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Seismic bearing capacity of shallow footings on cement-improved soils

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Abstract. A single rigid footing constructed on sandy-clay soil was modeled and analyzed using FLAC software under static conditions and vertical ground motion using three accelerograms. Dynamic analysis was repeated by changing the elastic and plastic parameters of the soil by changing the percentage of cement grouting (2, 4 and 6 %). The load-settlement curves were plotted and their bearing capacities compared under different conditions. Vertical settlement contours and time histories of settlement were plotted and analyzed for treated and untreated soil for the different percentages of cement. The results demonstrate that adding 2, 4 and 6 % of cement under specific conditions increased the dynamic bearing capacity 2.7, 4.2 and 7.0 times, respectively.

Keywords: vertical ground motion; cement-improved soils; dynamic vertical settlement; shallow footing; normal stress

1. Introduction

Estimating earthquake motion at the site of a structure is the most important phase of seismic design and retrofitting. Classical methods for structural analysis assume that motion in the foundation level of structure is equal to ground free field motion. However, this assumption is correct only for structures resting on rock or very stiff soils. Foundation motion is usually different from free field motion than for structures constructed on soft soils. In addition, a rocking component caused by support flexibility on the horizontal motion of the foundation must be accounted for.

Traditionally, in the analysis of rigid base structures, input motion at the base of the structure is assumed to be equal to the free field ground motion. For a flexible-base structure, in addition to the added rocking component, part of the structure’s vibrating energy will transmit to the soil layer and can dissipate via radiation damping resulting from wave propagation and hysteresis damping of the soil. However, in classical methods, for rigid-base structures, this energy dissipation is not

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considered.

Previous studies have sought to determine the dynamic behavior of shallow footings to provide quick, applicable data for structure design. Many researchers have investigated the dynamic behavior of footings under the horizontal component of earthquakes. The current research studied the dynamic behavior of shallow footings located on treated or untreated soils under the vertical component of ground motion. Experimental techniques were employed to formulate equations and charts for the numerical modeling of soil improved by grouting.

1.1 Bearing capacity

Early investigations on the settlement and bearing capacity of soil under footings include those of Terzaghi (1943), Meyerhof (1951), Caquot and Kerisel (1953), Vesic (1973), Chen (1975). The behavior of earthquakes has traditionally been a geotechnical topic and seismic concerns about soil were less a focus than were static conditions. If a structure is subjected to an earthquake, the applied load on its footing comprises cyclic vertical loads, cyclic horizontal loads and cyclic moments about one or more axes. Earthquakes also cause inertia forces within the mass of soil under the footing that affect settlement of the footing.

Richards *et al.* (1993) indicated that inertia forces within soil mass subjected to the horizontal component of an earthquake decreased the available strength of the soil and, thus, the bearing capacity of the shallow footing. Also, the transition of shear in the interface of the soil structure decreased bearing capacity. Dormieux and Pecker (1995) studied the failure surface of soil under a footing and indicated that the decrease in bearing capacity was mainly due to the inclined load and that inertia forces caused by soil mass had a minor effect on the bearing capacity.

Budhu and Al-Kerni (1993), Kumar and Mohan Rao (2002), Soubra (1994) and Sarma and Isossifelis (1990) studied the characteristics of failure surfaces under foundations. They calculated the ultimate load of the foundations and considered reduction ratios for the coefficients of the bearing capacities. They also proposed some charts to determine Nc, Nq and $N\gamma$ coefficients as a bearing capacity factors which are only functions of the soil friction angle. In this regard, Nc is cohesion factor, Nq is surcharge factor and $N\gamma$ is self weight factor. Tatsuoka *et al.* (1989) also modeled the reduction in bearing capacity from an earthquake by decreasing the internal friction angle of the soil using $i=\tan^{-1}k_h$.

