

# Review on Determining Seismic Bearing Capacity of Shallow Strip Footing

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**ABSTRACT:** Seismic bearing capacity for shallow strip footing has been determined indirectly due to increase in static allowable soil pressure in the form of pseudo-static bearing capacity. This paper shows a review on principle and methodology for determining the seismic bearing capacity of shallow strip footing by Analytical method with aspect of both limit equilibrium and limit equilibrium analysis theory and also simplified procedure that are generally used in practice discussed here.

**KEYWORDS:** Seismic Bearing Capacity, Earth pressure, shallow strip footing

## I. INTRODUCTION

Image Seismic events have a capacity to fail mat footings, isolated column footing, strip footing and pile foundations. These failures are mostly due to liquefaction (a condition where soil loses its shear strength). Liquefaction alone is so unlikely explanation due to a number of failures have occurred at a place where field condition show only partial saturation or a denser soil condition. Reduction in seismic bearing capacity is main reason for seismic settlement of above foundations. For example, the foundation of several oil storage tanks suffered from bearing capacity failure in MIYAGIHEN - OKI earthquake of magnitude 7.8 on June 12, 1978 northeast of Sendai, Japan. The sub soil for oil storage tank has a layer of 65 m thick fine sand consolidated by vibrofloatation before construction of tank. In the United States Of America, the settlement of Jensen Filtration Plant at the time of San Fernando earthquake is a well-picturized example of large seismic settlement (about 100 mm). Such site observations of earthquake settlements of foundations due to bearing-capacity reduction instead of densification or liquefaction are perhaps most simply explained in terms of the theory of seismic fluidization written recently by Richards et al. (1990). This theory, showed in a general way by shake-table tests at SUNY, shows that significant reduction in the bearing capacity and constant settlements should be anticipated for every type of foundations at moderate and severe earthquake intensities even if they are on denser sand. Moreover, this shear fluidization due to inertia stresses do not depend over water content and that is why independent of any liquefaction potential (which only intensifies this phenomenon). No specific calculations for either seismic bearing capacity or earthquake settlements of foundations were attempted in the earlier work. Only the limit analysis solution for inertial fluidization of sand layer is presented and settlement of test structures were discussed in terms of time averaged viscous flow of the sand from below the footing at very low Reynolds numbers. The present work makes up the deficiency by considering bearing capacity and settlements. First, the standard static bearing capacity formula is reviewed briefly in order to develop a new comparable dynamic limit analysis model. This, in turn, allows the development of seismic bearing capacity factors  $N_{\sim e}$ ,  $N_{qe}$ , and  $N_{ce}$  for comparison of static values. At the same time, this study will show how the pattern of slip surface changes with increasing acceleration to explain the shear flow and settlements observed in both the laboratory and the site conditions. Calculation of settlements for any earthquake condition is then possible using a sliding-block method that will allow the engineer to check existing foundations to design statically or, for new ones, to use a seismic design procedure to eliminate settlements or reduce them to a level of acceptance. It should be clearly noted that the seismic reduction in bearing capacity considered in the present work is that associated with both the seismic degradation of soil strength and the lateral inertial force transmitted by shear to foundation through structure and surcharge. The same problem is

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analyzed by Sharma and Iossifelis (1990) using a method for slope stability based on method of inclined slices. In the present work, however, a simpler Coulomb-type mechanism, using Mononobe-Okabe analysis (Mononobe and Matsuo 1929; Okabe 1926) is taken which leads to the presentation of a straight forward method for both analysis and design.

## II. LITERATURE REVIEW

Sarma and Iossifelis (1990) has been analyzed using a technique for slope stability based on a method of inclined slices. In the present paper, however, a simpler Coulomb-type mechanism, using Mononobe-Okabe analysis (Mononobe and Matsuo 1929; Okabe 1926) is used leading to the presentation of a simple procedure for both analysis and design.

Terzaghi (1943), Meyerhof (1951), Hansen (1970), Vesic (1973), Chen (1975), Das (1987), Shin et al. (2002), Kumar (2003), Dewaikar and Mohapatro (2003) and many others have extensively studied the bearing capacity of shallow footings for static loading case. Budhu and Al-Karni (1993) and Kumar and Kumar (2003) considered the seismic forces both on the structure and on the supporting soil mass. Dormieux and Pecker (1995) and Soubra (1997, 1999) used upper bound limit analysis to develop seismic bearing capacity factors for shallow strip footings. Kumar and Rao (2002, 2003) carried out the analysis for seismic bearing capacity of shallow foundations by using method of characteristics without using the vertical component of seismic acceleration.

