modeling framework for interfaces that is consistent with that for the soil, has not been developed.

Under loading, a saturated sand with certain range of (initial) density may experience microstructural particle motions that can lead to states of local and global instabilities. The latter can lead to liquefaction and resulting failure of the foundation soil. A great number of empirical and mechanistic models have been proposed and developed for liquefaction in sands. However, the issue of liquefaction at and in the vicinity of the interface, that can occur before or after that in the neighboring soils, has not been investigated before.

Scope

The scope of this paper entails the following items:

(a) A unified constitutive model, called the disturbed state concept (DSC), that allows for elastic, plastic and creep responses, relative particle motions and microcracking leading to softening and liquefaction, and stiffening or healing,

(b) Use of the DSC model for soils and interfaces,

(c) Laboratory testing for calibration of parameters for soils and interfaces,

(d) Validation of DSC models for soils and interfaces,

(e) Implementation of DSC model is nonlinear coupled static and dynamic finite element procedure, and

(f) Verification and analysis of typical problems tested in the laboratory and field.

In view of the length and time limitations, the descriptions presented below are brief, and related directly to the DSC. As a result, no detailed review of other literature is included; it is available in various references cited.

The Disturbed State Concept

The DSC represents a unified constitutive modeling approach that allows characterization of the behavior of "solids" (soils, rocks, concrete, ceramics, metal alloys), and interfaces and joints with the same mathematical framework as for solids. It permits, in a hierarchical manner, elastic, plastic and creep responses, relative particle motions, microcracking and fracture leading to softening, and healing or stiffening under thermomechanical loading.

The DSC possesses a number of advantages compared to other available models:

(1) Its hierarchical character provides the user the flexibility to choose a version(s), elastic, elastoplastic, viscoplastic and disturbance (damage and

softening), depending on the behavior of a given material(s) for specific application. In other words, only parameters relevant to the selected version are required to be input.

(2) It is based on fundamental mechanistic considerations, and yet is simplified for practical applications. For example, for a given capability, it involves lesser number of parameters compared to other available models.

(3) Its parameters have physical meaning and can be determined from standard laboratory tests.

Details of the DSC and its hierarchical (hierarchical single surface – HISS plasticity) versions are given elsewhere (Desai, 1995, 1999; Desai and Ma, 1992; Desai, et al., 1997a, 1998a, 1998b; Desai and Toth, 1996; Katti and Desai, 1995; Shao and Desai, 1998; Park and Desai, 1999). The basic constitutive equations for the DSC are given below.

$$d\underset{\sim}{\sigma}^a = (1-D)d\underset{\sim}{\sigma}^i + D\underset{\sim}{\sigma}^c + dD\left(\underset{\sim}{\sigma}^c - \underset{\sim}{\sigma}^i\right) \tag{1a}$$

or

$$d\underset{\sim}{\sigma}^a = (1-D)\underset{\sim}{C}^i\, d\underset{\sim}{\varepsilon}^i + D\underset{\sim}{C}^c\, d\underset{\sim}{\varepsilon}^c + dD\left(\underset{\sim}{\sigma}^c - \underset{\sim}{\sigma}^i\right) \tag{1b}$$

where $\underset{\sim}{\sigma}$ and $\underset{\sim}{\varepsilon}$ = stress and strain vectors, respectively, a, i and c denote observed, RI and FA states, respectively, d denotes increment or rate, $\underset{\sim}{C}$ = constitutive matrix and D = (scalar) disturbance, which can be expressed as a tensor if appropriate laboratory tests are available (Desai and Toth, 1996). In the DSC, the stresses and strains in the RI and FA states can be different; however, if they are assumed to be compatible (i.e., $d\underset{\sim}{\varepsilon}^a = d\underset{\sim}{\varepsilon}^i = d\underset{\sim}{\varepsilon}^c$), the formulation is simplified and only Eqs. (1) are required. Equations (1) include elastic, elastoplastic (or viscoplastic) models or versions as special cases when D = 0. If D ≠ 0, disturbance (microcracking and damage) is allowed, Desai and Toth (1996), Desai (1995, 1999).

In the DSC, it is assumed that during deformation, the material transforms from the initial *continuum* or relative intact (RI) state to the fully adjusted (FA) state as a result of continuing changes in the material's microstructure. Hence, at any state during deformation, the material element is composed of a mixture of parts in the RI and FA states. The observed or actual response of the material is then expressed in terms of the behavior under the RI and FA states through disturbance (D), which acts as an interpolation and coupling mechanism (Desai, 1995, 1999).

