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Jae-Soon Choi
Yonsei University, Seoul, Korea

Inn-Joon Park
Yonsei University, Seoul, Korea

Soo-Il Kim
Yonsei University, Seoul, Korea

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Jae-Soon Choi
Yonsei University
Shinchon Seodaemoon-Seoul-Korea

Inn-Joon Park
Yonsei University
Shinchon Seodaemoon-Seoul-Korea

Soo-Il Kim
Yonsei University
Shinchon Seodaemoon-Seoul-Korea

ABSTRACT

In this study, the behavior of reclaimed soils including the effects of excess pore water pressure and the loss of strength under dynamic loads or earthquakes are investigated and simulated. The constitutive model based on the disturbed state concept (DSC) is introduced and DSC-DYN2D program is utilized in a numerical analysis. In the laboratory test, quasi-static and cyclic triaxial tests were carried out to determine parameters for the numerical analysis. Field tests were executed in Inchon International Airport with a 10 tons hydraulic hammer. In the field test, the data of dynamic loads and excess pore water pressure were measured using a 3-D geophone and a pressure transducer respectively. The data of field tests showed the accumulation of excess pore water pressure when rapid dynamic loads were applied. Especially, a back-prediction program based on DSC model is developed and verified its accuracy using various parameters from the cyclic triaxial test.
As the results of numerical simulation, the predicted trends for excess pore water pressure compare well with the observed data. Based on the result of this research, it is found that the numerical analysis based on the DSC model is compatible to predict the softening behavior of saturated reclaimed soils under dynamic loads.

INTRODUCTION

For modeling the behavior of saturated sands under dynamic load, it is necessary to characterize the dynamic nonlinear behavior of saturated geological materials. Although a number of models have been proposed to characterize behavior of dry geological materials including elastic, plastic, and cyclic loading responses, few constitutive models have been developed for the behavior of fully saturated soils under dynamic loading. Such realistic constitutive models play an important role in analyzing and predicting the response of saturated geological materials.
The purpose of this research is to study for simulation of reclaimed soil behavior under dynamic loads based on the disturbed state concept (DSC). The initial idea for this theory was introduced by Desai (1974) to characterize the softening response of an over-consolidated soil by expressing the observed response in terms of its response in its normally consolidated state as the reference state. DSC model has been successfully verified with respect to other materials such as dry sands and the liquefaction of saturated sand and interface (Desai et al., 1998; Park, 1997). In this research, DSC model is calibrated and verified with cyclic triaxial test for the saturated reclaimed soils of Inchon International Airport in Korea and back-prediction scheme is developed with incremental integration methods. Especially, when the artificial dynamic vibrations are loaded, the vibrations and excess pore water pressures are measured in the site of Inchon International Airport. Excess pore water pressure is compared with the result from the numerical analysis.

BASIC THEORY

In the DSC, it is assumed that applied forces cause disturbance or change in the material's microstructure. As a result, an initially relative intact (RI) material modifies continuously, through a process of natural self-adjustment, and a part of it approaches the fully adjusted (FA) state at randomly disturbed locations in the material.

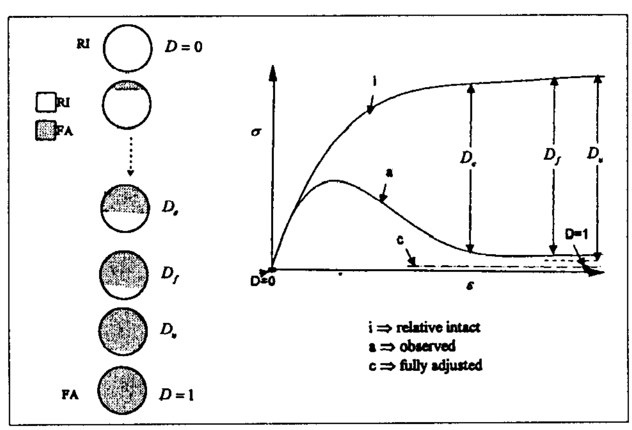

Fig. 1. Relative intact and fully adjusted state in DSC

Thus, the observed or average response can be represented in terms of the responses of the materials in RI and in FA state, which are called reference states as shown in Fig 2.

