Numerical simulation of a single pile under the combined effects of axial and lateral loading in liquefiable soil

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Abstract: In this research, a numerical investigation was carried out using FLAC 2D software to develop an understanding of the effects of combined (axially and laterally) loading on a single pile subjected to different earthquake motions. Several parameters were studied, including vertical displacement at pile head and soil surface lateral displacement with pile depth, and pore water pressure ratio (ru) at the near and far-fields within the soil model; these were then compared with the soil shaking table test, and the numerical model was solved with the Finn Model and Mohr-Coulomb to capture the non-linear effects of saturated sand under earthquake motions in Ali Algharbi and Kobe. The main test results from this numerical analysis showed that the lateral load on the pile headsignificantly reduced pile lateral displacement, yet had substantial effect on both pile and soil vertical displacement even at higher earthquake magnitudes. A comparison of pore pressure ratios at near and far-field between the numerical and experimental work showed a good agreement, with only a slight difference in overall values. The importance of this study and the test results imported from the numerical model lies in the generation of a reliable base for liquefaction analysis in more complex cases.

Keywords: Numerical Analysis, FLAC 2D, Earthquake motion, Liquefaction, Single pile

1. Introduction

Liquefaction occurs where saturated soil is subjected to shear stress cycles, combined with any excessive decrease in volumetric strain [1, 2, and 3]. Collapse damage resulting from such liquefaction is often observed in pile foundations during and after strong earthquake motions, despite the large factor of safety implemented in such designs [4]. Pile foundation failure characteristics also occur during earthquake motion in liquefiable soil, with large settlement, pure bending, maximum shear, and progressive pore pressure [5, 6, 7, and 8]. For example, during the Kobe earthquake in 1995, liquefaction was a major concern, which destroyed many piles, bridges, and structures. The Shin–Shukugawa bridge in particular was exposed to extreme pile foundation movement due to liquefaction [9]. In this context, the current study was based on a numerical investigation of the effect of combined loading in such circumstances, specifically lateral loading support, as previous studies have focused only on axial load and pile bending behaviours, with limited attention to combined loading. However, there will always be some lateral load effect on piles within soil resulting from inertia, and lateral spreading could increase the lateral displacement of such piles. Ghorbani et al. [10] investigated single piles in liquefiable soils using the finite difference method in the FLAC 2D environment. The results were verified with experimental data and showed good agreement. The model was found to be suitable in term of
examining axial load on the pile and other parametric studies; they also noted that axial load applied to the pile was a liquefaction risk reduction factor as it imposed higher effective stress around the pile, resulting in lower excess pore water pressure ratios. Naeini et al. [11] studied the behaviours of single pile bending moments and vertical and horizontal displacements within liquefied soil via numerical simulation using a FLAC 2D program. They pointed out that increasing the predominant frequency reduced the risk of liquefaction incidence, while the maximum lateral displacement at the pile head decreased when the earthquake frequency value decreased. The maximum displacement was much greater with higher acceleration, while the highest vertical displacement value corresponded to the lowest predominant frequency. Choobbasti et al. [12] discussed the response of a single end-bearing pile in liquefiable soil based on applying the two-dimensional finite difference program FLAC 2D using two earthquake motions with different predominant frequencies. Based on the results, they observed that the predominant frequency and the Arias' intensity of the earthquake positively affected the lateral displacement and the bending moment. Under critical conditions, these can lead to poor performance of the pile and structural damage.

In the last two decades, the Iraqi region has became more prone to seismic activity, leading local researchers to investigate pile foundations in sandy soil under different types of earthquake motions [13 and 14]. Al-Tameemi [14] studied the effects on single pile under different earthquake histories using flexible laminar shaking table tests and FLAC 3D numerical studies with different pile diameters and real earthquake motions, from weak to very strong, at various factors of safety. He noted that pile settlement in loose sand increased significantly as the acceleration of the earthquake increased, while, pile lateral displacement was maximised at maximum acceleration and remained constant with earthquake duration. That study also confirmed that the numerical simulation using FLAC 3D was appropriate for predicting liquefaction based on pore pressure generation and dissipation under dynamic loads.