The RI response can be characterized by using such continuum theories as elasticity, elastoplasticity and viscoplasticity, with thermal effects. Here, the elastoplastic model in the HISS approach (Desai, et al., 1986) is used. In the FA state, the material parts can carry no stress at all, as they act as cracks or voids, like in the classical damage model, or it can carry hydrostatic stress but no shear stress like a constrained liquid, or it can carry shear stress reached up to that state for a given mean pressure and deform in shear without volume change as in the critical state concept (Roscoe, et al., 1957; Desai, 1995, 1999). The latter two are considered to be realistic and provide for important interaction between the RI and FA parts, and are used often. In this paper, the FA state is characterized by using the critical state concept.

The DSC formulation includes in its framework the coupling between the RI and FA parts; as a result, it is not necessary to add external enrichments such as microcrack interaction, Cosserat and gradient theories. Because of this, the DSC model allows for nonlocal effects and avoids spurious mesh dependence (Desai, et al., 1997b; Desai, 1999).

Unloading and Reloading

Details of the simplified procedures to simulate unloading and reloading are given elsewhere (Desai, et al., 1997a; Shao and Desai, 1998). Their characterization requires two additional parameters, the slope of stress-strain curve at the end of unloading (E^u) and the irreversible strains during unloading cycle (ε^p).

Interfaces

The foregoing formulation for solids can be specialized to characterize behavior of interfaces (or joints) idealized as thin layers between structural and geologic materials (Desai, et al., 1984). The thin-layer interface element is treated as a "solid" element; however, the parameters for the DSC model are determined on the basis of static or cyclic interface tests. Details are given by Desai and Ma (1996), Desai and Fishman (1991), Desai and Rigby (1997), Park and Desai (1999), and Shao and Desai (1998).

Parameters

The parameters in the DSC model have physical meanings as they are related to specific states during deformation. Their number is lower than that in other available models of comparable capabilities. For example, for the general disturbance model that allows for elastic and plastic strains including continuous yielding, and microcracking leading to softening, the parameters involved are two elastic (E, ν); four plasticity (γ, β, a_1 and η_1), three critical state (e_o, λ, $\bar{m}$) and three for disturbance (A, Z and D_u). These parameters can be found from standard triaxial (or multiaxial) tests; the disturbance parameters are found based on the softening or cyclic degradation response.

Validations

The DSC model and its versions have been applied successfully to characterize behavior of cohesionless soils, cohesive saturated soils, rocks, concrete, ceramic composites, metal alloys like solders in electronic packaging problems, and silicon with dislocation and impurities, and interfaces and joints (Desai, 1995, 1998; Desai and Fishman, 1991; Desai and Ma, 1992; Katti and Desai, 1995; Desai and Toth, 1996; Desai, et al., 1998a, 1998b; Desai and Rigby, 1997; Park and Desai, 1997; Shao and Desai, 1998). The model has been implemented in static and dynamic finite element procedures for dry and saturated materials and interfaces (Desai, et al. 1997; Shao and Desai, 1998; Park and Desai, 1999).

Finite Element Procedure

The generalized Biot's theory is used to formulate the finite element (FE) procedure for coupled deformation and pore water pressure responses. The procedure allows different versions; e.g., dynamic, consolidation and static. The DSC model for loading, unloading and reloading is implemented in the FE procedure. The results from the code include time dependent displacements, strains, stresses (total and effective), pore water pressures and disturbance. Contours of disturbances with time in the finite element mesh provide identification of zones in which the critical disturbance, D_c, is reached (Desai, et al., 1998b; Desai, 2000). It denotes initiation of liquefaction, and allows tracing of the growth of disturbance and liquefaction under subsequent loading cycles.

