Earth Science Research; Vol. 1, No. 2; 2012
ISSN 1927-0542 E-ISSN 1927-0550
Published by Canadian Center of Science and Education

Liquefaction Potential Variations Influenced by Building Constructions

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Received: January 13, 2012 Accepted: January 29, 2012 Online Published: May 24, 2012

doi:10.5539/esr.v1n2p23 URL: http://dx.doi.org/10.5539/esr.v1n2p23

Abstract

In this paper, a sand deposit with liquefaction potential was simulated in *FLAC* using Finn constitutive model. Equivalent loads of 5, 10, 15 and 20-story buildings were applied to the deposit. As a cyclic loading, acceleration time history of Bam earthquake was used. To examine pore water pressure variations in the soil mass during the earthquake loading, parameter r_u (pore water pressure ration, which equals pore water pressure increment divided by the initial mean effective stress in the soil mass) was defined for the software by a Fish function. Static analyses show that by increasing the applied loading due to building construction, the values of effective and shear stress increase and it generally can be a factor to influence liquefaction potential. Furthermore, dynamic analyses show that there is a shallow longitudinal area beneath 15 and 20-story buildings in which liquefaction potential increases due to high confining effective stress.

Keywords: liquefaction potential, FLAC, pore water pressure ratio, standard penetration test, Finn constitutive model

1. Introduction

When saturated sandy soils are loaded by earthquake shaking, pore water pressure in the soil mass starts to progressively build-up leading to loss of soil shear strength. Liquefaction is the main consequence of this phenomenon. At the occurrence of soil liquefaction, the effective stress in the soil mass becomes zero and pore water pressure in the soil becomes equal to the initial static effective stress.

Most of the time, liquefaction phenomenon occurs at loose saturated sandy soils situated on regions which are prone to having earthquakes. When a saturated loose sand deposit is subjected to earthquake shaking, applied shear stress on the soil mass leads to reduce the volume of the soil mass and because the soil mass is saturated, this tendency to reduction of volume leads to an increase in pore water pressure. In these cases during dynamic loading, pore water pressure in the soil mass starts to progressively build-up and effective stress inclines to zero and the soil mass shows semi-liquid behavior and does not have enough shear strength.

A parameter named r_u (pore water pressure ratio) can be defined for examination of this process, equals the pore water pressure increment divided by the initial static mean effective stress in the soil. During cyclic loading in the loose saturated sandy soil, pore water pressure starts to progressively build-up and shear strain of the soil mass progressively increases and parameter r_u inclines to 1.0. In this situation, the soil mass might experience maximum shear strains and deformations because there is not enough shear strength.

Recently, some researchers have done examinations about the influence of external loading (such as building and embankments) on liquefaction potential. Most of the researches are classified in two major categories; the influence of building constructions on i) soil properties and ii) liquefaction potential, and they are discussed in the next part of the introduction.

1.1 The Influence of Building Constructions on Soil Properties

Rollins and Seed (1990) introduced three factors to evaluate the effect of overburden pressure on liquefaction potential. These factors are static shear stress, effective confining pressure and over consolidated ratio (OCR).

1.1.1 Initial Shear Stress

Overburden pressure and slope situation may induce anisotropy consolidated condition and cause initial static shear stress in the soil mass. According to related studies, static shear stress may affect soil liquefaction potential directly. Lee and Seed (1967) indicated that the liquefaction resistance of soil increases by increase of static shear stress. Increase of initial static shear stress in the soil mass may lead to increase of soil settlement and compression and subsequently, it leads to increase of Cyclic Resistant Ratio (CRR).

$$CRR = \frac{\tau_{cy}}{\sigma'_{v0}} \qquad (1)$$

where: τ_{cy}= cyclic shear strength, σ'_{v0}= vertical effective stress.

1.1.2 Effective Confining Pressure

Using the results of dynamic tri-axial testing, Peacock and Seed (1968) indicated that cyclic shear stress increases by increase of effective confining pressure, but Cyclic Resistance Ratio (CRR) is contrary. Mulilis et al. (1975) denoted that Cyclic Resistance Ratio (CRR) may slightly decrease by increase of effective confining pressure. Hynes and Olsen (1998) concluded that several factors such as method of deposition, stress history, aging effects and density may affect the influence of confining stress variations on the CRR.

1.1.3 Over-consolidation Ratio (OCR)

According to related studies, over-consolidation state is an important effect for soil liquefaction potential. If a soil mass has experienced stresses higher than its current state, it is an over-consolidated soil (OCR>1).

$$OCR = \frac{p_c}{p_0} \qquad (2)$$

where: p_c=over consolidation stress, p_0=current existing stress.

