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Vijay K. Puri
SIU Carbondale, IL

Shamsher Prakash
Missouri University of Science and Technology, prakash@mst.edu

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SHALLOW FOUNDATIONS FOR SEISMIC LOADS: DESIGN CONSIDERATIONS

Vijay K. Puri
Professor, Civil Engineering
SIU Carbondale, IL
puri@engr.siu.edu

Shamsher Prakash
Professor Emeritus
MST, Rolla ,MO
prakash@mst.edu

ABSTRACT

The seismic design of foundations for structures depends on dynamic bearing capacity, dynamic settlements and liquefaction susceptibility of soil. The dynamic bearing capacity problem has been attracting the attention of researchers during the last about fifty years. Till today (2013), there is no accepted dynamic bearing capacity theory. Most analysis for design of shallow foundations under seismic loads are based on the assumption that the failure zones in soil occur along a static failure surface. This is the pseudo-static approach. An attempt has been made in this paper to summarize the currently available information on design of shallow foundations under seismic loading. The case of a foundation resting on an upper non-liquefying layer overlying a layer susceptible to liquefaction is also included. The methods for determining the foundation settlements are also discussed.

INTRODUCTION

Shallow foundations may experience a reduction in bearing capacity and increase in settlement and tilt due to seismic loading as has been observed during several earthquakes. The foundation must be safe both for the static as well for the dynamic loads imposed by the earthquakes. The earthquake associated ground shaking can affect the shallow foundation in a variety of ways:

(1) Cyclic degradation of soil strength may lead to bearing capacity failure during the earthquake.
(2) Large horizontal inertial force due to earthquake may cause the foundation to fail in sliding or overturning.
(3) Soil liquefaction beneath and around the foundation may lead to large settlement and tilting of the foundation.
(4) Softening or failure of the ground due to redistribution of pore water pressure after an earthquake which may adversely affect the stability of the foundation post-earthquake.

Bearing capacity failures of shallow foundations have been observed in Mexico City during Michoacan earthquake of 1985 (Mendoza and Avunit (1988), Zeevart (1991)) and in city of Adapazari due to 1999 Kocaeli earthquake (Karaca (2001), Bakir et.al. (2002) and Yilmaz et. al (2004)).Typical examples of bearing capacity failure in Adapazari are shown in Fig. 1. The surface soils at the site of foundation damage belong to CL/ ML group which are generally considered non-liquefiable. Settlements of as much as 0.5-0.7m have been observed in loose sands in Hachinohe during the 1968 Tokachioki earthquake of magnitude 7.9. Settlements of 0.5 - 1.0 m were observed at Port and Roko Island in Kobe due to the Hygoken Nanbu (M=6.9) earthquake. Foundation failures may occur due to reduction in bearing capacity, excessive settlement and tilt, both in liquefying and non-liquefying soils.

CONSIDERATIONS IN FOUNDATION DESIGN

Foundation design depends on the several factors like site location and conditions, soil parameters and nature of applied loads on the foundation . The foundation must be safe which can be ensured by meeting the design criteria. Foundation must be safe for the static condition as well as for the seismic condition. The information on seismic design of shallow foundations is presented below for four different cases:

(1) Shallow Foundations on Soils Not Prone to Liquefaction.
(2) Settlement of Shallow Foundations on Soils Not Prone to Liquefaction.
(3) Shallow Foundation on Soil Prone to Liquefaction.
(4) Settlement of Shallow Foundations on Soil Prone to Liquefaction.

The pseudo-static approach is commonly followed for design of foundation under seismic conditions. Therefore a brief review of commonly used bearing capacity theories is given first.

(a)

(b)

Fig.1 Examples of Bearing Capacity Failures of Shallow Foundations in Adapazari (Yilmaz et. al. 2004).

STATIC CASE

The static loads covers loads like self-weight of the structure, soil loads, surcharge loads and live loads. The calculations then involve estimation of the safe bearing capacity of the footing and the amount of settlement. The conventional design procedure involves selection of allowable bearing capacity as the smaller of the following two values; the safe bearing capacity, based on ultimate capacity and the allowable bearing pressure and based on tolerable settlement. Terzaghi (1943), Meyerhof (1951), Hansen (1970), Vesic (1973), Kumar (2003), Dewaikar and Mohapatro (2003) and many others have done research in this area and either proposed new design equations or proposed correction factors for the prevalent equations.

Terzaghi's Analysis

The static approach based on Terzaghi's general shear failure is shown in Fig.2. For a continuous or strip foundation, the ultimate bearing capacity is obtained as:

$$q_u = c\,N_c + q\,N_q + 0.5\,\gamma\,B\,N_\gamma \tag{1}$$

c = Cohesion of soil
γ = unit weight of soil
q = Surcharge Pressure = γ D
B=width of the foundation
D= depthe of the foundation.

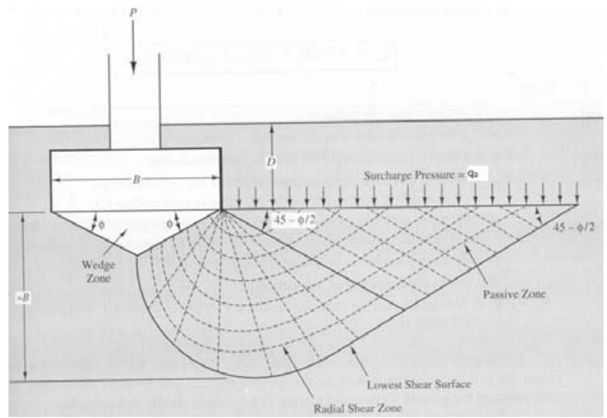

Fig. 2 Failure mechanism suggested by Terzaghi (1943)

N_c, N_q, N_γ = Bearing capacity factors (depend only on the soil friction angle ø) . These bearing capacity factors can be obtained from Table 1.

The ultimate bearing capacity for various foundation shapes can be obtained as follows:

For square footing:

$$q_u = 1.3\ c\ Nc + qD\ Nq + 0.4\ \gamma\ B\ N\gamma \tag{2}$$

For circular footing:

$$q_u = 1.3\ c\ Nc + qD\ Nq + 0.3\ \gamma\ B\ N\gamma \tag{3}$$

For rectangular footing:

$$q_u = c\ Nc\ (1+0.3\ B/L) + qD\ Nq + 0.4\ \gamma\ B\ \gamma \tag{4}$$

Where B= width or diameter of the footing and L=length of the footing.

Meyerhoff's Analysis

The Terzaghi's (1943) equation for ultimate bearing capacity was modified by Meyerhoff (1963) to give a more general solution. The value of q_u is obtained as (Meyerhoff ,1963):

$$q_u = c\ N_c s_c\ d_c\ i_c + q\ N_q\ s_{qs}\ d_q\ i_q + 0.5\ \gamma\ B\ N_\gamma\ s_\gamma\ d_\gamma\ i_\gamma \tag{5}$$

s_c, s_q, s_γ = Shape Factors
d_c, d_q, d_γ = Depth Factors
i_c, i_q, i_γ = Load Inclination Factors.

The values of bearing capacity factors for use in Eq. 5 may be obtained from Eqns. 6 through 8.

Table 1. Terzaghi's Bearing Capacity Factors (General Shear Failure)

Ø	N_c	N_q	$N_Ø$
0	5.7	1	0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	35.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5

$$N_q = e^{\pi \tan\varphi} \tan^2 (45^0 + Ø/2) \quad (6)$$
$$N_c = (N_q - 1) \cot Ø \quad (7)$$
$$N_\gamma = (N_q - 1) \tan (1.4 Ø) \quad (8)$$

The shape, depth and inclination factors can be calculated using equations given in Table 3.

SEISMIC CASE

Shallow Foundation on Soils Not Prone to Liquefaction

The design of foundations in earthquake prone areas requires different design approach involving earthquake forces along with the usual dead and live loads considered in the static analysis. The design approach involving limit equilibrium method or equivalent static method with consideration of pseudo-static seismic forces along with other static forces has been used as a primary method for the design of shallow foundations in seismic areas. Reduction in bearing capacity of the underlying soil and increase in settlement and tilt are the main causes of failure of a shallow foundation when subjected to seismic loading (Sarma and Iossifelis (1990), Richards et. al. (1993) and Budhu and Al-Karni (1993), Kumar and Kumar (2003) Choudhury and Rao (2005)). So, the main interest lies in first determining the soil parameters and then soil-structure interaction and seismic behavior to determine the nature of failure and finally, estimate the seismic bearing capacity of the footing as accurately as possible. A good design approach would require consideration of all possible factors such as soil parameters, seismic vulnerability, nature of applied loads and seismic soil-foundation interaction for an effective estimation of the seismic bearing capacity.

