tan\emptyset$and linearly with increasing B at a certain $R_f$. The relationship between $\mathbf{q_{ultd}}$and B for the different values of φ, $k_h$ and $R_f$, may be represented by the following expression

$$q_{ult} = aB\gamma e^{b\tan\emptyset}$$

where a, b are coefficients obtained by regression formula depend on $R_f$, $k_h$and listed in Table NO.1

**Table 1.**
a and b coefficients and deduced formula

| $R_f$ | coefficients | Horizontal accelerations coefficients | | | | Deduced formula $k_h \geq 0.2$ |
|---|---|---|---|---|---|---|
| | | 0.0 | 0.2 | 0.4 | 0.6 | |
| 0.0 | a | 0.572 | 0.154 | 0.120 | 0.117 | $0.170-0.092k_h$ |
| | b | 6.390 | 6.347 | 5.861 | 4.648 | $7.317-4.247k_h$ |
| 0.5 | a | 0.880 | 0.403 | 0.316 | 0.348 | $0.410-0.136k_h$ |
| | b | 5.632 | 5.039 | 4.344 | 3.594 | $5.77-3.612k_h$ |
| 1 | a | 1.09 0 | 0.728 | 0.715 | 0.847 | $0.644+0.297k_h$ |
| | b | 5.572 | 4.706 | 3.858 | 3.133 | $5.472-3.932k_h$ |
| 1.5 | a | 1.180 | 0.917 | 1.186 | 1.592 | $0.556+1.687k_h$ |
| | b | 5.750 | 4.684 | 3.595 | 2.815 | $5.5670-4.672k_h$ |
| 2 | a | 1.200 | 0.673 | 0.944 | 1.164 | $0.436+1.227k_h$ |
| | b | 5.846 | 5.222 | 4.017 | 3.337 | $6.07- 4.707k_h$ |

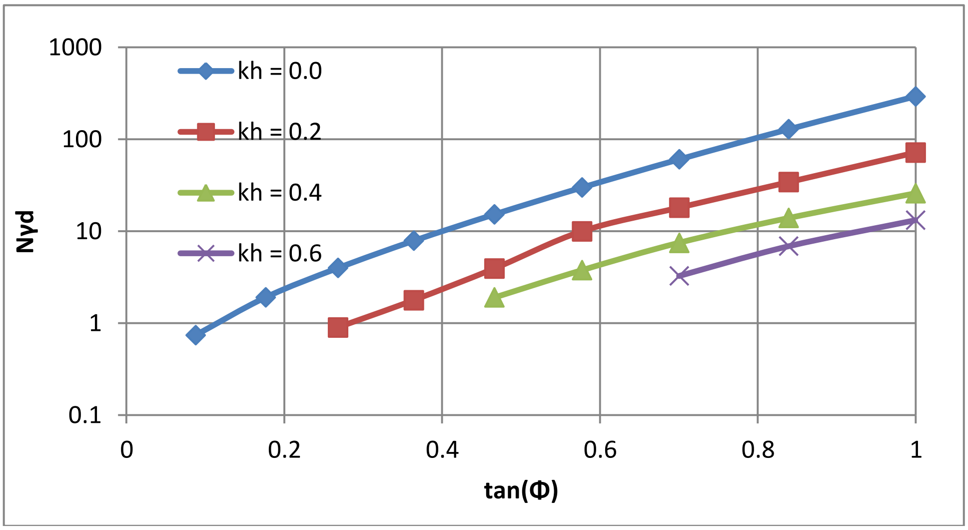


**Fig. 12.** Seismic bearing capacity factor $N\gamma_d$ versus tan (φ) at different $k_h$

*6.4.2.Bearing capacity factor $N_{\gamma d}$*

The ultimate seismicbearing capacity of footing at the ground surface forcohesionless soil can be expressed by the following equation

$$q_{ult} = 0.5\, B\gamma N_{\gamma d}$$

Where $\boldsymbol{N_{\gamma d}}$ is the bearing capacity factor depend on angle of internal friction of soil, and $k_h$ ,as shown in Fig. 12, may be represented by the following equation

$$N_{q\gamma} = ce^{d \tan \emptyset}$$

Where c, d are coefficient obtained by regression formula depend on $k_h$ and are listed in Table No. 2

