Unified modeling of sand under different conditions and its application to numerical simulation of boundary value problems

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UNIFIED MODELING OF SAND UNDER DIFFERENT CONDITIONS AND ITS APPLICATION TO NUMERICAL SIMULATION OF BOUNDARY VALUE PROBLEMS

砂の異なる条件下での統一的なモデル化および
境界値問題の数値解析への適用

A dissertation submitted in partial fulfillment of the requirements for the Doctoral Degree in Civil Engineering

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Preface

Liquefaction of sandy ground during earthquake is a major problem that may cause big catastrophes to infrastructures. In seismic evaluation, it is important to describe correctly the mechanical behavior of soils subjected to cyclic loading in order to avoid or reduce the damages caused by liquefaction.

Many researches have been done on the liquefaction of soils at an element level to develop a suitable constitutive model based on numerical simulation using finite element method (FEM) applied to a boundary value problem (BVP) to simulate the liquefaction of soils. Some constitutive models can describe the stress-strain relationship of sand during liquefaction to some extent, but the common disadvantage among these constitutive models is that it is necessary to determine different values of material parameters for sands with various densities. This is not reasonable because in a liquefaction process, the density of ground is not constant and will increase because pore water flows out from the ground. However, Zhang et al. (2007) proposed a new constitutive model for sand in which mechanical behavior of fictional sand subjected to different loadings in variant drainage conditions are simulated to verify whether the model is suitable to describe the general behavior of sand with one set of definite parameters. In this study, element simulations under different loading and drainage conditions are conducted to confirm the overall mechanical behaviors of Toyoura sand with modified elastoplastic constitutive model (Zhang et al., 2007) with a minor change.

On the other hand, in order to conduct a precise prediction of a BVP, it is necessary to determine the values of soil parameters involved in a constitutive model by element simulation based on laboratory tests in element level. In the element simulation, though stress-strain relation is regarded as uniform within the soil specimen, it is usually non-uniform in reality. Due to limitation of element test devices in laboratory tests, a perfect element test is impossible in reality because of some factors such as an initial imperfection of test specimen, friction between loading heads and the specimen, existence of gravitational stress field and etc. Although some researches concerning these factors under monotonic loading can be found in literatures, few studies have been
done about the nonuniformity of the soil specimen under undrained cyclic loading.

The main purpose of this dissertation is to study the mechanical behavior of Toyoura sand (TS) under different drainage conditions and loading conditions with laboratory tests, theoretical simulation and numerical calculation based on the modified model developed in Zhang et al. (2007).

In the experimental study, undrained cyclic loading tests are conducted to confirm the behavior of saturated TS by the newly adopted A/D board. The effect of density of the specimen, confining stress, amplitude of cyclic loading and frequency are discussed in detail. The precision and the reproducibility of the test method are verified as well.

Following the experimental study, the constitutive model proposed by Zhang et al. (2007) is modified in its evolution equation for overconsolidation in order to describe the overall mechanical behaviors of TS in a unified way in which the eight parameters for describing the sand will be constant no matter what kinds of loading conditions or drainage conditions may be. The performance of the model is confirmed with different triaxial tests considering both drained and undrained conditions subjected to monotonic and cyclic loadings.

2D and 3D soil-water coupling finite difference-finite element (FD-FE) analyses are conducted to simulate the mechanical behavior of sand specimen subjected to cyclic load in conventional triaxial test under undrained condition using a program named as DBLEAVES (Ye, 2007). It is investigated in the aforementioned problems using FEM considering the element test as a BVP, in which several factors affecting the nonuniformity of the soil specimen are taken into consideration in detail.

At last, three-dimensional (3D) finite element analyses of a real-scale group-pile foundation subjected to horizontal cyclic loading is conducted using the DBLEAVES. In the simulations, nonlinear behaviors of ground and piles are described by subloading $t_{ij}$ model (Nakai and Hinokio, 2004) and AFD model (Zhang and Kimura, 2002) respectively. The numerical analyses are conducted by the total stress and the effective stress methods. The material parameters of soils are determined based on the undisturbed and remolded specimens. It is conducted (a) to verify the applicability of the proposed numerical method by comparing the numerical results with the test results, and (b) to confirm the importance of determining the material parameters. In addition, numerical experiments on seismic performance of reinforcement by ground
improvement around an existing pile foundation are also conducted to determine the optimum pattern in the size, the location and the shape of ground improvement zone.
The work for this dissertation is completed at the Department of Architecture, Civil Engineering and Industrial Management Engineering, Nagoya Institute of Technology during the period from April 2007 to March 2010. The completion of this dissertation could not have been achieved without the support and encouragement from professors and colleagues, to whom I would like to express my wholehearted thanks and appreciation.

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1.1 Background and objectives

Liquefaction is a physical process taking place during earthquakes that may lead to ground failure. The most intense damage of liquefaction is confined to areas where buildings and other structures that are situated on top of loosely consolidated, water saturated soils. Loosely consolidated soils tend to amplify shaking and increase structural damage. Water saturated soils complicate the problem due to their susceptibility to liquefaction and corresponding loss of bearing strength. Some damages of this phenomenon are shown in Figure 1-1.

![Niigata, Japan in 1964 and Kobe, Japan in 1995](image)

(a) Niigata, Japan in 1964 (b) Kobe, Japan in 1995

Figure 1-1 Damages caused by liquefaction (JGS, 2004 & Kobe city homepage)

To avoid or reduce the damages caused by liquefaction, researchers have made great efforts trying to clear out the mechanical behaviors of soil subjected to cyclic loading. Many researches have done the liquefaction of soils experimentally and theoretically at an element level to develop a suitable constitutive model based on numerical simulation using finite element method (FEM) applied to a boundary value problem (BVP) to simulate the liquefaction of soils. There are a lot of constitutive models which can describe the liquefaction and cyclic mobility behaviors to some extend, but there are common disadvantages among these models:

1. The constitutive models need to determine different values of material
parameters in order to describe the various behaviors of sands with variant initial densities. However, it is not reasonable to determine the values of material parameters according to the initial density of sand because normally the density of sands will change during the loading process.

(2) In the most of models, it is necessary to decide the result if the ground liquefies or not before conducting a numerical simulation.

(3) Most of these models adopt non-associated flow rule thus the formulation of the models is quite complicated and more parameters have to be used in the models. However, Zhang et al. (2007) proposed a new constitutive model for sand in which mechanical behaviors of fictional sand subjected to different loadings in variant drainage conditions are simulated to verify whether the model is suitable to describe the general behaviors of sand with one set of definite parameters. In this study, element simulations under different loading and drainage conditions are conducted to confirm the overall mechanical behaviors of Toyoura sand with modified elastoplastic constitutive model (Zhang et al., 2007) with a minor change.

Moreover, in order to conduct a precise prediction of a BVP, it is vital to determine the values of soil parameters involved in a constitutive model by element simulation based on laboratory tests at the element level. In element simulation, though stress-strain relation is regarded as uniform within the specimen of soil, it is usually non-uniform in reality due to some inevitable imperfection or nonuniformity of both the test specimen and the test device. It is, therefore, necessary to confirm that the results of an element test commonly used in laboratory test are still useful to determine the mechanical behaviors of the soils, even if the so-called element test does not usually show a uniform behavior in its occupied area. Also it is needed to define the influence of the imperfection on the overall mechanical behavior of the soil specimen which is usually regarded as an element behavior clearly.

On the other hand, there have been many reports on damage to pile foundations caused by earthquake such as Hyogoken-Nanbu earthquake in 1995, e.g. Syamoto et al. (1995). Furthermore, it should be noted that once a foundation was damaged, its repair is technically difficult and the cost is expensive. Sometimes it is even forced to demolish the entire building even if the upper structure is intact. Thus the foundations of a building that may be subjected to seismic force should be designed so as to maintain
structural safety equivalent to or exceeding that of the upper structure. Therefore, it is important to conduct an investigation according to the state of the site with respect to possible ground deformation such as liquefaction and landslide, and measures such as appropriate soil improvement should be taken as needed. In many improvement methods, the partial ground solidification applicable to existing group-pile foundation is a very desirable method because it has several benefits such as the cost minimization, time minimization and workable construction. However, the optimum pattern in the size, the location and the shape of ground improvement zone has not been discussed yet.

Therefore, the research tried to achieve the following objectives:
1) describing the overall mechanical behaviors of Toyoura sand with a minor change for the elastoplastic constitutive model (Zhang et al., 2007);
2) confirming whether the results of an element test showing non-uniform behavior are useful in determining the mechanical behaviors for the numerical simulation and defining the influence of the nonuniformity on the overall mechanical behavior of the soil specimen;
3) investigating the mechanical behaviors of sand specimen subjected to cyclic load under undrained conventional triaxial test;
4) evaluating the mechanical behavior of a pile foundation subjected to cyclic lateral loading up to an ultimate state and determining the optimum size and location of the partial ground reinforcement material.

1.2 Review of previous works

1.2.1 Element tests for sand

Understanding the mechanical behavior of sand is very important for designing against various loading conditions. The mechanical deformation characteristics of sand should be predicted by a constitutive model in order to design many typical geotechnical structures such as levees, foundations, tunnels, dams using numerical simulations.

