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DYNAMIC BEARING CAPACITY AND SETTLEMENT OF FOUNDATIONS

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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. This may happen due to following reasons:

- Cyclic degradation of soil strength may lead to bearing capacity failure during the earthquake.
- Large horizontal inertial force due to earthquake may cause the foundation to fail in sliding or overturning.
- Soil liquefaction beneath and around the foundation may lead to large settlement and tilting of the foundation.
- 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

EXAMPLES OF FOUNDATION FAILURES DUE TO EARTHQUAKES

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

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

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OUTLINE

- Objectives
- Shallow Foundations on Soils Not Prone to Liquefaction.
- Settlement of Shallow Foundations on Soils Not Prone to Liquefaction.
- Shallow Foundation on Soil Prone to Liquefaction.
- Settlement of Foundations on Soil Prone to Liquefaction

- ***OBJECTIVES***

- Safe Foundation Design → No Bearing Capacity Failure
  → No Excessive Settlement

- Estimation of Bearing Capacity and Settlement

Static Case

- Terzaghi (1943)
- $q_u = c\ N_c + q\ N_q + 0.5\ \gamma\ B\ N_\gamma$ (1)
- q_u = Ultimate bearing capacity
- c = Cohesion of soil
- γ = unit weight of soil
- q = Surcharge Pressure = γ D
- B=width of the foundation
- D= depth of the foundation.
- $N_{c,}\ N_{q,}\ N_\gamma$ = Bearing capacity factors (depend only on the soil friction angle ø)

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Meyerhoff (1963)

General Bearing Capacity Theory

- $q_u = c\ N_c s_c\ d_c\ i_c + q\ N_q\ s_{qs}\ d_q\ i_q + 0.5\ \gamma\ B\ N_\gamma\ s_\gamma\ d_\gamma\ i_\gamma$ (2)
- N_c, N_q and N_γ are Meyerhoff's bearing capacity factors
- $s_{c,}\ s_{q,}\ s_\gamma$ =Shape Factors
- d_c, d_q, d_γ = Depth Factors
- $i_{c,}\ i_{q,}\ i_\gamma$ = Load Inclination Factors.

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Table 1. 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 $\emptyset = 0°$ $S_q = S_y = 1.0$	(i) For $\emptyset = 0°$ $d_q = d_y = 1.0$	$i_y = (1 - \frac{\alpha}{\emptyset})^2$
(ii) For $\emptyset \geq 10°$ $S_q = S_y = 1 + 0.1\, K_p \frac{B}{L}$	(ii) For $\emptyset \geq 10°$ $d_q = d_y = 1 + 0.1 \sqrt{K_p} \frac{D}{B}$	α = angle of resultant measured from vertical axis $K_p = \tan^2\left(45° + \frac{\emptyset}{2}\right)$

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- FOUNDATIONS ON SOILS NOT PRONE TO LIQUEFACTION

- **Pseudo-static Approach**
- Additional Forces and Moments due to Earthquake are included.
- Failure surface below the foundation is assumed to be the same as for the static case.

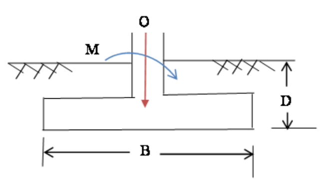

Fig. 2. Using Terzaghi's Theory

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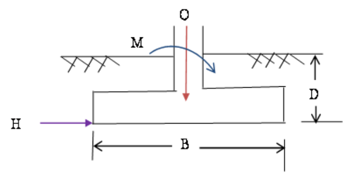

Fig. 3. Using General Bearing Capacity Theory

- To account for the effect of dynamic nature of the load, the bearing capacity factors are determined by using dynamic angle of internal friction which is taken as 2-degrees less than its static value (Das, 1992).
- Building Codes generally permit an increase of 33 % in allowable bearing capacity when earthquake loads in addition to static loads are used in design of the foundation. This recommendation may be considered reasonable for dense granular soils, stiff to very stiff clays or hard bedrocks but is not applicable for friable rock, loose soils susceptible to liquefaction or pore water pressure increase, sensitive clays or clays likely to undergo plastic flow (Day, 2006).

