Bearing capacity and settlement of shallow footings on liquefiable soil subjected to seismic loading

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Abstract. In liquefying soils, shallow foundations may experience a reduction in bearing capacity and increase in settlement and tilt due to seismic loading. Therefore, the buildings on shallow footings may settle and tilt excessively. In this paper, the bearing capacity and settlement of shallow foundation on liquefiable deposits has studied. The study includes a shaking table test, reviewing of theoretical equations (available in literature), which estimating settlement of shallow footings due to seismic loading, calibration, and verification of these equations with data from the shaking table test for improved soil by grouting and unimproved soil. It is important to note that the grouting materials used in this study are the CKD and Bentonite slurries. A modification to the seismic bearing capacity and settlement equations had been done to account for the liquefaction state. Good convergence has been gotten between predicted and measured settlement.

Keywords: Theoretical evaluation, CKD, Grouting, Liquefaction, bearing capacity, Settlement, Bentonite, granular piles.

1. Introduction
Problem of the seismic bearing capacity of shallow foundations is far from being considered solved. Many researchers stated that the response of a footing to dynamic loads is affected by:
(1) Nature and magnitude of dynamic loads,
(2) Number of pulses and
(3) The strain rate response of soil.
Shallow foundations for seismic loads are usually designed by the equivalent static approach. The foundations are considered as eccentrically loaded and the ultimate bearing capacity is accordingly estimated. 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 [1].
International Building Code permits 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 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 [2].

2. Review of related previous works
The seismic effects on the bearing capacity of foundation were studied by Meyerhof during the middle of last century and many other researchers, who adopted a pseudo-static approach. By applying
horizontal and vertical accelerations to the center of gravity of the structure, the problem is reduced to a static case of bearing capacity with inclined and eccentric loads. Nevertheless, in these methods the inertial effects on the soil mass beneath the foundation are not considered and this could have important effects on the overall system response. Isolated column footings, strip footings, mat footings, and even pile foundations all may fail during seismic events. Such failures are generally attributed to liquefaction. However, a number of failures have occurred where field conditions indicate there was only partial saturation or a dense soil and therefore liquefaction alone is a very unlikely explanation. Rather, the reasons for the seismic settlements of these foundations seem to be that the bearing capacity was reduced [3]. Richards et al [3] proposed a simplified approach to estimate the dynamic bearing capacity and seismic settlement of a strip footing for assumed failure surfaces as in equation (1). There is hardly any experimental verification of these theoretical solutions. There is a need for such validation [1].

\[ S_{eq} = 0.174 \frac{V^2}{A_g} \left( \frac{k_h}{A} \right)^{-4} \cdot \tan \alpha AE \]  

Where: \( S_{eq} \) is the settlement due to earthquake, \( V \) = peak velocity for the design earthquake (m/sec), \( A \)=acceleration coefficient for the design earthquake, \( g \) = acceleration due to gravity (9.81 m/sec2), the values of \( k_h \) and \( \tan \alpha AE \) can be obtained from some well-known figures which related the required factor with angle of internal friction and bearing capacity as mentioned in [4] as well as in table 1 [4]. Generally, the behavior of any structure during earthquake depend on many items such as earthquake magnitude, acceleration, duration, height of the structure/width of the foundation (aspect ratio), foundation boundaries, soil properties and condition, and probability of liquefaction triggering [1]. In present study the specified structure has an aspect ratio < 0.8 according to this ratio and as cited in many researches the structure will suffer from vertical settlement only without any visible tilting [5], while in dry soil the foundation will suffer from excessive settlement and sliding in the wards of shaking. The propagation of the acceleration amplitude through soil depth will cause additional stress and rearrangement to the soil particles to produce more dense soil, which causes a settlement and sliding failure to the shallow structures and infrastructures.