Many researchers have worked on investigating sand behavior. For instance, failure pattern of undrained sand subjected to cyclic loading may take the form of
liquefaction or cyclic mobility according to the density of the sand (Castro, 1969). Tatsuoka et al. (1982) reported that cyclic undrained strength of loose sand can be rather easily determined while those for dense sand is much more difficult to determine not only because of its density but also the way of sample preparation which is related to the structure formed in the preparation. Loose sand subjected to cyclic loading may liquefy under undrained condition but may be compacted to a denser state under drained condition (Asaoka, 2003). The strain history, also called as stress-induced anisotropy, has great influence on the behavior of undrained sand (Finn and Pickering, 1970). In the works by Ishihara and Okada (1978, 1982), the effect of pre-shearing on cyclic behavior of sand was carefully investigated and it is concluded that liquefaction resistance is dependent not only on the magnitude of the pre-shearing but also on its initial direction. As to the influence of stress-induced anisotropy, Oda et al., (2001) investigated the influence of orientation in the sedimentation of sand both on the stress-strain relation and the liquefaction resistance with macroscopic viewpoint, and the potentiality of re-liquefaction of sand that had experienced liquefaction previously with microscopic viewpoint. Kato et al. (2001) conducted a series of undrained triaxial compression tests on sand with anisotropic consolidation and found that the stress induced anisotropy does not affect the state variables at critical state or steady state but that stress-strain relation will be more contractive with larger anisotropy. Hyodo et al., (1994) conducted a series of tests on undrained sand with reversal and non-reversal cyclic loadings and found that non-reversal cyclic loadings may also cause large deformation and that initial deviatoric stress may increase phase transform strength where contractive behavior changes to dilative behavior.

Bidirectional simple shear tests were also conducted to investigate influence of superimposing cyclic shear stress in one direction on undrained behavior of sand subjected to monotonic loading in another direction in the works by Meneses et al. (1998). It is concluded that the magnitude and frequency of superimposing cyclic loading may affect the undrained behavior of sand and that loose sand is more susceptible to a small superposing cyclic loading than dense sand. Verdugo and Ishihara (1996) investigated systematically a so-called confining-stress dependency of sand in undrained triaxial test in which samples having the same void ratio but different confining stress were tested. It is concluded that critical state is not affected by initial
effective confining stress. Meanwhile, the critical state evaluated from drained tests is in good agreement with those evaluated from undrained tests.

Watanabe et al. (2008) evaluated three-dimensional displacement and strain properties of sand under drained triaxial compression test using X-ray CT, as shown in Figure 1-2. The strain localization in sand was visualized in three dimensions. According to the images, the shape of the shear band inside specimen could be well investigated using X-ray CT as a density change region.

Compared with undrained cyclic loading tests, drained cyclic loading tests are much fewer in literature, among which Hinokio (2000) conducted drained cyclic loading tests on dense sand under constant-mean-effective-stress. In the test, confining pressure of the sand was kept constant and a maximum principal stress ratio ($\sigma_1/\sigma_3$) was cyclically loaded up to 4.0. It is found that dense sand subjected to relative large cyclic shearing will contract to some extent but will not contract further even if the cyclic shearing continues.

![Force-displacement relationship](image)

(a) Force-displacement relationship

![Vertical cross-sectional images](image)

(b) Vertical cross-sectional images

Figure 1-2 Strain localization in sand obtained from X-ray CT (Watanabe et al., 2008)
Under undrained condition, it is commonly known that effective stress path is not affected by loading path such as conventional triaxial loading, true triaxial loading or plane-strain loading. Under drained condition, however, the loading path may greatly affect the mechanical behavior of sand. In the works by Nakai and Mihara (1984); Nakai (1989 & 2006); Nakai and Hinokio (2004), systematic investigation were conducted to clear out the drained behavior of sand subjected to very complicated loading paths with computer-controlled true triaxial loading device designed by the authors.

1.2.2 Constitutive models for sand and numerical simulations of boundary value problems

Above reviews just gave a brief description of the experimental researches on mechanical behavior of sand. Some researches related to constitutive model for liquefaction of sandy soils and corresponding numerical analysis using FEM can be found in the works by Oka and Ohno (1989) and Oka et al. (1992, 1999), in which a constitutive model for sand, using the kinematic hardening rule, was proposed and applied to BVP by a soil-water coupling finite difference-finite element (FD-FE) analysis with the code of LIQCA (Yashima et al., 1991; Oka et al., 1994).

In recent years, some developments in constitutive modeling should be emphasized, e.g., the concept of subloading proposed by Hashiguchi and Ueno (1977) and the concept of superloading proposed by Asaoka et al. (1998). These developments make it possible not only to describe remolded soils (Roscoe et al., 1963; Schofield and Wroth, 1968), but also naturally deposited soils in which the density, the stress-induced anisotropy and the structure of soils control the mechanical behaviors of soil.

K. Nakai (2008) reported a very interesting study on prediction and countermeasure of earthquake-induced consolidation deformation of river dike, with the code of GeoAsia (All Soils All States All Round Geo-analysis Integration, Asaoka et al., 2000,2002, Asaoka and Noda, 2007), where the analysis was carried out by FEM using an elasto-plastic constitutive model proposed by Asaoka et al. (2002) employing the collapse of soil skeleton structure (the concept of superloading), the loss in
overconsolidation (the concept of subloading), and the development of anisotropy during shearing in SYS Cam-clay model, the model that firstly proposed the concept of superloading by which the structure of soil formed in its deposition can be properly described. In the analysis, the settlement of the dike was caused mainly by the consolidation of the soils in which large excess pore-water pressure occurred due to cyclic loading in earthquake regardless the types of soils.

In the researches, influence of intermediate stress is carefully investigated which leads to the establishment of $t_{ij}$ models (Nakai, 1989). The stress-path dependency in general stress condition was properly described with subloading $t_{ij}$ model (Nakai and Hinokio, 2004).

Zhang et al. (2007) proposed a new constitutive model for sand in which apart from the concept of superloading related to the soil skeleton structure (Asaoka et al., 1998) and the concept of subloading related to the density (Hashiguchi and Ueno, 1977), the authors introduced a new approach to describe the stress-induced anisotropy. As a matter of fact, the concept of anisotropy was firstly formulated into a constitutive model by Sekiguchi (1977). Later, it was introduced to the extended subloading model by Hashiguchi and Chen (1998) using the expression of rotating hardening. Zhang et al. (2007) pointed out that the change of density or overconsolidation, is dependent not only on plastic stretching and elastic unloading, but also on the stress-induced anisotropy, which was formerly regarded as being only related to the plastic stretching when losing overconsolidation and the elastic unloading when gaining overconsolidation. Moreover, a natural limitation for the development of the stress-induced anisotropy was defined.

Based on the model, mechanical behavior of fictional sand subjected to different loadings in different drainage conditions are simulated to verify if the model is suitable to describe the general behavior of sand with one set of definite parameters. Particular attention was paid to the description of sand subjected to cyclic loading under undrained conditions, that is, for loose sand, liquefaction happens without transition from contractive state to dilative state; for medium dense sand, cyclic mobility occurs while for dense sand, liquefaction will not occur. Figure 1-3 shows element simulations of Toyoura sand at loose, medium-dense and relatively-dense states subjected to cyclic loading under undrained condition.
Based on the model proposed by Zhang et al. (2007), Ye et al. (2007) conducted a series of numerical analyses using a FEM code named as DBLEAVES (Ye, 2007), to simulate experimental results of shaking-table tests on saturated sandy ground with repeated liquefaction-consolidation process. In the tests, a sandy ground made of Toyoura sand in a laminate box placing on the shaking table was firstly shook to liquefaction and then the liquefied ground was allowed to settle without shaking due to the consolidation. The process of liquefaction-consolidation settlement was repeated three times to identify the mechanical behaviors of sand in different densities and stress histories (stress-induced anisotropy). In the simulation, the repeated process has been considered for repeated static and dynamic analyses using the same parameters of the constitutive model in all calculations, with a FD-FE scheme and considering soil-water coupled problems for both infinitesimal and finite deformation algorithms. It is known from the calculation that the numerical simulation is capable of reproducing uniquely the different responses of liquefied grounds with different densities and stress-induced anisotropy during repeated shaking and consolidation, which shows the potentiality of the method to solve the BVP for liquefaction of soil ground in a precise way.

Figure 1-3 Element simulation of sands with different densities ($p_0' = 298$ kPa)

1.2.3 Seismic behavior of group-pile foundation

In recent year, the limit state design method became predominant in the design of
foundations of bridges and other structures. The investigation on the mechanical behavior of pile foundations subjected to lateral cyclic loadings up to an ultimate state, therefore, it is very important to provide evidences for the designing method.

Numerous tests either in model scale or in real-scale on group-pile foundation subjected to lateral loading can be found in literatures in order to elucidate the ultimate state of the foundations during strong earthquakes, e.g., Tokimatsu et al. (2007) conducted the shaking table test using E-defense, one of the largest shaking table of 20×15m was built in 2005, to estimate the effects of dynamic soil-pile-structure interaction on pile. However, with such a large shaking table, when dynamic behavior of a whole structure including its foundation is examined, a prototype has to be scaled down due to the limitation of the shaking table’s capacity (Kawashima, 2008). Needless to say, a full-scale loading test is the most accurate way to determine the mechanical behaviors of deep foundations though it might be extremely expensive and time consuming.