Hardin and Black (1968), and also Hardin and Drnevich (1972), based on extensive experimental data, established indispensable relations between the shear wave velocity, void ratio, and shear rigidity of soils. Ohkubo and Terasaki (1976) supplied various expressions relating the seismic wave velocities to weight density, permeability, water content, unconfined compressive strength and modulus of elasticity. Imai and Yoshimura (1976), Tatham (1982), Willkens et al. (1984), Eberhart-Phillips et al. (1989), Keceli (1990), Jongmans (1992), Sully and Campanella (1995), and Pyrak-Nolte et al. (1996). Campanella and Stewart (1992) determined various soil parameters by digital signal processing, while Butcher and Powell (1995) supplied practical geophysical techniques to assess various soil parameters related to ground stiffness. Abd El-Rahman et al. (1992), for the ultimate bearing capacity of soils, using the logarithm of shear wave velocity. Sieffert and Bay-Gress (2000). Turker (2004), based on extensive case studies, supplied an explicit expression for the allowable bearing pressure, using shear wave velocity. Massarsch (2004) determined deformation properties of fine-grained soils from seismic tests.

Paolucci and Pecker (1997) and Dormieux and Pecker (1995), or a nonlinear Hoek-Brown failure criterion, as in Serrano and Olalla (1994), Saada et al. (2008, 2011), and Yang and Yin (2006), in which the effects of intermediate principal stress on rock strength are not considered. rock masses. Serrano and Olalla (1994) studied the bearing capacity of strip footings on weightless rock masses using the Hoek-Brown failure criterion (Hoek and Brown, 1980). Serrano et al. (2000) used the modified Hoek-Brown failure criterion (Hoek et al., 1992) to evaluate the static bearing capacity of strip footings on weightless rock masses. Yang et al. (2003) employed the lower bound limit analysis method to calculate the bearing capacity of strip footings on rock masses. Yang (2009) used the upper bound theorem of limit analysis and the modified Hoek-Brown criterion to calculate the seismic bearing capacity of strip footing on rock slopes. Saada et al. (2011) assessed the seismic bearing capacity of strip footings near rock slopes. Sokolovski et al. (1960) has been successfully used to solve many geotechnical problems, including the bearing capacity of foundations. Choudhury & Subba Rao (2005) have studied the problem of the seismic bearing capacity of shallow strip foundations under vertical loads by using limit equilibrium and considering the inertia effects in both soil and structure. as Dormieux & Pecker (1995), Paolucci & Pecker (1997), Soubra (1997; 1999), Zhu (2000) and Ghosh (2008) have addressed this problem in a more rigorously way by using the upper bound theorem of limit analysis theory (Chen & Scawthorn, 1970; Chen, 1975) for an associated flow rule Mohr-Coulomb material.

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## III. METHODOLOGY

### R. Richards equation

Coulomb failure mechanism can be directly extended to the dynamic Earthquake situation.

$$K_{AE} = \frac{\cos^2(\phi - \theta)}{\cos\theta \cos(\delta + \theta) \left\{ 1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \theta)}} \right\}^2} \dots\dots\dots (1a)$$

$$K_{PE} = \frac{\cos^2(\phi - \theta)}{\cos\theta \cos(\delta + \theta) \left\{ 1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \theta)}} \right\}^2} \dots\dots\dots (2a)$$

Where  $K_{AE}$  and  $K_{PE}$  are seismic earth pressure coefficients.  $K_{AE}$  is seismic active earth pressure coefficient.  $K_{PE}$  is seismic passive earth pressure coefficient.