Research on the effects of earthquakes have traditionally addressed the horizontal load applied to the structure of a footing as part of a surcharge and the inertia caused in the underlying soil of that footing was disregarded (e.g., Meyerhof 1951, Sokolovski 1960). Indeed, such research can be considered the result of the behavior of inclined loads (e.g., Singh *et al.* 2007). Sarma and Isossifelis (1990), for example, studied the influence of the lateral component of an earthquake on the footing and surcharge and on parts of the failure area of foundation soil. Extensive calculations that incorporated the effect of the vertical component of an earthquake have also been formulated by Kumar and Rao (2002).

In recent years, the significance of the vertical component on accelerograms has been recognized and investigated, including work by Shafiee and Jahanandish (2010) and Varadhi and Saxena (1980). They constructed a circular footing on sandy soil and studied the vertical transition of the load under the footing. Vertical stresses at various depths within the soil were compared under static and dynamic conditions with and without a footing. These researchers concluded:

• Maximum dynamic vertical stress under a footing was ten times greater than vertical stress under static conditions.

• The influence of the mass of a footing on vertical stress distribution was very small at depths of twice the diameter of the footing (2*B*).

The internal friction angle of uncompacted saturated sands during an earthquake, based on the New Zealand Code (1996), can be obtained from Eqs. (1) and (2)

$$\varphi_d = \tan^{-1}[(1 - C\frac{a_{max}}{g})\tan\varphi] \tag{1}$$

where

$$C = \frac{D}{-3B} + \frac{4}{3} \tag{2}$$

In these equations, a_{max} is the maximum acceleration of an earthquake on the surface of the earth, g is the ground acceleration, φ is the internal friction angle of soil under static conditions, B is the width of the footing and D is the distance between the base of the footing to the groundwater level ($D \leq 2B$). If $D > 2B$, the generation of pore water pressure during an earthquake can be disregarded.

Another method suggested by the New Zealand Code (1996) is the use of static equations to calculate bearing capacity where a factor of safety other than the static condition is used. The code recommends that the static bearing capacity of the consolidation clays increase about 33% during the earthquake.

The same equations applied to determine bearing capacity under static conditions can be employed conservatively for dense, saturated sands and unconsolidated clays. According to the India Code (2005), bearing capacity increases about 50% for well-graded sand and gravel and for clayey sands using standard penetration test (SPT) ratings of greater than 30. Furthermore, for poorly-graded sands and soft soils, the bearing capacity increases about 25%. It was noted that, in loose saturated sands and sensitive saturated clays, bearing capacity during an earthquake drops to about zero.

1.2 Grouting

Grouting has been in use for the last 200 years. The inventor of this technique was French engineer Charles Berigny who injected clay and lime grout into a masonry wall. Grouting materials were traditionally clay, pozolan and hydraulic lime with the addition of Portland cement later on.

In England, Kinniple first used grouting in gravel (Nonveiller 1989). Collin (1841) and Beaudemoulin (1839) in France and Varten in the United States (Nonveiller 1989) were among the first researchers to study grouting. Lugeon and Lufrane also Dayakar *et al.* (2012) determined the permeability of rocks and soils (Lacaster-Jones 1975). These researchers and Hvorslev (1951) have done significant research in the field of permeability.

Currently, grouting is employed to increase the bearing capacity of foundations (Khan *et al.* 2013), to construct cut-off curtains, for groundwater control, the stabilization of slopes and more (Zahiri and Majidi 2006). The history of grouting is shown in Table 1.

In the present research, a numerical model for footings was simulated using FLAC software to study settlement-load curves, displacement contours and time-histories of displacement plots under different grouting conditions and vertical components of earthquakes.