**Table 2.**
c and d coefficients

| $k_h$ | 0.0 | 0.2 | 0.4 | 0.6 | Deduced formula $k_h \geq 0.2$ |
|---|---|---|---|---|---|
| Coefficient c | 0.632 | 0.172 | 0.158 | 0.130 | $0.1953-0.105k_h$ |
| Coefficient d | 6.40 | 6.35 | 5.26 | 4.65 | $7.126-4.26k_h$ |

### *6.4.3.Bearing capacity factor $N_{qd}$*

Based on the results, the relation between $q_{ultd}$ and $\tan\emptyset$ is drawn for different values of $k_h$ = 0.0, 0.2, 0.4, and 0.6 and $R_f$ = 0.0, 0.5, 1, 1.5 and 2 as shown in Figs. (3-5). From all cases the ultimate seismic bearing pressure increases exponentially with $\tan\emptyset$ increasing and linearly with increasing B at a certain $R_f$. The ultimate seismic bearing capacity of soil can be divided into two parts. First part for surcharge load while second part unit weight of soil. The relationship between $q_{ultd}$ and B for the different values of $k_h$, ϕ and $R_f$, may be represented by the following expression

$$q_{ultd} = R_f B\gamma N_{qd} + 0.5B\gamma N_{\gamma d}$$

$$N_{qd} = \frac{q_{ultd - 0.5\,B\gamma N_{\gamma d}}}{R_f B\gamma}$$

The value of $N_{qd}$ versus tan (φ) is plotted for different values of $k_h$ as shown in Fig. 13. It is clear that with increasing tan (φ) the value of $N_{qd}$ increases and decreasing with increases $k_h$. The relationship between $N_{qd}$ and tan (φ) may be represented by the following expression:

$$N_{qd} = f e^{g\tan\emptyset}$$

Where f, g are coefficients obtained by regression formula depend on $k_h$ and listed in Table No. 3

**Table 3.**
f and g coefficients

| $k_h$ | 0.0 | 0.2 | 0.4 | 0.6 | Deduced formula $k_h$>0.0 |
|---|---|---|---|---|---|
| Coefficient f | 0.678 | 0.437 | 0.37 | 0.47 | $-16.42k_h^2 + 8.955k_h + 3.272$ |
| Coefficient g | 4.96 | 4.41 | 4.23 | 2.736 | $5.46k_h^2 - 3.612k_h + 0.941$ |