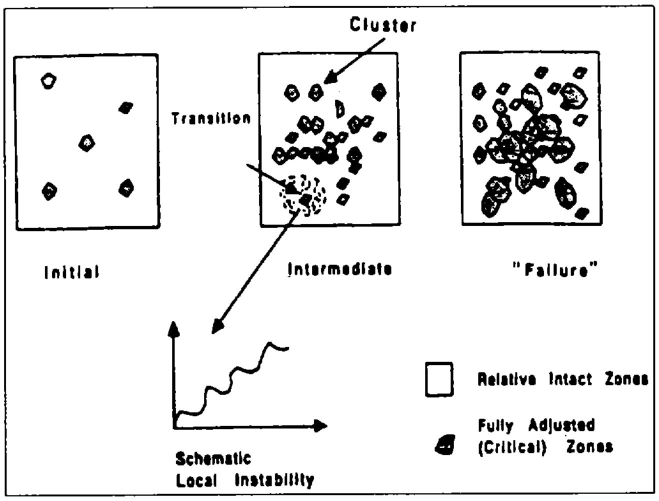

Fig. 2. Representation of DSC (Desai et al., 1998)

In this concept, RI state can be characterized as elastic, elastic-perfectly plastic, or any constitutive model like the Hierarchical Single Surface (HiSS) model (Desai et al 1986; Desai and Wathugala, 1987). FA state can be assumed to imply the state in which the material can continue to carry the shear stress level reached up to that state under given initial hydrostatic stress and can continue to deform in shear with constant volume-- critical state (Roscoe et al., 1957).

Relative intact state

In this study, relative intact state is characterized by using an elasto-plastic model that includes only hardening behavior. Here the basic $\delta_0$model in the HiSS Family is used to represent the behavior of the elasto-plastic material in the RI state. The $\delta_0$model is based on associative flow rule and isotropic hardening. In this model, the single yield surface function proposed by Desai et al. (1986) and Desai and Wathugala (1987) is given by

$$F = J_{2D}/P_a^2 - F_b F_s = 0 \qquad (1)$$

where J_{2D} = the second invariant of the deviatoric stress tensor, p_a = atmospheric pressure, F_b= the basic function, and F_s = the shape function and the value in the associative flow rule is unity.

$$F_b = -\alpha\,[J_1 / P_a]^{\,n} + \gamma\,[J_1 / P_a]^{\,2} \qquad (2)$$

where J_1=first invariant of the stress tensor, J_{3D}=the third invariant of the deviatoric stress tensor, n=phase change parameter, and γ =material parameters associated with the ultimate behavior. The hardening function, α, can be defined in terms of the plastic strain trajectory and hardening parameters.

$$\alpha = h_1 / \xi^{\,h_2} \qquad (3)$$

where ξ is the plastic strain trajectory and h_1 and h_2 are the hardening parameters. Fig. 3 describes the shape of yield surface in the J_1-$\sqrt{J_{2D}}$ space.

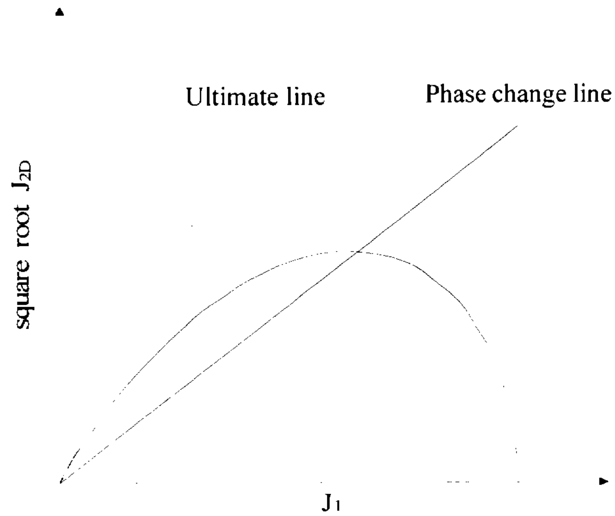

Fig. 3. The yield surface of HiSS model

Fully adjusted state

The FA state is modeled using critical state concept. At the critical state condition, shear deformations can continue indefinitely without further changes in volume. Such critical state often lies on a straight line called critical state line (CSL) with the slope $\bar{m}$:

$$(J_{2D})^{0.5} = \bar{m}\, J_1^c \qquad (4)$$

During shearing, volume and initial void ratio in the materials change in volume and initial void ratio changes. Finally, at the critical state the void ratio, e^c, is given by

$$J_1^c = 3\,P_a \times \exp\left[(e_o^c - e^c)/\lambda\right] \qquad (5)$$

where e_o^c = the value of critical state of void ratio at J_1^c =3 P_a and λ is a slope of the critical state line.

Disturbance function

The disturbance function can be dependent on microstructural deformations, initial conditions, temperature, and moisture content (Desai et al., 1998). D is defined as

$$D = M_s^c / M_s \qquad (6)$$

where M_s^c is the mass of solids in the FA state and M_s is the total mass of solids in the materials. Assuming the density of solid particles is constant in time and space, disturbance function, D, becomes

$$D = V_s^c / V_s = A_s^c / A_s \qquad (7)$$

where V and A are volume and area for a constant thickness. Initially with no disturbance the material is assumed to be entirely in the RI state, so D is zero. With full disturbance the material can be assumed to be fully in the FA state, and at the ultimate state, $D = D_u \leq 1$. Theoretically, the disturbance, D, varies between zero and unity, but many materials fail before D reaches unity. The disturbance function, is given by Armaleh and Desai (1990):

$$D = D_u [1 - \exp (- A \xi_D^Z)] \qquad (8)$$

where A and Z are DSC parameter and D_u is the ultimate disturbance of material. Fig. 4 shows a schematic of the disturbance function.

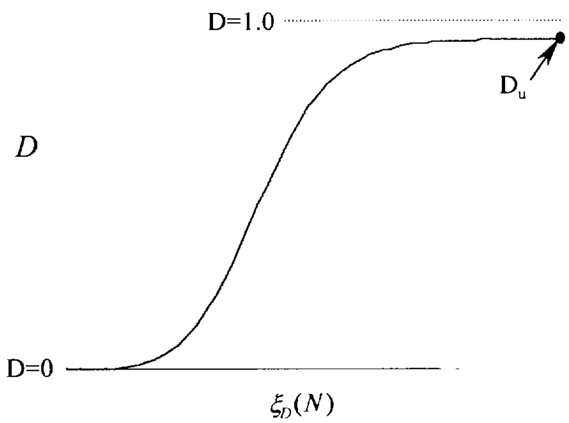

Fig. 4. Schematic of DSC function

Based on the equilibrium forces in the observed, RI, and FA state, the incremental constitutive equations to describe the observed response are derived for DSC-DYN2D.

$$d\sigma^a = (1\text{-}D)\, C^i d\varepsilon^i + D\, C^c d\varepsilon^c + dD(\sigma^c - \sigma^i) \quad (9)$$

where a, i, and c denote observed RI and FA responses, respectively, C=constitutive matrix and σ and ε=effective stress and strain vector, respectively, and dD= rate of D.

TESTS FOR DYNAMIC PARAMETERS

In this study, the field tests were conducted to measure the vibration and excess pore water pressure of saturated sandy soils under the dynamic loads that is generated by the hydraulic hammer and several laboratory tests were carried out to determine Hiss and DSC parameters.

Field test

Before the field test, the several ground investigations including SPT were carried out. The site condition of the reclaimed site is shown Fig. 5.