Table 1 History of grouting as a technique of improvement

Researcher	Year	Country	Type of grouting
Charles Britney	1802	French	Preventing the corrosion between soil and footing
Kalin	1838	French	Injection for improvement of bridge foundation
Worthen	1845	USA	Application of impact pump for improvement of bridge pier
Kinniple	1856	British	Injection of clay and cement grout to seal the foundation of dam on the Nile river
Barlow	1864	British	Filling the distance between cover of tunnel and its wall
-----	By 1904	Europe and America	Extensive application of injection for stabilization of underground walls and protection of them against underground waters and leakage of water
Fransois	1902	USA	Following results have been obtained from laboratory testing of injection: - The more the number of boreholes and the less their depth, the more optimum the operation of injection. - The injecting pressures up to about 50 to 200 atmosphere have more efficiency than the low pressures of 15 to 20 atm.
Joosten	1925	Germany	Chemical grout (Sodium silicate) and injection in the cavity in order to improve the soil foundations

2. Numerical model

The dimensions and geometry of the current model were chosen to eliminate the possibility of a boundary effect on the results of the analyses. The soil model extended to the bedrock and the earthquake acceleration model was applied to bedrock soil. To investigate the accuracy of model predictions, model dimensions, including soil mass, were set at 30×30 m. A footing with a width of 2 m and height of 50 cm was placed in the middle of the upper boundary of the model. Mesh was generated as a rectangle of 0.5×0.5 m adjacent to the footing and 0.5×1 m and 1×1 m in areas far from the footing (Massumi *et al.* 2008, Kholdebarin *et al.* 2008).

Elastic behavior for the footing and the Mohr-Coulomb failure criterion were considered valid to simulate the stress-strain behavior of the soil. The Mohr-Coulomb model is a perfect elastoplastic model that only dilates at failure and not at densification during cyclic loading at stress below failure. Thus, using FLAC software, the Mohr-Coulomb model was modified to model the changes of permanent volumetric strains in drained cyclic loading or pore pressure in undrained cyclic loading. The simplicity of the model was one reason for its selection. The main parameters are C and φ, which were determined using conventional soil mechanics tests.

In this model, the soil parameters were density (ρ), bulk modulus (K), shear modulus (G), cohesion (C), internal friction angle (φ) and dilation angle (ψ). Sandy-clay soil was used for the soil beneath the footing. Since the values of shear modulus and bulk modulus were considered inputs of the program, they were calculated as (Itasca Consulting Group 2000)

$$K = \frac{E}{3(1-2\nu)} \tag{3}$$

$$G = \frac{E}{2(1+\nu)} \tag{4}$$

where E and υ are the elasticity modulus and the Poisson's ratio, respectively.

The footing model was elastic and behaved like a moment-resisting element. The parameters of elastic modulus, density, area and moment of inertia were assigned to it. A boundary element or common interface was employed to consider the discontinuity boundary in the space between the soil and footing. This element has a thickness of zero and was connected to the beam element on one side and the soil mesh on the other. Normal stiffness and shear stiffness of this element were ten times greater than the equivalent stiffness of smallest zone of the adjacent mesh, calculated as (Itasca Consulting Group 2000)

$$K_n \approx K_s = 10\max\left(\frac{K + \frac{4}{3}G}{\Delta z_m}\right) \tag{5}$$

The interface model was a perfect elastic structure and plastic soil (Mohr-Coulomb). The parameters for the interface were cohesion (C), friction angle (φ), dilation angle (ψ), normal stiffness (K_n), shear stiffness (K_s) and tensile strength (T).

History of acceleration is a loading option for FLAC software. The vertical components of three accelerograms were employed to analyze the model under vertical acceleration. The Maku earthquake with a magnitude of 6.2 (Richter) and a maximum acceleration of 0.051 g in the bedrock, the Shalamzar earthquake with a magnitude of 5.0 (Richter) and maximum acceleration of 0.266 g and the Zanjiran earthquake with a magnitude of 5.9 (Richter) and a maximum acceleration of 0.983 g in the bedrock were used in the dynamic analyses. General specifications of earthquakes including the magnitude and Peak Ground Acceleration of selected records are