Table 2. Meyerhof's Shape, Depth and Inclination factors

Shape Factors	Depth Factors	Inclination Factors
$S_c = 1 + 0.2\, K_p \frac{B}{L}$	$d_c = 1 + 0.2 \sqrt{K_p} \frac{D}{B}$	$i_c = i_q = (1 - \frac{\alpha}{90°})^2$
(i) for Ø = 0° $S_q = S_\gamma = 1.0$	(i) For Ø = 0° $d_q = d_\gamma = 1.0$	$i_\gamma = (1 - \frac{\alpha}{Ø})^2$
(ii) For Ø ≥ 10° $S_q = S_\gamma = 1 + 0.1\, K_p \frac{B}{L}$	(ii) For Ø ≥ 10° $d_q = d_\gamma = 1 + 0.1 \sqrt{K_p} \frac{D}{B}$	α = angle of resultant measured from vertical axis $K_p = \tan^2\left(45° + \frac{Ø}{2}\right)$

Pseudo-static Approach. This analysis technique uses limit equilibrium methods in which the inertial forces generated on the structure due to shaking of the ground are simply accounted for by an equivalent unidirectional horizontal and vertical forces, is termed as the Pseudo-static Approach. The equivalent forces are taken as the mass of the body multiplied by coefficients of acceleration for both horizontal and vertical directions. These coefficients are termed as seismic acceleration coefficients, K_h and K_v, for horizontal and vertical direction respectively. The horizontal force may also produce a moment. The foundation may thus, be treated as being subjected to combined action of vertical and, horizontal loads and moments. If the foundation is subjected only to vertical loads and moments, then it may be designed as eccentrically loaded foundation. The eccentricity 'e' is defined as;

$$e = \frac{M}{V} \quad (9)$$

In which, V = vertical load and,
M = Moment.

The effective width $\acute{B} = B - 2e$

The ultimate bearing capacity may be obtained using Eqs. 1-5 by replacing B with $\acute{B}$

When the foundation is subjected to a combination of vertical loads, horizontal loads and moments, it may be designed as foundation subjected to inclined eccentric load.

The angle inclination with the vertical 'α' is given by:

$$\alpha = tan^{-1}\left(\frac{H}{V}\right) \quad (10)$$

In which, H = horizontal load.

In this case Eq. 5 should be used to calculate the value of the ultimate bearing capacity. It may be noted that in this approach, the bearing capacity is estimated using the static bearing capacity factors and any effects of the earthquake loads on the supporting soil are not considered. This implies that the failure surface below the foundation for the earthquake load is assumed to be the same as for the static case. The estimated bearing capacity should, therefore, be considered as approximate only. Attention has been given in recent years to better define the failure surface below the foundation for the seismic case and estimate the bearing capacity factors and still following the pseudo-static approach. This is discussed below:

Developments in Determination of Seismic Bearing Capacity

Sarma and Iossifelis (1990), Richards et. al. (1993) and Budhu and Al-Karni (1993) made changes in Meyerhof's (1963) model and used a different approach based on limit equilibrium method with consideration of the upper bound solutions only. These solutions were dependent on the predetermined failure mechanism. Pecker and Salencion's, (1991) considered the soil inertial force to be independent to correctly account for its influence. They (Pecker and Salencion ; 1991) considered this inertial force to estimate the reduction in the bearing capacity of the foundation and on top of that they also considered the same seismic horizontal coefficient K_h for both soil and structure . This led to somewhat erroneous conclusions. This approach was later modified by Dormieux and Pecker (1995), who determined load inclination and eccentricity on the foundation to be the main cause for the reduction of bearing capacity rather than soil inertial force. Moreover, unlike previous researches which used limit equilibrium method (Dormieux and pecker (1995), Soubra (1997, 1999)) used upper bound limit analysis for the estimation of the seismic bearing capacity factors. Later a new approach was introduced by Kumar and Rao (2002, 2003) to determine seismic bearing capacity of footing using method of characteristics. Their analyses also didn't consider the effect of the vertical component of the ground acceleration. Up to this time, the effect of ground shaking was only considered in the horizontal direction, in other word, only horizontal acceleration due to earthquake was taken into account. Among others, Sarma and Iossifelis (1990), Richards et. al. (1993) and Budhu and Al-Karni (1993), Kumar and Kumar (2003) assumed focus of the log spiral surface to be at the edge of the footing. Choudhury and Rao (2005) proposed a new approach, which also used the limit equilibrium method to find the seismic bearing capacity factors including the seismic forces on both soil and structure and considered planar and log-spiral failure surfaces below the foundation.. They also calculated seismic bearing capacity factors for cohesion, surcharge and unit weight of soil for various soil friction angles and seismic acceleration coefficients. Unlike previous researches, Choudhury and Rao (2005) considered the seismic acceleration in both horizontal and vertical directions and also determined the critical failure surface.
Some of these significant developments in estimation seismic bearing capacity determination are discussed below.

Estimation of Seismic bearing capacity

Richards, Elms and Budhu (1990, 1993)

Richards, Elms and Budhu's (1990) developed the concept of 'Dynamic Fluidization of Soils' which implies an increase in the shear flow in soil with an increase in ground acceleration. They observed that, although dynamic fluidization looks similar to liquefaction, it is an altogether different phenomenon. Their work shows that in dynamic fluidization the shear flow takes place at finite levels of effective stress, whereas liquefaction is accompanied by the reduction in the effective stress to zero due to increase in pore pressure. The difference is also shown in the displacements, which are unbounded in case of liquefaction and finite and incremental in case of fluidization. Richards, Elms and Budhu (1993) used the concept of dynamic fluidization of soil to formulate equations for the seismic bearing capacity of foundation. They modified Prandl's bearing capacity analysis using planar failure surfaces.

Budhu and Al_Karni (1993)

Logarithmic failure surfaces shown in Fig. 3 were assumed by Budhu and Al-karni (1993) to determine the seismic bearing capacity of soils. They suggested modifications to the commonly used (Terzagh's) equations for static bearing capacity to obtain the dynamic bearing capacity as follows:

$$q_{ud} = c\ N_c\ s_c\ d_c\ e_c + q\ N_q\ s_q\ d_q\ e_q + 0.5\ \gamma\ B\ N_\gamma\ s_\gamma\ d_\gamma\ e_\gamma \quad (11)$$

Where,

N_c , N_q, N_γ are the static bearing capacity factors.
s_c, s_q, s_γ are static shape factors.
d_c, d_q, d_γ are static depth factors
e_c , e_q and e_γ are the seismic factors estimated using following equations

$$e_c = \exp\left(-4.3k_h^{l+D}\right) \quad (12)$$

$$e_q = (1-k_v)\exp\left[-\left(\frac{5.3k_h^{1.2}}{1-k_v}\right)\right] \quad (13)$$

$$e_\gamma = (1-\frac{2}{3}k_v)\exp\left[-\left(\frac{9k_h^{1.2}}{1-k_v}\right)\right] \quad (14)$$

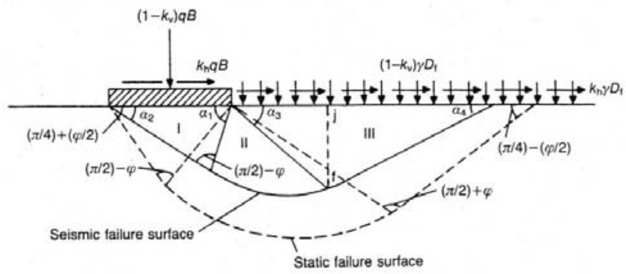

Fig. 3. Failure Surfaces used by Budhu and al-karni (1993) for Static and Seismic Case

Where,

K_h and K_v are the horizontal and vertical acceleration coefficients respectively.

H= depth of the failure zone from the ground surface and

$$D = \frac{\gamma H}{C} \frac{0.5B}{\cos\left(\frac{\pi}{4} + \frac{\phi}{2}\right)} \exp\left(\frac{\pi}{2}\tan\phi\right) + D_f \qquad (15)$$

D_f = depth of the footing and
φ = angle of internal friction
c=cohesion of soil

Budhu and Al-Karni's (1993) also compared the effects of K_h and K_v on N_{cE}/N_c , N_{qE}/N_q and $N_{\gamma E}$ /N_γ for various angles of friction and also with results of other researchers. The comparisons are shown in Figs. 4 through 8.