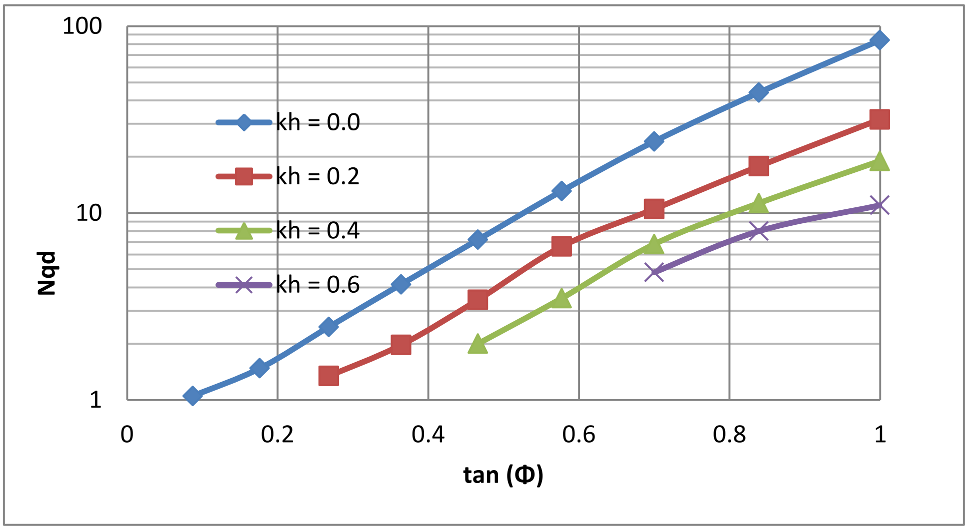


**Fig. 13.** Seismic bearing capacity factor $Nq_d$ versus tan(φ) at different $k_h$

### *6.4.4. Bearing capacity factor $N_{cd}$*

Based on the results, the relation between $q_{ultd}$ and $\tan\emptyset$ is drawn for different values of c =0, 2, 4, 6 and 8t/m$^2$,kh = 0.0, 0.2, 0.4, and 0.6 and $R_f$ = 0.0, 0.5, 1, 1.5 and 2 as shown in Figs.(6-7). From all cases the ultimate seismic bearing pressure increases exponentially with $\tan\emptyset$ increasing and linearly with increasing c at a certain $k_h$. The ultimate seismic bearing capacity of soil can be divided into three parts. First part for cohesion, second part for surcharge loads while third part unit weight of soil for friction. The relationship between $\mathbf{q_{ultd}}$and B for the different values of c, φ and $R_f$, may be represented by the following expression

$$q_{ultd} = cN_{cd} + R_f B\gamma N_{qd} + 0.5B\gamma N_{\gamma d}$$

$$N_{cd} = \frac{q_{ultd - R_f B\gamma N_{qd} + 0.5B\gamma N_{\gamma d}}}{c}$$

The value of $N_{cd}$ versus $\tan\emptyset$is plotted for different values $R_f$ as shown in Fig.(7). It is clear that with increasing $\tan\emptyset$ the value of $N_{cd}$ increases, and slightly decreases with increasing $R_f$. the relationship between $N_c$ and $\tan\emptyset$ may be represented by the following expression:-

$$N_{cd} = he^{z\tan\emptyset}$$

Where h, z are coefficients obtained by regression formula depend on $k_h$and listed in Table No. 3

**Table 3.**

h and z coefficients

| $k_h$ | 0.0 | 0.2 | 0.4 | 0.6 | Deduced formula $k_h$>0.0 |
|---|---|---|---|---|---|
| Coefficient h | 5.283 | 3.56 | 2.571 | 1.984 | $4.281 - 3.94k_h$ |
| Coefficient z | 2.858 | 2.455 | 2.173 | 1.95 | $2.694 - 1.25k_h$ |