Seed and Idriss (1971) showed that the liquefaction resistance increases by increase of the OCR values. Using cyclic torsion shear test, Ishihara and Takatsu (1979) showed the relations between OCR, K_0 and cyclic shear strength. As shown in their results, under constant K_0, the cyclic stress ratio increases by increase of the OCR value.

1.2 The Influence of Building Constructions on Liquefaction Potential

Using shaking table tests facilitates, Yoshimi and Tokimatsu (1978) denoted that soil liquefaction strength decreases beneath the building and liquefaction potential becomes more in this region. Whitman and Lambe (1986) also concluded the similar results by centrifuge model tests. Lopez and Modaressi (2008) preformed a numerical modeling about this issue and indicated that the pore water pressure distribution at the end of the earthquake motion is modified by the presence of the structure, even if the soil profile is far from it.

This study is about a general model ground and it is tried to determine the influence of building construction on liquefaction potential and consider pore water pressure distribution in the soil mass. For this aim, a sand deposit with liquefaction potential was simulated in *FLAC* using Finn constitutive model. Finn model unifies equations represented by Martin et al. (1975) and Byrne (1991) into the standard Mohr-Coulomb plasticity model. Parameter $(N_1)_{60}$ is the main factor for Byrne (1991) formula, so in the present study some parameters required for the Mohr-Coulomb plasticity model were defined for the program by equations which relate them with $(N_1)_{60}$. Considering effect of building construction on liquefaction potential, equivalent loads of 5, 10, 15 and 20-story buildings were applied to the deposit. Furthermore, as a cyclic loading, acceleration time history of Bam earthquake was used.

2. Finn Constitutive Model and Modeling Procedure

In this study, *FLAC* software which is a Finite Difference Method-based program (FDM) was used. According to *FLAC* guidance manual, there are several constitutive models that facilitate soil behaviors under static and dynamic loadings. Calculation of excess pore water pressure in the soil mass due to dynamic loading is the main factor in the modeling process of liquefaction phenomenon. *FLAC* has a constitutive model named Finn model which unifies equations represented by Martin et al. (1975) and Byrne (1991) into the standard Mohr-Coulomb plasticity model. Using this model, it is possible to calculate pore water pressure generation by calculating irrecoverable volumetric strains during dynamic analysis. The void ratio in this model is supposed to be constant, also it can be calculated as a function of volumetric strain and other parameters can be defined by void ratio.

Martin et al. (1975) described initially the effect of cyclic loading on increase of pore water pressure as a result of irrecoverable volume contraction in the soil mass. In these situations, because the matrix of grains and voids is filled by water, the pressure of pore water increases. Later, Byrne (1991) presented a simpler equation which corresponds irrecoverable volume change and engineering shear strain with two constants. In this model, a soil mass with liquefaction potential was modeled using $(N_1)_{60}$ parameter as a main factor to the Finn model, so all of the soil properties needed for the model were defined for the program by $(N_1)_{60}$.

2.1 Definition of Soil Mass and Mesh Model

In this study, the soil profile consists of two types: First type was assumed with 30 m height saturated loose sand and second type of the soil was assumed with 2 m height dry and relatively dense sand that overlies the loose sand, so water level is at depth of 2 m from the surface. Defining dense upper layer helps the model to be close to reality. Furthermore, the length of soil mass was assumed 400 m (how this length is selected will be discussed later). According to Figure 1, grid size in the middle region of this mesh was selected finer enough to satisfy the analysis exactness and equivalent load of the building was imposed on this area to examine the effect of it on liquefaction potential of the soil deposit.

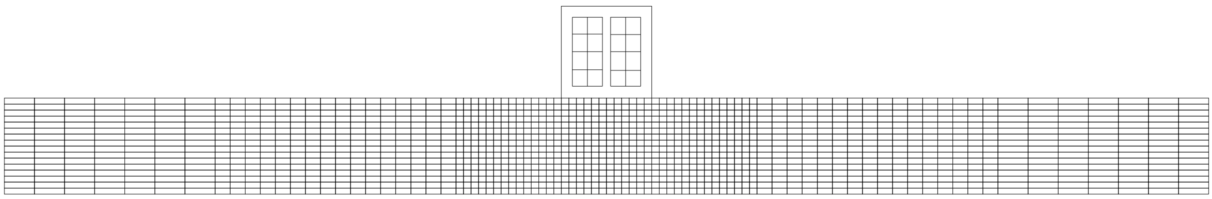

Figure 1. Physical schema of the soil mass mesh and location of loaded area

3. Imported Numerical Quantities for the Soil Mass

For the soil materials, based on the state of sandy soil density it was tried to define properties of materials according to their weight-volume relationships.