On the other hand, numerical simulation also plays a very important role in determining the behaviors and a large number of numerical studies have been done in this field. Kimura et al. (1991) developed a three-dimensional static finite-element-analysis program GPILE-3D in which the stress-strain relationship of the ground is simulated with the Drucker-Prager model under a monotonic loading condition. Kimura and Zhang (1999) developed a three-dimensional static and dynamic finite element analysis code (DGPILE-3D) in order to investigate the static and dynamic interaction between soil and pile foundation. In the simulation of pile, the usual method in the finite element analysis is to use a beam element which does not have volume. However, in the finite element analysis of pile, the pile-size influence cannot be neglected, especially if the diameter of the pile is very large. Zhang et al. (2000a) have proposed a hybrid element that consists of both beam and solid elements, considering pile volume effects on soil-pile interaction. The beam element bears most of the force acting on the pile, while the neighboring solid elements bear less and exist as measures of the pile diameter effects. It is also reported that the adequate constitutive model adopted in the finite element analysis, such as $t_{ij}$ model which is composed of $t_{ij}$ clay model (Nakai and Matsuoka, 1986) and $t_{ij}$ sand model (Nakai, 1989), is crucially important to properly describe the mechanical behavior of a pile foundation and its
surrounding ground under cyclic loading condition. Zhang et al. (2000b) conducted a 3-D FEM analysis of a laterally cyclic-loaded, real scale 9-pile foundation based on an axial-force dependent hysteretic model for RC. In the analysis, the field measurements can be simulated to a quite accurate level. A new beam theory (AFD model) of a reinforced concrete material was proposed by Zhang et al. (2002). It is based on a weak form in which the axial-force dependency in the nonlinear moment-curvature relation is considered.

1.3 Outline of the dissertation

The outline of each chapter is described below.

Chapter 2 presents experimental study on sand under undrained cyclic loading. The effects of density of the specimen, confining stress, amplitude of cyclic loading and frequency are discussed in detail.

In Chapter 3, based on the model proposed by Zhang et al. (2007), a minor modification for the evolution equation of overconsolidation is carried out at first and then a unique description of the overall mechanical behaviors of Toyoura sand is confirmed. In the theoretical simulation, based on the conventional drained triaxial compression tests and undrained triaxial cyclic loading tests, the material parameters of Toyoura sand are determined. The capability of the model to describe uniquely the overall behaviors of the sand under different drainage and loading conditions with one set of fixed parameters is verified.

In Chapter 4, based on the modified elastoplastic model (Zhang et al., 2007), 2D and 3D FD-FE analyses considering the soil-water coupling problems are conducted to simulate element test of sand specimen subjected to cyclic load in undrained triaxial test. In the simulation, the specimen is considered as a boundary value problem, in which several factors affecting the nonuniformity of the soil specimen are taken into consideration in detail.

In Chapter 5, numerical simulation of real-scale group-pile foundation subjected to horizontal cyclic loading is conducted. In the analysis, nonlinear behaviors of ground and piles are described by subloading $t_{ij}$ model and AFD model which considered the
axial-force dependency in the nonlinear moment-curvature relation. It is conducted to verify the applicability of the proposed numerical method by comparing the numerical results with the experimental test results. In addition, numerical experiments on seismic performance of reinforcement by ground improvement around an existing pile foundation are also conducted to determine the optimum pattern in the size, the location and the shape of ground improvement zone.

The conclusions are summarized in Chapter 6. Future direction of this research is discussed.
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Kobe city homepage:
http://www.city.kobe.lg.jp/safety/hanshinawaji/data/photo/p-index.html


Photographs and motion picture of the Niigata City immediately after the earthquake in 1964 (2004): published by the Japanese Geotechnical Society.


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CHAPTER 2 Experimental Study on Sand under Undrained Cyclic Loading

2.1 Introduction

During earthquake, liquefaction is one of the most dangerous threats to the structures constructed on the sandy ground. Due to the strong motion during an earthquake, sandy ground may liquefy and cause great disasters. For example, footings of structure may settle, buildings may tilt, and light structures buried within the ground may float out. To avoid or reduce the damages caused by liquefaction, researchers have made great efforts trying to clear out the mechanical behaviors of soil subjected to cyclic loading. Undrained cyclic loading test is one of the useful ways which is widely used in geotechnical engineering to predict the stress-strain behavior of saturated sand under cyclic loading.

In this chapter, experimental study under undrained cyclic loading is conducted to confirm the behavior of saturated sand. The effect of density of the specimen, confining stress, amplitude of cyclic loading and frequency are discussed in detail. The precision and the reproducibility of the test method are verified as well.

2.2 Test device and procedures

2.2.1 Triaxial cyclic loading test device and preparation of specimen

The material of specimen used in this experiment is Toyoura sand, composed mainly of subangular quartz and feldspar, whose particle size is extremely uniform and is widely used in geotechnical experiments in Japan. The physical properties of Toyoura sand are shown in Table 2-1 and the grading distribution curve is shown in Figure 2-1.

Saturated specimens, 10 cm in height and 5 cm in diameter, are prepared in a medium dense state of packing by a method of sedimentation. Freshly boiled saturated sand is poured using a spoon into a split mold filled with de-aired water and compacted in ten and seven layers. By this procedure, the void ratios between 0.73 and 0.80 (Dr=49
are obtained in the forming mold. After each specimen is set up in the cell with 3 kN/m$^2$ vacuum, the confining stress is raised to 49, 98, 196 or 294 kN/m$^2$. In all the tests in this chapter, the effective confining stress at the start of shear tests is 49, 98, 196 or 294 kN/m$^2$. This pressure is maintained for 40 minutes to ensure full consolidation. Continue to increase back pressure and confining pressure until pore pressure response indicates a B value of at least 0.97. A scheme of the used triaxial device is presented in Figure 2-2.

In the experiments, detailed method followed ‘Laboratory soil testing methods (2001)’ and ‘Method for cyclic undrained triaxial test on soils: JGS 0541-2000’ which is a standard established for laboratory testing method by the Japanese Geotechnical Society.

![Graph showing particle size distribution curve of Toyoura Sand](image)

**Figure 2-1** Particle size distribution curve of Toyoura Sand

| Specific gravity of soil $G_s$ (g/cm$^3$) | 2.65 | Uniformity coefficient $U$ | 1.37 |
| Maximum grain size (mm) | 0.425 | Maximum density, $\rho_{max}$ (g/cm$^3$) | 1.647 |
| Minimum grain size (mm) | 0.102 | Minimum density, $\rho_{min}$ (g/cm$^3$) | 1.347 |
| 60% grain size (mm) | 0.281 | Maximum void ratio, $e_{max}$ | 0.967 |
| 30% grain size (mm) | 0.241 | Minimum void ratio, $e_{min}$ | 0.609 |
| 10% grain size (mm) | 0.206 |
2.2.2 Confirmation of test precision

In the previous research, Yamaguchi (2009) conducted undrained cyclic loading tests with the same machine and sand material used in this research. A data logger which is an electronic device, model number of TDS-302, is used for recording the data over time in the tests. In his research, it was confirmed the degree of accuracy on the test machine, as shown in Figure 2-3. The plotting data at frequency loading 0.1 Hz is very coarse to describe the exact stress-strain behavior. On the other hand, the plotting data at 0.01 Hz is very clear to present the deviator stress wave. Because typically data logger has slower sample rates, it is not appropriate to record high frequency. The maximum loading frequency can measure with a data logger is established as 0.05 Hz in the experiment. It can also be described the accurate wave under high confining stress 198 kPa in the low loading frequency, as shown in Figure 2-3(d).

The A/D board (model PCI-3165) having 16 channels and A/D converter is newly adapted in this laboratory test for collecting experimental data having high sample rates. The data obtained from the same test at 0.1 Hz are organized by the data logger and the A/D board, shown in Figure 2-4. It should be pointed out that, in the figures throughout this paper, deviator stress means stress difference $q$ ($\sigma_1 - \sigma_3$) and shear strain $\epsilon_s$ means axial strain. The plotting data organized by the data logger is very coarse to describe the exact stress-strain behavior, as shown in Figure 2-4(a). On the other hand, the degree of precision obtained by the A/D board is quite good, although the volume of data is large.
(a) $f=0.1$ Hz, $p'=98$ kPa

(b) $f=0.05$ Hz, $p'=98$ kPa

(c) $f=0.01$ Hz, $p'=98$ kPa

(d) $f=0.01$ Hz, $p'=196$ kPa

Figure 2-3 Time histories of deviator stress (Yamaguchi, 2009)

(a) Test result obtained from the data logger at 0.1 Hz

(b) Test result obtained from the A/D board at 0.1 Hz

Figure 2-4 Comparison of the same test result organized by data logger and A/D board
Every 0.1, 0.2 and 0.6 sec is read out to confirm the effect of output time interval, as shown in Figure 2-5. All cases shown in the figure are the same test result at 0.1 Hz organized by A/D board. In the stress-strain relation of 0.6 sec, the plot lines near small range of deviator stress show relatively inaccurate behavior. Therefore, throughout this research, time interval 0.2 sec that showed satisfactory results is used in the subsequent tests for decreasing the volume of data. Detailed specifications of two types of data recording devices are listed in Appendix 1.
2.3 Discussion of the test results

2.3.1 Influence of initial void ratio

In this verse, the influence of initial void ratio on the mechanical behaviors of soil is discussed. The frequency of the cyclic loading is 0.1 Hz. The detail conditions of the test are listed in Table 2B2. \( D_r \) is the relative density that is commonly identified as a principal factor influencing the deformation and strength characteristics of sand. \( N_c \) means that number of cycles shear strain corresponds to 5\% in double amplitude. Figure 2-6 shows the stress paths, stress-strain relations under confining stress of 196 kPa. The initial void ratios of relatively loose and dense sands are from 0.88 to 0.75. The loose sand is reconstituted without tamping. It is clear from the figure that the effective stress of loose sand substantially decreases in the first cycle having a few numbers of cycles before cyclic mobility occurs.