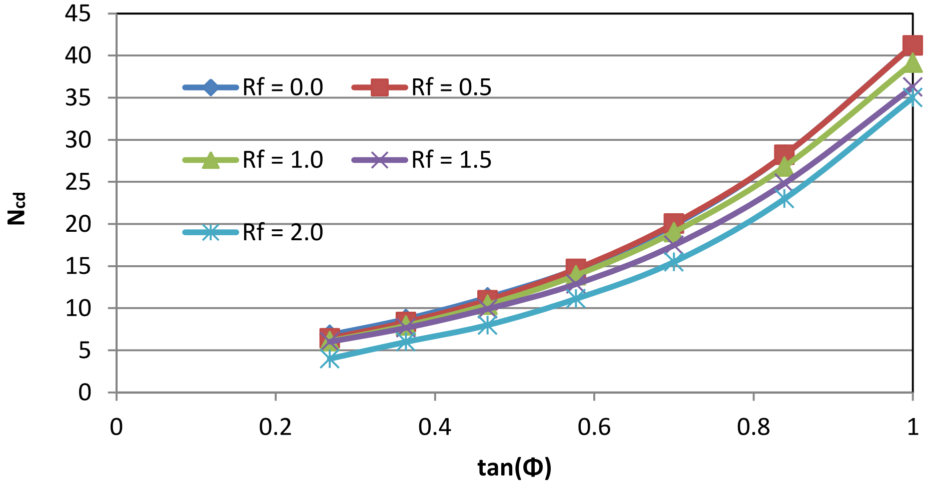


**Fig.14.**Seismic bearing capacity factor ($N_{cd}$) versus tan φ at $k_h$= 0.2for different Rf

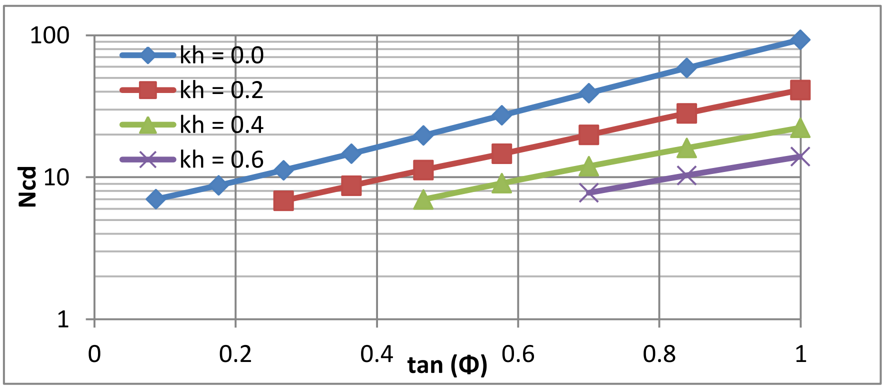


**Fig.15.**Seismic bearing capacity factor ($N_{cd}$) versus tan φ for different $k_h$

## 7. Ratio of Seismic to Static Bearing Capacity Factors

The seismic to static bearing capacity factors depend on the acceleration ratio $\frac{k_h}{1-k_v}$where $k_h$and $k_v$ are the horizontal and vertical seismic coefficients with the failure zone. In analysis subscripts d and s signify earthquake and static condition. A simple approach to account for seismic effects is to reduce the static bearing capacity factors $r_c, r_q and r_\gamma$ where:

$$r_c = \frac{N_{cd}}{N_{cs}}, r_q = \frac{N_{qd}}{N_{qs}}, r_\gamma = \frac{N_{\gamma d}}{N_{\gamma s}}.$$

The ratio of seismic to static bearing capacity factors are given in Table No. 4

**Table 4.**
Ratio of seismic to static factors and simple formula

| Ratio | $k_h$ | Equation | Average | Simple formula $k_h \geq 0.2$ |
|---|---|---|---|---|
| $r_c = \frac{N_{cd}}{N_{cs}}$ | 0.2 | $0.676 - 0.234 \tan\varphi$ | 0.535 | $0.91e^{-2.797k_h}$ |
| | 0.4 | $0.4567 - 0217 \tan\varphi$ | 0.301 | |
| | 0.6 | $0.3104 - 0.1604 \tan\varphi$ | 0.175 | |
| $r_q = \frac{N_{qd}}{N_{qs}}$ | 0.2 | $0.58 - 0.21 \tan\varphi$ | 0.46 | $0.785e^{-2.489k_h}$ |
| | 0.4 | $0.326 - 0.09 \tan\varphi$ | 0.26 | |
| | 0.6 | $0.37 - 0.23 \tan\varphi$ | 0.17 | |
| $r_\gamma = \frac{N_{\gamma d}}{N_{\gamma s}}$ | 0.2 | $0.2403 - 0.0413 \tan\varphi$ | 0.265 | $0.603e^{-4.13k_h}$ |
| | 0.4 | $0.165 - 0.07 \tan\varphi$ | 0.114 | |
| | 0.6 | $0.077 - 0.0304 \tan\varphi$ | 0.051 | |

Form the Table No. 4 ratio of seismic to static bearing factors decreases with increasing tan (φ) at a certain $k_h$ by small value, therefore taking the average value and use the simple formula.All the deduced formula can be easily calculated by ordinary calculator.