According to the Fish function written in *FLAC* program, for definition of liquefaction properties, Byrne (1991) formula was used.

$$\frac{\Delta\epsilon_{vd}}{\gamma} = C_1 \exp\left(-C_2\left(\frac{\epsilon_{vd}}{\gamma}\right)\right) \tag{3}$$

where: $\Delta\epsilon_{vd}$ is the increment of volume decrease and γ is cyclic shear-strain amplitude. This equation has two constants C_1 and C_2. Byrne (1991) notes that the constant, C_1, can be derived from normalized standard penetration test values, $(N_1)_{60}$,

$$C_1 = 8.7(N_1)_{60}^{-1.25} \tag{4}$$

C_2 is then calculated from $C_2 = 0.4/C_1$.

For determining the relative density (D_r) by $(N_1)_{60}$, Idriss and Boulanger (2004) was used

$$D_r = \sqrt{{(N_1)_{60}}/{46}} \tag{5}$$

According to ASTM-D2040 we determined e_{max} and e_{min} for several sandy soil specimens and the average values of them were: e_{max}=0.8 and e_{min}=0.25, so the quantity of "*e*" for each state of relative density according to Equation (6) was calculated.

$$D_r = (e\_max - e)/(e\_max - e\_min\,) \tag{6}$$

The values of "*e*" for each $(N_1)_{60}$ values can be calculated.

Table 1 summarizes above-mentioned soil properties.

Table 1. Numerical values of soil properties for the model

Soil condensation	Loose soil	Medium dense soil	Overlaid soil
$(N_1)_{60}$	8	15	30
D_r	0.417	0.571	0.81
γ_d (kN/m^3)	17	18	19.6
γ_{sat} (kN/m^3)	20	20.5	22.2
e	0.570	0.485	0.354

As mentioned in introduction, to examine the pore water pressure variations in the soil mass during the earthquake loading, the parameter r_u was defined for the software by a Fish function. Theoretically, if r_u inclines to 1.0, effective stress inclines to zero and liquefaction should occur. But r_u=1.0 is only theoretical definition for liquefaction occurrence. To make a valid comparison between in situ and laboratory pore pressure responses, Hazirbaba and Rathje (2004) carried out series of strain controlled- undrained cyclic simple shear tests on the soils which have same situations in the field. Results of the pore pressure generation show a smooth progressive pattern until the pore pressure ration reaches the value of about 0.9 and liquefaction occurs in that point. Therefore, lower boundary of r_u values which define liquefaction occurrence is 0.9 and if r_u reaches a value greater than 0.9 it is assumed that liquefaction happens.

3.1 Initial Conditions and Boundary Condition

The base boundary of the model was fixed along horizontal and vertical directions in both static and dynamic analyses. Right and left boundaries of the mesh were horizontally fixed for static analysis. In dynamic analyses, enough distance between the structure and right and left boundaries should be considered to prevent the reflection of waves contacting the boundaries. Choosing adequate dimensions for the model plays an important role in modeling process. For this aim, the length of mess was assumed 400m by trial and error method (data are not shown).

In *FLAC*, the dynamic input can be applied in one of the following ways: (a) an acceleration history; (b) a velocity history; (c) a stress (or pressure) history; or (d) a force history. In this study, for applying earthquake loading, acceleration time history of Bam earthquake with magnitude M=6.5 and maximum acceleration a_{max}=0.42 g was used. Duration of earthquake was chosen 37 seconds.

3.2 External Loading

Assuming both building and its foundation loading equal to 20 kPa per each m^2 for each story of building, numerical values of extended load applied to the soil surface for each above mentioned buildings are according to Table 2.

Table 2. Numerical values of applied extended loads for the model

Number of building stories	5	10	15	20
Equivalent load (kPa)	100	200	300	400

4. Results and Discussions

4.1 Examination of Results Due to Building Load on Soil Properties after Static Analysis

4.1.1 Shear Stress Distribution in the Soil Profile

In this stage, building construction loads were applied to loose sand with $(N_1)_{60}$=8 and medium dense sand with $(N_1)_{60}$=15. Figure 2 shows shear stress contours after applying static loading under 20-story building in soil with $(N_1)_{60}$=8. According to this figure, the values of shear stress start to expand from two edges of buildings and the influence of shear stress becomes wider by depth. The influence of shear stress distribution under the foundation due to weight of the building and its effect on liquefaction potential will be discussed.