Developments in Determination of Seismic Bearing Capacity

- *EFFORTS TO DEFINE THE FAILURE SURFACE*
- Selig and McKee (1961). Small scale tests
- Dynamic Bearing Capacity about 30% smaller than for static case and large settlements at failure.
- **Richard et. al (1993)**

Used plane failure surfaces below the footing to determine the Seismic Bearing Capacity

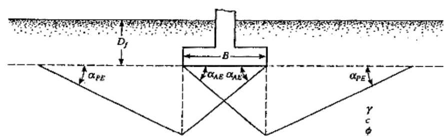

Fig.4. Failure surface in soil for seismic bearing capacity (After Richards et. al, 1993)

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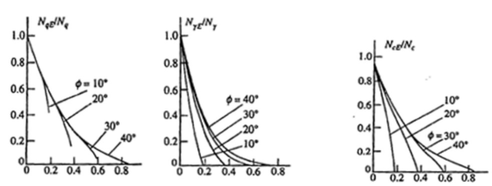

$\tan\psi = k_h/(1-k_v)$ $\tan\psi = k_h/(1-k_v)$ $\tan\psi = k_h/(1-k_v)$,

Fig.5. Values of N_{qE}/N_q, $N_{\gamma E}/N_\gamma$ and N_{cE}/N_c (After Richard et. al: 1993)

$q_{uE} = cN_{cE} + qN_{qE} + \frac{1}{2}\gamma BN_{\gamma E.}$
q_{uE} = Seismic bearing capacity
D_f = Depth of the foundation and $q = \gamma D_f$
N_{cE}, N_{qE}, and $N_{\gamma E}$ = Seismic bearing capacity factors which are functions of φ and $\tan\psi = k_h/(1-k_v)$
k_h and k_v are the horizontal and vertical coefficients of acceleration due to earthquake.

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Budhu and AlKarni (1993)

- Logarithmic failure surfaces shown in Fig. 6 were assumed by Budhu and Al-karni (1993) to determine the seismic bearing capacity of soils.

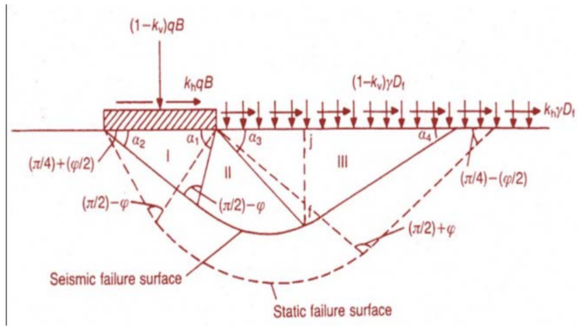

Fig. 6. Failure Surfaces Used by Budhu and al-karni (1993)

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$$q_{ud} = c\, N_c\, s_c\, d_c\, e_c + q\, N_q\, s_q\, d_q\, e_q + 0.5\, \gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, e_\gamma \qquad (3)$$

- 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

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

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

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

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D= c/ γH

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

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Chaudhury and SubbaRao (2005, 2006)

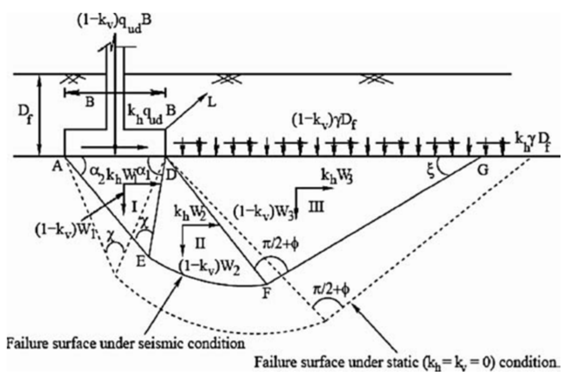

Fig.7. Failure Surfaces under static and Seismic Loading (Chaudhury and Subba Rao ; 2005)

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$$q_{ud} = c\,N_{cd} + q\,N_{qd} + 0.5\,\gamma\,B\,N_{\gamma d} \qquad (4)$$

- Where, N_{cd}, N_{qd} and $N_{\gamma d}$ are seismic bearing capacity factors which may from Fig.8.