## 8. Application of the Program and Deduced Formula and Comparison with Others

Some examples were solved using program and the formulas given by author, comparison with the references given in Figs (16-17). Fig. 16 shows the $q_{ultd}$ versus tan (φ) at B =1 m, Rf = 1, c = 2t/m$^2$ and γ = 1.8t/m3using different methods. It is clear that the $q_{ultd}$by current method (modified Krey's method) less than others methods. Fig.17 shows the ratio between seismic to static bearing capacity factors from researches and current method.

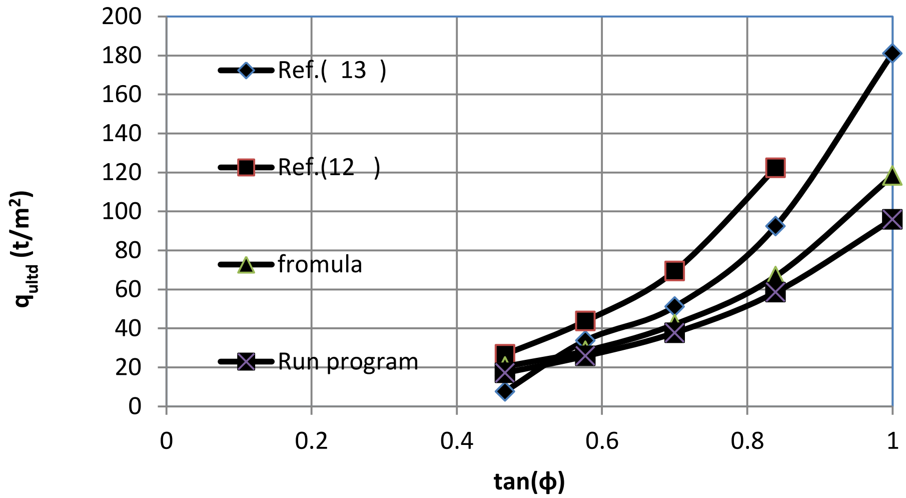


**Fig.16.**Ultimate seismic bearing capacity versus tan Ø at B = 1.0 m, $R_f$ = 1, c = 2 t/m$^2$ using different methods

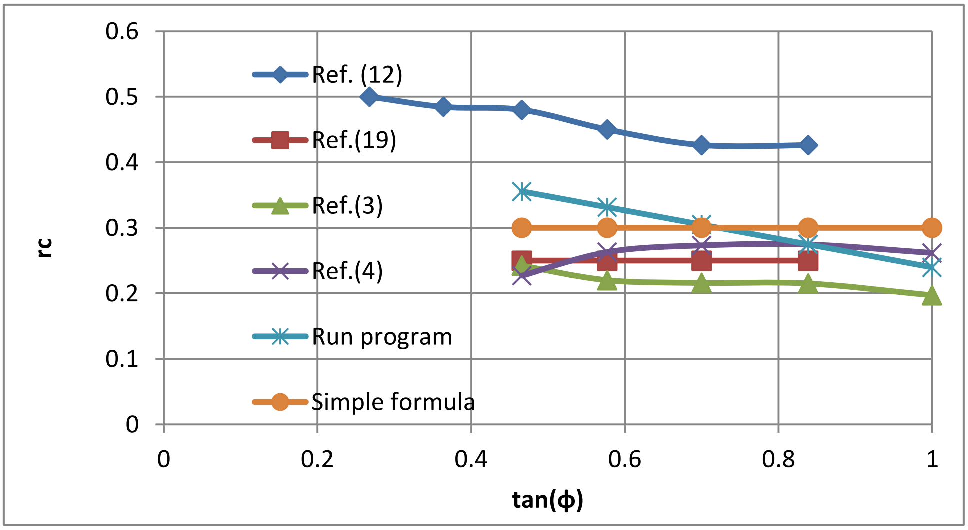


**Fig. 17 a.** Bearing capacity factor ($N_{cd}$) versus tan φ using different methods

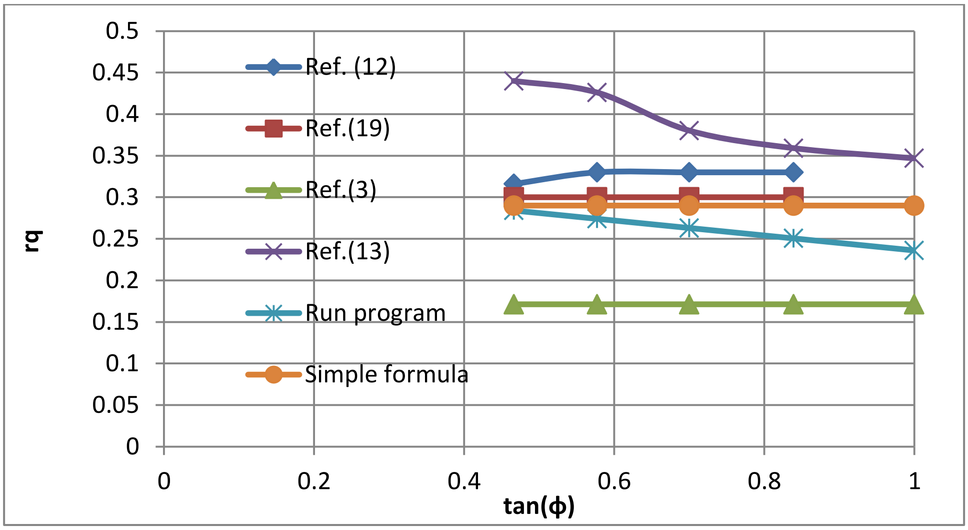


**Fig.17 b.** Bearing capacity factor ($N_{qd}$) versus tan φ using different methods

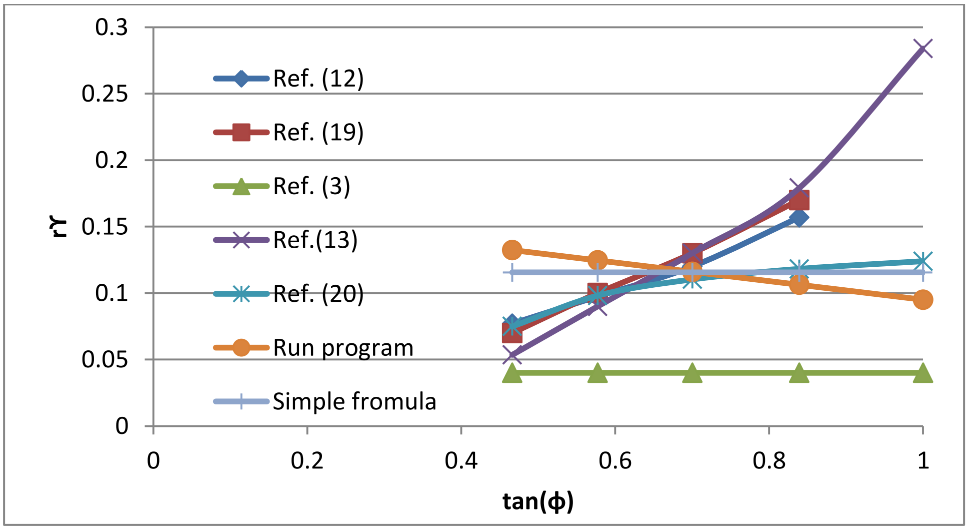


**Fig.17 c.** Bearing capacity factor ($N_{\gamma d}$) versus tan φ using different methods

## 9. Conclusions

The horizontal acceleration, as soil properties (c, φ), effect on the seismic bearing capacity coefficients significantly. The problem of the ultimate seismic bearing capacity of strip footing using modified Krey's method (friction circle method) on (c-φ ) soil can be easily analyses