In the current research, a numerical investigation was carried out to develop an understanding of the effect of combined (axially and laterally) loading on a single pile subjected to different earthquake motions using FLAC 2D software. Several parameters were studied, including vertical displacement at pile head and soil surface, lateral displacement along with pile depth, and pore water pressure ratio (ru) at the near- and far-field within the soil model; these results were compared with the soil shaking table test.

2. Numerical Modelling

2.1 Finite Difference Model

FLAC 2D (Fast Lagrangian Analysis of Continua, version 8.0) [15] computer software was used to solve fully coupled flow stress problems using finite difference procedures under an explicit time scheme. A soil model of 21 m length and 28 m height consisting of two layers, loose sand (liquefied) and dense sand, was simulated using 2D plane strain analysis with eight-node block elements. The analysis was then applied in different stages. In the static stage, the construction of the proposed soil model with Mohr-Coulomb properties and fixed boundary conditions (pinned) at the base and roller boundary at the side of the domain to prevent vertical motion and to allow for equilibrium was completed. After that, the phreatic surface was initiated in order to define a pressure distribution as shown in figure 1. The second dynamic stage involved the application of the real time acceleration history of an earthquake by applying quiet boundary conditions along the base and free failed boundaries at the side of the model. The use of a large soil model was intended to minimise reflected waves and energy radiation, with material damping absorbing most of the energy in the waves reflected from distant boundaries [15]. The pile element model was created with two different segments to allow for normal and shear behaviours, installed to 17.5 m and embedded 14 m below the surface, while the pile cap was constructed using a beam element to ensure superstructure loading.
2.2 Soil Domain and Pile
The soil domain grid was generated and executed using the graphical panel in FLAC 2D. Two different layers were implemented: loose sand with 33% relative density (liquefied), extending to 10 m below the surface, and dense sand (67% relative density) from 10 m below the surface to the base of the model, as shown in figure 1. Domain compatibility was illustrated in the way both frequency and input wave speed characteristics of the system affected the accuracy of wave transmission in the numerical model [16]; the element size of the domain had to range between 0.125 and 0.1 $\lambda$ where $\lambda$ is the wavelength of the vibration. The input wave used was based on the acceleration recorded with a maximum frequency of 10 Hz for a scaled earthquake at Kobe with an amplitude of 0.783 g. The mesh size was designed so that the largest zone size was equal to 0.4, so that the maximum frequency could be directed accurately, as shown in equation 1 [16]:

$$F = \frac{Cs}{10 \Delta l} = \frac{44}{10 \times 0.5} = 11 \text{ Hz}$$

where: $Cs$= shear wave speed, $\Delta l$= smallest wavelength, and $F$= maximum frequency

To enable accurate representation of seismic wave propagation through a model, the spatial element size, $\Delta l$, must be smaller than approximately 1/8 of the wavelength related to the highest frequency of the input wave. Based on the soil properties listed in table 1, the upper layer was assigned a lower shear
wave speed of $Cs = 44\text{ m/sec}$, based on shear modulus $= 4E6\text{ Pa}$ and saturated density $= 1,990\text{ kg/m}^3$. The concrete pile had a diameter of 0.56 m and length of 17.5 m, embedded to a depth of 14 m, to support an overall load from the superstructure of 858 kN total allowable axial load and 175 kN as a total allowable lateral load acting on the x-direction of the pile cap. The model was first imposed on geostatic stress equilibrium under gravity loading before pile installation and pore pressure development at the phreatic level, as shown in figure 2. Table 2 lists the pile and beam properties used in the numerical simulation.