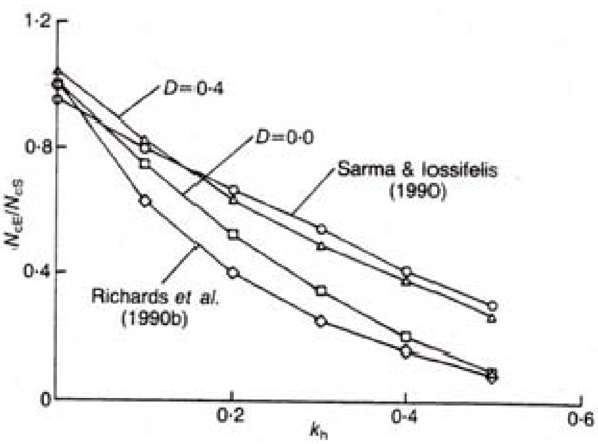

Fig. 4. Effect of k_h on N_{cE}/N_c for Ø = 30° (Budhu and Al-Karni, 1993)

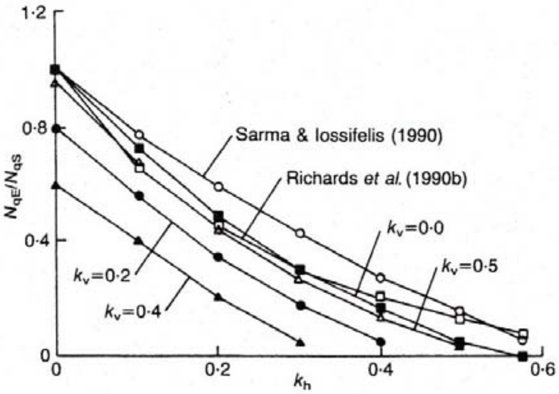

Fig. 5. Effect of k_h on N_{qE}/N_q for Ø = 30° (Budhu and Al-Karni;1993)

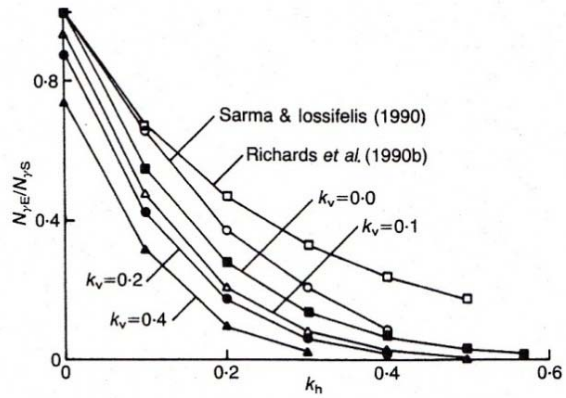

Fig. 6. Effect of k_h and k_v on N_{qE}/N_q for ϕ =30°; (Budhu and Al-Karni ; 1993)

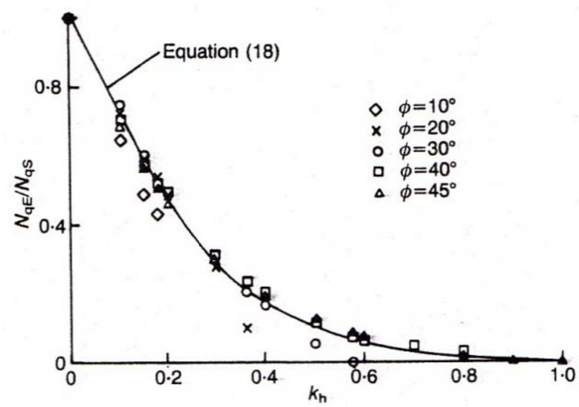

Fig. 7. Effect of k_h on $N_{\gamma E}/N_\gamma$ for Various Ø values (Budhu and Al-Karni ;1993)

Chaudhury and Rao (2005, 2006)

A study of the seismic bearing capacity of shallow strip footing was conducted by Chaudhury and Subba Rao (2005,2006). The failure surfaces for the static and seismic case are shown in Fig. 9. They used the limiting equilibrium approach and the equivalent static method to represent the seismic forces and obtained the seismic bearing capacity factors.

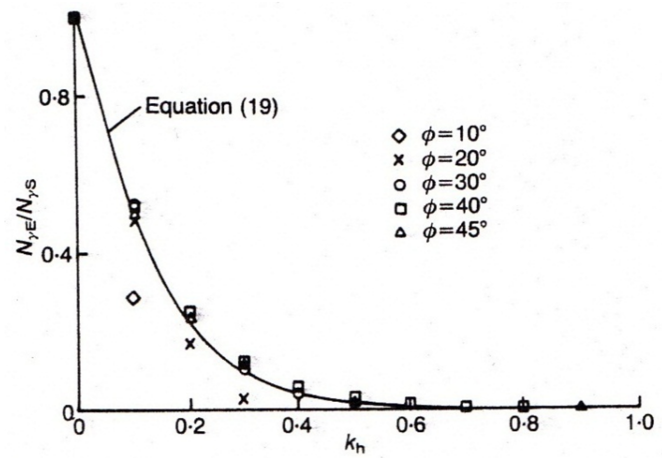

Fig. 8. Effect of k_h and k_v on $N_{\gamma d}/N_\gamma$ for various angles of friction by Budhu and Al-Karni's (1993)

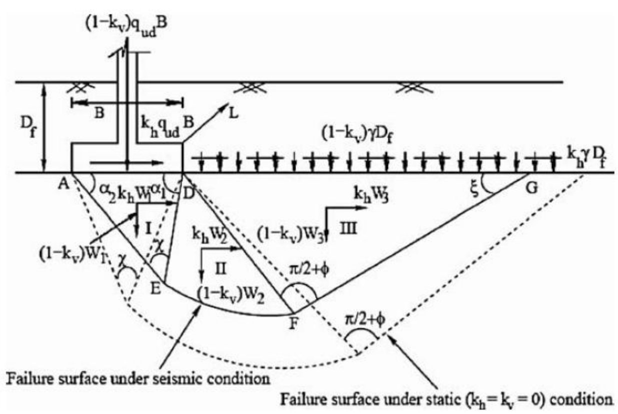

Fig. 9. Failure mechanism Assumed by Chaudhury and Rao (2005, 2006)

Using equilibrium of all vertical forces Choudhury and Rao (2005, 2006) formulated the final expression for the ultimate seismic bearing capacity q_{ud}.

$$q_{ud} = c\,N_{cd} + q\,N_{qd} + 0.5\,\gamma\,B\,N_{\gamma d} \qquad (16)$$

Where, N_{cd}, N_{qd} and $N_{\gamma d}$ are seismic bearing capacity factors which are quantified using equilibrium of all the forces in the horizontal direction. The expressions are as follows:

$$N_{cd} = \frac{1}{k_h}\left[\frac{\frac{K_{pcd1}}{\cos\phi}\sin(\alpha_1-\phi) - \frac{mK_{pcd2}}{\cos\phi_2}\sin(\alpha_2-\phi_2)}{\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}} + \frac{\sin\alpha_1\,\tan\phi_2\cos\alpha_2}{\sin(\alpha_1+\alpha_2)\tan\phi} - \frac{\sin\alpha_2\cos\alpha_1}{\sin(\alpha_1+\alpha_2)}\right] \qquad (17)$$

$$N_{qd} = \frac{1}{k_h}\left[\frac{\frac{K_{pqd1}}{\cos\phi}\sin(\alpha_1-\phi) - \frac{mK_{pqd2}}{\cos\phi_2}\sin(\alpha_2-\phi_2)}{\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}}\right] \qquad (18)$$

$$N_{\gamma d} = \frac{1}{k_h}\left[\frac{\frac{K_{p\gamma d1}}{\cos\phi}\sin(\alpha_1-\phi) - \frac{mK_{p\gamma d2}}{\cos\phi_2}\sin(\alpha_2-\phi_2)}{\left(\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}\right)^2}\right] - \frac{1}{\left(\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}\right)} \qquad (19)$$

$$N_{cd} = \frac{1}{1-k_v}\left[\frac{\frac{K_{pcd1}}{\cos\phi}\cos(\alpha_1-\phi) + \frac{mK_{pcd2}}{\cos\phi_2}\cos(\alpha_2-\phi_2)}{\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}} + \frac{\sin\alpha_1\,\tan\phi_2\sin\alpha_2}{\sin(\alpha_1+\alpha_2)\tan\phi} + \frac{\sin\alpha_2\sin\alpha_1}{\sin(\alpha_1+\alpha_2)}\right] \qquad (20)$$

$$N_{qd} = \frac{1}{1-k_v}\left[\frac{\frac{K_{pqd1}}{\cos\phi}\cos(\alpha_1-\phi) + \frac{mK_{pqd2}}{\cos\phi_2}\cos(\alpha_2-\phi_2)}{\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}}\right] \qquad (21)$$

$$N_{\gamma d} = \frac{1}{1-k_v}\left[\frac{\frac{K_{p\gamma d1}}{\cos\phi}\cos(\alpha_1-\phi) + \frac{mK_{p\gamma d2}}{\cos\phi_2}\cos(\alpha_2-\phi_2)}{\left(\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}\right)^2}\right] - \frac{1}{\left(\frac{1}{\tan\alpha_1}+\frac{1}{\tan\alpha_2}\right)} \qquad (22)$$

Where, ϕ values considered in the analysis are to satisfy the relationship given by

$$\phi > \tan^{-1}\left[\frac{k_h}{1-k_v}\right]$$

The variation of bearing capacity factors for various values seismic coefficients given by Chaudhury and Rao (2005, 2006) are shown in figures 10, 11 and 12.