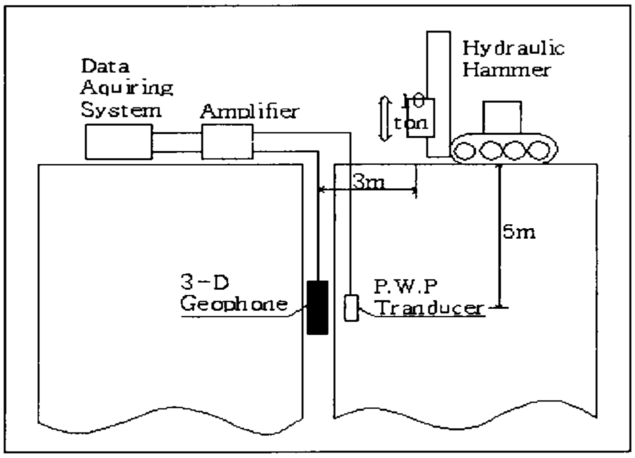

Fig. 5. Figure of Site condition in field test.

From SPT tests, the average SPT value of depth 4 to 7m was 12. In the field test, a pore water pressure transducer and 12mm diameter steel pipe for 3-D geophone's measuring were set up in the ground. A 10-ton hydraulic hammer was used as a vibrator for dynamic source and the data of test were gained with a 3-D geophone and pore water pressure transducer at a specific depth, respectively. The data from both instruments were measured with an interval of 0.001 second. Fig. 6 shows a typical data of field tests from a pore water pressure transducer.

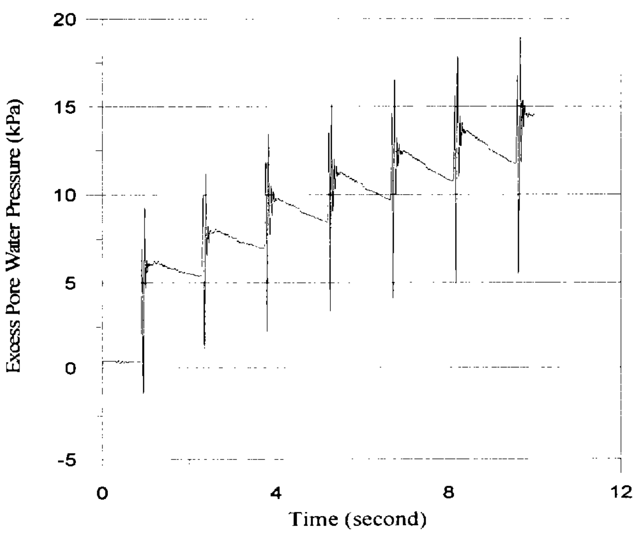

Fig. 6. Figure of the results from the field test

In Fig. 6, as generating the excess pore water pressure under dynamic loads is not enough to generate the liquefaction behavior, the data of field tests showed the accumulation of excess pore water pressure when rapid dynamic loads were applied.

Laboratory test

In the laboratory test, the parameters concerned with RI and FA were gained by quasi-static triaxial tests and DSC parameters were gained by cyclic triaxial tests. In the triaxial tests, the sample was remolded with reclaimed soils from Inchon National Airport. Relative density for remolding was 50% and its value was determined as a representative value for considering average SPT-N value (N = 12). The condition of cyclic triaxial test and properties of remolded sample are summarized in Table 1.

Table 1. Laboratory test conditions and sample properties

Test condition	Sample property	
Load control : sterss	Relative density	50%
control	Fine content (≤#200)	20%
Drainaged condition:	Specific gravity	2.65
undrained	Average grain size	0.20 mm
Load type : sinusoidal	Uniformity coefficient	2.60
Frequency : 1Hz	Curvature coefficient	1.20
Effective confining	Max. unit weight	1.64 g/cm^3
Pressure : 100 kPa	Min. unit weight	1.31 g/cm^3

Table 2 shows the input parameters of the numerical analysis based on the laboratory tests.