Table 2 General specifications of earthquakes

Name of earthquake station	Record No.	Date	Corrected Peak Ground			Magnitude		ED* (km)	Database
			Acc.	Vel.	Disp.	mb	Ms		
Maku	BHRC** 1046-1	11/24/1976	L 86 V 42 T 63	5.15 1.35 2.63	0.26 0.07 0.16	6.2	7.3	53	NEIC***
Shalamzar	BHRC 1226-2	06/01/1984	L 299 V 245 T 227	10.01 5.52 7.34	0.47 0.84 0.99	5.0	4.2	31	NEIC
Zanjiran	BHRC 1502-9	06/20/1994	L 841 V 576 T 1018	32.11 14.32 39.83	3.84 1.61 2.76	5.9	5.7	12	NEIC

*ED: Epicentral Distance
**Building and Housing Research Center
***National Earthquake Information Center

shown in Table 2. The time history of acceleration curve for these records and their power amplitudes are illustrated in Fig. 1. This figure illustrates that the predominant frequencies in the first two earthquakes are located between 0-10 Hz, whereas this frequency range in the third record is located between 10-20 Hz. The last 10 sec of the strong motion of the seismic accelerograms were applied for modeling (Azzam 2014).

Static and dynamic soil parameters were calculated from the geotechnical reports of the Iran-Khak Consulting Engineers Co. from the Chabahar gas power plant and are shown in Tables 3 and 4 (Irankhak Consulting Engineers 2008). To determine the geotechnical characteristics of the soil underlying the site, 14 boreholes were drilled by wash-boring and 2 boreholes were drilled by continuous coring. Standard plate loading tests were performed at 10 points at depths of 0.5, 2.0 and 4.0 m. The soil stratum was obtained from results of static plate load tests. Geo-electric tests and geophysical seismic surveys were conducted in the field. Direct shear, triaxial, and consolidation tests were performed on samples in the laboratory to determine the physical, chemical and mechanical properties of the soil underlying the site.

Parameters for a soil for which the Mohr-Coulomb constitutive model is valid are the internal friction angle (φ), cohesion (C), tensile strength (T) and dilation angle (d_i). Rayleigh damping was used for dynamic modeling assuming the coefficient of damping as D=0.05 and the central frequency as f=1.6 Hz. For modeling the foundation, moment-resisting beam elements where the nodes were connected to the calculative mesh were used. After analyzing the model, the influence of different parameters on the bearing capacity of shallow footing was studied (Massumi *et al.* 2008).

3. Results of numerical modeling

The static and dynamic behavior of improved soil varies after grouting with changes in the percentage of cement and the ratio of water to cement. In the present research, the impact of these variations on the static and dynamic soil parameters will be considered by using some charts. These charts have been introduced based on basic assumptions in the experimental models (Rosquoet *et al.* 2003, Ro 1994).

The grouting was done by penetration so that the overall structure of the soil was not altered and the grains did not experience relative displacement. The average confining pressure of the soil was σ^3=115 kPa, the ratio of water to cement was 2 and the injection pressure was 50 kPa. Static analysis results showed the maximum axial strain to be 10^{-2}%. The change in volume of the cemented soil with respect to normal soil (Δv) was zero. Since the soil strains were very small, the shear strength of soil was calculated with dominant cohesion and a small contribution from the angle of friction (Dano *et al.* 2004). The Poisson ratio for normal soil was about 0.37. After improvement, the Poisson ratio remained unchanged (Dano *et al.* 2004).

The dilation angle was assumed to be zero for the different percentages of cement (Ro 1994).

Table 3 Elastic parameters of footing

Area	Density	Elasticity modulus	Moment of inertia
A (m^2)	ρ (kg/m^3)	E (kPa)	I (m^4)
1.00	2500	2.5×10^7	0.0208

