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Behavior of dry and saturated soils under impact load during dynamic compaction

Le comportement de sols secs et saturés sous le chargement d'impact pendant le compactage dynamique

A. Pak, H. Shahir & A. Ghassemi
Department of Civil Engineering, Sharif University of Technology, Iran

ABSTRACT
Dynamic compaction is a widely used soil improvement method in dry and/or saturated soils. Despite its vast application, its design basis is still empirical and the mechanisms that are involved in the procedure are not fully understood. A fully coupled dynamic finite element code has been developed in order to clarify the ambiguities in the process and predict the strain/displacement field in the ground, determine depth and degree of improvement, and also calculate the pore pressure variation during the process. This model can be used as a rational design tool for dynamic compaction projects.

RÉSUMÉ
Le compactage dynamique est une méthode d'amélioration de sol largement utilisée dans et/ou sec les sols saturés. Malgré son application vaste, sa base de conception est toujours empirique et les mécanismes qui sont impliqués dans la procédure ne sont pas entièrement compris. Un code d'élément fini, dynamique et entièrement couplé a été développé afin de clarifier les ambiguités dans le procédé et prédit le champ de tension/déplacement dans le sol, déterminer la profondeur et le degré d'amélioration, et aussi calculer la variation de pression de pore pendant le procédé. Ce modèle peut être utilisé comme un outil de conception rationnel pour les projets de compactage dynamiques.

1 INTRODUCTION

Dynamic compaction (DC) is a soil improvement method that is routinely used worldwide. The method involves the repeated application of high energy impacts on the soil surface using tampers commonly weighing 100-250 KN, dropped from heights of 10-30m using heavy crawler cranes, compacting the soil strata to considerable depths. The tampers are dropped repeatedly at each impact point on a predetermined pattern over the treatment area.

In practice, DC design is based on empirical formulas and past experiences are enhanced by a series of trial field compactions to make sure about the effectiveness of the DC pattern (grid spacing and impact energy).

In recent years, several analytical studies have been carried out to understand dynamic compaction mechanism and also to provide a practical design method. The majority of the proposed analytical models are one-dimensional. Scott and Pearce (1975) explained the main features of the falling weight impact on the surface of a soil column using mechanical principles established by Kolsky (1963), and Lysmer and Richart (1966). Employing this model, they discussed the behavior of some specific types of soils under impact. In order to study the deceleration of the falling tamper, Chow et al. (1990) utilized the approach originally proposed by Lee et al. (1988) to analyze the forces induced during pile driving. In the subsequent studies, they could predict the depth and degree of improvement (Chow et al. (1992)).

1-D models are not able to determine the lateral improvement extension in the ground. The other main problem with these models is that they can not take the groundwater effects into account directly. However, Ganaratne et al. (1996) introduced a 1-D model that could calculate the pore pressure generation and dissipation in a saturated soil column under impact. Based on their experimental studies, the authors showed that the vertical and horizontal distribution of stresses below the impact center is in agreement with the elastic theory.

Despite the great advances that have occurred in the numerical procedures and computer technology in recent decades, only a few two-dimensional models have been introduced in dynamic compaction literature. Poran and Rodriguez (1992) modeled dynamic compaction in dry sand deposits using finite element codes DYNA2D and IMPACT. They analyzed impact effects assuming large deformation formulation and two different elasto-plastic models. Pan and Selby (2002) used ABAQUS to numerically analyze the soil response to rigid body impacts of an axisymmetric elasto-plastic F.E. representation of the dry soils. They divided the FE mesh into three separate zones in depth with different stiffness in order to simulate the densification of soil deposit during successive drops. Based on valuable findings of Poran and Rodriguez (1992), Gu and Lee (2002) described the dry sand behavior under impact loads utilizing the FE program CRISDYN. They employed a cap model to simulate the sandy soil behavior under impact. However, their numerical model was unable to consider dynamic consolidation in saturated soils, but their results are very useful to find out the mechanisms that are involved in a DC process.

2 BEHAVIOR OF SOIL UNDER IMPACT LOAD

Dynamic compaction has been applied to dry, moist, and saturated soil layers. For saturated soils this technique is useful if soil is free draining. The reason is contribution of pore water in load carrying mechanism of the two-phase system of saturated soil due to very low compressibility of water comparing with the soil skeleton.

Van Impe et al. (1993) have confined the applicability of compaction improvement methods to silty, sandy, and gravelly soils due to their high coefficient of permeability. For impervious deposits such as clayey soils with a plasticity index greater than 8, dynamic compaction is not recommended at high degree of saturation. High pore water pressure that is generated in the impervious saturated soils diminishes the effectiveness of the energy of impact and the outcome of the compaction procedure would not be desirable.

For dry soils, DC involves two different mechanisms: displacement of soil particles in the vicinity of the impact, and deformation due to the propagation of stress waves.