Table 2. Input parameters in numerical analysis

Material state	Group	Parameter	Value
Relative	Elastic	E (kPa)	178000
		ν	0.380
Intact	Plastic	γ	0.199
		n	2.623
		h_1	0.118
State		h_2	0.032
Fully	Critical	$\bar{m}$	0.341
Adjusted	state	λ	0.019
State		e_o^c	0.654
Observed	Disturbance	D_u	0.991
	Function	A	3.325
		Z	0.531

VERIFICATION

A back prediction program was developed with a incremental integration scheme to validate DSC model and model parameters. The program can define the strain softening with DSC model. DSC model was adapted in the back prediction of the compression and extension behavior and linear elastic theory was adapted in the expression of unloading behaviors. Fig. 7 shows the excess pore water pressure from the cyclic triaxial test and the back prediction.

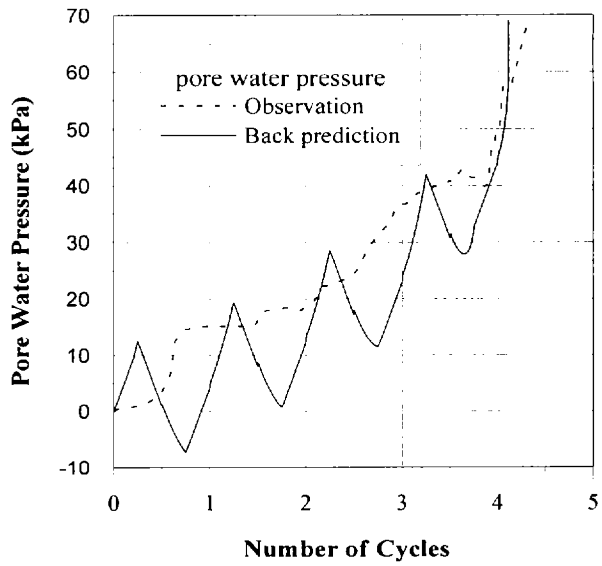

Fig. 7. Excess pore water pressure from the back prediction (Jung, 2000)

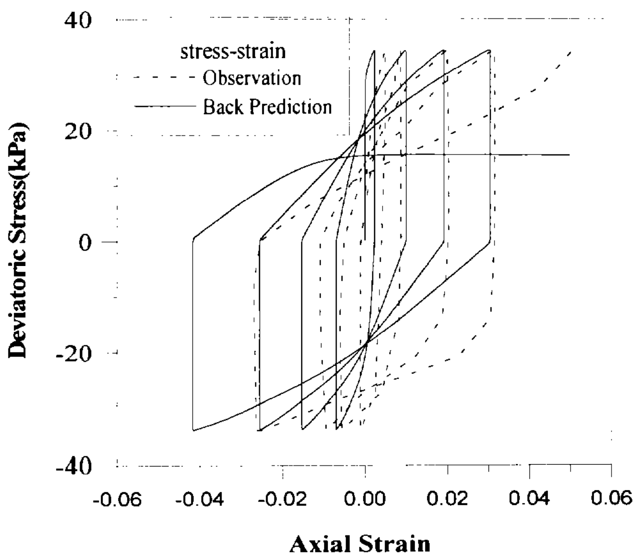

Fig. 8. Stress-strain relationship from the back prediction (Jung, 2000)

From the Fig. 7, it is shown that the back prediction has good agreement in representation of the peak point of excess pore water pressure. Especially, the test result was not more conservative in expressing the excess pore water pressure with respect to number of cycle because the pore water pressure instrument was little sensible about synchronizing compression and extension. In Fig. 8, the relationship of axial strain and deviatoric stress form the back prediction is pretty similar with the result that was gained from cyclic triaxial tests.

NUMERICAL SIMULATION

In the numerical simulation, DSC-DYN2D(Park 1997) based on the Hiss δ_0 model and DSC model is utilized. The input loads were strains calibrated from the measured from 3-D geophone. Fig. 9 shows the input strain loads and Fig. 10 shows the finite element mesh in the numerical simulation.