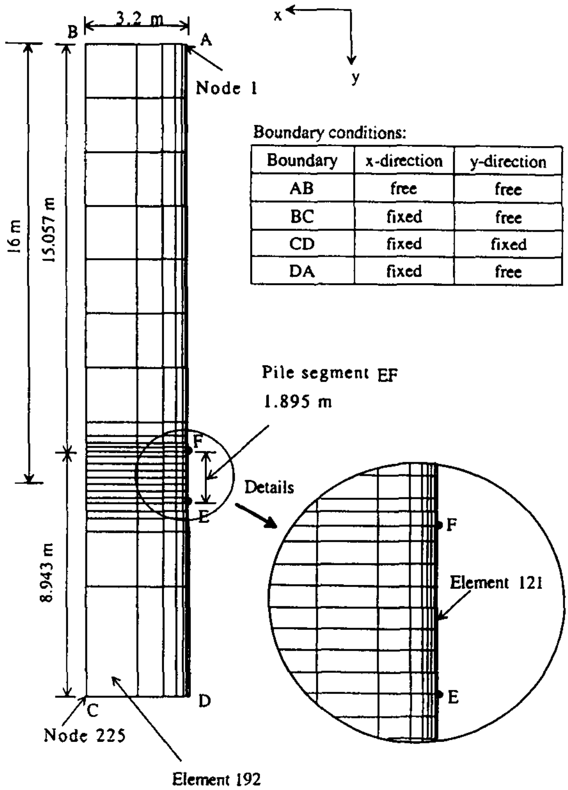
Applications

A number of problems involving static, repetitive and dynamic loading with elastoplastic, viscoplastic and DSC models have been solved by using the computer code. In most cases, the computer predictions are compared with observations in the field and simulated laboratory models. Here, brief descriptions of three typical examples involving a pile in clays, pile in sand, and dynamic soil-structure interaction in a shake table test are presented.

Pile in Marine Clay

Instrumented pile segments were tested in the field at Sabine, Texas by Earth Technology Corporation (ETC, 1986). The field program included tests for measurement of *in situ* stresses, installation of pile segments with different diameters and cutting shoes, monitoring consolidation, performing axial tension tests at different levels of consolidation and cyclic axial load tests at the end of consolidation. The measurements included total lateral stresses and pore water pressures at the pile wall, and shear transfer versus pile displacement. Undisturbed specimens of soil were obtained by using Shelby tubes and specially designed rectangular tubes for cyclic triaxial and multiaxial testing of the marine clay, respectively. Interface tests between the marine clay and pile (steel) were conducted by using the cyclic multi degree-of-freedom shear device (Desai and Rigby, 1997).

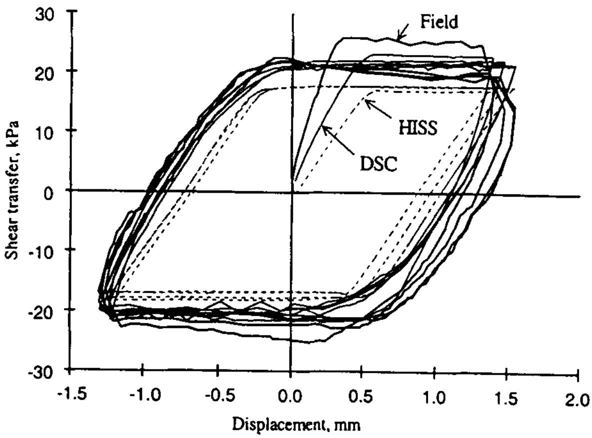
The results presented here are based on the coupled finite element procedure including the disturbed state model for loading, unloading and reloading (Desai, et al., 1997; Shao and Desai, 1998). Various sequences such as initial conditions, pile driving, consolidation, axial tension tests and cyclic loading were simulated in the finite element analysis. The FE mesh is shown in Fig. 1. The inside nodes on the soil-pile interface (along EF) were subjected to cyclic (vertical) displacements as it was applied in the field.



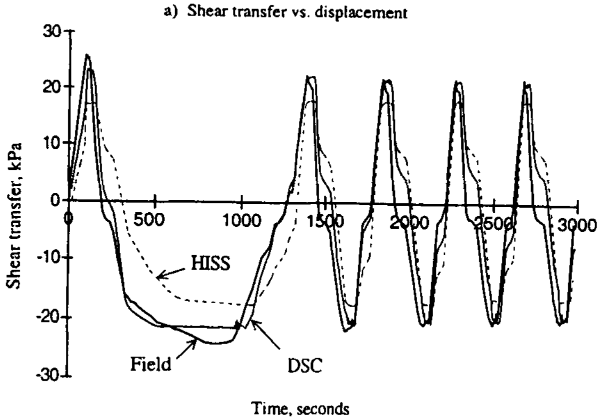
Boundary	x-direction	y-direction
AB	free	free
BC	fixed	free
CD	fixed	fixed
DA	fixed	free

Figure 1. Finite Element Mesh of Field Pile Test

Figures 2 (a) to (c) show typical comparisons between the predictions in terms of shear transfer versus displacement, and shear transfer and pore water pressures versus time from the DSC model and field behavior under two-way cyclic loading, respectively. They also show predictions by using anisotropic hardening plasticity (HISS) model without the provision for degradation (disturbance) (Wathugala and Desai, 1993; Desai, et al., 1993). It can be seen that the DSC predictions correlate very well with the field data, and the provision for degradation (disturbance) provides improved predictions compared to those from the HISS-plasticity model.