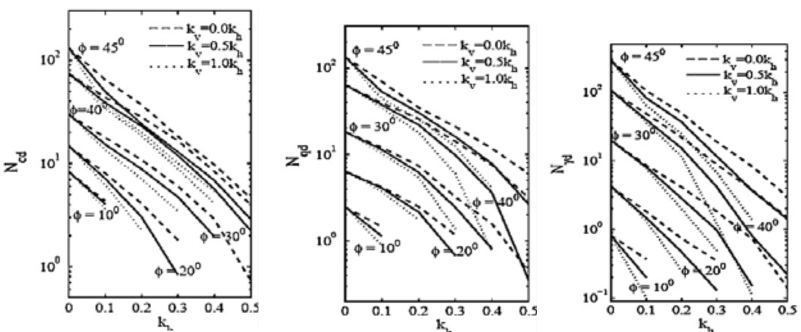

Fig. 8 Values of N_{cd}, N_{qd} and $N_{\gamma d}$ (Chaudhury and Rao; (2005, 2006)

Comparison of Chaudhary and Subba Rao's (2005 Bearing Capacity Factors With Other Researchers.

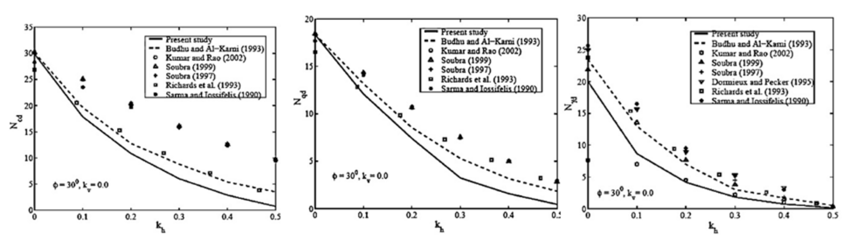

Fig. 9. N_{cd} , N_{qd} and $N_{\gamma d}$ by Chaudhury and Rao (2005, 2006) and Other Researchers

SETTLEMENT OF SHALLOW FOUNDATIONS ON SOILS NOT PRONE TO LIQUEFACTION

The settlement due seismic loading may, in general, occur due to:

1. Loads and Moments imposed on the foundation.
2. Settlement of the soil deposit due to shaking.

The settlement due to (l) is discussed here and due to (2) will discussed along with settlement of shallow foundations on soils prone to liquefaction.

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Settlement due to Loads and Moments imposed on the foundation.

The settlement and tilt may occur due additional loads and moments on the foundation and also due to degradation of soil strength.

When foundations are designed following the Pseudo-static approach, the settlement and tilt are generally estimated using the static methods.

Whitman and Richart (1967) and Prakash and Saran (1977) proposed simple empirical methods to estimate settlement and tilt of foundations. Richards, et al. (1993) developed a method to determine the vertical settlement due to seismic loading.

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Prakash and Saran (1977) Method.

- This method estimates the settlement and tilt as follows:

$$\frac{S_e}{S_o} = 1.0 - 1.63\frac{e}{B} - 2.63\left(\frac{e}{B}\right)^2 + 5.83\left(\frac{e}{B}\right)^3 \qquad (5)$$

$$\frac{S_m}{S_o} = 1.0 - 2.31\frac{e}{B} - 22.61\left(\frac{e}{B}\right)^2 + 31.54\left(\frac{e}{B}\right)^3 \qquad (6)$$

$$S_m = S_e + \left(\frac{B}{2} - e\right)\sin t \qquad (8)$$

S_o = settlement below the center of the foundation for vertical load only.
S_e = settlement at the center of the eccentrically loaded foundation.
S_m = maximum settlement of the eccentrically loaded foundation.
e= eccentricity given by $e = \frac{M}{Q}$, Q = vertical load and M = moment.

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Richards, et al. (1993) Method

$$S_{Eq}(m) = 0.174\frac{V^2}{Ag}\left|\frac{k_{h^*}}{A}\right|^{-4}\tan\alpha_{AE} \qquad (9)$$

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 $\tan\alpha_{AE}$ in Eq (9) depends on φ and k_h*.