### Table 1. Variation of \( \tan \alpha AE \) with \( k_h \) and soil friction angle \( \phi \) (Compiled from [3])

| \( k_h \) | \( \phi = 20^\circ \) | \( \phi = 25^\circ \) | \( \phi = 30^\circ \) | \( \phi = 35^\circ \) | \( \phi = 40^\circ \) |
|---|---|---|---|---|---|
| 0.05 | 1.10 | 1.24 | 1.39 | 1.57 | 1.75 |
| 0.10 | 0.97 | 1.13 | 1.26 | 1.44 | 1.63 |
| 0.15 | 0.82 | 1.00 | 1.15 | 1.32 | 1.51 |
| 0.20 | 0.71 | 0.87 | 1.02 | 1.18 | 1.35 |
| 0.25 | 0.56 | 0.74 | 0.92 | 1.06 | 1.23 |
| 0.30 | 0.47 | 0.66 | 0.84 | 0.98 | 1.10 |
| 0.35 | 0.32 | 0.55 | 0.73 | 0.88 | 1.01 |
| 0.40 | 0.27 | 0.42 | 0.63 | 0.79 | 0.94 |
| 0.45 | 0.22 | 0.36 | 0.57 | 0.72 | 0.87 |
| 0.50 | 0.18 | 0.30 | 0.49 | 0.64 | 0.79 |
| 0.55 | 0.15 | 0.25 | 0.40 | 0.55 | 0.70 |
| 0.60 | 0.12 | 0.21 | 0.32 | 0.45 | 0.57 |

In the settlement equation, it is clear that it neglects the effect of saturation or PWP generation, changes in soil density due to shaking, mitigation, and changes in effective stress or bearing capacity failure. To understand the effect of these factors, it is very important to understand the effect of each item on soil behavior individually and together. As well as the settlement formula [3] suggested a simplified approach to evaluate the dynamic bearing capacity \( q_{ue} \) and seismic settlement \( S_{eq} \) of a strip footing for assumed failure surfaces. The proposed seismic bearing capacity \( q_{ue} \) is given by equation (2) as follow:

\[ q_{ue} = cN_{cE} + qN_{qE} + \frac{1}{2} \gamma B N_{\gamma E} \]  

Where, \( \gamma \) = is the soil unit weight, \( q=\gamma D_f \), \( D_f \) is the depth of the foundation,
\( N_{cE}, N_{qE}, \) and \( N_{\gamma E} \) are the factors of seismic bearing capacity.

These factors are related to the magnitude of dynamic angle of internal friction of the soil \((\phi_{dyn})\) and \(\tan \psi = \frac{k_h}{1 - k_v} \)

\( k_h \) and \( k_v \) are the horizontal and vertical coefficients of acceleration due to earthquake.

For static case, \( k_h = k_v = 0 \) and equation (1) becomes:

\[ q_u = cN_{c} + qN_{q} + \frac{1}{2} \gamma B N_{\gamma} \]  

(3)

In which \( N_{c}, N_{q}, \) and \( N_{\gamma} \) are the static bearing capacity factors.

In practice and according to many investigations that carried out and recorded in many works of literature, the shallow foundation may or may not suffer from a reduction in bearing capacity during dynamic and/or earthquake loading.

At the same time of bearing capacity changes, there is an increase in total and differential settlement under shallow foundation during earthquake loading, this behavior is due to the densification of the soil strata and destroys the soil structure during shaking.

The previous explanation is for the non-liquefaction condition of the soil, in case of liquefaction it occurs both behaviors of bearing capacity and settlement are changed as follow:

- Bearing capacity of shallow foundation during earthquake loading and liquefaction of the underneath soil will decrease sharply after a few time period or loading cycles.
- The amount of total and differential settlement will be an increase in an excessive manner when the liquefaction has happened.
- Thus, the bearing capacity at earthquake loading will not increase as suggested in some researches if liquefaction triggering may occur in loose soils, friable rocks, and sensitive clay [4].

In dynamic and seismic loading, the angle of internal friction will change with the progress of loading cycles (decrease until the failure state will occur due to bearing capacity failure or due to excessive deformations and settlements).

In this paper, the bearing capacity and settlement of shallow foundation on liquefiable deposits has studied. The study includes a shaking table test, reviewing of theoretical equations (available in literature), which estimating settlement of shallow footings due to seismic loading, calibration, and verification of these equations with data from the shaking table test for improved soil by grouting and unimproved soil. A modification to the seismic bearing capacity and settlement equations had been done to account for the liquefaction state. Find out an adequate formula for estimating footings settlement so as to introducing the effect of liquefaction process which accompany with generation of PWP and increasing in the acceleration amplitude within soil column.

3. Suggested modifications on dynamic Bearing Capacity and Settlements of Shallow Footing on Liquefiable Soil formulas.

In liquefaction state, the reduction in the value of the angle of internal friction becomes noticeable during a few seconds of the earthquake which produce a rapid bearing capacity failure. After checking the soil susceptibility to liquefaction, the selection of the suitable angle of internal friction must be achieved to avoid sudden bearing capacity failure.