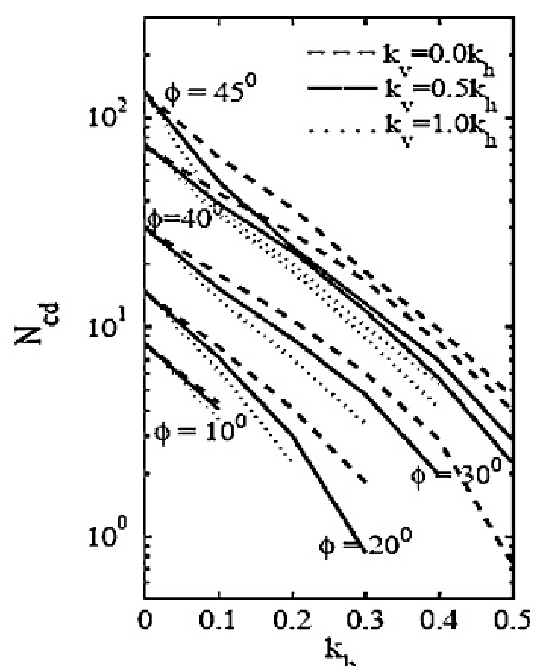

Fig. 10. Variation of N_{cd} with k_h. by Chaudhury and Rao (2005, 2006)

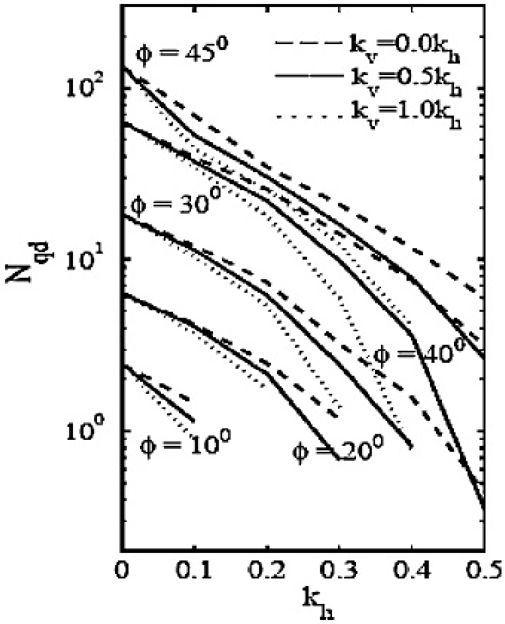

Fig. 11. Variation of N_{qd} with k_h by Chaudhury and Rao (2005, 2006)

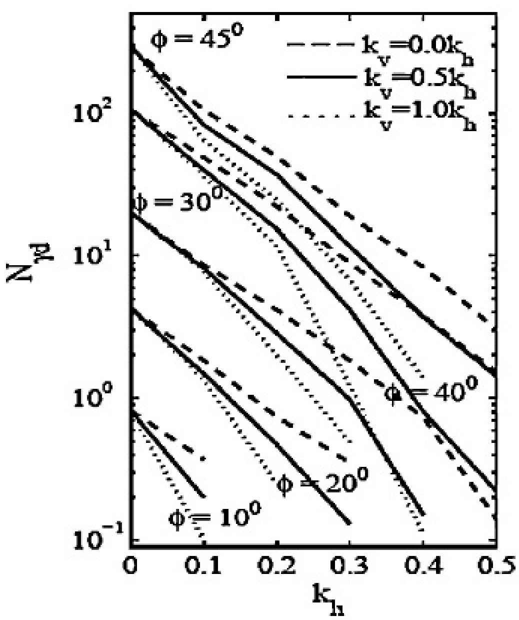

Fig. 12. Variation of N_{cd} with k_h by Chaudhury and Rao (2005, 2006)

Chaudhury and Rao (2005, 2006) also made a comparison of seismic bearing capacity factors obtained by them with those reported by other researchers. Typical such comparisons with other investigations are shown in Figs.13, 14 and15.

It is quite apparent from the comparisons shown in Figs. 13-15 that the values for the seismic bearing capacity factors suggested by Chaudhury and Rao (2005, 2006) are somewhat smaller than those suggested by other previous researchers. Hence, the comparison concludes that Chaudhury and Rao's work yields conservative seismic bearing capacities of a shallow footing.

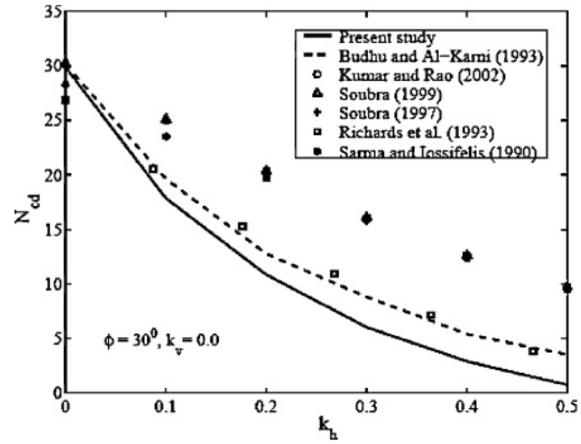

Fig. 13. Comparison of N_{cd} by Chaudhury and Rao (2005, 2006) with other studies in seismic case for ø = 30° and k_v = 0

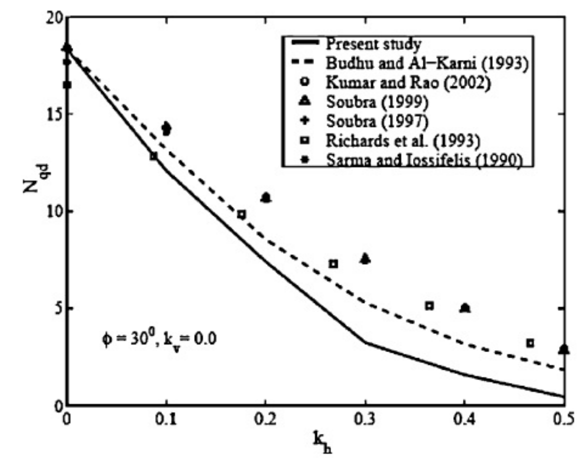

Fig. 14. Comparison of N_{qd} by Chaudhury and Rao (2005, 2006) with other studies in seismic case for ø = 30° and k_v = 0

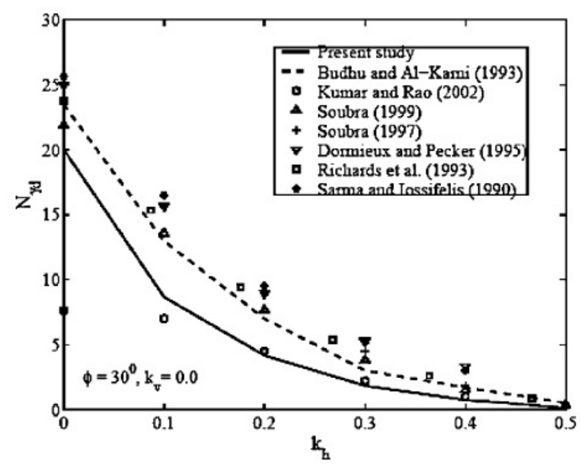

Fig. 15. Comparison of $N_{\gamma d}$ by Chaudhury and Rao (2005, 2006) with other studies in seismic case for ø =

Code Provisions

The relevant guidelines given by Eurocode and International Building Code regarding design of foundations in seismic areas are briefly enumerated here. Unlike Euro-code 8, IBC 2006 doesn't provide us with the equations for the bearing capacities but rather provides prerequisites in selecting parameters for designing foundation under earthquake loads.

Eurocode 8 - Part 5. Eurocode 7 mainly covers the specifications for the static geotechnical designs. The dynamic design and analyses are covered in Eurocode 8 and the earthquake resistive design criteria for foundations, retaining structures and geotechnical aspects are covered in Part 5 of Eurocode 8. The code suggests the procedure to check the stability of the shallow strip foundation under seismic bearing capacity failure for different types of soils.