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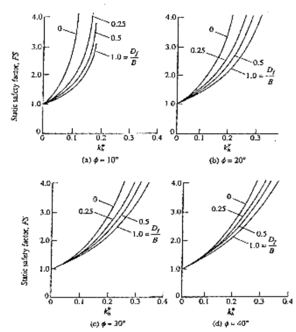

Fig.10. Values of Critical Horizontal Acceleration (Richards, et al. ; 1993)

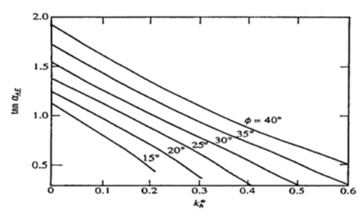

Fig. 11. Variation of tan α_{AE} with k_h* and φ (Richards, et al.; 1993)

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FOUNDATIONS ON SOILS PRONE TO LIQUEFACTION

General Considerations

- The foundation must not bear directly on soil layers that will liquefy.
- There must be an adequate thickness of un-liquefiable soil layer to prevent damage due to sand boils and surface fissuring.

Types of Analysis

- 1. Punching Shear Analysis.
- 2. Reduction in Bearing Capacity due to Build Up of Pore water Pressure

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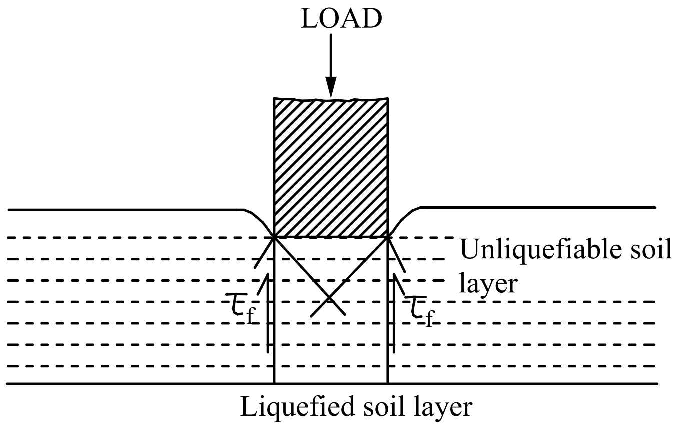

Fig 12. Schematic Sketch Illustrating Punching Shear

1.Punching Shear Analysis

Factor of Safety= FS= R/P (10)

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- R= 2(B+L) T* τ (11)
- For clays:
- $\tau = s_u$ (12a)
- For clayey sands:
- $\tau = c + \boldsymbol{\sigma}_h \tan Ø$ (12b)
- s_u = un-drained shear strength of cohesive soil
- c & Ø are un-drained shear strength parameters
- $\boldsymbol{\sigma}_h$ = Normal stress on the failure surface
- Use effective stresses and effective strength parameters if upper non-liquefiable layer is sand

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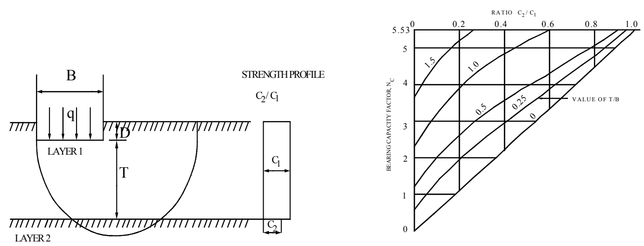
2.Reduction in Bearing Capacity due to Build Up of Pore Water Pressure

- Upper non-liquefiable Layer is clay use total stress analysis



(a) (b)

Fig 13. Bearing Capacity Factor N_c for two layer soil system (Day, 2002)

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- $Q_{ult}=s_u N_c (1+0.3 B/L)$ (13)
- Use Fig.13(b) ($c_2/c_1 =0$) to obtain N_c
- Upper Non-Liquefiable Layer is Cohesionless Sand
- $q_{ult}= (½) (1- r_u)\gamma_b BN_\gamma$ (14)
- $r_u=u_e/\sigma'$
- (FS_L) = Factor of safety against liquefaction.