3 NUMERICAL SIMULATION OF DC

For numerical simulation of soil behavior under impact loads, general class of formulation which governs the physical phenomenon of wave propagation through dry and saturated soils is considered. Densification behavior of soil due to the body waves induced by impact is modeled using suitable constitutive laws.

3.1 *Governing equations*

In a two phase saturated system, pore water pressure and deformation of solid particles in the soil are inter-related. For a fully coupled analysis of this system, equilibrium or momentum balance for the soil-fluid mixture, momentum balance for the fluid phase and finally mass balance for the system must be satisfied (Lewis and Schrefler, 1998). The spatially discretized form of these equations in the simplified U-P form is as follows (Zienkiewicz et al., 1999):

$$M\ddot{U} + C\dot{U} + \int_V B^T \sigma' \, dV - QP - f^{(1)} = 0 \quad (1)$$

$$Q^T \dot{U} + HP + S\dot{P} - f^{(2)} = 0 \quad (2)$$

Where M is the mass matrix, C the viscous damping matrix, U the solid displacement vector, B the strain-displacement matrix, σ' the effective stress tensor (determined by soil constitutive model which will be discussed later), Q the discrete gradient operator coupling the motion and flow equations, P the pore pressure vector, S the compressibility matrix, and H the permeability matrix. The vectors $f^{(1)}$ and $f^{(2)}$ include the effect of body forces and prescribed traction, and fluid flux, respectively.

By solving the above system of equations, soil deformation and generated pore pressure can be determined at any desired point in the soil mass. The generated pore pressure in the saturated soil mass and the induced settlements are inter-related through the term Q in (1) and (2).

When impact loads are applied to dry soils, no pore pressure is generated. For numerical modeling of the process in this case, the terms containing P are eliminated and the above two equations are reduced to the familiar equation of equilibrium in dynamic form.

3.2 *Finite element program "PISA"*

There are several requirements that have to be fulfilled in a numerical model for simulating DC in dry and/or saturated soils. Namely, incorporation of dynamic effects, contact between tamper and soil surface, stress-wave propagation through the soil body and interaction between soil skeleton and fluid phase. Available commercial geotechnical softwares usually do not include one or more characteristics mentioned above. So, in this study a developed finite element program PISA was used for analytical purposes.

Chan and Morgenstern (1988) developed the original version of this multi-purpose geotechnical code. The subsequent versions of this program provided more possibilities for analyzing a wide variety of geotechnical problems. Pak (1997) increased the program capabilities by amending the formulation for analyzing thermal hydro-mechanical (THM) problems. Shahir (2001) added the dynamic analysis ability to the program and used PISA to model liquefaction phenomenon in loose saturated sand deposits. Ghassemi (2004) developed two special cap models in the program and investigated the effects of different constitutive laws in numerical modeling of dynamic compaction.

3.3 *Finite element mesh*

The equations (1) and (2) can be solved in two or three dimensions, however, since the geometry and loading configuration are symmetric around the axis of falling tamper (load centerline), a two-dimensional axisymmetric simulation usually yields satisfactory results (Fig 1).

Discretized domain should be chosen large enough that reflection of the stress waves from the boundaries is limited. The elements size especially at the vicinity of the impact should be small to show the intense stress and deformation gradients. The elements size was determined based on the method proposed by Zerwer et al. (2002).

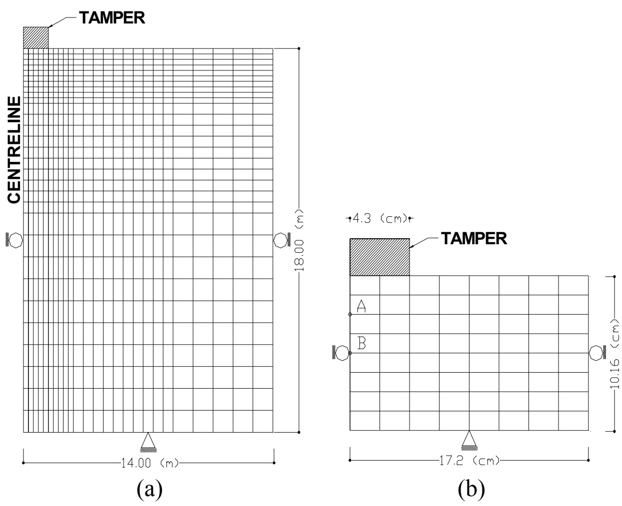
3.4 *Modeling of impact*

To simulate the stress waves propagating through the soil deposit during dynamic compaction, the impact of tamper on the ground surface should be accurately modeled. The most accurate procedure is the contact formulation between two or more moving bodies. Here, for the sake of simplicity the rigid body method is used for modeling the impact, i.e. the input of the program is the initial velocities of tamper nodes that can be determined from free fall equation.