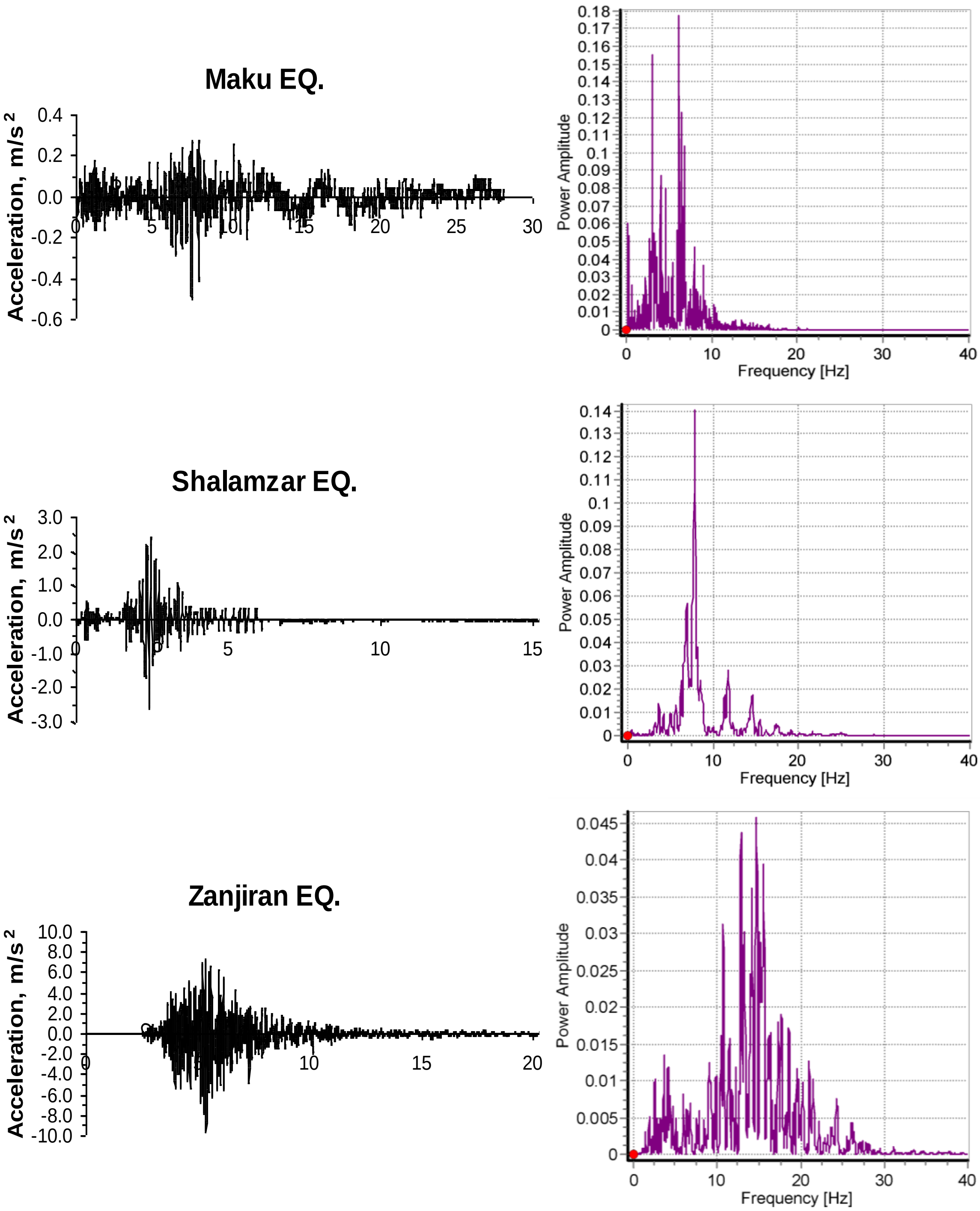


Fig. 1 Accelerograms and Fourier spectra of vertical components of earthquakes used in time-history analyses

Table 4 Plastic parameters of soil in static and dynamic conditions

Condition	K (kPa)	ρ (kg/m^3)	G (kPa)	C (kPa)	φ (degree)	T (kPa)	*di* (ψ) (degree)
Static	1.26e4	1830	3.58e3	4.90	20	0	0
Dynamic	1.60e5	1830	4.68e4	7.35	18	0	0

Table 5 Comparison of the parameters for normal and improved soil with different percentages of cement