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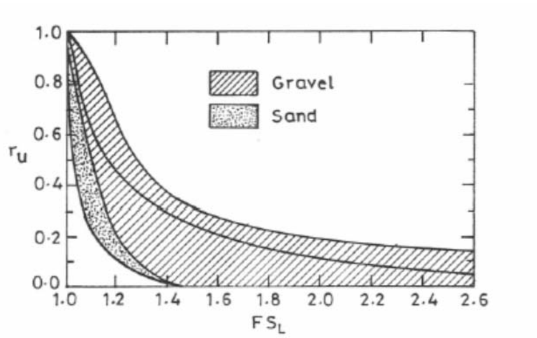

Fig.14. Residual Excess Pore water Pressure r_u versus Factor of Safety against Liquefaction (Marcuson and Hynes ; 1990).

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SETTLEMENT OF FOUNDATIONS ON LIQUEFYING SOIL

Simplified Procedures for the Evaluation of Settlements of Structures During Earthquakes (Ishihra and Tokimatsu, 1988).

- $S_{st} = S_v + S_e$ (15)

  S_v = settlement due to volumetric strain.

  S_e = immediate settlement due to change in soil modulus.

  S_{st} = total settlement of the structure due to earthquake shaking.

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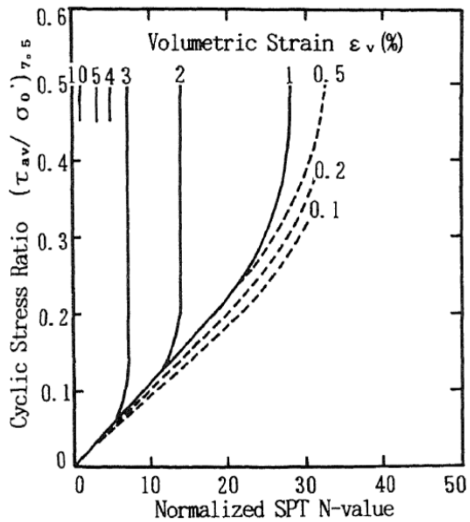

Fig 15. Cyclic Stress ratio, (N1)60 vs. Volumetric Strain (Tokimatsu and Seed; 1984)

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$$\left(\frac{\tau_{av}}{\sigma'_o}\right)_{7.5} = \left\{0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma'_o}\right) r_d\right\} r_m \qquad (16)$$

$\left(\frac{\tau_{av}}{\sigma'_o}\right)_{7.5}$ = 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

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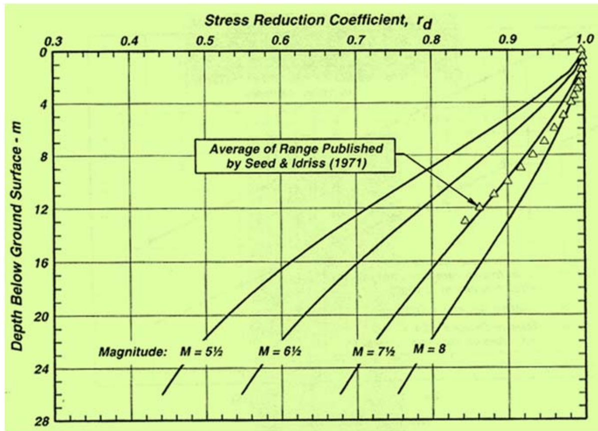

Fig. 16 Variation of stress reduction Factors (Idriss; 1999)

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Table.2. Scaling Factor for Effect of Earthquake Magnitude

Earthquake Magnitude, M	Scaling Factor for Stress Ratio ,r_m	Scaling Factor for Volumetric Strain r_v
8-1/2	1.12	1.25
7-1/2	1.0	1.0
6-3/4	0.88	0.85
6	0.76	0.6
5-1/4	0.67	0.4

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Immediate settlement caused by the change in soil modulus can be computed as:

$$S_e = q\,.B\,.I_p\left(\frac{1}{E_2} - \frac{1}{E_1}\right) \qquad (17)$$

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 Modulii of soil before and during earthquake shaking respectively.