The critical angles of rupture are

$$P_{AE} = a + \tan^{-1} \left\{ \frac{\sqrt{(1 + \tan^2 a)[1 + \tan(\delta + \theta)\cot a]} - \tan a}{1 + \tan(\delta + \theta)(\tan a + \cot a)} \right\} \dots\dots\dots (3a)$$

$$P_{PE} = -a + \tan^{-1} \left\{ \frac{\sqrt{(1 + \tan^2 a)[1 + \tan(\delta - \theta)\cot a]} + \tan a}{1 + \tan(\delta + \theta)(\tan a + \cot a)} \right\} \dots\dots\dots (4a)$$

where  $\theta = \tan^{-1} \frac{K_h}{(1 - K_v)}$  and  $a = \phi - \theta$

The seismic bearing capacity that a footing can sustain is usually calculated by superposition where the contributions from soil cohesion  $c$ , soil unit weight  $\gamma$  and surcharge loading  $q$  are added together. Each contribution can be expressed as a seismic bearing-capacity factor  $N_i$  to give the total capacity.

$$P_{LE} = CN_{CE} + \gamma d N_{qE} + \frac{1}{2} \gamma B N_{\gamma E} \dots\dots\dots (5a)$$

Where  $N_{\gamma E}$ ,  $N_{qE}$  and  $N_{CE}$  are seismic bearing capacity factor. Use the value  $N_{\gamma E}$ ,  $N_{qE}$  and  $N_{CE}$  and put in equation of seismic bearing capacity. To find out seismic bearing capacity of soil.

$$N_{\gamma E} = \tan p_A \left( \frac{K_P}{K_A} - 1 \right)$$

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$$N_{qE} = \frac{K_P}{K_A}$$

$$N_{cE} = (N_{qE} - 1) \cot \phi$$

**Deepankar Choudhury and Kanakapura S. Subba Rao Equation:**

The seismic bearing capacity  $q_{ud}$  are expressed in the following form:

$$Q_{ud} = cN_{cE} + \gamma d N_{qE} + \frac{1}{2} \gamma B N_{\gamma E}$$

where  $N_{cd}$ ,  $N_{qd}$  and  $N_{\gamma d}$  are the seismic bearing capacity factors.

where  $K_{pcd1}$ ,  $K_{pqd1}$ ,  $K_{pcd2}$  and  $K_{pqd2}$  are the seismic passive earth pressure coefficients for unit weight, surcharge and cohesion components acting normally on the faces DE and AE, respectively.

By considering the horizontal equilibrium of all the forces (Figure 2(b))

Are  $N_{cd}$ ,  $N_{qd}$ , and  $N_{\gamma d}$  calculated

$$N_{cd} = \frac{1}{K_h} \left[ \frac{\frac{K_{pcd1} \sin(\alpha_1 - \phi) - \frac{mK_{pcd2} \sin(\alpha_2 - \phi_2)}{\cos \phi_2}}{\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}} + \frac{\sin \alpha_1 \tan \phi_2 \cos \alpha_2}{\sin(\alpha_1 + \alpha_2) \tan \phi} - \frac{\sin \alpha_2 \cos \alpha_1}{\sin(\alpha_1 + \alpha_2)} \right] \dots (1b)$$

$$N_{qd} = \frac{1}{K_h} \left[ \frac{\frac{K_{pqd1} \sin(\alpha_1 - \phi) - \frac{mK_{pqd2} \sin(\alpha_2 - \phi_2)}{\cos \phi_2}}{\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}} \right] \dots \dots \dots (2b)$$

$$N_{\gamma d} = \frac{1}{K_h} \left[ \frac{\frac{K_{pyd1} \sin(\alpha_1 - \phi) - \frac{mK_{pyd2} \sin(\alpha_2 - \phi_2)}{\cos \phi_2}}{\left(\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}\right)^2}}{\left(\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}\right)} \right] \dots \dots \dots (3b)$$

By considering the vertical equilibrium of all the forces (Figure 2(b))

Are  $N_{cd}$ ,  $N_{qd}$ , and  $N_{\gamma d}$  calculated

$$N_{cd} = \frac{1}{1 - K_v} \left[ \frac{\frac{K_{pcd1} \cos(\alpha_1 - \phi) - \frac{mK_{pcd2} \cos(\alpha_2 - \phi_2)}{\cos \phi_2}}{\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}} + \frac{\sin \alpha_1 \tan \phi_2 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2) \tan \phi} + \frac{\sin \alpha_2 \sin \alpha_1}{\sin(\alpha_1 + \alpha_2)} \right] \dots (4b)$$