No.	Percentage of cement (C%)	Water / Cement ratio	Shear modulus (kPa)	Bulk modulus (kPa)	Damping percentage (%)	Dominant frequency (Hz)	Cohesion (kPa)	Velocity of shear wave (m/s)
1	0.0	0.0	4.68e4	1.60e5	5.0	1.60	7.35	192.0
2	2.0	2.0	9.36e4	3.28e5	5.6	1.88	30.64	226.2
3	4.0	2.0	1.33e5	4.67e5	6.4	2.25	55.34	269.6
4	6.0	2.0	2.09e5	7.34e5	12.1	2.82	102.50	337.9

In penetrating grouting, the change in the density of treated soil compared with normal soil was very small (Rosquoet *et al.* 2003). Variation in cohesion for different percentages of cement and the water/cement ratio (2) is shown in Table 5. After improvement by grouting, the dynamic parameters of damping percentage (D), shear modulus (G) and, velocity of shear wave and bulk modulus changed (Itasca Consulting Group 2000).

The model was used to investigate the impact of an increase in cement grouting on the increase in bearing capacity of the soil (Kumar *et al.* 2013). The results showed that, by increasing the cement grout 2%, bearing capacity increased 2.66 for the Maku earthquake, 2.65 for the Shalamzar earthquake and 2.74 for the Zanjiran earthquake. The increase for a soil with 4% cement increase for these earthquakes was 4.2, 4.2 and 4.75 times, respectively. For a soil with a 6% cement increase, it was 7, 6.9 and 7.16 times, respectively (Fig. 2). The dynamic load-settlement curves of this figure indicate that the graphs gradually become horizontal after 100 to 200 mm settlements. This range of settlements is almost equal to 0.1B, which is proportional to the well known maximum dynamic bearing capacity of the foundations (Das and Ramana 2011).

Variations in normal stress versus depth in dynamic analysis (Fig. 3) demonstrate that the presence of a footing increased the maximum normal stresses eightfold compared with the static condition (Bousinesque equation). To test this, a linear 150 kPa surcharge was applied to a footing with a width of 2 m. The average vertical stress under the footing in a dynamic condition was 5.4 e5Pa for the accelerograms of the three earthquakes. As depth increased, the footing gradually lost its effectiveness to the extent that, at 4B (B=depth of footing), the concrete footing had no effect on the normal stresses in the soil.

To test the influence of increased stress on the dynamic settlement of a shallow footing, a constant surcharge was applied to compare settlement contours. Fig. 4 shows the settlement contours for dynamic analysis at a surcharge of 200 kPa. Maximum settlement (14 cm) occurred under the footing. At a point far from the center of the footing, semi-circle contours showed smaller settlements. At a depth of about 5B, settlement decreased to 2 cm.

Fig. 4 also illustrates the time-history for soil treated with 2, 4 and 6% cement, where the maximum stress under the footing under a constant surcharge decreased to 7, 6 and 3.5 mm, respectively. Also, increasing the percentage of cement caused maximum displacement under the footing to decrease.