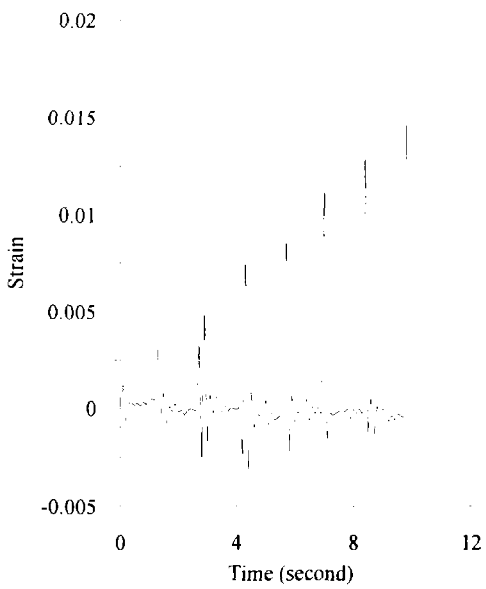

Fig. 9. Input loads in the numerical analysis

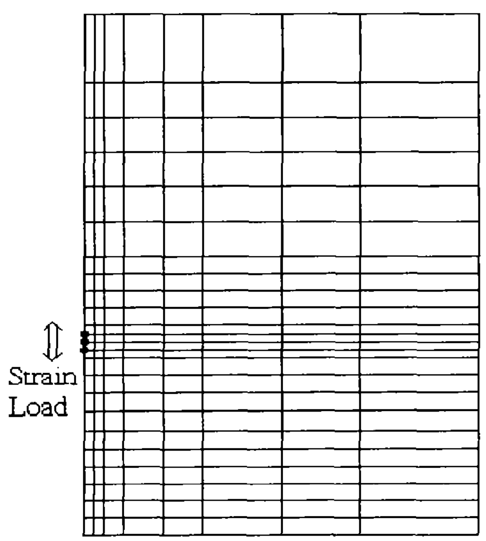

Fig. 10. FEM mesh in the numerical analysis

In the numerical analysis, the DSC-DYN2D program using finite element method (FEM) was utilized. The result of numerical analysis with DSC-DYN2D was compared with the field test result previously referred. The output of DSC-DYN2D is shown Fig. 11.

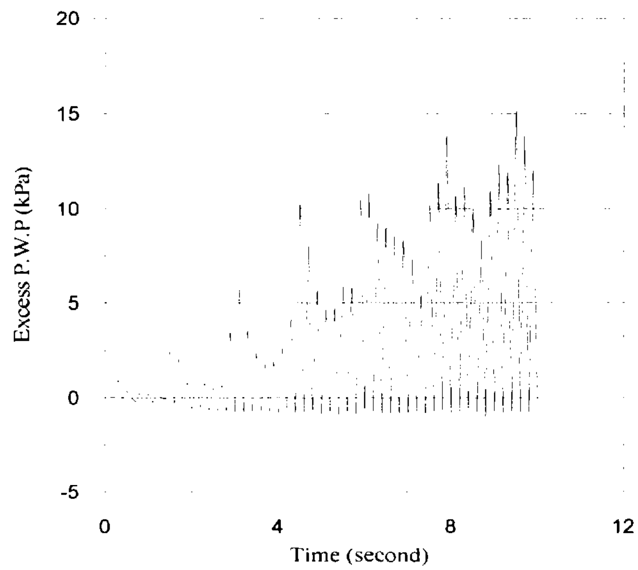

Fig. 11. The results from numerical analysis

In the Fig. 11, the result from numerical analysis shows the similar accumulation of excess pore water pressure with Fig. 6. In comparing each result, it is found that there is the measuring time lack in field test because a excess pore water pressure transducer was more insensible than 3-D geophone. Overall, the correlation is good. However, there are differences between observations and predictions. These differences can be due to the fact that the parameter was found from tests with different load condition.

In the results of numerical analysis, it is found that DSC model can capture the behavior of saturated reclaimed soils under dynamic loads using the effective stress analysis concept.

CONCLUSION

In this research, the numerical analysis using the DSC model is studied. From the laboratory tests, the input parameters are determined in the field test with a 10 tons hydraulic hammer. Comparing the results between the field test and numerical analysis, the trends of pore water pressure between test results and numerical results are similar. Therefore, the numerical analysis using DSC model can predict the softening behavior of saturated soils under dynamic loads.

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