The general expression for the check is given as

$$\frac{\left(1-e\overline{F}\right)^{c_T}\left(\beta\overline{V}\right)^{c_T}}{\left(\overline{N}\right)^{a}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{b}}+\frac{\left(1-f\overline{F}\right)^{c'_M}\left(\gamma\overline{M}\right)^{c_M}}{\left(\overline{N}\right)^{c}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{d}}-1\leq 0 \qquad (23)$$

Where,

$$\overline{N}=\frac{\gamma_{Rd}N_{Ed}}{q_{ud}}\ ,\quad \overline{V}=\frac{\gamma_{Rd}V_{Ed}}{q_{ud}}\ ,\quad \overline{M}=\frac{\gamma_{Rd}M_{Ed}}{B\,q_{ud}}$$

$\overline{F}$ = dimensionless inertia force
N_{Ed}, V_{Ed}, M_{Ed} = the design action effects at the foundation level
γ_{Rd} = model partial factor
q_{ud} = Ultimate bearing capacity of the foundation under a vertical centered load
a, b, c, d, e, f, m, k, k', c_T, c_M, c'_M, β, γ are numerical parameters depending on type of soils
The expressions and values for these entities are defined later for different types of soils.

For purely cohesive soil

$$q_{ud}=(\pi+2)\frac{\overline{c}}{\gamma_{Rd}}B \qquad (24)$$

Where,
$\overline{c}$ = the un-drained shear strength of soil, c_u, for cohesive soil, or the cyclic un-drained shear strength, τ_u, for cohesion-less soils

γ_{Rd} = the partial factor for material properties

Now, $\overline{F}$ is given by

$$\overline{F}=\frac{\rho\cdot a_g\cdot S\cdot B}{\overline{c}} \qquad (25)$$

ρ = the unit mass of the soil
$a_g = \gamma_I\, a_{gR}$ = the design ground acceleration on type A ground
a_{gR} = the reference peak ground acceleration on type A ground
γ_I = the importance factor
S = the soil factor can be obtained from Table 3 as defined in EN 1998-1:2004, 3.2.2.2

Table 3. The Soil Factor (S) for different elastic response spectra

Ground Type	*Soil Factor (S)*	
	Type 1 elastic response spectra	Type 2 elastic response spectra
A	1.0	1.0
B	1.2	1.35
C	1.15	1.5
D	1.35	1.8
E	1.4	1.6

and $0<\overline{N}\leq 1$, $\left|\overline{V}\right|\leq 1$ constraints are to be satisfied.
For purely cohesion-less soil

$$q_{ud}=\frac{1}{2}\rho\, g\left(1\pm\frac{a_v}{g}\right)B^2 N_\gamma \qquad (26)$$

$\overline{F}$ is given by

$$\overline{F}=\frac{a_g}{g\ \tan\phi'_d} \qquad (27)$$

a_v = may be taken equal to 0.5 a_g.S
N_γ = the bearing capacity factor, as a function of the shearing angle Ø

and $0<\overline{N}\leq\left(1-m\overline{F}\right)^{k'}$ constraint is to be satisfied.

The values for the numerical parameters and model partial factor are given in Tables 4 and 5.

In most situations $\overline{F}$ may be taken as being equal to 0 for cohesive soils and may be neglected for cohesion-less soils if $a_g.S < 0.1g$ (i.e. if $a_g.S < 0.98\ m/s^2$)

IBC (2006). IBC 2006 provides provisions for designing foundations under seismic loading conditions closely in relation to ASCE 7. Chapter 18 of IBC 2006 deals with Soils and Foundations. As per ASCE code the structures are categorized into six Seismic Design Categories A, B, C, D, E and F. These categories are based on the use, importance and size of the structures. The IBC 2006 makes use of this categorization and suggests necessary provisions for structures and footing falling in the respective categories.

Table 4. Values of Numerical Parameters Eurocode 8-5

	Purely cohesive soil	Purely cohesionless soil
a	70 0	92
b	29 1	25
c	14 0	92
d	81 1	25
e	21 0	41
f	44 0	32
m	21 0	96
k	22 1	00
k'	00 0	39
c_T	00 1	14
c_M	00 1	01
c'_M	00 1	01
β	57 2	90
γ	85 2	80

Table 5. Values of the model partial factor γ_{Rd} Eurocode 8-5

Medium-dense to dense sand	Loose dry sand	Loose saturated sand	Non sensitive clay	Sensitive clay
1.00	1.15	1.50	1.00	1.15

For Seismic Design Category C

IBC 2006 suggests for conducting an investigation and evaluation of the potential earthquake hazards like slope instability, liquefaction and surface rupture due to faulting or lateral spreading for the structures determined to be in the this category.

For Seismic Design Category D, E or F

According to IBC 2006 the structures falling under the Seismic Design Category D, E or F are subject to additional soil investigation requirements on top of that suggested by Seismic Design Category C. These investigations can be listed as follows:

- A determination of lateral pressures on basement and retaining walls due to earthquake motions.
- Assessment of potential consequences of any liquefaction and degradation of soil strength along with estimation of differential settlement, lateral movement and reduction in bearing capacity.
- This provision also addresses mitigation measures. Measures range from soil stabilization to the selection of appropriate type and depth of foundation
- The potential for liquefaction and loss in soil strength shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motion. The peak ground motion is as specified by ASCE 7.

Other provisions

- Interconnected ties are to be provided for the individual spread footings supported on the soil defined as Site Class E or F. IBC2006 specifies standards for these ties to be capable of bearing a force equal to the product of the larger footing load times the seismic coefficient, divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slab on grade.

Settlement of Shallow Foundations on Soils Not Prone to Liquefaction

The settlement of the foundation due to applied loads is one of the most important considerations in ensuring the safe performance of the supported structure. A foundation subjected to seismic load may undergo vertical settlement, tilt and may also experience sliding. The settlement and tilt of the foundation is commonly obtained by using same procedures as for a foundation subjected static vertical loads and moments. The following methods can be conveniently used in this case.

<u>Prakash and Saran (1977) Method</u>

A procedure to determine the settlement and tilt of foundations subjected vertical load and moment was developed by Prakash and Saran (1977) which uses Eqs. (28) and (29)

$$\frac{S_e}{S_o} = 1.0 - 1.63\frac{e}{B} - 2.63\left(\frac{e}{B}\right)^2 + 5.83\left(\frac{e}{B}\right)^3 \quad (28)$$

$$\frac{S_m}{S_o} = 1.0 - 2.31\frac{e}{B} - 22.61\left(\frac{e}{B}\right)^2 + 31.54\left(\frac{e}{B}\right)^3 \quad (29)$$

Where, S_o = settlement at the center of the foundation for vertical load only

S_e = settlement at the center of the eccentrically loaded foundation (combined ction of vertical load and moment)

S_m = maximum settlement of the eccentrically loaded foundation

B= width of the foundation

e= eccentricity given by e = $\frac{M}{Q}$,

Q = vertical load and M = moment.

The tilt of the foundation 't' may then be obtained from the following equation:

$$S_m = S_e + \left(\frac{B}{2} - e\right) \sin t \qquad (30)$$

S_e , S_m and 't' can thus be obtained if S_o can be determined. Prakash and Saran (1977) have suggested the use of plate load test to determine S_o. The value of S_o can also, be obtained any other procedure commonly used for determination of elastic settlement of foundations.

<u>Richards et al, (1993) Method</u>

Richards et al, (1993) suggested the use of the following equation to estimate the seismic settlement of a strip footing.

$$S_{Eq}(m) = 0.174 \frac{V^2}{Ag} \left| \frac{k_h*}{A} \right|^{-4} \tan \alpha_{AE} \qquad (31)$$

where S_{Eq} = seismic settlement (in meters) , V = peak velocity for the design earthquake (m/sec), A = acceleration coefficient for the design earthquake, g = acceleration due to gravity (9.81 m/sec^2). The value of tan α_{AE} in Eq (31) depends on φ and k_h*. Figure 16 shows the variation of tan α_{AE} with k_h* for φ values from 15° - 40°.

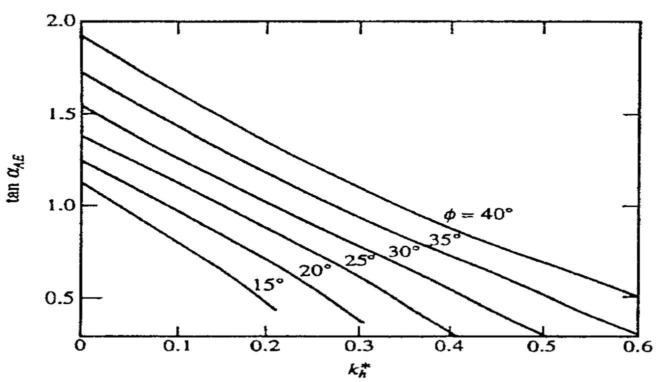

Fig. 16. Variation of tan α_{AE} with k_h and φ (Richards et al 1993)*

Whitman and Richart (1967) and Georgiadis and Butterfield (1988) have suggested procedures for determining the settlement and tilt of the foundations subjected to static vertical loads and moments.