Another test with different initial density is conducted under confining stress of 294 kPa. The detail conditions of the test are listed in Table 2-3. The initial void ratios of relatively loose and dense sands are 0.83 and 0.72, respectively. Figure 2-7 shows the stress paths, stress-strain relations under confining stress 294 kPa. It is known from the figure that relatively loose sand is easy to liquefy. The number of cycles shear strain corresponds to 5\% in double amplitude \( N_c \) of the relatively loose sand is more smaller than those of the dense sand.

Furthermore, a very dense sand specimen having initial void ratio less than 0.7 and a very loose sand specimen having initial void ratio more than 0.9 have been made in this research. Moist placement method which is the air inside the specimen replaces \( \text{CO}_2 \) before passing water is used when making a very loose specimen. The mechanical behaviors of very dense sand and very loose sand specimen will be discussed in the future study.

| \( q/\sigma_0 \) | 0.20 | 0.20 | 0.20 |
| \( \sigma_3 \) (kPa) | 196.60 | 195.23 | 195.90 |
| \( f \) (Hz) | 0.1 | 0.1 | 0.1 |
| Initial void ratio \( e_0 \) | 0.88 | 0.80 | 0.75 |
| \( D_r \) (%) | 25 | 49 | 62 |
| \( N_c \) (DA=5\%) | 2.1 | 4.64 | 15.52 |
(a) Loose sand (e_0=0.88)

(b) Medium loose sand (e_0=0.80)

(b) Medium dense sand (e_0=0.75)

Figure 2-6 Influence of initial void ratio under confining stress 196 kPa

Table 2-3 Conditions of the test shown in Figure 2-7

<table>
<thead>
<tr>
<th></th>
<th>Figure 7 (a)</th>
<th>Figure 7 (b)</th>
</tr>
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<tbody>
<tr>
<td>q/2e_0</td>
<td>0.2</td>
<td>0.20</td>
</tr>
<tr>
<td>σ_3 (kPa)</td>
<td>294.21</td>
<td>294.68</td>
</tr>
<tr>
<td>f (Hz)</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial void ratio e_0</td>
<td>0.83</td>
<td>0.72</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>39</td>
<td>70</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
<td>21.18</td>
<td>108.34</td>
</tr>
</tbody>
</table>
2.3.2 Influence of confining stress

The influence of confining stress on the mechanical behaviors of soil is discussed in detail. Table 2-4 lists the conditions of the tests shown in Figure 2-8. The experiments conducted under two different confining stresses 98 kPa and 196 kPa having the same shear stress ratio 0.15. Figure 2-8 present the stress paths, stress-strain relations. Table 2-5 lists the conditions of the tests shown in Figure 2-9. The experiments conducted under four different confining stresses from 49 kPa to 294 kPa having the same shear stress ratio 0.2. Table 2-6 lists the conditions of the tests shown in Figure 2-10. The experiments conducted under two different confining stresses 98 kPa and 196 kPa having the same shear stress ratio 0.25. From the results, it is clear that the results under low confining stress have smaller number of cycles \( N_c \) compared to the results under high confining stress. It means that specimen under low confining stress is easy to liquefy with a large strain. This tendency can be observed no matter what the shear stress ratio is.

Figure 2-7 Influence of initial void ratio under confining stress 294 kPa
Table 2-4 Conditions of the test shown in Figures 2-8

<table>
<thead>
<tr>
<th></th>
<th>Figure 8 (a)</th>
<th>Figure 8 (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>98.21</td>
<td>196.42</td>
</tr>
<tr>
<td>$f$ (Hz)</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.76</td>
<td>0.79</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>60</td>
<td>50</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
<td>34.96</td>
<td>43.42</td>
</tr>
</tbody>
</table>

(a) $\sigma_3= 98$ kPa

(b) $\sigma_3= 196$ kPa

Figure 2-8 Influence of confining stress ($q/2\sigma_0= 0.15$)

Table 2-5 Conditions of the test shown in Figures 2-9

<table>
<thead>
<tr>
<th></th>
<th>Figure 9 (a)</th>
<th>Figure 9 (b)</th>
<th>Figure 9 (a)</th>
<th>Figure 9 (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>49.95</td>
<td>98.78</td>
<td>195.90</td>
<td>294.68</td>
</tr>
<tr>
<td>$f$ (Hz)</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.76</td>
<td>0.76</td>
<td>0.75</td>
<td>0.72</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>59</td>
<td>60</td>
<td>62</td>
<td>70</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
<td>5.12</td>
<td>12.48</td>
<td>15.52</td>
<td>108.34</td>
</tr>
</tbody>
</table>
(a) $\sigma_3 = 49$ kPa

(b) $\sigma_3 = 98$ kPa

(c) $\sigma_3 = 196$ kPa

(d) $\sigma_3 = 294$ kPa

Figure 2-9 Influence of confining stress ($q/2\sigma_0 = 0.20$)
Table 2-6 Conditions of the test shown in Figures 2-10

<table>
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<tr>
<th></th>
<th>Figure 10 (a)</th>
<th>Figure 10 (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q/2\sigma_0 )</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>( \sigma_3 ) (kPa)</td>
<td>98.18</td>
<td>198.30</td>
</tr>
<tr>
<td>( f ) (Hz)</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial void ratio ( e_0 )</td>
<td>0.73</td>
<td>0.77</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>69</td>
<td>57</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
<td>4.6</td>
<td>11.1</td>
</tr>
</tbody>
</table>

![Graph showing influence of confining stress](image)

(a) \( \sigma_3 = 98 \text{ kPa} \)

(b) \( \sigma_3 = 196 \text{ kPa} \)

Figure 2-10 Influence of confining stress \((q/2\sigma_0=0.25)\)
2.3.3 Influence of amplitude (q/2 $\sigma_0$) of cyclic loading

The influence of amplitude of cyclic loading on the mechanical behaviors of soil is discussed, in this verse. Table 2-7 lists the conditions of the test shown in Figure 2-11. The tests are conducted under the same confining stress 98 kPa and cyclic loading frequency 0.01 Hz at three different amplitudes of shear stress ratios 0.15, 0.20 and 0.25. Figure 2-11 shows the stress paths, stress-strain relations. The higher the shear stress ratio is, the quicker the soil specimen enters into the cyclic mobility loop. Cyclic mobility occurs even if the specimen liquefies during the first loading cycle at 0.25 amplitude of cyclic loading. Moreover, it seems that the amplitude of shear stress ratio has no effect on the occurrence of shear strain during the test. It is because the shear strains occur irrespective of amplitude of cyclic loading.

Table 2-8 lists the conditions of the tests shown in Figure 2-12. Figure 2-12 shows stress paths, stress-strain relations at the 0.1 Hz under confining stress 98 kPa. The other conditions are the same as Table 2-7. The stress paths indicate that the results at high frequency are comparable to that of the low frequency except for a minimal difference in the occurrence of large shear strain on the compression side.

Table 2-9 lists the conditions of the tests shown in Figure 2-13. Figure 2-13 shows stress paths, stress-strain relations at the 0.1 Hz under confining stress 196 kPa. Same tendency can be seen in the results under high confining stress. The higher the shear stress ratio is, more quickly the soil specimen goes into cyclic mobility loop.
Table 2-7 Conditions of the test shown in Figure 2-11

<table>
<thead>
<tr>
<th></th>
<th>Figure 11 (a)</th>
<th>Figure 11 (b)</th>
<th>Figure 11 (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$</td>
<td>0.15</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>98</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>$f$ (Hz)</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>63</td>
<td>61</td>
<td>62</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
<td>62</td>
<td>44</td>
<td>23</td>
</tr>
</tbody>
</table>

Figure 2-11 Influence of amplitude ($q/2\sigma_0$) of cyclic loading at 0.01Hz under $\sigma_3=98$ kPa
Table 2-8 Conditions of the test shown in Figure 2-12

<table>
<thead>
<tr>
<th></th>
<th>Figure 12 (a)</th>
<th>Figure 12 (b)</th>
<th>Figure 12 (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$</td>
<td>0.15</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>98.2</td>
<td>98.78</td>
<td>98.2</td>
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<tr>
<td>$f$ (Hz)</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.76</td>
<td>0.76</td>
<td>0.73</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>60</td>
<td>60</td>
<td>69</td>
</tr>
<tr>
<td>$N_c$ (DA=5%)</td>
<td>34.96</td>
<td>12.48</td>
<td>4.6</td>
</tr>
</tbody>
</table>

(a) $q/2\sigma_0= 0.15$

(b) $q/2\sigma_0= 0.20$

(c) $q/2\sigma_0= 0.25$

Figure 2-12 Influence of amplitude ($q/2\sigma_0$) of cyclic loading at 0.1Hz under $\sigma_3=98$ kPa
Table 2-9 Conditions of the test shown in Figure 2-13

<table>
<thead>
<tr>
<th></th>
<th>Figure 13 (a)</th>
<th>Figure 13 (b)</th>
<th>Figure 13 (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$</td>
<td>0.15</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>196.42</td>
<td>195.90</td>
<td>198.30</td>
</tr>
<tr>
<td>$f$ (Hz)</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.79</td>
<td>0.75</td>
<td>0.77</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>50</td>
<td>62</td>
<td>57</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
<td>43.42</td>
<td>15.52</td>
<td>11.1</td>
</tr>
</tbody>
</table>

Figure 2-13 Influence of amplitude ($q/2\sigma_0$) of cyclic loading at 0.1Hz under $\sigma_3=196$ kPa
2.3.4 Influence of loading frequency

In this verse, influence of loading frequency on the mechanical behaviors of soil is discussed. Table 2-10 lists the conditions of the tests shown in Figure 2-14. The experiments are conducted under same confining stress 98 kPa at different loading frequencies between 0.01 Hz and 0.5 Hz. Figure 2-14 shows the stress paths and stress-strain relations. Though the effect of loading frequency cannot be seen in the stress paths, it can be seen that at a high frequency of cyclic loading (0.5 Hz), the deviatoric stress decreased after the stress path entered the cyclic mobility, in spite of the fact that external load is still kept constant. It is known from the figure that this phenomenon cannot be observed in the results of low cyclic loading frequency (0.01 Hz & 0.1 Hz).