After contact the acceleration decreases rapidly until the time that tamper stops and then starts to come up. This procedure causes the acceleration to change sign; consequently the tamper elements pull the soil elements and they undergo high tensile stresses. To avoid this unreal tension in soil column, the tamper elements should be eliminated from analytical procedure after the acceleration reduces to zero.

3.5 *Dynamic analysis parameters*

The time increment must be carefully chosen to maintain numerical stability and accuracy. It may cause the solution to diverge if time increment is too large. Conversely, a very short time increment can cause spurious oscillations and also



(a) (b)

Figure 1. Finite element meshes used in this study

more cost. The time increment in all analyses of this study was considered 0.5 msec.

Another important factor influencing the accuracy of finite element modeling is material damping. In PISA, the viscous damping for soil material is applied using the Rayleigh damping equation as follows:

$$[C]=\alpha[M]+\beta[K] \tag{3}$$

Where α and β are constants multiplied by mass and stiffness matrices, respectively. α=0 and β=0.01 have been used for this study based on the suggestions made by Gu and Lee (2002) and Rix et al. (2000).

4 CONSTITUTIVE LAWS

In this study two plastic cap models have been used. The first one is the Modified Cam-Clay model. Following the critical state concept, this model is a certain type of elasto-plastic model in which the isotropic hardening is a function of plastic volumetric strain. Success of this constitutive law in modeling soil behavior (especially cohesive soils) has been proved during recent years. In this model, the yield locus takes the following form (Desai and Siriwardane (1984)):

$$f = M^2 p^2 - 2M^2 p p_0 + q^2 = 0 \tag{4}$$

In this equation, p_0 is half of the ellipse bigger diameter and M is the slope of the critical state line in p-q space.

The second model chosen for modeling granular soils is one of the cap model series introduced by Dimaggio and Sandler (1971). In this model, the fixed surface is assumed to be composed of an initial portion of the Drucker-Prager envelope joined smoothly to the subsequent Von-Mises surface (Fig. 2). The logic for adopting the Von-Mises surface at higher stresses is based on the observation that at higher stresses the soil behaves like a metal. The expressions for shear failure and cap locus (f_1 and f_2) are given by:

$$f_1 = \sqrt{J_{2D}} + \gamma e^{-\beta J_1} - \theta = 0 \tag{5}$$

$$f_2 = (J_1 - l)^2 + R^2 J_{2D} - (x-l)^2 = 0 \tag{6}$$

Where γ, β, θ and R are material parameters. *l* is the J_1 value of the intersection point of shear failure locus and the cap obtained by a simple numerical process. The hardening parameter x is dependent on plastic volumetric strain as follows (W, D and x_0 are material parameters):

$$x = \frac{-1}{D} Ln\left(1 - \frac{\varepsilon_V^P}{W}\right) + x_0 \tag{7}$$

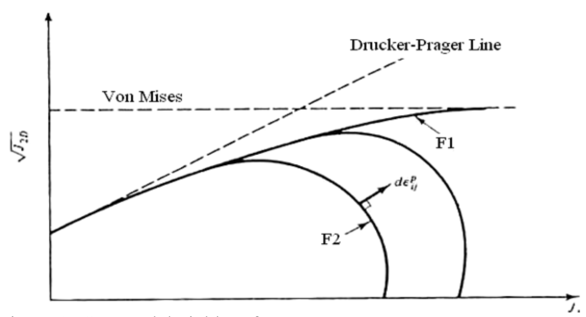


Figure 2. Cap model yield surface

5 VERIFICATION EXAMPLES

5.1 *Dry soil*

In order to verify the numerical model for dry soils, comparisons have been made with Takada and Oshima's (1994) centrifuge model tests conducted on nearly dry sand with water content of 4% under 100g acceleration.

The equivalent prototype parameters are tamper area 4 m^2, tamper mass 20 tons, and drop height 20 m. Miniature CPTs were conducted before and after the tamping, and the relative density (D_r) was deduced from the cone resistance.

The 2-D axisymmetric FE mesh shown in Fig. 1.a was used for modeling of the test in prototype dimensions. The cap model has been used herein to model dry sand behavior. The parameters of cap model were chosen based on the values which Gu and Lee (2002) had applied in their model. These parameters are presented in Table 1. Lee and Gu (2004) noted that if the state of the sand characterized in terms of relative density, then the influence of sand type and properties on the final state would have already been largely accounted for. The results of the numerical analysis and the experiment for final increase in relative density after 5 drops are shown in Fig. 3. As can be seen, reasonable agreement is obtained between computed and experimental results.