Figure 2. Shear stress distribution under 20-story building in soil with $(N_1)_{60}$=8

4.1.2 Confining Effective Stress Distribution in the Soil Profile

Figure 3 from top to bottom, shows confining effective stress contours in case $(N_1)_{60}$=8 related to 5-story building (Part(A)), 10-story building (Part (B)), 15-story building (Part (C)), 20-story building (Part (D)), respectively. According to Figure 3, parts C and D, applying heavy loads on the soil mass can lead to stress concentration on the edges of loaded area and increase of initial confining effective stress in this region. This process may lead to increase in growth potential of ΔU values in equation $r_u = \Delta U/\acute{\sigma}_0$ and therefore, liquefaction potential in the regions near foundation of tall buildings may increase.

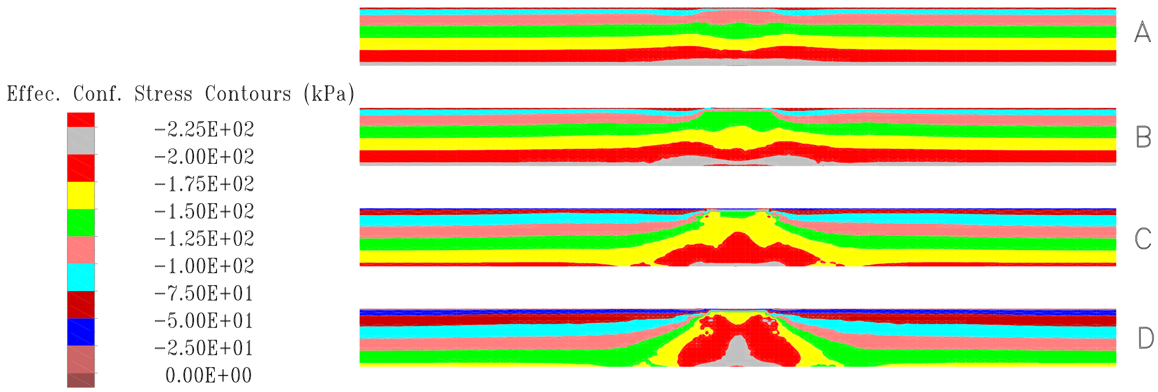

Figure 3. Confining effective stress contours in case $(N_1)_{60}$=8 before dynamic analyses

As described in the introduction, according to previous works preformed by several researchers, the influence of shear stress distribution and confining effective stress on liquefaction potential is contrary. Increase of these two parameters in the soil mass may affect the liquefaction resistance.

4.2 Examination of Results after Dynamic Analysis

After static analyses, as a dynamic loading, acceleration time history of Bam earthquake was applied into the model. Figures 4 and 5, from top to bottom, show maximum values of r_u related to 5-story building (Part(A)), 10-story building (Part (B)), 15-story building (Part (C)), 20-story building (Part (D)), respectively.

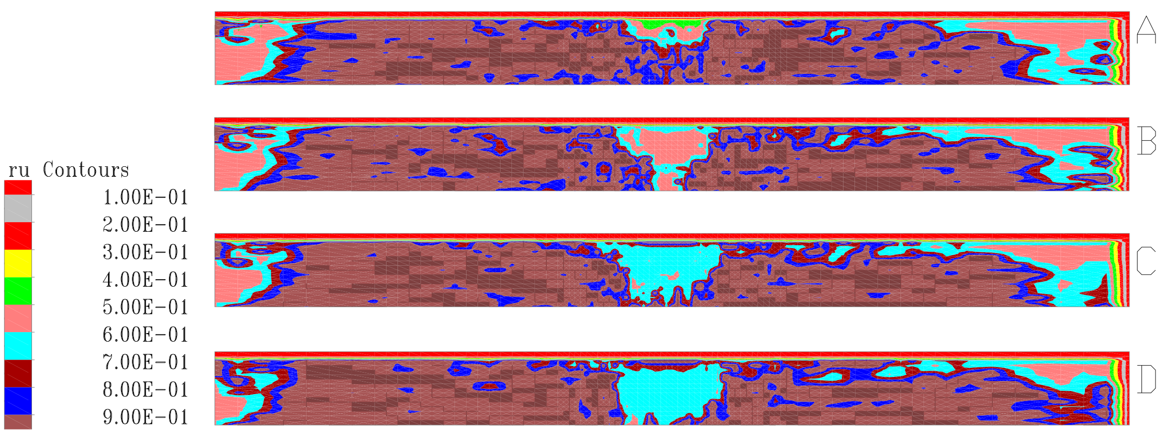

Figure 4. Maximum values of r_u in loose sand at presence of building loads induced by dynamic loading