The same tendencies are observed in related studies, e.g. Furuta (2003). He conducted an experimental study on Kasumigaura sand under undrained cyclic loading. The confining stress was 49 kPa, and Dc was 70% in the test. Figure 2-15 shows stress path and stress-strain relations at 0.01 Hz and 0.5 Hz. It is known that the phenomenon, the deviatoric stress becomes small after the stress path entered into the cyclic mobility, cannot be observed in the results at cyclic loading at 0.01 Hz shown in Figure 2-15(a) while the deviatoric stress decreased after stress path entered into cyclic mobility at high frequency of cyclic loading 0.5 Hz. However, he did not mention anything about the phenomenon.

On the other hand, it is clarified from the test results under the confining stress 196 kPa. The phenomenon, the deviatoric stress decreases after the stress path enter the cyclic mobility, can not be observed at the high confining stress, as shown in Figure 2-16. The reason why this phenomenon happens under the low confining stress should be explained in the future study.
Table 2-10 Conditions of the test shown in Figure 2-14

<table>
<thead>
<tr>
<th></th>
<th>Figure 14 (a)</th>
<th>Figure 14 (b)</th>
<th>Figure 14 (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>98.00</td>
<td>98.78</td>
<td>100.14</td>
</tr>
<tr>
<td>$f$ (Hz)</td>
<td>0.01</td>
<td>0.1</td>
<td>0.5</td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.75</td>
<td>0.76</td>
<td>0.76</td>
</tr>
<tr>
<td>$Dr$ (%)</td>
<td>61</td>
<td>60</td>
<td>58</td>
</tr>
<tr>
<td>$Nc$ (DA=5%)</td>
<td>44</td>
<td>12.48</td>
<td>37</td>
</tr>
</tbody>
</table>

**Figure 14** (a) $f=0.01$ Hz

**Figure 14** (b) $f=0.1$ Hz

**Figure 14** (c) $f=0.5$ Hz

*Figure 2-14 Influence of loading frequency*
Figure 2B15 Test results of Kasumigaura sand at different frequencies

(Furuta, 2003)
### Table 2-11 Conditions of the test shown in Figure 2-16

<table>
<thead>
<tr>
<th></th>
<th>Figure 16 (a)</th>
<th>Figure 16 (b)</th>
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</thead>
<tbody>
<tr>
<td>$q/2\sigma_0$ (kPa)</td>
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<td>0.2</td>
</tr>
<tr>
<td>$\sigma_1$ (kPa)</td>
<td>196.4</td>
<td>197.79</td>
</tr>
<tr>
<td>$f$ (Hz)</td>
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</tr>
<tr>
<td>Initial void ratio $e_0$</td>
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<td>0.73</td>
</tr>
<tr>
<td>Dr (%)</td>
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<td>67</td>
</tr>
<tr>
<td>Nc (DA=5%)</td>
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<td>8.6</td>
</tr>
</tbody>
</table>

![Graphs](image)

(a) $f=0.5$ Hz

(b) $f=0.5$ Hz

**Figure 2-16 Influence of loading frequency**

### 2.4 Summary

In this chapter, experimental study on sand under undrained cyclic loading is conducted to confirm the behavior of saturated sand. The precision and the reproducibility of the test method are verified as well. The following conclusions can be given:

1. The A/D board is newly adapted in the laboratory test for collecting experimental data with high sample rates. The plotting data organized by data logger is very
coarse to describe the exact stress-strain behavior. However, the degree of precision obtained by the A/D board is quite good, although the volume of data is large.

2. Influence of initial void ratio on the mechanical behaviors of soil is discussed. It is clear that the effective stress of loose sand substantially decreases in the first cycle having a few numbers of cycles before cyclic mobility occurs. The same tendency can be seen in the results regardless of the confining stress.

3. Influence of confining stress on the mechanical behaviors of soil is discussed under four confining stresses from 49 kPa to 294 kPa. It is clear that the results under low confining stress have smaller $N_c$ compared to the results under high confining stress. It means that specimen under low confining stress is easy to liquefy with a large strain. This tendency can be observed no matter what the shear stress ratio is.

4. Laboratory tests are conducted to confirm the influence of amplitude of cyclic loading on the mechanical behaviors of soil at three different amplitude of shear stress ratio 0.15, 0.20 and 0.25. The higher the shear stress ratio is, the quicker the soil specimen goes into cyclic mobility loop. Cyclic mobility occurs even if the specimen liquefies during the first loading cycle at 0.25 amplitude of cyclic loading. Moreover, it seems that the amplitude of shear stress ratio has no effect on the occurrence of shear strain during the test. It is because the shear strains occur irrespective of amplitude of cyclic loading.

5. Influence of the loading frequency on the mechanical behaviors of soil is discussed at loading frequency ranging between 0.01 Hz and 0.5 Hz. It can be seen that at a high frequency of cyclic loading 0.5 Hz under low confining stress, the deviatory stress decreased after stress path entered into cyclic mobility, in spite of the fact that external load is still kept constant. It is also known that the results at low loading frequency and 0.5 Hz under high confining stress, that this phenomenon cannot be observed. The reason why this phenomenon happens should be explained for future study.

6. The mechanical behaviors of very dense sand specimen having initial void ratio less than 0.7 and very loose sand specimen having initial void ratio more than 0.9 will be discussed in the future study. Furthermore, it will be discussed that nonuniformity of the specimen and the influence of the nonuniformity on the overall mechanical behavior of the soil specimen.
References


CHAPTER 3  A Unified Description of Toyoura Sand

3.1 Introduction

Mechanical behavior of clean sand has been investigated for years and so many reports on this topic can be found in literatures that it is hard to list them completely within limited pages of references. The reason why so many researchers have spent much effort to learn it, is that various mechanical behaviors of clean sand are dependent not only on shape of particles, angular or round, but also on its density, strain history, and even on degree of structure formed in its deposition (Asaoka et al., 1998). Sand may behave totally differently under different above-mentioned conditions.

Zhang et al. (2007) proposed a new constitutive model for sand in which in addition to the concept of superloading related to the soil skeleton structure (Asaoka et al., 1998) and the concept of subloading related to the density (Hashiguchi and Ueno, 1977), the authors introduced a new approach to describe the stress-induced anisotropy.

In this chapter, though it might be a little ambitious, the author tries to use the model (Zhang et al., 2007), with a minor change for the evolution equation of overconsolidation, to describe the overall mechanical behaviors of Toyoura sand (TS), a typical clean sand, in a unified way in which the eight parameters for describing a sand will be constant no matter what kinds of loadings or drainage conditions may be.

3.2 Modification of original constitutive model

In the original model proposed by Zhang et al. (2007), apart from the concepts of subloading (Hashiguchi and Ueno, 1977) and superloading (Asaoka et al., 1998), a new evolution rule for the development of overconsolidation was proposed. Here we use the words of ‘development of overconsolidation’ instead of ‘loss of overconsolidation’ just want to emphasize that during plastic loading the degree of overconsolidation sometimes may even increase, not always the case in which overconsolidation only develops in elastic unloading process. In the original model, the changing rate of overconsolidation is assumed to be controlled by two factors, one is the plastic component of stretching that was employed as the only factor in the work by Asaoka et
al. (2002), and the other is the increment in anisotropy, namely
\[
\dot{R} = JU \left\| \mathbf{D}^p \right\|^\frac{3}{2} \frac{R}{MD} \frac{\partial f}{\partial \mathbf{\beta}} \dot{\mathbf{\beta}}
\]  

(1)
in which, \( \dot{\mathbf{\beta}} \) is proportional to the norm of the plastic component of stretching \( \left\| \mathbf{D}^p \right\| \), and \( U \) is given by the following relation as:

\[
U = -\frac{m}{D} \left( \frac{p'}{p_0} \right)^2 \ln R \quad (p'_0 = 98.0 \text{ kPa, reference stress})
\]  

(2)

The problem arisen in Equation(2), however, is that when a confining stress becomes very high, then the value of \( \left( \frac{p'}{p_0} \right)^2 \) will be even much larger, which may result in a very quick development of overconsolidation in large confining stress. Obviously this is not a realistic behavior. To solve this problem, a new function is proposed in the paper as shown in the following relation:

\[
U = -\frac{m}{D} \left( \frac{(p'/p_0)^2}{(p'/p_0)^2 + 1} \right) \ln R \quad (p'_0 = 98.0 \text{ kPa, reference stress})
\]  

(3)
in which, the function \( y = \frac{x^2}{(x^2 + 1)} \) satisfying the requirement that when \( x \) is close to zero, \( y = x^2 \) while if \( x \) is very large, \( y \) will be equal to 1. This modification can then solve the problem perfectly without affecting the performance of the model in describing the cyclic mobility (Zhang et al., 2007). A brief description of the model can be found in Appendix II.