and solved to find the ultimate seismic bearing capacity , $q_{ultd}$, bearing capacity factors ($N_{cd}$, $N_{qd}$ and $N_{\gamma d}$) , maximum extent of failure surface and maximum depth of the failure from ground surface ((w/B) and ($d_o$/B)) by a simple program by author instead of a graphical methods used before in this method. The recommend program based on footing characteristic and soil properties described in details of the case study. Simple formulas were deduced base on results obtained from run of computer program for the case study to calculate easily by a calculator ($q_{ultd}$, $N_{cd,}$ $N_{qd}$ ,$N_{\gamma d}$, $r_c$, $r_q$, and $r_\gamma$). A comparison was made between results of present work and researches to evaluate to mention items. The obtained results approximately well agree with some pervious work. In this provides the designer a means of evaluating the seismic bearing capacity factors from the static ones.

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# اقصى قوة تحمل سيزمية لاساس شريطى باستخدام طريقة كيرى المعدلة (طريقة دائرة الاحتكال)


## الملخص العربى

فى الابحاث الجيوتقنية تحديد اقصى قوة تحمل سيزمية للتربة يعتبر من اهم المهام حيث ان قوة تحميل التلربة الاستاتيكية اسفل الاساس تم دراسته بتوسع ومن الاعمال البكرة برانـدال (1921) و تصميم الاساس فى منطقة الزلازال يحتاج الى اعتبارات خاصة بالنسبة لحالة الاستاتيكية و الاداء الغير كافى للمنشأت اثناء