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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 is:

$$\gamma_{eff}\left(\frac{G_{eff}}{G_{max}}\right) = 0.65.\left(\frac{a_{max}}{g}\right).\ \sigma_o\,.r_d\,.\left(\frac{1}{G_{max}}\right) \qquad (18)$$

G_{max}= Shear modulus at low shear strain level

G_{eff} = effective shear modulus at induced shear strain level

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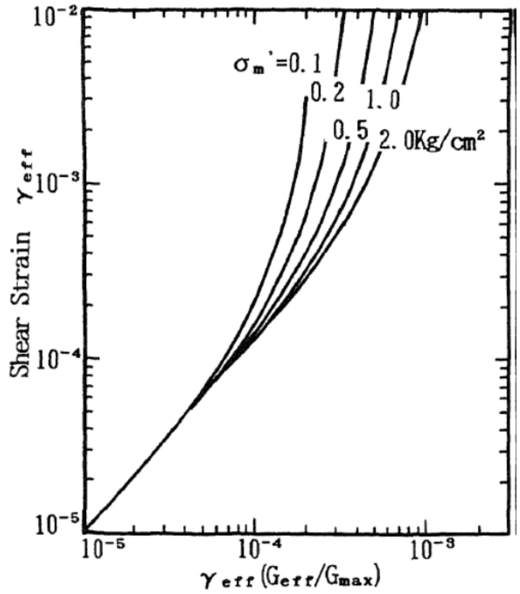

Fig.17.Determination of induced Shear Strain
Tokimatsu and Seed (1984)

- For case of large strains
  $S_{st} = S_v .r_b$ (19)

  r_b = scaling factor for shear deformation.

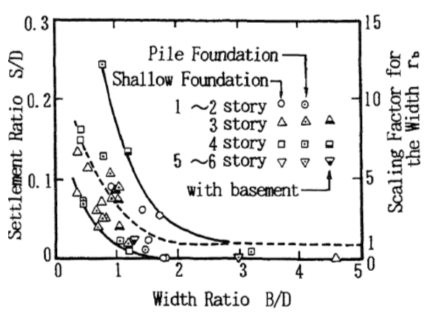

Fig.18. Scaling factor vs. width ratio

Ishihara and Yoshmine (1992)

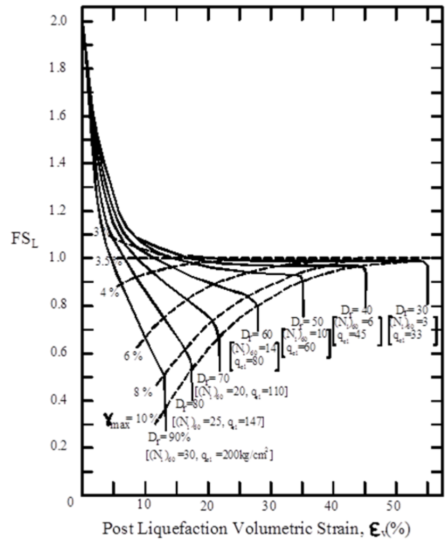

Fig. 19. Chart for Post Liquefaction Volumetric Strain (After Ishihara and Yoshimine, 1992)

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$S = H \in_v$ (19)

S= settlement

H= thickness of the deposit

$\in_v$ = volumetric strain

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SOME SIGNIFICANT OBSERVATIONS

CODES-SOME FALLACIES

Codes recommend higher allowable pressure under shallow footings during earthquakes!

Euro-Code

- According to EC8-5:

"*For the majority of usual building structures, the effects of SSI tend to be beneficial, since they reduce the bending moments and shear forces acting in the various members of the superstructure*".

- The importance of accounting for SSI effects has been often dismissed in most cases, to be on the safe side.

Due to soil or seismological factors, an increase in the fundamental period due to SSI may lead to increased response (despite a possible increase in damping), which contradicts the provision of a conventional code.

- For Example, Mexico earthquake was particularly destructive to 10 –to 12- story buildings founded on soft clay; their period apparently increased from about 1 sec (under the fictitious assumption of a fixed base) to nearly 2 seconds in reality.