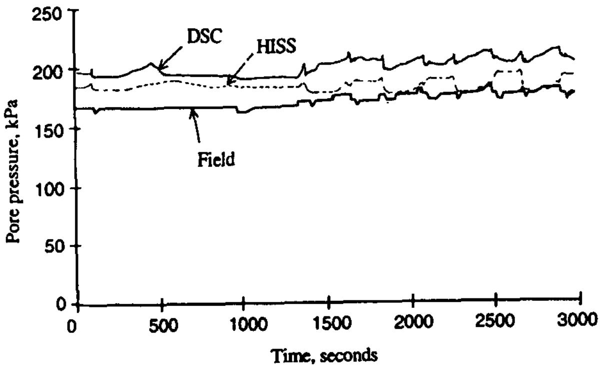


a) Shear transfer vs. displacement



b) Shear transfer vs. time

Figure 2. Comparison between Field Measurements and Predictions From DSC and HISS Models: Two-way Cyclic Load Test



c) Pore pressure vs. displacement

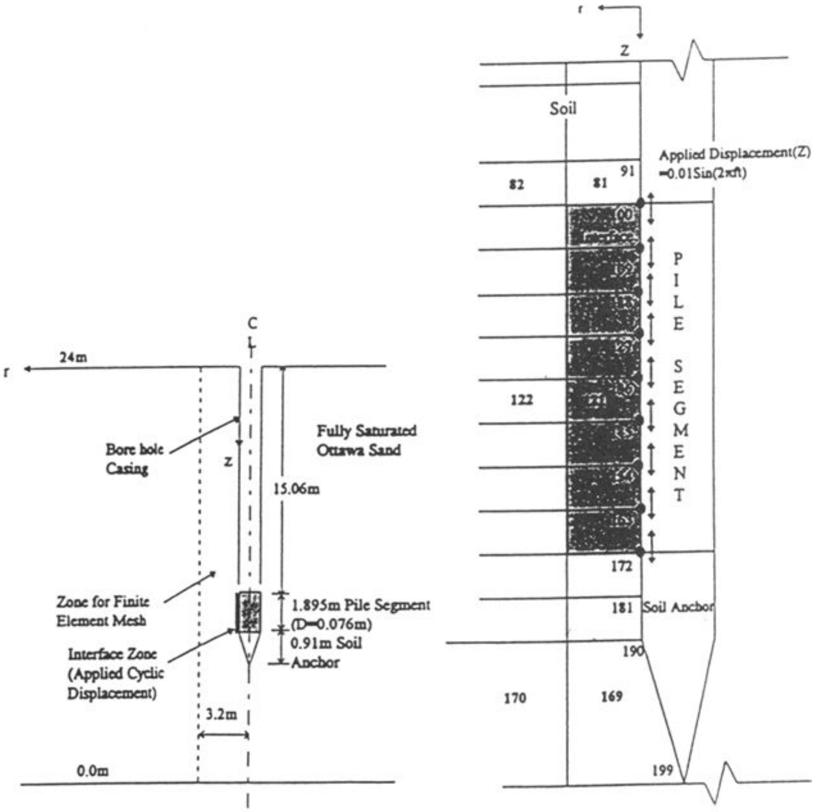
Figure 2 (continued)

Pile in Saturated Sand

In order to study the interaction and liquefaction behavior, a steel pile in saturated Ottawa sand was simulated by using the DSC model and the finite element procedure. Figure 3(a) shows the pile-sand details, and Fig. 3(b) shows mesh details near the interface in which sinusoidal displacement loading was applied at the nodes (Park and Desai, 1997).

The cyclic behavior of the soil was characterized from a series of tests using the multiaxial test device with 10 x 10 x 10 cm saturated specimens (Gyi, 1996). The interface DSC behavior was characterized based on a series of interface tests between steel and sand using the CYMDOF-P device (Desai and Rigby, 1997).