3.3 Simulation of Toyoura sand in different loadings and drainage conditions

Among the eight parameters involved in the model, five parameters, \( M, N, \lambda, \kappa, \) and \( \nu \) are the same as in the Cam-clay model. The other three parameters and their functions are listed below,

- \( a \) : parameter that controls the collapse rate of structure
- \( m \) : parameter that controls the losing rate of overconsolidation
- \( b_r \) : parameter that controls the developing rate of stress-induced anisotropy

These three parameters have clear physical meanings and can be determined by
undrained triaxial cyclic loading tests and drained triaxial compression tests. In particular, the first two parameters are exactly the same as those in the model proposed by Asaoka et al. (2002). Therefore the method to determine them is also the same. Parameter $b_r$ can be determined based on the developing rate of the stress-induced anisotropy when the soil is subjected to shearing or compression. The higher the developing rate is, the larger the parameter $b_r$ will be. Detailed description about the physical meanings of the structure, overconsolidation and stress-induced anisotropy can be referred to the work by Asaoka et al. (2002) and Zhang et al. (2007).

3.3.1 Determination of the material parameters of Toyoura sand

Undrained triaxial cyclic loading tests and its simulation

In this verse, the material parameters of TS are determined in above-mentioned tests. The test conditions are listed in Table 3-1. Cyclic loading with sine wave is applied with 0.01 Hz under confining stress of 98 kPa and the sand samples are prepared with the method of sedimentation within water. Loose specimen was prepared by depositing the saturated sand slowly in de-aired water using a funnel with an opening of 3mm. Medium dense specimen was prepared by pouring the saturated sand into a mold in several layers, each of which is compacted with a 6mm-diameter rod in prescribed number of times.

Figure 3-1 shows experimental results of undrained triaxial cyclic loading test with three different amplitude of cyclic loading ratios $q/2p_0$ ($= 0.15, 0.20$ and $0.25$), in which $p'$ is mean effective stress and $q$ is stress difference. It is seen from the test results that because the samples are medium dense sand, cyclic mobility dose happen in all three tests and that the larger the cyclic loading ratio is, the faster the samples come into the cyclic mobility loop, an typical behavior that can be found in any literature available.

Figure 3-2 shows the theoretical simulations of the undrained triaxial cyclic loading test with different cyclic loading ratios. The effective stress paths and stress-strain relations predicted the model are qualitatively coincident with the test ones in general, while the cyclic number necessary for causing cyclic mobility is less than the test results.
Table 3-1 Test conditions of TS specimens subjected to cyclic loadings with different amplitudes

<table>
<thead>
<tr>
<th>Amplitude of shear stress ratio ($q/2p'$)</th>
<th>0.15</th>
<th>0.2</th>
<th>0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.748 (Dr=0.62)</td>
<td>0.753 (Dr=0.61)</td>
<td>0.77 (Dr=0.56)</td>
</tr>
<tr>
<td>Initial mean effective stress $p'$ (kPa)</td>
<td>98</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Cyclic loading frequency $f$ (Hz)</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Note: $e_{\text{max}}=0.97$; $e_{\text{min}}=0.61$

Figure 3-1 Test results of undrained triaxial cyclic loading test with different shearing ratios

(a) $q/2p'=0.15$  
(b) $q/2p'=0.20$  
(c) $q/2p'=0.25$

Figure 3-2 Theoretical simulations of undrained triaxial cyclic loading test with different cyclic loading ratios

(a) $q/2p'=0.15$  
(b) $q/2p'=0.20$  
(c) $q/2p'=0.25$
Table 3B2 Material parameters of TS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression index $\lambda$</td>
<td>0.05</td>
</tr>
<tr>
<td>Swelling index $\kappa$</td>
<td>0.0064</td>
</tr>
<tr>
<td>Critical state parameter $\Omega$</td>
<td>1.30</td>
</tr>
<tr>
<td>Void ratio $N (p''=98$ kPa on N.C.L.)</td>
<td>0.87</td>
</tr>
<tr>
<td>Poisson’s ratio $\nu$</td>
<td>0.30</td>
</tr>
<tr>
<td>Degradation parameter of overconsolidation state $m$</td>
<td>0.01</td>
</tr>
<tr>
<td>Degradation parameter of structure $a$</td>
<td>0.5</td>
</tr>
<tr>
<td>Evolution parameter of anisotropy $b_{r}$</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table 3B3 Initial conditions of TS samples subjected to cyclic loadings with different amplitudes

<table>
<thead>
<tr>
<th>Amplitude of shear stress ratio $(q/2p'')$</th>
<th>0.15</th>
<th>0.2</th>
<th>0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.81</td>
<td>0.81</td>
<td>0.81</td>
</tr>
<tr>
<td>Initial mean effective stress $p'$ (kPa)</td>
<td>98</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Initial degree of structure $R_0^*$</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>Initial degree of overconsolidation $1/R_0$</td>
<td>70</td>
<td>70</td>
<td>70</td>
</tr>
<tr>
<td>Initial anisotropy $\zeta_0$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

In the simulation, the set of material parameters of TS with unique value, listed in Table 3-2, are determined with above cyclic loading tests and drained compression tests (Nakai and Hinokio, 2004). The values of the parameters, except $a$ and $m$, are the same as those in the works by Ye et al. (2007). Parameter $a$ is determined to be 0.5 while parameter $m$ is determined to be 0.01. Detailed way to determine the parameters can be found in the work by Ye (2007).

Compared to the material parameters, it is rather difficult to determine initial conditions of state parameters for sand because the values are dependent not only on present stress state but also on its history. The initial conditions of the state parameters of TS are listed in Table 3-3. Initial anisotropy $\zeta_0$ is assumed to be zero, which means that the sand is isotropic at the beginning of shearing. Structure $R^*$ which influenced by the process in depositing the sand, usually collapses quickly during shearing and never recover. Therefore initial degree of structure $R_0^*$ is assumed to be a relatively large value $R_0^*=0.75$ for the medium dense sand. The initial degree of overconsolidation $1/R_0$ is determined to be 70 according to its initial void ratio.
Drained triaxial compression test and its simulation

Simulation of drained triaxial compression test under constant mean principal stress is conducted to determine the material parameters. In the test (Nakai et al., 2004), the initial void ratio of loose sand is $e_0=0.851$ and medium dense sand is $e_0=0.666$.

It is, however, very difficult to identify the reference void ratio $N$ in a small confining stress condition. Therefore, by extending the $e$-ln $p$ relation to small stress range, the reference void ratio $N$ of TS is determined to be 0.87. Combined with the undrained triaxial cyclic loading test in previous section, the material parameters of TS are fully determined and listed also in Table 3-2. The initial values of the state parameters for the drained triaxial compression test, however, are different from the cyclic loading test and listed in Table 3-4.

Figure 3-3 shows the comparison of experimental and theoretical results of triaxial compression test with different densities. The test result of loose sand, as shown in Figure 3-3(a), is reproduced quite well quantitatively. The result of dense sand, as shown in Figure 3-3(b), is reproduced relatively well before peak strength while in the residual state, a discrepancy between the test and the theory exists. On the whole, however, the model can describe the behaviors of the sand to some extent in these tests.

| Table 3-4 Initial conditions of TS samples with different densities |
|---|---|---|
| | Loose sand | Dense sand |
| Initial void ratio $e_0$ | 0.78 | 0.69 |
| Initial mean effective stress $p^'(kPa)$ | 196 | 196 |
| Initial degree of structure $R_0^*$ | 0.99 | 0.99 |
| Initial degree of overconsolidation $1/R_0$ | 4.0 | 30.0 |
| Initial anisotropy $\zeta_0$ | 0.0 | 0.0 |
3.3.2 Numerical prediction of sands with different densities

The behavior of sand is known to be dependent on its density. In order to verify the influence of the initial density on the behavior of sand, we now consider numerically a set of sands with different densities which is prepared from a very loose sand using the same method proposed by Asaoka (2003) and Nakai K. (2005). Table 3-5 lists the initial conditions of the loose sand before compaction. From the initial values of $R_r$, $R_r^*$, and $ζ_r$, it is understood that the sand is originally normally consolidated and highly structured loose sand without stress-induced anisotropy and with a very large void ratio. In preparing the set of sand samples with different densities, the very loose sand is compacted by a small vibration load along vertical direction with amplitude of 2.3 kPa under a small confining pressure of 10 kPa. After the compaction, these sands with different densities are isotropically consolidated to a prescribed confining pressure of 294 kPa. The set of sands with eight different densities are prepared by different numbers of vibration compaction, as shown in Figure 3-4. Table 3-6 lists the compaction numbers for preparing the sands with different densities and the state variables of these sands after they are isotropically consolidated to the confining pressure of 294 kPa.