![Figure 2: Geostatic step and pore pressure development at the phreatic level](image)

### Table 1: Soil properties used in numerical analysis

| Property                      | Loose Sand | Dense Sand |
|-------------------------------|------------|------------|
| Dry Density, kg/m$^3$         | 1590       | 1768       |
| Bulk Modulus K, Pa            | 6.7E6      | 8.3E8      |
| Shear Modulus G, Pa           | 4E6        | 3.8E8      |
| Poisson’s ratio, v            | 0.25       | 0.3        |
| Porosity                      | 0.4        | 0.4        |
| Friction, degree              | 30         | 40         |
| Wet Density, kg/m$^3$         | 1990       | 2168       |
| Cohesion, Pa                  | 1000       | 4000       |
| Dilation Angle                | 0          | 10         |
| Shear wave speed, Cs m/sec    | 44.8       | 421.2      |

### Table 2: Structural properties for pile and beam elements
2.3 Soil-Pile-Interface

FLAC 2D provides a unique interface utilising Coulomb sliding and tensile and shear bonding. Interfaces properties thus include friction, cohesion, dilation, and tensile shear bond strength; in this way, pile contact with soil interface can be defined by slippage and separation during an earthquake in terms of normal and shear coupling spring as defined by stiffness, which can be obtained by applying equation 2 (Itasca Consulting Group Inc):

\[ K_n \text{ or } K_s = 10 \times \max \left\{ \frac{K + \frac{G}{\Delta Z_{\min}}}{} \right\} \] (2)

where \( K_n \) and \( K_s \) are the normal or shear stiffness, \( \Delta Z_{\min} \) is the minimum smallest dimension of \( K \), and \( G \) is the bulk and shear modulus of soil.

The exposed perimeter of a pile, along with properties of the coupling springs, were chosen to represent the behaviours of the pile/soil domain interface, as listed in table 3.

| Property                       | Pile cap | Pile |
|-------------------------------|----------|------|
| Normal Stiffness, N/m/m       | 1E7      | 1E7  |
| Shear Stiffness, N/m/m        | 1E7      | 1E7  |
| Normal Cohesion, Pa           | 1000     | 4000 |
| Shear Cohesion, Pa            | 1000     | 4000 |
| Normal Friction, degree       | 30       | 40   |
| Shear Friction, degree        | 30       | 40   |

2.4 Dynamic Pore Pressure

Finn-Byrne is the most effective elastic-plastic model for liquefied soil behaviours under dynamic loading. Pore water pressure generation is used to calculate the irrecoverable change in the volumetric strain as a function of volumetric strain in the presence of a constant void ratio [17]. Equation 3 thus states the relationship of shear strain to irrecoverable volume [18].
\[ \Delta \varepsilon_{vd} = C_1 \exp \left( \frac{-C_2 \varepsilon_{vd}}{\gamma} \right) \] (3)

where \( C_1 \) and \( C_2 \) are derived from relative density as shown in equations 4 and 5 [17]

\[ C_1 = 7600 \times (Dr)^{2.5} \] (4)

\[ C_2 = 0.4/C_1 \] (5)

where \( \Delta \varepsilon_{vd} \) = incremental volume strain, \( \gamma \) = shear stress, and \( Dr \) = relative density

The behaviours of the liquefiable soil layer in this study were controlled via the Finn model, a constitutive model built-in to FLAC 2D, using coupled model work combining Mohr-Coulomb with volumetric change based on pore water pressure generation. Mohr-Coulomb was selected for the static stage, while the loose sand was defined using the Finn model. The modelling stage progressed through the phreatic process based on using a water modulus of 2E8 Pa, one order of magnitude smaller than the modulus of pure water to account for dissolved air within the soil mixture at 99% saturation [15].

2.5 Boundary Condition

Roller boundary conditions were applied at the bottom of the model for the static stage, while a compliant boundary was applied at the base of the soil model to reflect shear stress by converting the velocity of the input wave \( \sigma_s = -2\rho C_s V_s \) based on the free-field at the lateral side boundaries [16].

2.6 Input Acceleration

Two different ground actual motion records were considered for the analysis; these were based on the local motion of Ali Algharbi, with peak ground acceleration (PGA) 0.1 g, and the scaled Kobe earthquake at 0.782 g. Figure 3 shows the time acceleration and Fourier spectra records, while Table 4 presents information about earthquakes.