In present work, the calculation of bearing capacity during an earthquake executed (simulated by shaking table test for the soil layering and sensors distribution shown in figure 1). The liquefiable soil considered in this study is poorly graded sand with relative density of 33%. The soil was improved by permeation grout using cement kiln dust (CKD) and Bentonite slurries. The grouting process was started from bottom to top by gradual steps with uniform injection process and very low injection pressure (less than 0.05 bars), using grouting machine connected to air pressure compressor so as to permit to the grouting material to pumping through pumping hose, depending on the depth of the injection and overburden pressure of the soil.

The magnitude of design acceleration must be taken at foundation level, that producing from the transfer of acceleration amplitude from the deep origin. The correct selection of the acceleration magnitude will prevent any undesirable or unexpected failure.
The comparison between acceleration values at different levels in the shaking table test shows the effect of foundation level on its behavior during shaking.

The acceleration at the foundation level will become larger than its value at the source due to the following:

- Effect of inertia mass motion.
- Effect of loose soil strata behavior during shaking.
- Effect of generation of PWP and variation of stress states.

Figure 2 presents the variation of acceleration with the depth of soil. It is clear that when the acceleration at hard strata is 0.2g the resulting surface acceleration at foundation level become more than 0.35g.

When saturated soil (the water table is close to ground level) exposed to an earthquake, the pore water pressure will increase. If pore water pressure continues to increase, liquefaction will take place in a few seconds as shown in figure 3. This effect will result in a decrease in bearing capacity of the soil. Therefore, it is proposed to include the effect of pore water pressure in the determination of the dynamic bearing capacity of the soil.

**Figure 1.** Shaking table schematic diagram explain soil layers and distribution of transducers.
Figure 2. (a) Acceleration (g) Vs time (sec.) at bottom saturated sand layer for 1Hz.; (b) Acceleration (g) vs time (sec.) at 1st saturated sand layer for 1Hz.; (c) Acceleration (g) Vs time (sec.) at foundation for 1Hz.
Figure 3. Generation of PWP (bar) Vs time (sec.) at top layer of saturated soil model.

The more suitable and meaningful expression which relate the effects of PWP effective stress changes is the pore water pressure ratio ($r_u$), this factor can be introduced in the bearing capacity equation as a control parameter for all cases as follow:

$$qu = (1 - ru)^* (C. Nceu + q. Nqse .. + \frac{1}{2} B. N^e)$$  \hspace{1cm} (4)

When the soil is dry $r_u$ value become equal to zero thus there is no effect on the original equation, at $r_u$ equal to 0.5 which mean that the resulting bearing capacity will reduce to the half magnitude due to the generation of PWP and reduction in effective stress, and when $r_u$ become equal or near to 1 which mean that the liquefaction has occurred in this case the bearing capacity will has zero value which compatible with actual or field situation. When liquefaction mitigation methods applied on the liquefiable soils the following changes and improvement recognized:
- The soil strength increases to resist the shaking.
- There are specific reductions in the generation of pore water pressure during shaking.
- There is an increase in soil density due to filling voids as shown in table 2.

These effects will lead to maintain the propagation of acceleration along the soil column during shaking which leads to fixing the acceleration value from bottom to top layers.
- The above changes in mitigated soil produce a noticeable increase in soil stability and the bearing capacity of the mitigated soils.

| Sat. sand unit weight kN/m$^3$ | Sand + Bentonite slurry grout unit weight, kN/m$^3$ | Sand + CKD slurry grout unit weight, kN/m$^3$ | Granular pile equivalent unit weight kN/m$^3$ |
|-----------------------------|-----------------------------------------------|-----------------------------------------------|-----------------------------------------------|
|                             | 3%                                            | 5%                                            | 10%                                           | 3%                                            | 5%                                            | 10%                                           | 3%                                            | 6%                                            | 3%                                            | 6%                                            | 3%                                            | 6%                                            | 3%                                            | 6%                                            | 3%                                            | 6%                                            |
|                             | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           | 3/1                                           | 6/1                                           |
| 18.5                        | 19.3                                          | 18.92                                         | 19.5                                          | 19.24                                         | 19.3                                          | 19.4                                          | 19.25                                         | 19.4                                          | 19.25                                         | 19.4                                          | 19.25                                         | 19.4                                          | 19.25                                         | 19.4                                          | 19.25                                         |

The presence of water within soil strata will make the soil settle due to lubrication of the particles which tend to become closer to each other. This phenomenon recognized in the experiment when the saturated soil left for one day leading to settlement. When the shaking or earthquakes are applied there are noticeable generations of PWP which cause soil liquefaction.