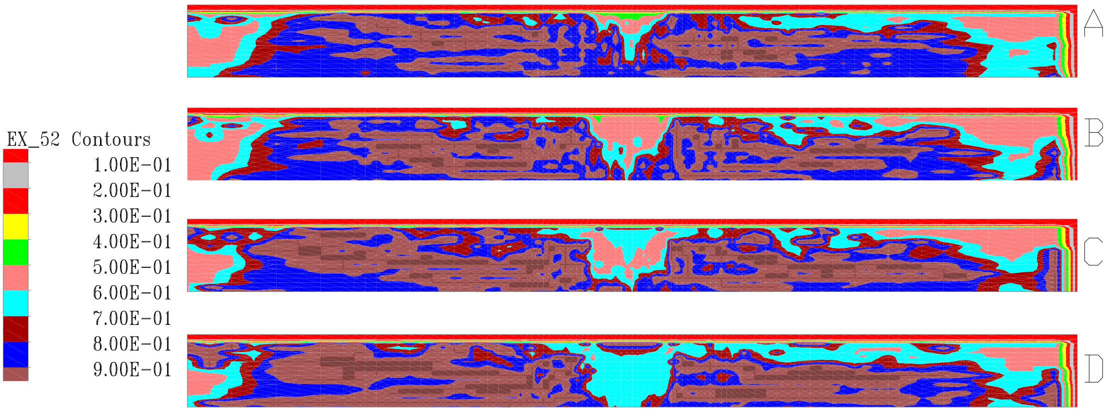

Figure 5. Maximum values of r_u in medium dense sand at presence of building loads induced by dynamic loading

According to Figures 4 and 5 in all cases, in general, construction loading leads to reduce maximum values of r_u in the underlying deposits. The main reason of this result is the influence of applied loading on the soil mass and increase of effective stress in the soil deposit before earthquake loading. This loading leads to an increase in both relative density and shear stress values in the soil mass. Also it can be seen that by increase of external loading values, safe area (regarding to occurrence of liquefaction phenomenon) becomes larger. Therefore, occurrence of liquefaction phenomenon under buildings is generally cancelled.

4.3. Discussion

According to Figures 4 and 5, building construction can be a reduction factor of liquefaction potential in the soil mass, but under 15 and 20-story buildings (parts C and D), there is a shallow longitudinal region under loaded area (marked by dark blue) in which the maximum values of r_u reach about 0.8 to 0.9. In this region, amount of effective stress increment due to external loading theoretically inclines to the overburden value. In this case, it should be elaborated that a change in soil mass mechanism is occurred and soil liquefaction strength (or CRR) decreases significantly beneath the building. The main reason of this phenomenon might be confining pressure built-up in upper layers due to heavy overburden pressure.

The influence of shear stress distribution and effective confining stress on liquefaction potential is inverse, so considering Figures 2 to 5, increase of liquefaction potential in upper layers due to higher confining stresses is dominant than decrease of liquefaction potential due to shear stress distribution in this layer and therefore, maximum values of r_u are close to 0.9 and liquefaction probability beneath tall buildings becomes more. Conversely, decrease of liquefaction potential in lower layers due to shear stress distribution is dominant than increase of liquefaction potential due to higher confining stresses and therefore, probability of liquefaction occurrence in lower layers of the soil mass becomes less.

5. Conclusions

Based on numerical results, the following conclusions can be obtained.

1). Results show that by increasing the applied loading due to building construction, the values of effective confining and shear stress in the soil mass increase and it generally can be a factor to influence liquefaction potential in the deposit. In general, overburden pressure due to building construction leads to reduction in maximum values of r_u in the underlying deposits.

2). One of the other effective factors about liquefaction phenomenon is generation of effective confining stress in the soil mass situated under loaded area with high overburden pressure. Due to external loading, the value of excess effective stress in upper layers inclines theoretically to the applied overburden loading. The increase of effective confining stress in this region may lead to an increase in growth potential of ΔU in upper layers. The increase of ΔU in equation $r_u = \Delta U/\acute{\sigma}_0$ may lead to an increase in r_{umax} values and therefore liquefaction potential in upper layers is high.

3). Generally, the stress concentration and high effective confining stress in upper layers of the soil mass can be named as the factors to increase of liquefaction potential due to building construction. Construction of tall buildings on the alluvium applies heavy load into the soil and changes soil mechanism due to stress concentration on two sides of loaded area. Therefore, liquefaction potential in the regions near foundation of building increases.

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