![Figure 3](acceleration_fourier.png)

**Figure 3:** Acceleration and Fourier spectra records for scaled Kobe and Ali Algharbi earthquakes
Table 4: Earthquake inputs for numerical simulation

| Earthquake        | Ali-Algharbi | Kobe |
|-------------------|--------------|------|
| Magnitude, (Mw)   | 4.9          | 6.9  |
| Maximum acceleration, (g) | 0.1          | 0.782 |
| Shake duration, (sec) | 160          | 48   |
| Region            | Iraq         | Japan |
| Sampling frequency, (Hz) | 10           | 50   |
| Epicentre depth, (km)  | 10           | 17.9 |

2.7 Case analysis

Numerical analysis using FLAC 2D was used to investigate the effect of combined loading on a pile in liquefiable soil. A saturated soil domain was subjected to different earthquake motions, based on earthquakes in Kobe and Ali Algharbi, and pile foundations with combined axial and lateral loadings. The constant mass of the structure was 885 kN at the pile head, while the lateral load varied between 50 and 100% of the allowable load imposed at the pile head, equivalent to 175 kN and 87.5 kN, respectively. The numerical model held the superstructure load under gravity force in the static stage extending this to the dynamic stage, while the lateral load introduced fixity to reduce pile head motion in the x-direction during the dynamic stage. Table 5 lists the cases analysed in the test runs and the nomenclature applied to each case.

Table 5: Cases analysed in FLAC 2D

| Earthquake     | Ali-Algharbi           | Kobe               |
|----------------|------------------------|--------------------|
| Lateral Load   | 100 % axial, 0 lateral = A1 | 100 % axial, 0 lateral = K1 |
|                | 100% axial, 50% lateral = A2 | 100% axial, 50% lateral = K2 |
|                | 100 %axial, 100 %lateral = A3 | 100 %axial, 100 %lateral = K3 |
| Pore Pressure ratio (ru) | Far-Field and Near Field at a depth of 3 and 7.5 m |
| Settlement     | Pile head- Soil surface |
| Displacement   | Pile head- mid-span of the pile - the quarter of the pile - Pile tip |
| Model Validation | Soil Shaking table, as a reference to compare the actual test result with numerical |
| Scale factor, $\lambda$ | 35 |

Table 5 shows the different parameters investigated in the numerical model; all test results were then scaled to $\lambda = 35$ to ensure best fit with the experimental work, based on a shaking table manufactured by AL-Tameemi. [14], which was used to validate the results exported from FLAC 2D. The soil shaking table was 600 x 800 mm with a maximum acceleration of ± 3 g at frequencies from 0 to 100 Hz. The laminar soil box was constructed from 12 square steel laminate. Figure 4. shows the model setup and sensor locations.
Figure 4: Model soil shaking table set up with a sensor apparatus

3. Numerical test Analysis

3.1 Excess pore water pressure

Liquefaction is defined in terms of excess pore pressure (Δu) and reduced initial vertical stress in the soil. The possible occurrence of liquefaction is assessed using the pore pressure ratio (ru), which is the difference between the current and hydrostatic pore pressures and the initial effective vertical stress (σv₀), as stated in equation 6:

$$ru = \frac{\Delta u}{\sigma v_0}$$  (6)

Figures 5 and 6 show the pore pressure ratio in the soil profile at the near and far-fields of the pile for 3 and 7.5 m depths in test cases K1 and A1, respectively. For test case K1, the liquefaction of soil decreased in the near field as compared with that in the far-field of the pile for different depths of saturated sandy soil, which may be attributed to the loss of shear deformation in the reinforcement area of the pile foundation. These results confirm the findings of Assadi and Sharifipour [20]. Furthermore, the magnitude of ru becomes greater than 1, and the mean effective vertical stress approaches zero, which means that the excess pore pressure equals the initial effective vertical stress; thus, the soil is liquefiable. The value of the pore pressure ratio in test case A1 reached about 0.8 and 0.9 for the near and far-fields at a depth of 3 m, with values of 0.62 to 0.7 for a depth of 7.5 m, which means that the sandy soil is already partially liquefied. Comparing the results with ones obtained from the shaking table for K1 and A1, as shown in table 5, good agreement is seen in the general trend, with only slight differences in overall values.
Table 5: Comparison between numerical and experimental results for pore pressure ratio (ru, %)

| Earthquake type | Ali Algharbi | Kobe |
|-----------------|--------------|------|
|                 | Experimental | Numerical | Experimental | Numerical |
| Pore pressure ratio at 3m | 0.91 | 0.82 | 1.56 | 1.20 |
| Pore pressure ratio at 7.5m | 0.80 | 0.65 | 1.77 | 1.51 |