By using the material parameters listed in Table 3-2 and the initial values of the state variables for the sands with different densities listed in Table 3-6, various kinds of triaxial tests under drained/undrained conditions subjected to monotonic and cyclic loading, are calculated systematically in the following sections.
Table 3-5 Initial conditions of TS samples before compaction

<table>
<thead>
<tr>
<th>Initial void ratio $e_0$</th>
<th>1.19</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial mean effective stress $p'$ (kPa)</td>
<td>10.0</td>
</tr>
<tr>
<td>Initial degree of structure $R_0^*$</td>
<td>0.1</td>
</tr>
<tr>
<td>Initial degree of overconsolidation $1/R_0$</td>
<td>1.0</td>
</tr>
<tr>
<td>Initial anisotropy $\zeta_0$</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Figure 3-4 Set of sands with different densities prepared from loose sand by vibration compaction and isotropic compression

Table 3-6 Initial conditions of TS samples prepared by compaction and isotropic consolidation

<table>
<thead>
<tr>
<th>No.</th>
<th>Vibration number $n$</th>
<th>Initial void ratio $e_0$</th>
<th>Initial degree of overconsolidation OCR</th>
<th>Initial degree of structure $R_0^*$</th>
<th>Initial anisotropy $\zeta_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[1]</td>
<td>2</td>
<td>0.903</td>
<td>1.30</td>
<td>0.103</td>
<td>6.44E-06</td>
</tr>
<tr>
<td>[2]</td>
<td>15</td>
<td>0.842</td>
<td>4.77</td>
<td>0.113</td>
<td>5.28E-06</td>
</tr>
<tr>
<td>[3]</td>
<td>30</td>
<td>0.785</td>
<td>15.99</td>
<td>0.124</td>
<td>7.30E-06</td>
</tr>
<tr>
<td>[4]</td>
<td>35</td>
<td>0.771</td>
<td>21.70</td>
<td>0.128</td>
<td>1.01E-05</td>
</tr>
<tr>
<td>[5]</td>
<td>40</td>
<td>0.758</td>
<td>28.14</td>
<td>0.132</td>
<td>1.70E-05</td>
</tr>
<tr>
<td>[6]</td>
<td>50</td>
<td>0.737</td>
<td>42.96</td>
<td>0.139</td>
<td>8.13E-05</td>
</tr>
<tr>
<td>[7]</td>
<td>70</td>
<td>0.707</td>
<td>78.88</td>
<td>0.152</td>
<td>1.75E-03</td>
</tr>
<tr>
<td>[8]</td>
<td>120</td>
<td>0.660</td>
<td>208.47</td>
<td>0.168</td>
<td>2.64E-02</td>
</tr>
</tbody>
</table>
**Mechanical behaviors of sand subjected to undrained/drained cyclic loading**

The eight sands with different densities listed in Table 3-6 are simulated in cyclic loading tests under confining stress of 294 kPa. The amplitude of the cyclic loading in shear stress ratio \(q/2p'_o\) is 0.12.

Figure 3-5 shows the stress paths and stress-strain relations of the sands with different densities in undrained tests. It is clear from the figures that very loose sands (No.1&2) generate a large failure strain along the path directly towards the zero effective stress state without transition from contractive state to dilative state, as shown in Figure 3-5(a) and (b). For relatively loose sands (No. 3&4), they also generate large failure strain at last but transition from contractive state to dilative state can be observed, as shown in Figure 3-5(c) and (d). For medium dense sands (No.5 to 7), however, cyclic mobility occurs and the strain increases gradually to a relatively larger scale, as shown in Figure 3-5(e) to 5(g). On the other hand, the dense sand (No.8) only generates a small amount of strain and never shows cyclic mobility, as shown in Figure 3-5(h). Therefore, the mechanical behaviors of sand subjected to undrained cyclic loading can be uniquely and properly described by the constitutive model under the condition that all material parameters are kept constant.

Figure 3-6 shows the simulated result of loose sand (No.2) subjected to cyclic loading under drained condition. Other simulate conditions are the same as aforementioned undrained test. As the cyclic loading goes on, the loose sand is compacted and experiences a large contractive volumetric strain.

The above-discussed simulations of undrained/drained cyclic loading tests in Figures 3-5& 6 coincide with the results in the works by Asaoka (2003) in which the author stated the fact that consolidation or liquefaction of sand due to cyclic loading is just dependent on the drainage condition of the test. If the loading is conducted under drained condition, then the consolidation of sand will happen; while under undrained condition, the liquefaction will occur, which is totally coincident with the reality.
Figure 3-5 Stress paths, stress-strain relations of the sand specimens with different densities subjected to cyclic triaxial test under undrained condition
Mechanical behaviors of sand under undrained/drained compression

The relationships between stress, strain and void ratio under undrained/drained triaxial compression tests on the sands listed in Table 3-6 are simulated in this section.

Under undrained conditions, three different types of typical stress-strain relations can be observed in the simulation, as shown in Figure 3-7. For loose sands [1] and [2], the sands reach a peak strength in small strain level and then collapse and flow rapidly toward the origin of the stress space, showing a typical strain-hardening/softening and contractive behavior. For medium dense sands [3] to [6], stiffness of the sands decreases abruptly in certain strain level where a typical transition from contractive to dilative occurs. Dense sands [7] and [8], however, only show strain hardening. In Figure 3-7(c), traces of $e$-$\ln p'$ states during shearing are plotted for all sands, showing that all sands finally move towards critical state line (C.S.L.), and that even for a very loose sand, transition process from contractive to dilative would occur. For dense sand, dilation will be predominant and the stress, that is needed to shear the sands to C.S.L., will be extremely large.

Figure 3-8 shows simulated stress-strain-dilatancy relations of the sands with different densities listed in Table 3-6 in drained triaxial compression tests with constant confining stresses. It is known that dense sands show typical strain hardening-strain softening and dilation while loose soils only show strain hardening along with monotonic contraction. The transition from contractive state to dilative state is just
dependent on the density of the sand. It is known from Figure 3-8(c) that all sands approach to the same point in $e$-$p$ space at critical state and have the same void ratio, irrespective of the different initial densities at the beginning of shearing, because they are originated from the same sand.

The above simulated density-dependent effect of sands in drained/undrained triaxial compression are well-known to the researches and have already been confirmed in laboratory tests that it is not necessary to give any comparison between the test and the simulation.

(a) Effective stress paths  
(b) Stress-strain relations  
(c) e-ln$p'$ relations

Figure 3-7 Simulated stress paths, stress-strain relations of the sands with different densities listed in Table 6 in undrained triaxial compression tests

(a) Stress-strain relations  
(b) Strain-dilatancy relations  
(c) e-$p'$ relations

Figure 3-8 Simulated stress-strain-dilatancy relations of the sands with different densities listed in Table 6 in drained triaxial compression tests
3.3.3 Confining-stress dependency of sand in undrained monotonic loading tests

![Graphs showing the stress paths and stress-strain relations of TS with the same void ratio but different confining stress in undrained triaxial compression test (Verdugo and Ishihara, 1996).](image)

Verdugo and Ishihara (1996) reported their experimental results of TS, in which undrained triaxial compression tests on sands with the same void ratio but different confining pressures were conducted under very high confining pressures (up to 3MPa).
The test results given in Figure 3B9 show that under the same void ratio, if a confining stress is large, the sand behaves like a loose sand, while if the confining stress is small, the sand behaves like a dense sand. Such a phenomenon is called as “confining-stress dependency of sand”, originally defined in the research by Ishihara (1993). Nakai (2005) also reported the same phenomenon in his tests on silica sands.

The parameters of the sand used in the simulation are the same as Table 3B2. In the tests, three group sands were considered. The sands in each group have the same void ratio, while for different group the void ratio is different. Therefore, in the simulation it is necessary to adjust the density for all the sands before shearing. The initial values of the void ratios are set to be equal to 0.78, 0.70 and 0.65 respectively and are listed in Table 3-7 with other initial conditions of the sands. Figure 3-10 shows that the simulated results on the whole, coincide well with the test results quantitatively and qualitatively.

It is also known for the simulation that the mechanical behavior of sands with the same density but different confining stresses can also be reproduced uniquely with one set of the same material parameters in all different conditions.

Table 3-7 Initial conditions of TS samples before undrained triaxial compression

<table>
<thead>
<tr>
<th>No.</th>
<th>Confining stress $p'$ (MPa)</th>
<th>Initial void ratio $e_0$</th>
<th>Initial degree of overconsolidation OCR</th>
<th>Initial degree of structure $R_0$</th>
<th>Initial anisotropy $\zeta_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>①</td>
<td>0.1</td>
<td>0.776</td>
<td>40.0</td>
<td>0.21</td>
<td>0.0</td>
</tr>
<tr>
<td>②</td>
<td>1.0</td>
<td>0.781</td>
<td>3.0</td>
<td>0.20</td>
<td>0.0</td>
</tr>
<tr>
<td>③</td>
<td>2.0</td>
<td>0.781</td>
<td>1.6</td>
<td>0.15</td>
<td>0.0</td>
</tr>
<tr>
<td>④</td>
<td>0.1</td>
<td>0.698</td>
<td>111.0</td>
<td>0.45</td>
<td>0.0</td>
</tr>
<tr>
<td>⑤</td>
<td>1.0</td>
<td>0.698</td>
<td>12.0</td>
<td>0.30</td>
<td>0.0</td>
</tr>
<tr>
<td>⑥</td>
<td>2.0</td>
<td>0.703</td>
<td>9.0</td>
<td>0.16</td>
<td>0.0</td>
</tr>
<tr>
<td>⑦</td>
<td>3.0</td>
<td>0.709</td>
<td>5.3</td>
<td>0.15</td>
<td>0.0</td>
</tr>
<tr>
<td>⑧</td>
<td>0.1</td>
<td>0.651</td>
<td>270.0</td>
<td>0.55</td>
<td>0.0</td>
</tr>
<tr>
<td>⑨</td>
<td>1.0</td>
<td>0.651</td>
<td>35.0</td>
<td>0.30</td>
<td>0.0</td>
</tr>
<tr>
<td>⑩</td>
<td>2.0</td>
<td>0.651</td>
<td>16.0</td>
<td>0.30</td>
<td>0.0</td>
</tr>
<tr>
<td>⑪</td>
<td>3.0</td>
<td>0.651</td>
<td>12.0</td>
<td>0.25</td>
<td>0.0</td>
</tr>
</tbody>
</table>
3.3.4 Dense sand subjected to drained cyclic loading

Finally, behaviors of dense sand subjected to drained cyclic loading under constant-mean-effective-stress are simulated. The confining pressure of the sand is 196 kPa and cyclic loading condition is that the mean effective stress is kept constant and a maximum principal stress ratio ($\sigma_1/\sigma_3$) is loaded to 4. Figure 3-11 shows the test results by Hinokio (2000), in which the stress-strain curves are plotted in terms of effective stress ratio $\sigma_1/\sigma_3$, a dimensionless normalized stress. The volumetric strain shows
dilatancy at the very beginning under cyclic loading and then turns to compression until it reaches a convergence state at which the compression almost stops, as shown in Figure 3-11 (b). For deviatory stress-strain relation, at the beginning, it shows a relatively large loop, as the cyclic loading number increases, however, the stiffness of the sand grows up and the stress-strain relation comes into an almost fixed loop as shown Figure 3-11(c).