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$$N_{qd} = \frac{1}{1-K_v} \left[ \frac{\frac{K_p q d_1}{\cos \phi} \cos(\alpha_1 - \phi) - \frac{m K_p q d_2}{\cos \phi_2} \cos(\alpha_2 - \phi_2)}{\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}} \right] \dots\dots\dots (5b)$$

$$N_{\gamma d} = \frac{1}{1-K_v} \left[ \frac{\frac{K_p \gamma d_1}{\cos \phi} \cos(\alpha_1 - \phi) - \frac{m K_p \gamma d_2}{\cos \phi_2} \cos(\alpha_2 - \phi_2)}{\left(\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}\right)^2} \right] \cdot \frac{1}{\left(\frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2}\right)} \dots\dots\dots (6b)$$

The central angle  $\chi$  of the wedge ADE is given by  $\chi = \pi - \alpha_1 - \alpha_2$

In this work, only Richards et al. (1993) and Budhu and Al-Karni (1993) had considered the effect of  $K_v$  on seismic bearing capacity factors while others had considered  $K_v = 0$  in their analysis

$K_h = 0.0, 0.1, 0.2, 0.3, 0.4, 0.5$

$K_v = 0.0K_h, 0.5K_h, 1 K_h$

$$m = \frac{\tan \phi_2}{\tan \phi}$$

where  $\phi_2$  is the mobilized friction angle ( $\phi_2 \leq \phi$ )

In the static case, full mobilization of passive earth pressure on both the sides AD and DE of the wedge ADE will occur due to symmetry. Here  $m = 1$ , i.e. ( $\phi_2 = \phi$ ) To evaluate the seismic bearing capacity factors, say  $N_{cd}$ , some value of  $\alpha_1$  is first assumed, and with known value of central angle  $\chi$   $\alpha_2$  is obtained. Iteration is carried out in the range of  $\phi$  to  $(180 - \chi - 1)$  with interval of 0.001 for  $\alpha_1$ . Again some value of  $c$  is assumed to obtain  $m$ . The values of  $\alpha_2$  are iterated from 0 to  $\phi$  with interval of 0.01. For only a particular set of  $\alpha_1$  and  $m$ , the two values of  $N_{cd}$  and  $N_{\gamma d}$  will be the same. This extensive numerical iteration technique is carried out in a supercomputer to obtain the result for  $N_{cd}$ . In a similar way  $\alpha_1$  and  $m$  are determined to obtain  $N_{qd}$  [using  $N_{cd}$  and  $N_{\gamma d}$ ] and Thus the ultimate bearing capacity of horizontal shallow strip footing in soils under seismic conditions are calculated.

### IV. CALCULATION AND RESULT

- Calculation of bearing Capacity given in tabular form.

Nc	Nq	Ny	Result
0	1	0	17.00000
6.2137	2.6649	1.17916	246.74859
13.04377	4.49506	3.12231	507.53857
23.50801	7.29895	6.73078	915.13999
40.5532	11.86621	13.54439	1591.01254
60.79213	19.7007	26.8577	2501.11148