The change in soil unit weight due to shaking in the dry state is differing from a change in soil density due to liquefaction mitigation process. In the first case, the densification causes a specific settlement due to soil particles rearrangement while in soil mitigation the change in density is due to replacement of the water inside the voids by grouting materials without and rearrangement or densification of soil particles. This behavior exactly recognized during shaking tests at all applied acceleration. The main difference between the non-mitigated soil and the mitigated one is the magnitude of the settlement.
It is important to note that the simplified approach proposed by [3] is derived to estimate settlement for non-liquefied soils. This may explain the divergence between experimental and the simplified approach, since the settlement measured experimentally for liquefied soil post liquefaction.

Also, at mitigated soil the estimation of settlement according to the same formula required a modification so as to adjust the settlement magnitude. Settlement formula didn’t take into account the soil density, degree of saturation, generation of PWP, and reduction in effective stress during shaking. A modified formula for settlement estimation suggested.

This modification makes Richards formula takes the effect of soil density and pore water pressure generation during an earthquake in consideration. See equations (5) and (6).

For frequency amplitude ≤ 0.75 Hz.

\[ S_{EQm} = S_{EQ} \times \left( \frac{\gamma_{soil}}{\gamma_{new}} \right)^a \times \left( \frac{b}{(c-r_u d)} \right) \quad (r_u \leq 0.90) \]  

(5)

a, b, c, and d: are an empirical coefficient from statistical analysis by SPSS software and it equal to (58.8, -1.85, 1, and -0.133 respectively) the regression coefficient (R² = 0.95).

For frequency amplitude > 0.75 Hz.

The empirical coefficients become as follow: (-35.1, -321.6, -468.8 and -3.83 respectively) the regression coefficient (R² = 0.86).

Where \( \gamma_{new} \) is the new or improved soil unit weight after mitigation or improvement process. In case of \( r_u \) (0.9-1) there are additional formula can be introduce which cover the effect of pore water pressure generation and the reduction in effective stress. This formula can be written as follow:

\[ S_{EQm} = S_{EQ} \times \left( \frac{\gamma_{soil}}{\gamma_{new}} \right)^e \times \left( \frac{1}{r_u} \right)^g \]  

(6)

Where e, is an empirical coefficient from statistical analysis which equal to 1.5 and \( g = (\gamma_{soil} / (1 - (\gamma_{new} - \gamma_{soil})) /100, \) the regression coefficient (R² = 0.89).

The above-modified formula has been obtained using statistical analysis by using SPSS software. A good convergence between modified formula and experimental measurement noticed in table 3 and Figures 4, 5, 6 and 7.

The settlement of the foundation during shaking before and after soil mitigation were recorded for some cases and calculated using original and modified formula. It is observed that the original formula is suitable for high acceleration levels without probability of liquefaction triggering.

| Table 3. Settlemnts of saturated pure, mitigated soils at different acceleration levels depending [3], modified, and experimental measurements. |
| Settlement formula | Saturated sand | Sand +5% Bentonite+6/1 | Sand +CKD +6/1 | Granular piles |
|-------------------|----------------|-------------------------|----------------|--------------|
| Formula of [3]    | 0.05g 0.1g 0.2g | 0.05g 0.1g 0.2g | 0.05g 0.1g 0.2g | 0.05g 0.1g 0.2g |
| Modified Richard  | 0.05 0.83 11.6 | 0.05 0.83 11.6 | 0.05 0.83 11.6 | 0.05 0.83 11.6 |
| Experimental      | 2.0 17.6 25.3 0.064 1.56 22.2 | 0.065 1.03 11.6 | 0.05 1.4 15.15 | 2.5 19.1 26.0 1.2 2.8 31.0 | 0.1 0.75 10.2 | 0.3 3.5 15.35 |
Figure 4. Relationship between settlement of foundation and acceleration on saturated pure sand.

Figure 5. Relationship between settlement of foundation and acceleration on saturated sand + 5% Bentonite + 6/1 dilution percent.

Figure 6. Relationship between settlement of foundation and acceleration on saturated sand + 5% CKD + 6/1 dilution percent.
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4. Conclusions
A shallow footing resting on liquefiable poorly graded saturated sand was subjected to horizontal acceleration by a manufactured controlled and instrumented shaking table. The acceleration of the foundation and of the sand deposit is quite different from the input table motion. A modification on bearing capacity equation was introduced for seismic design taking into account the buildup in pore water pressure. This modification makes the equation applicable on liquefiable soil. Another modification was prepared for [3] equation of settlement to take into account the liquefaction triggering due to the buildup in pore water pressure. Good convergence was observed between the predicted and measured settlement.

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Figure 7. Relationship between settlement of foundation and acceleration on saturated sand+ granular pile.
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