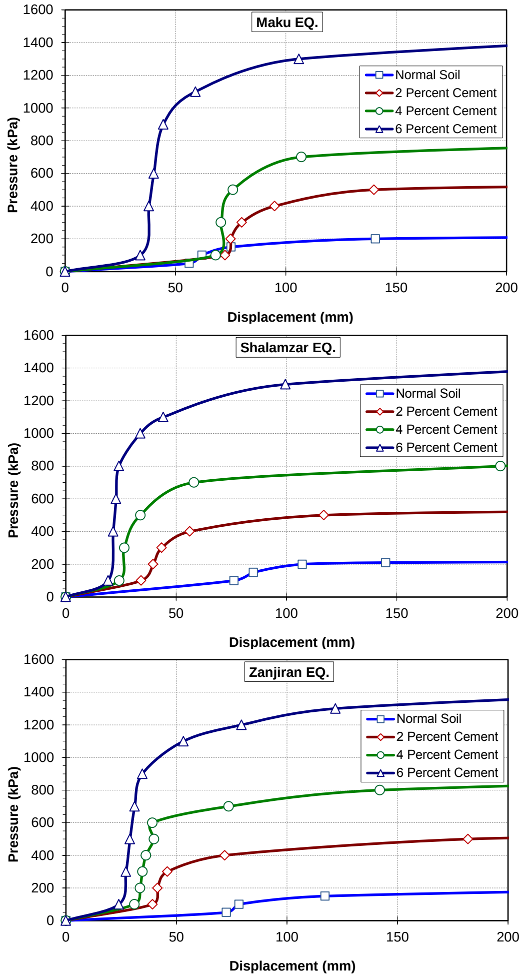


Fig. 2 Comparison of the settlement-pressure curves for normal and cemented soil with different percentages of cement (Maku, Shalamzar and Zanjiran earthquakes)

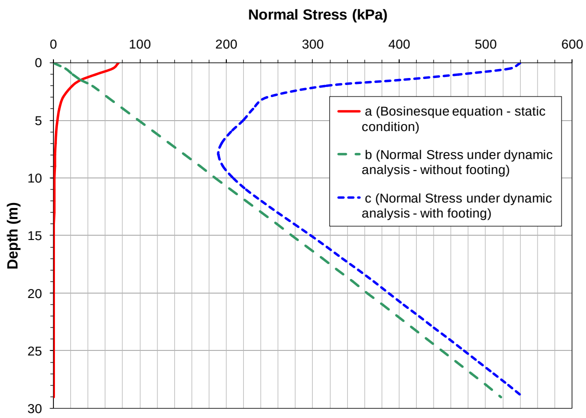
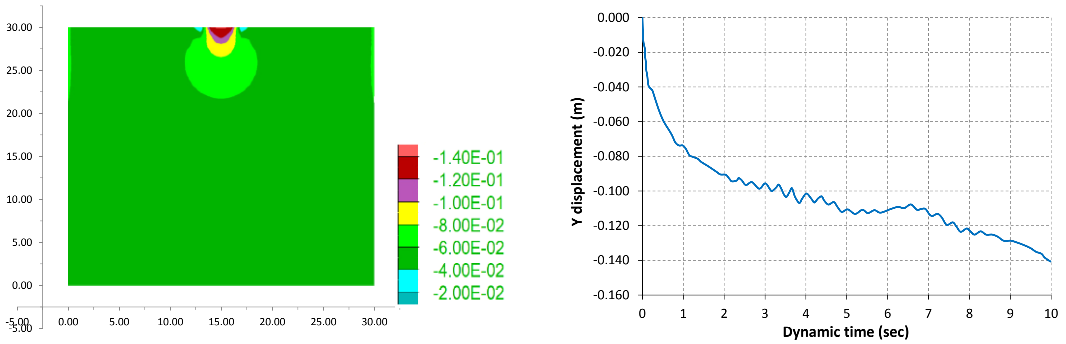
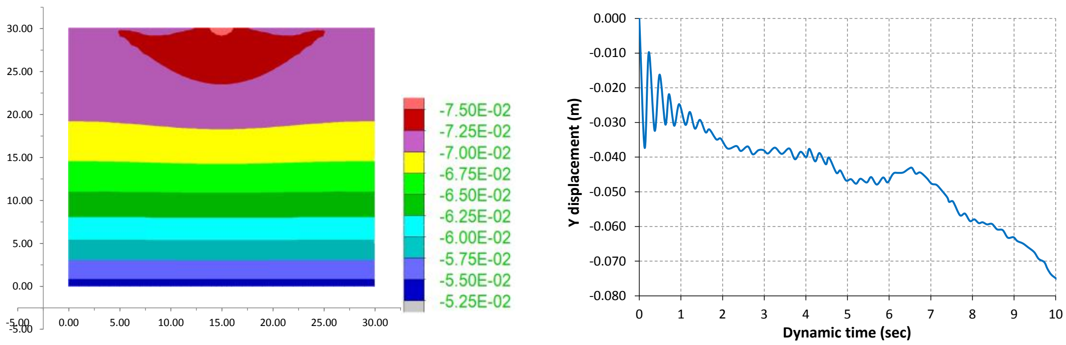