Front – 8 – Storied Building Collapsed
Back – 15 Storied Building DID NOT
(Mexico 1985)

- A Soil-structure system on soft soil (category D) will probably be subjected to a soil-amplification ground motion, with a more-or-less sharp spectral peak (at T_P)

If its fixed-base $T_1 < T_P$:

SSI is probably detrimental

But with the (AVERAGE) Design spectrum one predicts the opposite!

Gazetas et al(2006) present the Need and Feasibility of Inelastic Analysis of Soil-Foundation Interaction accounting for Uplifting and Bearing capacity Mobilization

Almost "Forbidden" Modes :

- ***Sliding at Soil–Foundation Interface***

- ***Separation and Uplifting of Footing from Soil***

- ***Mobilization of B.C. Failure Mechanism (s)***

GAZETAS(2006) Contd.

1. Seismic and Pseudo-static response of Structure foundation-soil systems are often vastly different.

2.Sliding and Uplifting: Often Beneficial to Structure and Foundation.

3. But Maximum and Permanent Deformations (displacement, rotation) and Increased Internal Forces m*ust satisfy the design criterion*

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- Gazezas et al (2004) studied tilting of buildings in 1999 Turkey earthquake.
- “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.
- 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.

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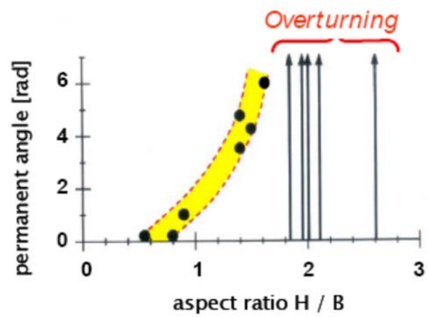


Fig. 20.The angle of permanent tilting as a unique function of the slenderness ratio H/B (Gazetas et al (2006)

Andrianopoulos et al., (2006)

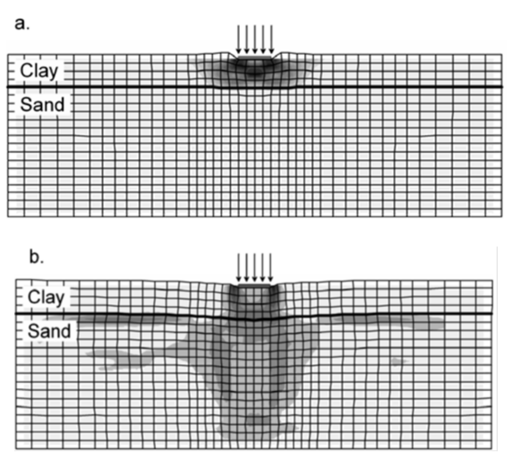
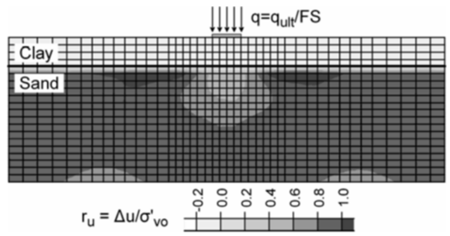


Fig. 21.Deformed mesh, shear strain increment contours and displacement vectors indicating the mode of (a) static and (b) dynamic failure, (Andrianopoulos et al., 2006)



Excess pore pressure ratio at the end of shaking (t=0.5s), (Andrianopoulos et al., 2006)

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- Liu and Dobry(1997), Bray and Dashti (2010), (Dashti et al. 2010) and Knappett and Madabhushi (2008) have explained the mechanism of progress of settlement with ground shaking for various soil, structure and ground motion parameters.

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CONCLUSION

1. 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.

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2. Shallow foundations subjected to combined static and seismic loads are commonly designed using the pseudo-static approach. Most research effort in recent years has been directed towards better defining the failure surface under combined static and seismic loading and efforts have been made to understand the behavior of the foundations under seismic loading.

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3. The codal provisions permitting 33% increase in static bearing capacity for the seismic case need to be re-examined in view of recent developments in this area.
4. Experimental and analytical research is continuing in the calculating response of foundations subjected to seismic shaking may which may result in better understanding of foundation behavior and improvement in design practice.

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