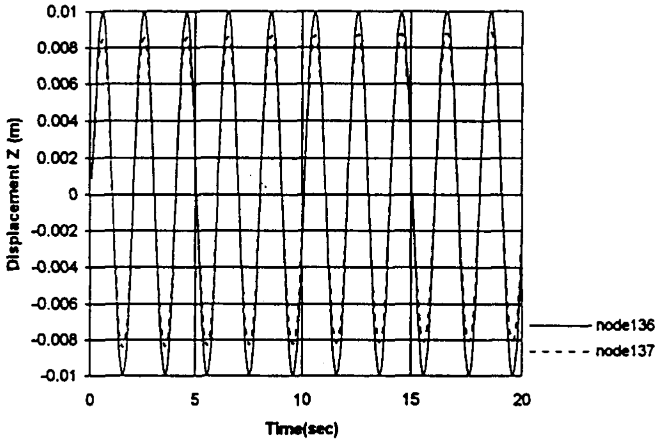
Computer analyses were performed for two conditions: (1) no interface, i.e., pile and soil are compatible, and (2) with interface, i.e., relative motions are allowed between the soil and pile. Figures 4(a) and (b) show computed (vertical) displacements with time at typical notes 136 and 137; the former is on the pile and the latter in the soil. Figures 5(a) and (b) show computed pore water pressures in typical (soil) element 121 with time. Figures 6(a) and (b) show disturbance in soil elements 121 and 122, Fig. 3(b), with time. It can be seen that provision of the interface, i.e., relative motions, modify the computed results significantly. There occur significant relative displacements between pile and soil with the interface provision. The variation in the magnitudes of cyclic pore water pressures are much lower with the interface, indicating that the relative motions cause "damping" in the pore pressures generated. The disturbances in the vicinity of the interface are generally smaller with the interface.



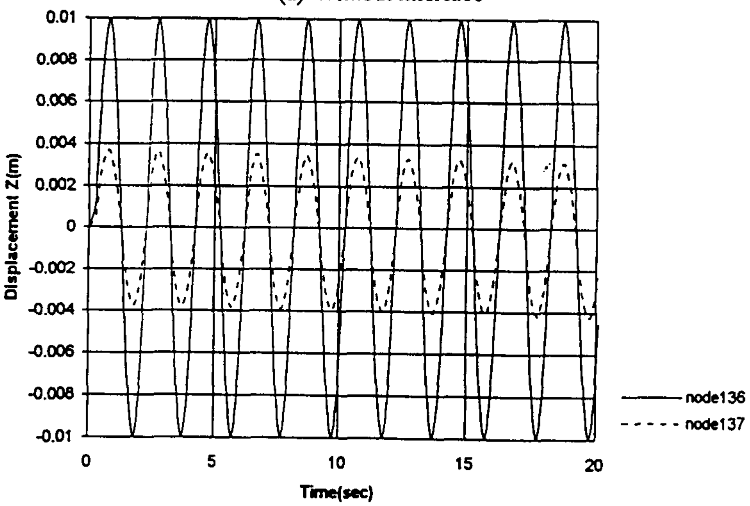
(a) Pile details

(b) Detailed mesh around interface

Figure 3. Pile in Saturated Sand (Park and Desai, 1997)

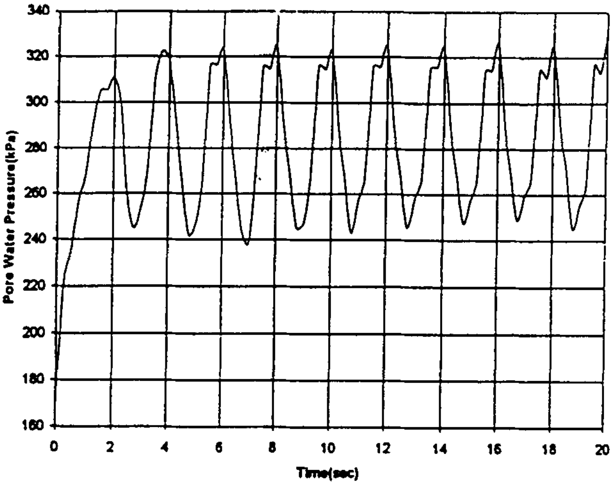


(a) Without interface

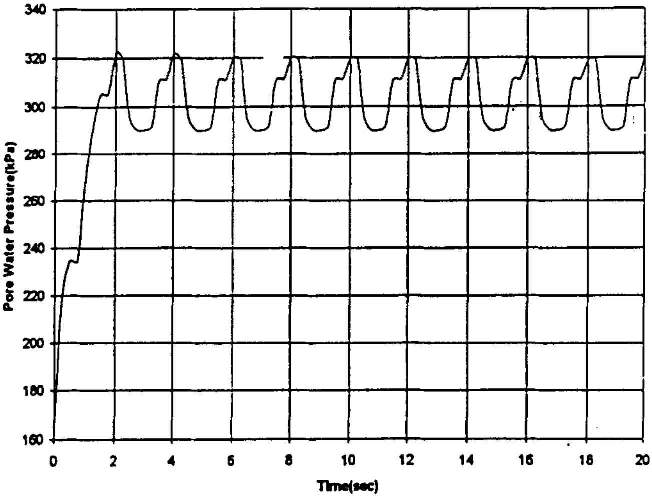


(b) With interface

Figure 4. Displacement at Typical Adjacent Nodes near Interface



(a) Without interface



(b) With interface

Figure 5. Pore Water Pressure in Element 121