Shallow Foundations on Soil Prone to Liquefaction

The most common cause of seismic bearing capacity failure is the liquefaction of the underlying soil. Localized failure due to punching can also lead to seismic bearing capacity failure. Liquefaction analysis can help determine the soil layers susceptible to liquefaction. This analysis involves the following two requirements:

1. The foundation must not bear directly on soil layers that will liquefy during the design earthquake. Even the lightly loaded foundations can sink in to the liquefied soil.
2. There must be an adequate thickness of un-liquefiable soil layer to prevent damage due to sand boils and surface fissuring. Otherwise, there could be damage to the shallow foundations.

If the above two conditions are not met, then the site-soil condition is highly susceptible to liquefaction and requires special design considerations such as the use of deep foundations or ground modification.
If the above two requirements are met, then there are two different types of bearing capacity analysis that can be performed.

Type I: Punching shear Analysis.
Type II: Reduction in Bearing Capacity due to Build up of Pore water Pressure.

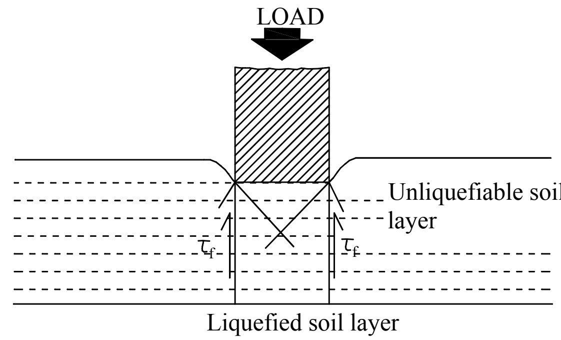

Fig. 17. Schematic Sketch Illustrating Punching Shear

These two analyses are discussed below:

Type I: Punching shear Analysis. Figure 17 shows the concept of punching shear failure occurring in a non-liquefying upper layer which is underlain by a liquefying layer. In this analysis, the footing will punch vertically downwards into the liquefied soil. This situation will arise when the upper non-liquefying layer is thin. The factor of safety FS against bearing capacity failure may be calculated as follows:

$$FS = R/P \qquad (32)$$

For Strip Footing
Where, R= shear resistance of soil per unit length of the footing

$$R = 2T^*\tau \qquad (33a)$$

τ = shear strength of unliquefiable soil layer
T= vertical distance from the bottom of footing to the top of liquefiable oil layer, m

P= Load per unit length of the footing. This load includes dead, live and seismic loads acting on footings as well as weight of footing itself.

For Spread footing:

$$R= 2(B+L)\ T^*\ \tau \qquad (33b)$$

There are two unknown parameters in the equations of factor of safety for each of the two types of footing, i.e. vertical distance from the bottom of footing to the liquefied soil layer and the shear strength of un-liquefied soil layer.
If the un-liquefiable upper soil layer consists of cohesive soil (eg: clay) or clayey sand, using total stress analysis, the following equations may be used to obtain τ.

For clays:

$$\tau = s_u \qquad (34a)$$

For clayey sands:

$$\tau= c+\boldsymbol{\sigma}_h \tan Ø \qquad (34b)$$

Where, s_u = undrained shear strength of cohesive soil
c & Ø are undrained shear strength parameters

$\boldsymbol{\sigma}_h$ = Normal stress on the failure surface.

Since shear surfaces are vertical, the normal stresses acting on shear surfaces will be the horizontal total stress. For cohesive soil, $\boldsymbol{\sigma}_h$ may be taken as $\sigma_v/2$.

If the unliquefied soil layer is cohesionless (sand), using effective stress analysis,

$$\tau = \boldsymbol{\sigma}'_h \tan Ø' = k_o \boldsymbol{\sigma}'_v \tan Ø' \qquad (35)$$

Where $\boldsymbol{\sigma}'_h$ = horizontal effective stress
Ø' = effective angle of internal friction.

$\boldsymbol{\sigma}_v'$= Effective vertical stress at (T/2 + footing depth from the ground surface)

Type II Reduction in Bearing Capacity due to Build Up of Pore Water Pressure. Terzaghi bearing capacity theory discussed earlier may be conveniently used for this purpose. For the situation of cohesive soil layer overlying sand which is susceptible to liquefaction, a total stress analysis is used and the equations used are:

For strip footing, $q_{ult}= cN_c = S_u\ N_c$ (36)
For spread footing, $q_{ult}=s_u\ N_c\ (1+0.3\ B/L)$ (37)

Where s_u= undrained shear strength=cohesion c
N_c = bearing capacity factor determined from Fig. 18 for the condition of un-liquefiable cohesive soil layer that is expected to liquefy during design earthquake. In Fig. 18, T represents the vertical distance from the bottom of the footing to the top of the liquefied soil layer.

B= width of footing & L= Length of footing

Since the liquefied soil layer has zero shear strength, c_2=0 & c_2/c_1 =0 for use in Fig. 18.

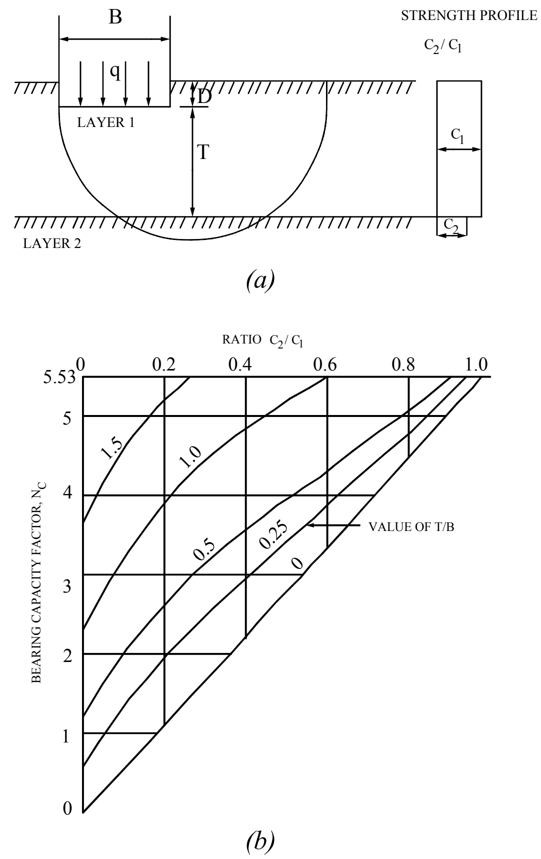

Fig. 18. Bearing Capacity Factor N_c for two layer soil system (Day, 2002)

The value for ultimate load Q_{ult} can be determined by multiplying the q_{ult} and the footing dimensions. FS= Q_{ult} / P Reduction in Bearing Capacity due to Build Up of Pore water Pressure.

Granular Soil. There are many factors involved in the determination of bearing capacity of soils that may liquefy during design earthquake. Distance from of bottom of footing to the top of the liquefied soil layer is an important consideration. This parameter is difficult to determine for soil that is below ground water table and has factor of safety against liquefaction that is slightly greater than one. The reason being earthquake might induce liquefaction or partial liquefaction of the upper layer as well. In addition to vertical loads, footing might also be subjected to the static and dynamic lateral loads during earthquake. They are usually dealt with separately. There may be reduction in the shear strength of the upper dense layer of granular soil due to an increase in the pore water pressure following liquefaction of the lower layer. Sands and gravels that are below the ground water table may have a factor of safety against liquefaction greater than 1.0 but less than 2.0. If the factor of safety against liquefaction is greater than 2.0, the earthquake induced pore water pressures will typically be small enough so that their effect can be neglected. For cohesionless soils, Terzaghi's ultimate bearing capacity can be expressed as:

$$q_{ult} = (½)\, \gamma\, B N_\gamma \qquad (38)$$

If the ground water table is at the bottom of the footing or closer to the bottom of the footing, the effective unit weight γ_b used in place bulk unit weight γ_t in Eq. (37). In order to account for the increase in the excess pore water pressure during the design earthquake, the term (1- r_u) can be inserted in Eq. (37) which becomes:

$$q_{ult} = (½)\,(1 - r_u)\gamma_b\, B N_\gamma \qquad (39)$$

The value for r_u can be obtained from the plot in Fig. 19 which is a plot of the pore water pressure ratio, i.e. $r_u = u_e/\boldsymbol{\sigma}'$ versus the factor of safety against liquefaction. To find r_u, the factor of safety against liquefaction (FS_L) of soil located below the bottom of the footing must be determined. Equation (39) established for the case with factor of safety against liquefaction greater than 1. If the value of (FS_L) is less than 1, the foundation design is not feasible unless counter-measures against liquefaction failure are adopted.