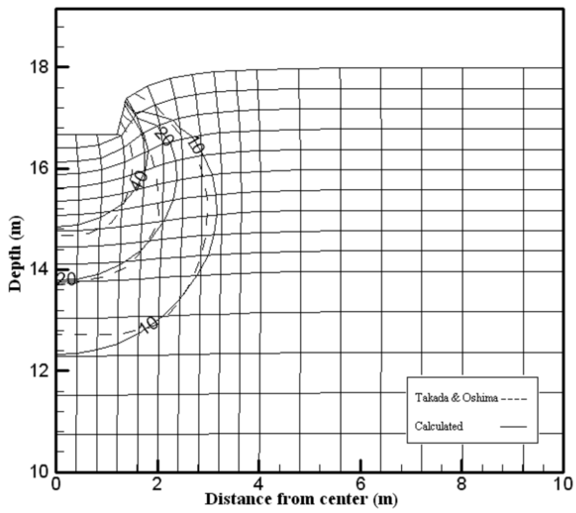


Figure 3. Final increase in relative density after 5 drops

Table 1: Cap model parameters for dry sandy soil

E (MPa)	γ	β	θ	R	W	D m^2/kN	x_0 (kPa)
25	500	0.0023	500	4.33	0.5	.00020	100

5.2 *Saturated soil*

To demonstrate the capability of the developed program for simulating DC procedure in saturated soil, results of the laboratory experiments performed on an organic soil by Gunaratne et al. (1996) were used. The organic soil had water content of 378 percent, wet density of 1064 kg/m^3 and organic content of 80 percent.

The FE mesh is shown in Fig. 1.b. All elements were considered saturated and drainage was allowed only from top surface. In this case Modified Cam-Clay model has been used for simulating organic soil behavior. The constitutive parameters and other model parameters are shown in table 2.

Tamper was dropped one time and a transducer recorded the pore pressure variation at 25 mm below the impact center. The experiment results and numerical prediction of pore pressure time history at this point are plotted in Fig. 4.

Two plates were embedded in soil mass at the depths of 25 and 50 mm (points A and B in Fig. 1.b, respectively). The clearance decrease between these points after the impact was reported 8.0 mm. The computed time history of decrease in AB distance is shown in Fig. 5.

Results on pore pressure variation depicted on Fig. 4 indicates that the maximum and minimum pore pressures calculated by the FE program PISA using modified cam-clay model is close to the values observed during the experiment. Although the numerical peak pore pressure and the subsequent dissipation occurs faster that those recorded in the experiment, the general pattern of pore pressure variation shows the similarity between the numerical values and experimental results.

In Fig. 5 the decrease in distance between A and B has an excellent conformity with the experimental observation.

6 CONCLUSIONS

Mechanisms of DC in dry and saturated soils and the mathematical formulation required for numerical simulation of DC procedure were explained. A fully coupled dynamic finite element code was developed which is able to simulate the DC treatment procedure in dry as well as saturated porous media, using U-P formulation. The results indicate that the model can predict the depth and degree of improvement, stress and strain fields at different stages of compaction and also the pore pressure variation in saturated soil under impact loads.

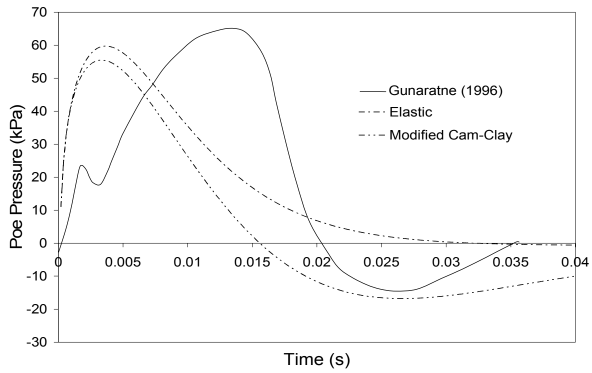


Figure 4. Pore pressure variation at 25 mm below the impact center

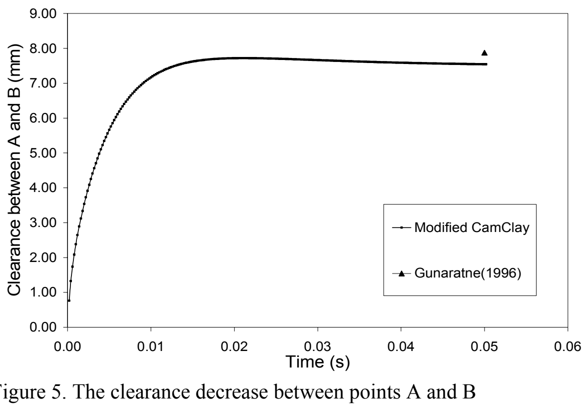


Figure 5. The clearance decrease between points A and B

Table 2: Modified cam-clay parameters for saturated organic soil

E (kPa)	ν	ρ (ton/m^3)	K (m/sec)	P_0 (kPa)	λ	κ
215	0.37	1.064	$.34\times10^{-8}$	0.5	0.1	0.01

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