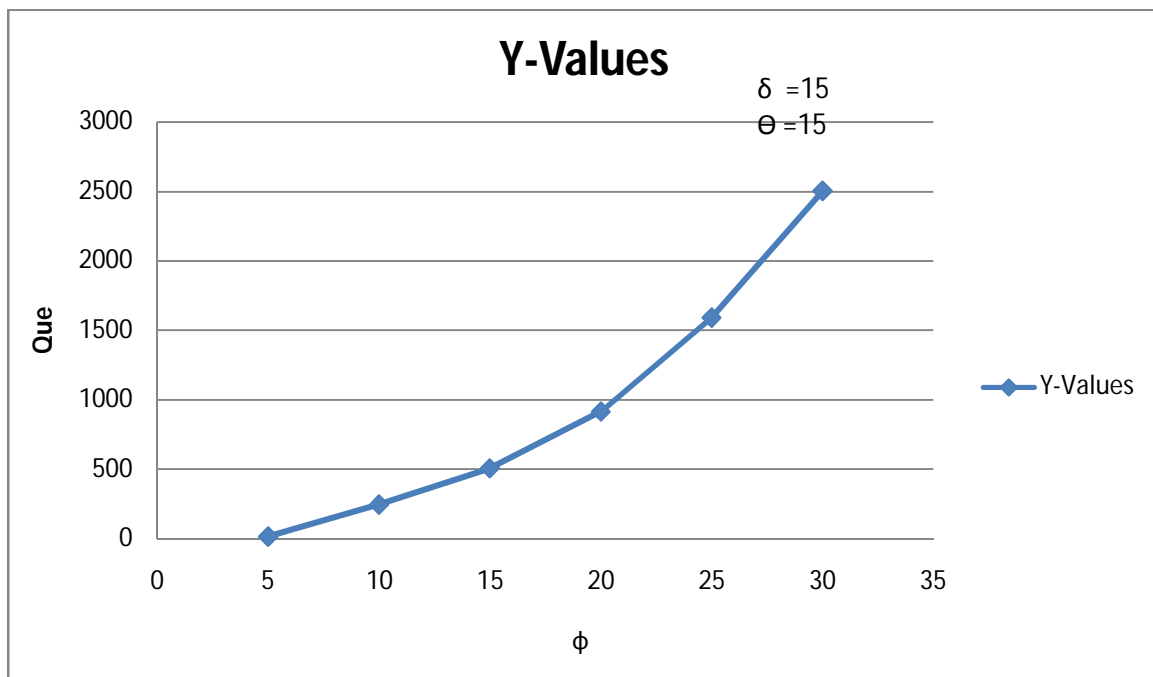
**Table: Calculation of Bearing Capacity**

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**Fig.:** Graph between Que and  $\phi$

Nc	Nq	Ny	Result
17.61603	15.18149	3.54995	831.82812
14.54987	12.4203	2.814804	683.52995
9.69991	8.280201	1.712402	453.59383
7.27493	6.21015	1.161201	338.62576
5.81995	4.96812	0.830479	269.64515
4.849959	4.1401	0.60999	223.65784

**Table:** Calculation of bearing Capacity by Richard equation

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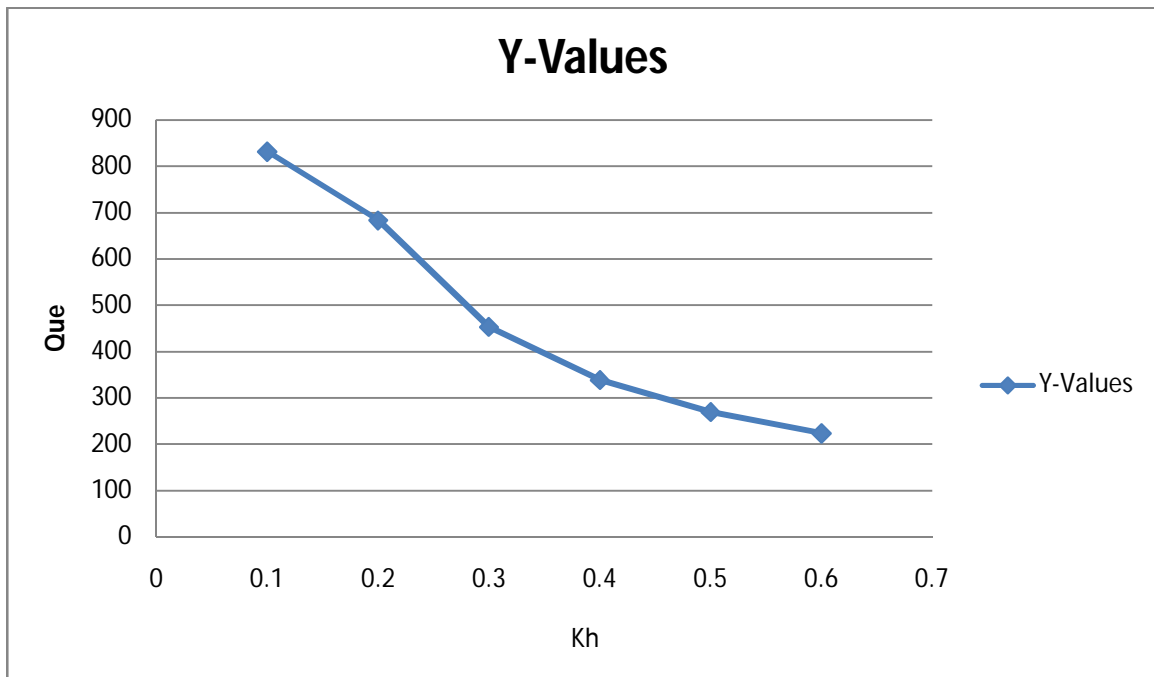