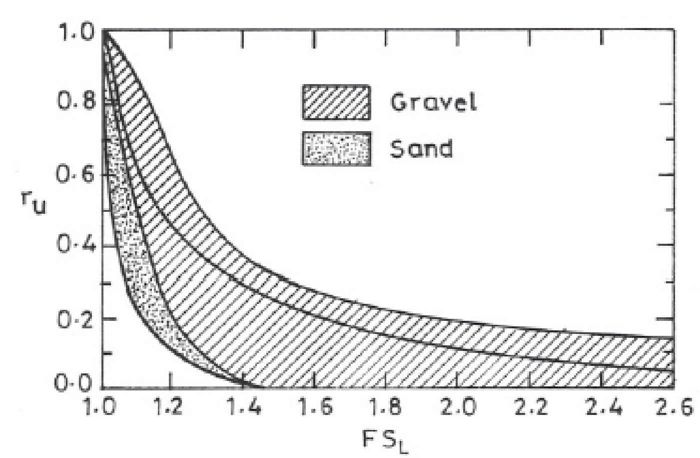

Fig.19. Residual Excess Pore water Pressure r_u versus Factor of Safety against Liquefaction. (Marcuson Hynes, 1990)

There is a need to be careful when dealing with foundation design in soils that may liquefy during the design earthquake. The site could experience liquefaction induced lateral spreading and flow slides. If the soil is softened and gets liquefied, ground deformations occur rapidly in response to static or dynamic loading. The amount of deformation is a function of loading conditions, amplitudes and frequencies of seismic waves, the thickness and extent of the liquefiable layer, the relative density and permeability of the liquefied sediment and the permeability of surrounding sediment layers. Despite the severity of damages, there has been relatively little progress towards the development of consistent methodology for the design of foundation systems under these circumstances. Usually, the presence of superstructure is neglected and calculations are performed for free-field conditions. Liquefaction is evaluated and empirical correlations developed for free field conditions are used. But, the presence of superstructure results in a significantly different response than that under free-field conditions.

<u>Settlement of Foundations in Liquefying Soil</u>

<u>Simplified Procedures for the Evaluation of Settlements of Structures During Earthquakes (Ishihara and Tokimatsu; 1988).</u> A procedure to determine earthquake induced settlements of structures on saturated sand deposits due to pore water pressure generation was developed by Ishihara and Tokimatsu (1988). To investigate the effectiveness of the proposed method, the observed values of settlement of structures were also compared with the values obtained from the proposed method.

The total settlement of the structure due to earthquake shaking (S_{st}) is given as:

$$S_{st} = S_v + S_e \qquad (40)$$

where, S_v = settlement due to volumetric strain caused by earthquake shaking

S_e = immediate settlement due to change in soil modulus

Knowing the value of the cyclic stress ratio developed in the soil during earthquakes and normalized $(N_1)_{60}$ value, the volumetric strain can be determined from Fig.20 below.

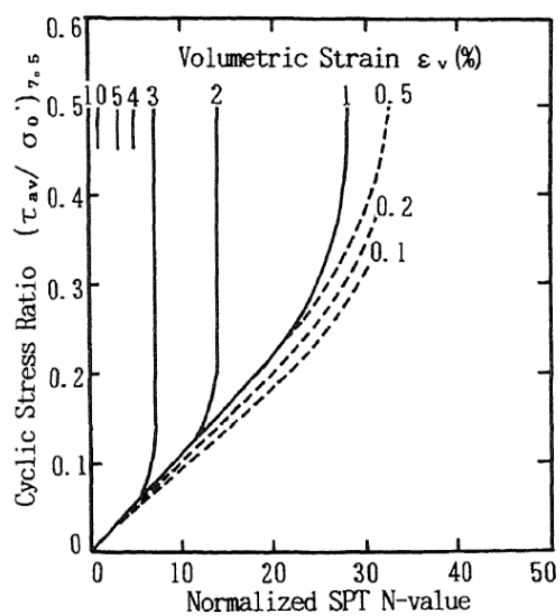

Fig. 20: Cyclic Stress ratio, (N1)60 vs. Volumetric Strain (Tokimatsu and Seed: 1987)

The relationship shown in Fig. 20 was proposed earlier by Tokimatsu and Seed (1987) which is based on the controlling factors like maximum pore pressure generated before initial liquefaction and the maximum shear strain after liquefaction.

The cyclic stress ratio developed in the soil during earthquakes is given as:

$$\left(\frac{\tau_{av}}{\sigma'_o}\right)_{7.5} = \left\{0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma'_o}\right)r_d\right\}r_m \qquad (41)$$

where, $\left(\frac{\tau_{av}}{\sigma'_o}\right)_{7.5}$ = Equivalent Shear Stress Ratio induced by the earthquake shaking of M = 7.5

a_{max} = maximum horizontal acceleration at the ground surface

σ_o =total overburden pressure at the depth considered.
r_d = Stress reduction factor that varies with depth.
r_m = Scaling factor for a stress ratio concerning the magnitude of earthquake .

By integrating the volumetric strains for different depths, the settlement of the structure can be computed. For values of M other than 7.5, magnitude scaling factors may be used.
Ishihara and Tokimatsu (1988) suggested that the immediate settlement caused by the change in soil modulus can be computed as:

$$S_e = q\,.B\,.I_p\left(\frac{1}{E_2} - \frac{1}{E_1}\right) \qquad (42)$$

Where, q = contact pressure of the structure
B = width of the structure
I_p = coefficient concerning the dimension of the structure, thickness of soil layer and poisson's ratio of soil.
E_1 and E_2 = Young's Modulus of soil before and during earthquake shaking respectively.

The reduction in the shear modulus of soil during earthquake shaking can be computed based on the effective shear strain (γ_{eff}) induced in the soil as given in Eq. (43) below:

$$\gamma_{eff}\left(\frac{G_{eff}}{G_{max}}\right) = 0.65.\left(\frac{a_{max}}{g}\right).\ \sigma_o\,.r_d\,.\left(\frac{1}{G_{max}}\right) \qquad (43)$$

where, G_{max}= Shear modulus at low shear strain level
G_{eff} = effective shear modulus at induced shear strain level
a_{max} = maximum horizontal acceleration at the ground surface
σ_o =total overburden pressure at the depth considered

Using the computed value of $\gamma_{eff}\left(\frac{G_{eff}}{G_{max}}\right)$ in Fig. 21, the value of corresponding effective shear strain (γ_{eff}) is obtained and G_{eff} can be computed.

They have further emphasized that the change in effective stress due to pore pressure generation as well as the shear strain level developed in the soil are highly influenced when there is liquefaction and therefore, do not recommend to use eq. (39) to compute the settlement of structure. In such condition, the settlement of the structure is affected due to the shear deformation of the soil strata and thus young's modulus can't be accurately determined. Accordingly, they have estimated an approximate relationship based on the field observations as given in Eq. (43).

$$S_{st} = S_v\,.r_b \qquad (44)$$

Where, r_b = scaling factor concerning the shear deformation which may be obtained from Fig. 22. Based on the studies of Niigata earthquake (1964) done by Yoshimi and Tokimatsu 1977, the importance of large width of the structure (compared to the thickness of the liquefied layer) on reducing the liquefaction induced settlement can be noted very clearly from figure 21. It can be seen from Fig (22) that appreciable settlement occurred where the width ratio was less than 2 whereas the settlement was small and constant where the width ratio exceeds 2 or 3. Ishihara and Tokimatsu (1988) developed parameter 'r_b' that is equal to the settlement ratio normalized by the settlement ratio at width ratio equal to 3. They found the computed values generally consistent with the observed values, and proposed that this simplified method of computation can be used as a first approximation to predict earthquake induced settlement of structures.