Figure 5: Development of excess pore water pressure ratio, ru% for test case K1

Figure 6: Development of excess pore water pressure ratio, ru% for test case A1

3.2 Lateral Displacement
Loss of soil stiffness and strength due to liquefaction during earthquake events cause the pile foundations to experience extensive deformation; thus, liquefiable soil significantly increases the lateral displacement of the pile head. Figures 7 and 8 show the effects of lateral load variation on horizontal displacement at different depths from pile head to tip for both earthquakes investigated. The maximum displacement occurred at the pile head and this decreased linearly towards the pile tip in the soil due to the natural interactions between the pile and soil and the excess pore water pressure generation in loosely saturated soil. The maximum lateral displacement of the pile head was about 60 mm for test case K1 and 12 mm for test case A1. Finally, the inclusion of lateral load on the pile head significantly reduced the lateral displacement on the pile head, by 50% for K2 and A2 and 80% for K3 and A3 in comparison.
with test cases K1 and A1, respectively. The variation of the lateral loading along the pile shaft had little effect on lateral displacement reduction.

**Figure 7:** Time history of lateral displacement along the pile for Kobe earthquake

**Figure 8:** Time history of lateral displacement along the pile for Ali Algharbi earthquake.
3.3 Vertical displacement

Figures 9 and 10 illustrate the time history of vertical displacement on the pile head and soil surface. Vertical pile displacement reached 20 and 1.6 mm at the end of earthquake motion with combined loading for the Kobe and Ali Algharbi earthquakes, respectively. As compared to the soil surface, the vertical soil displacement was less than the pile head, reaching values of 5 and 1 mm at the end of the Kobe and Ali Algharbi earthquakes under the same conditions. The vertical displacement of the soil surface was smaller than the pile head due to friction behaviours induced by the pile. These results suggest that lateral loading variations have no significant effect on either pile or soil vertical displacement, even at higher earthquake magnitudes.

![Figure 9: Time history of vertical displacement of pile and soil surface for Kobe earthquake](image-url)
Figure 10: Time history of vertical displacement of pile and soil surface for Ali Algharbi earthquake

4. Conclusion

In this study, a numerical analysis was carried out using FLAC 2D to investigate the effect of combined loading on a single pile in liquefiable soil under different earthquake motions. Vertical displacement at the pile head and soil surface and horizontal displacement along with pile depth and pore water pressure ratios at near and far-field were compared with a soil shaking table test. The results indicated that

- In the Kobe earthquake, the magnitude of ru became greater than 1, and the mean effective vertical stress approach zero, indicating that the excess pore pressure equalled the initial effective vertical stress; thus, the state of the soil was liquefiable. In the Ali Algharbi earthquake, the pore pressure ratio reached about 0.8 and 0.9 for the near and far-fields at a depth of 3 m and 0.62 to 0.7 for a depth of 7.5 m, suggesting that the sandy soil was partially liquefied.
- Comparison between the numerical and experimental work showed good agreement in the general trend, with slight differences in overall values for pore pressure ratio for K1 and A1.
- The inclusion of lateral load on the pile head significantly reduces pile lateral displacement behaviours, by 50% for K2 and A2 and 80% for K3 and A3. At the same time, variation of the lateral loading along the pile shaft has little effect on lateral displacement reduction.
- The lateral loading variations have no substantial effect on either pile or soil vertical displacement, even at higher earthquake magnitudes.
- The study, in particular the test results imported from the numerical model, presents a reliable basis for liquefaction analysis in more complex cases.

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