Fig.: Graph between Que and Kh

Nc	Nq	Ny	Result
6.06481	4.39061	0.67668	265.21234
9.90744	7.58322	1.52681	445.60477
10.49023	8.029293	1.645591	472.18620
11.14587	8.53112	1.779216	502.09014
11.88892	9.09986	1.930657	535.98110
12.73813	9.74985	2.103733	574.71395

Table: Calculation of Bearing Capacity by Deepakar equation

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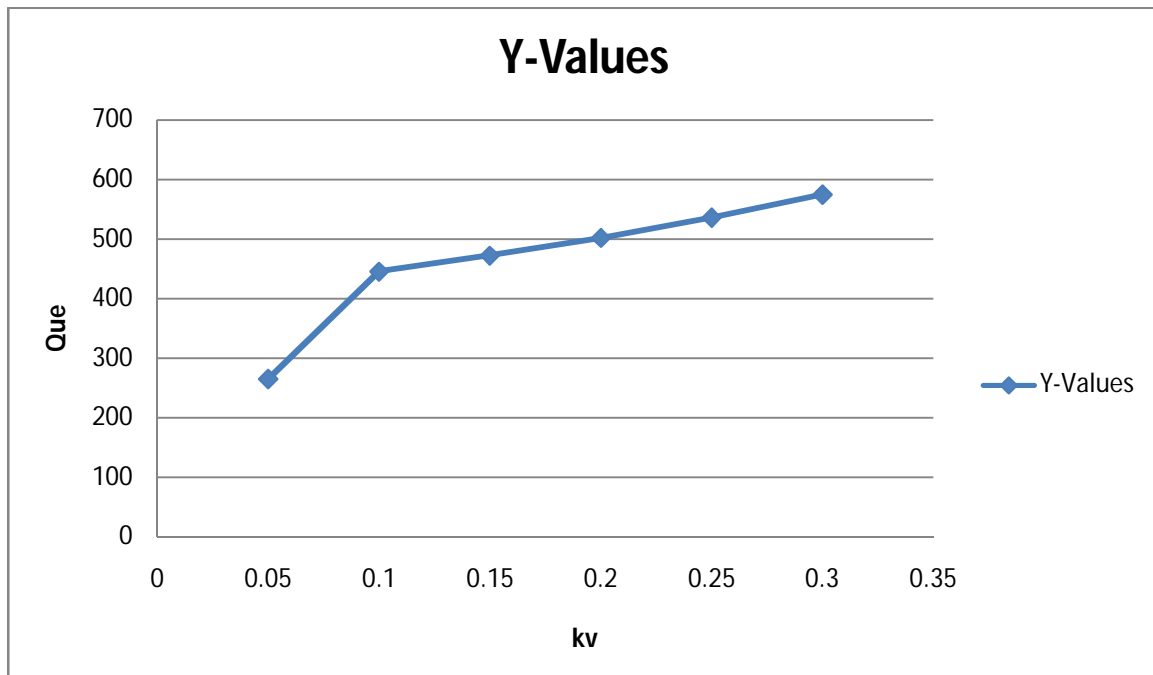


Fig.: Graph between Que and Kv

## V. CONCLUSION

Coulomb active-passive wedge mechanism is not exact for computing seismic bearing capacity. It allows straight forward development of seismic bearing capacity due to straight forward approach of this can serve better in the aspect of earthquake engineering. Seismic force analysis by pseudo-static approach leads to the minimum seismic bearing capacity factor. In this paper, the principles and methodology for Determining Seismic Bearing Capacity of Shallow Strip Footing were reviewed. And, the equation of R. Richards and Deepankar Choudhury and Kanakapura S. Subba Rao for determining Seismic Bearing Capacity for shallow strip footing are discussed. The research activities carried out in India and various part of world are also highlighted.

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