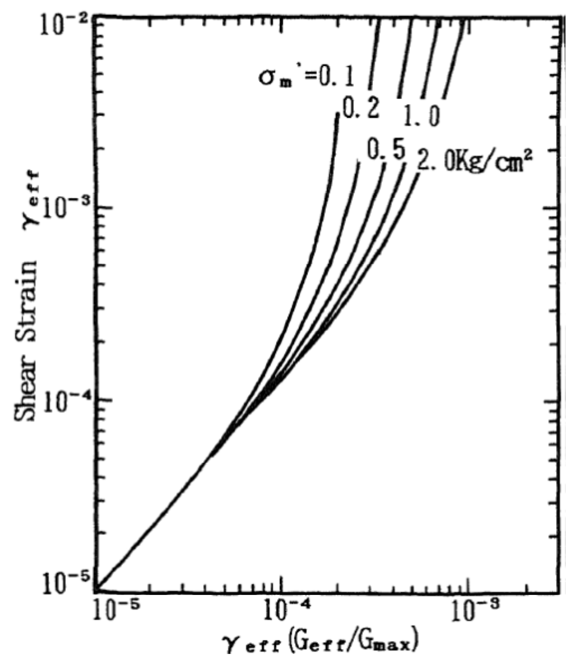

Fig. 21. Determination of induced Shear Strain (Tokimatsu and Seed, 1987)

Ishihara and Yoshmine (1992). Ishihara and Yoshmine (1992) have provided a chart to estimate the post-liquefaction volumetric strain of clean sand as function of factor of safety against liquefaction. This chart is shown in Fig. 23. This chart can be easily used if any of the corrected SPT values, cone resistance at the site or the maximum cyclic shear strain induced by the earthquake are known.

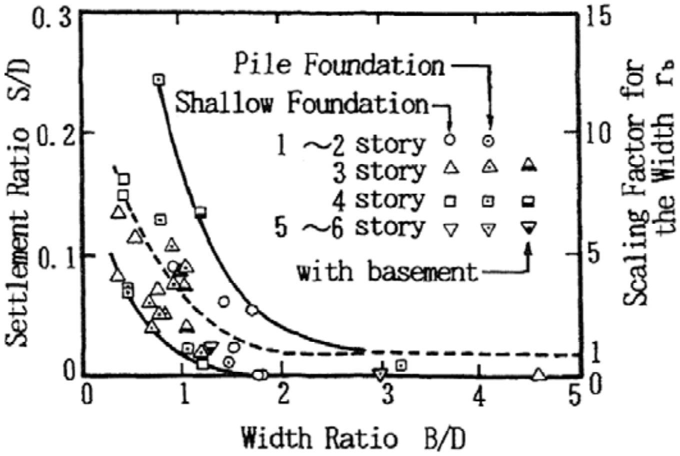

Fig. 22: Scaling factor vs. width ratio

The chart in Fig. 23 is convenient to use. The factor of safety against liquefaction failure is calculated and then the volumetric strain is determined using value of relative density of the deposit or its the corrected standard penetration resistance or cone penetration resistance. The settlement of the deposit may then be calculated as:

$$S = H \in_v \tag{45}$$

In which, S= settlement
H= thickness of the deposit
and $\in_v$ = volumetric strain.

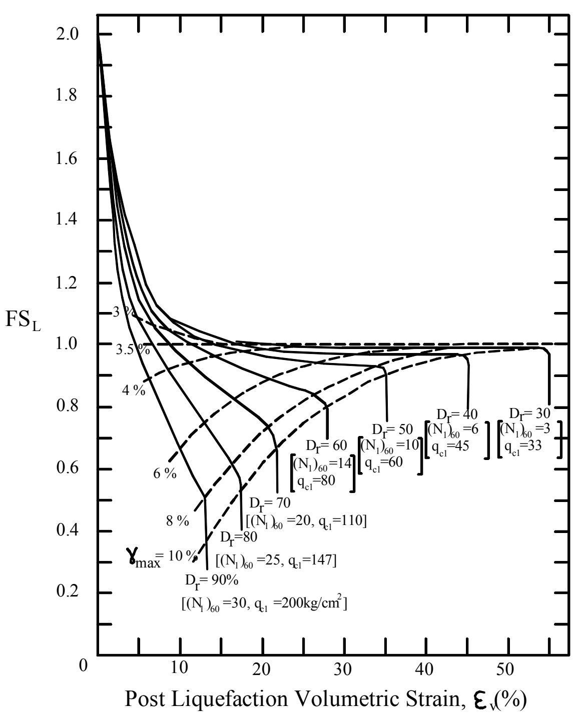

Fig. 23. Chart for Post Liquefaction Volumetric Strain (After Ishihara and Yoshimine, 1992)

For deposits consisting of various layers of saturated sand, the settlement for each layer may be calculated and the total settlement obtained as the sum of the settlements of each layer.

Additional Comments on Foundation Performance on Liquefied Soil

Gazetas et al (2004) studied tilting of buildings in it1999 Turkey earthquake. Detailed scrutiny of the “Adapazari failures” showed that significant tilting and toppling were observed only in relatively slender buildings (with aspect ratio: $H / B > 2$), provided they were laterally free from other buildings on one of their sides. Wider and/or contiguous buildings suffered small if any rotation. For the prevailing soil conditions and type of seismic shaking; most buildings with $H / B > 1.8$ overturned, whereas building with $H / B < 0.8$ essentially only settled vertically, with no visible tilting. Figure 24 shows a plot of H/B to tilt angle of building. Soil profiles based on three SPT and three CPT tests, performed in front of each building of interest, reveal the presence of a number of alternating sandy-silt and silty-sand layers, from the surface down to a depth of at least 15 m with values of point resistance $q_c \approx (0.4 – 5.0)$ MPa . Seismo–cone measurements revealed wave velocities Vs less than 60 m/s for depths down to 15 m, indicative of extremely soft soil layers. Ground acceleration was not recorded in Tigcilar. Using in 1-D wave propagation analysis, the EW component of the Sakarya accelerogram (recorded on soft rock outcrop, in the hilly outskirts of the city) leads to acceleration values between 0.20 g -0.30 g, with several significant cycles of motion, with dominant period in excess of 2 seconds. Even such relatively small levels of acceleration would have liquefied at least the upper-most loose sandy silt layers of a total thickness 1–2 m, and would have produced excess pore-water pressures in the lower layers Gazetas et al (2004).

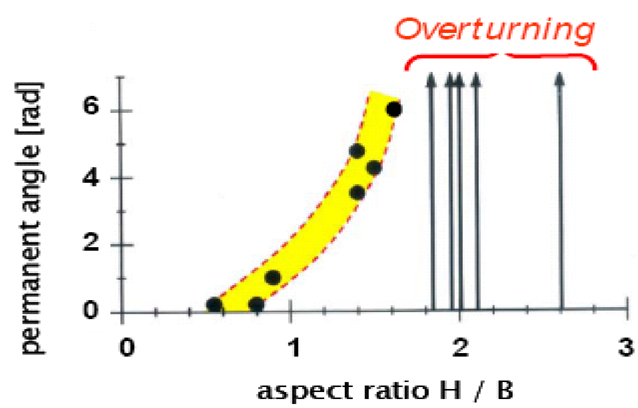

Fig.24. The angle of permanent tilting as a unique function of the slenderness ratio H/B (Gazetas et. al., 2004)

OVERVIEW ON SEISMIC DESIGN OF SHALLOW FOUNDATIONS

Estimation of seismic response of foundation during a strong earthquake is a complex task because soil behaves in a highly non linear manner when subjected to large cyclic strains. When loose soil deposits get saturated, it deforms substantially with large pore water pressure generation and eventually liquefies. It is very important to have a thorough understanding of the potential consequences of liquefaction, the need for ground improvement and the subsequent evaluation of the performance of the proposed mitigation scheme. The present practice of estimating liquefaction induced settlement based on post-liquefaction reconsolidation settlements under free field conditions might misrepresent and largely underestimate the consequences of liquefaction (Andrianopoulos et al. 2006, Dashti et al. 2010, Liu and Dobry, 1997).This practice ignores the deviatoric deformation (settlements due to the cyclic inertial forces acting on the structures within the liquefiable soil under a building's foundation as well as volumetric deformations due to localized drainage during shaking. Presently, well calibrated analytical tools and design procedures that identify, evaluate and mitigate the most critical mechanisms of liquefaction induced settlement are wanting. Due to the discontinuousness of soil skeleton and large amount of lost pore water and continued loss in soil stiffness, it is very difficult to exactly reflect the actual performance of buildings in liquefying soils (Liu, 1995).

Evaluation of foundation settlement for a wide range of soil, foundation and earthquake parameters in complicated. The empirical charts and relationships developed are based on the several assumptions and are limited to some specific conditions which cannot be generalized to other combinations of foundation load and diameter, density and thickness of the liquefiable sand layer and intensity and duration of shaking.

CONCLUSIONS

Considerable research effort has been devoted to define the failure surfaces below shallow foundations subjected to seismic loads as well as their settlements. However, the equivalent static approach is still commonly used for their design.

It may be emphasized here that for the case soils susceptible to liquefaction (i) the foundation should not rest directly on soil layers that may liquefy as even lightly loaded foundations can sink into the soil and (ii) adequate thickness of non-liquefiable soil should be there to prevent damage to the foundation due sand boils and surface fissuring. If these conditions are not met then the ground improvement may be needed or the deep foundation should be provided.

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