الزلازال دفع الباحثين الى مراجعة الطرق الجديدة و ايجاد طرق جديدة للتصميم المقاوم للزلازال حيث ان اساسات المبانى المبينة فى منطقة زلازال تعانى من حمل و تشكل اثناء الزلازال المحتمل حدوثها. ونتجة لحمل الزلازال يحدث نقص لقوة تحمل التربة و زيادة فى الهبوط. يوجد مصدرين للاحمال يجب ان ياخذو فى الاعتبار الحمل القصورى المتسبب بواسطة القوى العرضية الاتية من المنشآ العلوى و الحمل الكينمتيكى المتسبب بواسطة الحركة الارضية اثناء الزلازال. توجد عدة طرق تشتخدم لدراسة تأثير قوة الزلازال على قوة تحمل التربة مثل طريقة الاتزان الحدية , النهج الكنيمتكى لنظرية الخضوع , النهج التفاضلى و نظرية الاجهاد الموحد حيث ان هذه طرق تتطلب فرض شكل سطح الانهيار و قوة الزلازال تاخذ بقوة افتراضية تؤثر فى التربة والاساس مع الرغم ان طريقة العناصر المحددة و نظرية الخصائص لا تطلب فرض سطح الانهيارمسبقا.

فى هذا المقال اجريت دراسة نظرية باستخدام نظرية كيرى بعد تعديل نصف قطر دائرة الاحتكال لدراسة تأثير معاملات العجلة الزلازالية على قوة التحمل السيزمية للتربة بمساعدة برنامج كمبيوتر ثم تم مقارنة الدراسة الحالية بالدراسات السابقة المتاحة و استنتج فى هذه الدراسة ان قوة التحمل السيزمية للتلربة اقل من الدراسات السابقة و لسهولة الحسابات استخدمت معادلات مقترحة مبينة على النتائج التى تم الحصول عليها من البرنامج معتمدة على عرض الاساس ($B$) ونسبة عمق التأسيس الى عرض الاساس ($R_f=D_f/B$) و زواية الاحتكاك الداخلى للتربة ($\varphi$) و معامل تماسك التربة ($c$) و معامل العجلة السيزمية ($k_h$) حيث يمكن استخدامها بواسطة الة الحاسبة العادية.