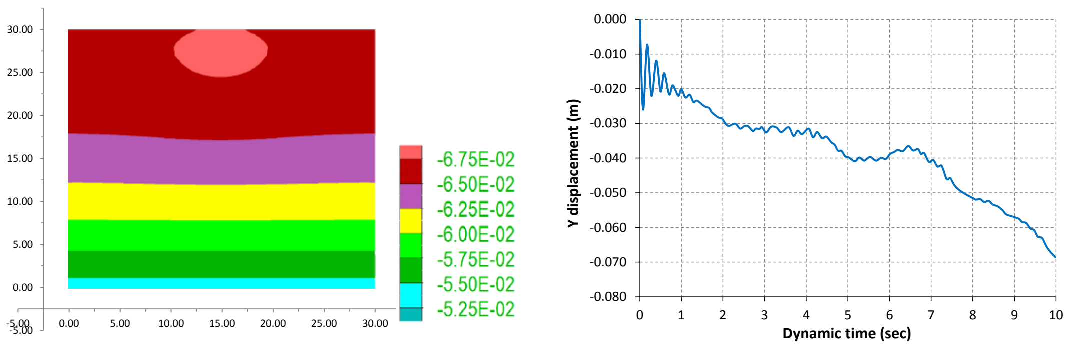
Fig. 3 Comparison between normal stress under footing in static and dynamic conditions vs. depth



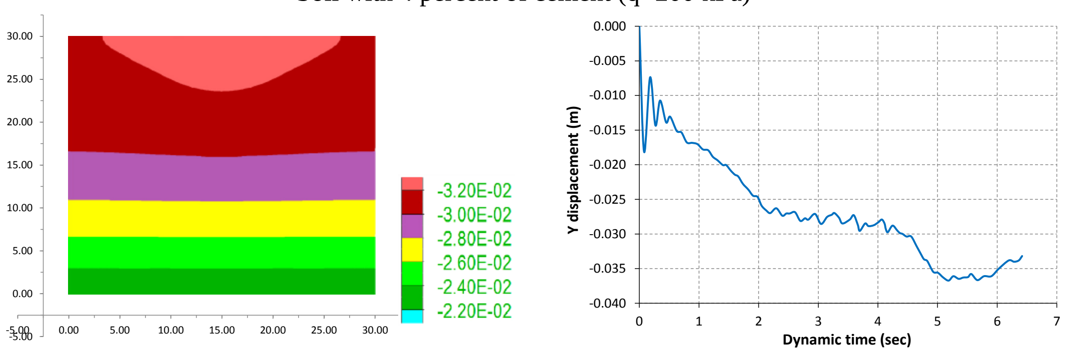
Normal soil (q=200 kPa)



Soil with 2 percent of cement (q=200 kPa)



Soil with 4 percent of cement (q=200 kPa)



Soil with 6 percent of cement (q=200 kPa)

Fig. 4 Displacement contours and time-history of displacement in the Y direction

4. Conclusions

Adding cement grout reduced the maximum normal stress and caused a decrease in settlement under a footing. It also caused the soil to reach failure for greater surcharges and its bearing capacity increased. The maximum normal stress under a footing under dynamic conditions was eight times greater than for static conditions.

• The shear wave velocity in sandy soil with 2% cement increased 1.18 times that for the same values in normal sand. The velocity for sandy-clay soils with 4% and 6% cement also increased 1.4 and 1.76 times, respectively.

• Adding 2% cement on average increased the dynamic bearing capacity 2.7 times compared with the bearing capacity of normal sand. The increase for sands with 4% and 6% cement were 4.2 and 7.0, respectively.

• Settlement distribution under the footing showed greater dispersion as the percentage of cement increased.

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