In the simulation, the parameters of the sand are the same as those listed in Table 3-2 and initial conditions of dense sand are shown in Table 3-8. In determining the initial conditions of the sand, it is assumed that the sand is well-remolded one with extremely low structure and relatively high OCR. The maximum principal stress ratio ($\sigma_1/\sigma_3$) is loaded to 5.4. As can be seen in Figure 3-12, the overall characteristics of the sand predicted by the present model, for instance, the changes in dilatancy and stress-strain relations, agree qualitatively well with the test results, but showing a slight over-estimation of volume strain. It should be emphasized here that in the simulation, volumetric compression also stopped automatically after certain cycles of loadings, which agrees well with the experimental results. The reason why the model can describe this behavior is quite simple. Taking a look at Figure 3-12(d) and (e), in which the changes of overconsolidation and stress-induced anisotropy are plotted, it is easy to find out that during plastic loading the degree of overconsolidation sometimes may even increase, not always the case in which overconsolidation only develops in elastic unloading process. One of the most important features of the model is that the changing rate of overconsolidation, or density, is assumed to be controlled by two factors, one is the plastic component of stretching and the other is the increment of stress-induced anisotropy. The physical meaning of the first part is very familiar to the readers and the second part, is just used to consider the influence of stress-induced anisotropy which is usually dependent on the roundness of soil particles and their orientation of deposition. It is conceivable that the sand with strong anisotropy will have a stronger resistance against volumetric change during shearing than those of weak one in which soil particiles deposit rather randomly. It is admitted, however, that above discussion is still hard to be confirmed with experiment at present time. It is clear from the figures that during cyclic shearing, overconsolidation gets higher and higher, in other words, the density is getting higher, resulting in the difficulty to further compression.
Figure 3-11 Test results of dense sand in drained cyclic loading tests (Hinokio, 2000)

Table 3-8 Initial conditions of TS sample before drained cyclic loading

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.661</td>
</tr>
<tr>
<td>Initial mean effective stress $p'$ (kPa)</td>
<td>196.0</td>
</tr>
<tr>
<td>Initial degree of structure $R_0^*$</td>
<td>0.99</td>
</tr>
<tr>
<td>Initial degree of overconsolidation $1/R_0$</td>
<td>55.0</td>
</tr>
<tr>
<td>Initial anisotropy $\zeta_0$</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Figure 3-12 Simulation of the test results in Figure 3-11
3.4 Summary

In this chapter, based on conventional drained triaxial compression tests and undrained triaxial cyclic loading tests, material parameters of Toyoura sand is determined. By using the uniquely determined material parameters, various tests under different loadings and drainage conditions are simulated by the constitutive model. The capability of the model to describe uniquely the overall behaviors of the sand under different drainage conditions and different loadings with one set of fixed parameters, is verified and the following conclusions can be given.

1. Eight parameters are needed to describe the behavior of sand, among which five parameters, \( M, N, \lambda, \kappa, \) and \( \nu \) are the same as in the Cam-clay model. Other three parameters, \( a \): the parameter controlling the collapse rate of structure, \( m \): the parameter controlling the losing rate of overconsolidation, and \( b_r \): the parameter controlling the developing rate of stress-induced anisotropy, have clear physical meanings and can be easily determined by undrained triaxial cyclic loading tests and drained triaxial compression tests. In order to give a unified description of TS, the eight material parameters are kept the same value for all the tests under different loadings and drainage conditions.

2. Simulations on a set of sands with different densities originally compacted from the same loose sand are conducted to verify the density-dependent behavior of the sand. The results reveal the fact that the mechanical behaviors of sand subjected to cyclic loading under drained/undrained condition can be uniquely and properly described by the constitutive model no matter what the density may be. It is confirmed theoretically that for loose sand, liquefaction happens without transition from contractive to dilative state; for medium dense sand, cyclic mobility occurs while for dense sand, liquefaction will not occur. Furthermore, it is confirmed that if the sand subjected to cyclic loading under drained condition, then the consolidation of sand will occur; while under undrained condition, the liquefaction might happen, depending on its density.

3. Under undrained triaxial compression test, loose sand exhibits a peak strength in small strain level and then collapses and flows rapidly toward the origin of the stress space, showing a typical strain-hardening/softening and contractive behaviors. For medium dense sand, stiffness of the sand decreases abruptly in certain strain
level where a typical transition from contractive to dilative state occurs. Dense sand, however, only shows strain hardening. In undrained tests, all sand samples finally move towards C.S.L., while in drained tests, all sands approach to the same point in $e-p$ space at critical state, irrespective of different initial densities at the beginning of shearing.

4. Confining-stress dependency of sand, a typical behavior of sands with the same density but different confining stresses, can also be simulated properly.

5. Dense sand subjected to rather large cyclic loading under drained and constant-mean-effective-stress conditions is also simulated. The overall characteristics of the sand is predicted well by the theory, for instance, the changes in dilatancy and stress-strain relations are qualitatively the same as the test results with a slight over-estimation of volume strain. Particular attention is paid to the volumetric contraction during cyclic loading, which shows in the test a small dilatancy at the very beginning and then turns to compression until it reaches a critical state at which the compression stops. The simulation also describes the same behavior automatically without changing the values of any parameters, which was impossible up till now.

6. It cannot say that the model can perfectly describe the various behaviors of TS, but that the model can give a unified description of TS qualitatively with only eight material parameters with fixed value. The fact that sometimes the calculated results do not coincides well with the experimental results quantitatively, implies that the accuracy of the model is still needed to be improved in further research. Meanwhile, if the influence of intermediate principal stress can be properly taken into consideration like the $t_{ij}$ concept (Nakai and Mihara, 1984), the model would be much better.
References


Nakai, K., Nakano, M., Noda, T. and Asaoka, A. (2004): Description of “Compaction” and “Liquefaction” behavior of sand based on evolution of soil skeleton structure, Proc. 2\textsuperscript{nd} International Workshop on New Frontiers in Computational Geotechnics, Fortaleza, Brazil, Zhang et al. (eds), 135-144.

Nakai, K. (2005): An elasto-plastic constitutive modeling of soils based on the evolution laws describing decay of soil skeleton structure, loss of overconsolidation and


CHAPTER 4 Numerical Simulation of Sand Subjected to Cyclic Load under Undrained Conventional Triaxial Test

4.1 Introduction

In order to conduct a precise prediction of BVP, it is necessary to determine the values of soil parameters involved in the constitutive model by element simulation based on the laboratory tests in element level. In the element simulation, though stress-strain relation is regarded as uniform within the specimen of soil, it is usually non-uniform in reality. Due to the limitation of element test devices for soils in laboratory tests, a perfect element test is impossible in reality because of some factors such as an initial imperfection of test specimen, friction between loading caps and the specimen, existence of gravitational stress field and etc. Some researches concerning these factors under monotonic loading can be found in literatures, such as the works by Higo (2003) about the instability and strain localization of saturated clay specimens in conventional triaxial compression tests simulated by FD-FE analysis based on elasto-viscoplastic model; the works by Miyata et al. (2003) and Miyata et al. (2004) in which different restraint conditions of test specimens in element tests were discussed in detail with two-dimensional (2D) and three-dimensional (3D) finite element analyses, concerning stress-strain relation, shear band formation and strain localization.

The same aforementioned problems can happen in the element tests subjected to cyclic loading or dynamic loading in which the loading speed is much faster than those to static loading tests where the frequency may reach a fraction of 1 Hz. Watanabe et al. (2006) conducted an experimental and numerical study on liquefaction-induced compaction for a specimen of sand in conventional triaxial tests with a static and dynamic FD-FE analysis, to find out the non-uniform mechanical behavior of the test specimen under so-called element test subjected to static and dynamic loading.

It is, therefore, natural to ask whether the results of an element test commonly used in laboratory test are still useful in determining the mechanical behaviors of the soils, as the so-called element test usually does not show a uniform behavior in its occupied area due to some inevitable imperfection or nonuniformity of both the test specimen and the test device. If the answer is yes, then we need to answer the question that how much is
the influence of the imperfection on the overall mechanical behavior of the soil specimen.

It is, therefore, necessary to clarify the following aspects in order to answer the above question by establishing a reliable numerical technique that is not only be able to determine the soil parameters of a suitable constitutive model based on conventional triaxial tests, but also be able to describe the characteristics of the non-uniform behavior of test specimen observed in the element test and explain the reasons why it happens;

(a) Qualitative and quantitative evaluation of the influence of nonuniformity in test specimen on the overall or average mechanical behavior of the soil specimen, e.g., friction between specimen and loading caps, non-uniform distribution of initial stress due to gravity of the specimen;
(b) Influence of loading conditions, e.g., influence of inertia force, dynamic loading frequency or loading rate, and amplitude of cyclic loading ratio

To achieve this main purpose, 2D and 3D soil-water coupling FD-FE analyses are conducted to simulate the mechanical behavior of sand specimen subjected to cyclic load in conventional triaxial test under undrained condition. The numerical analyses are conducted using a FEM code named as DBLEAVES (Ye, 2007). It has been developed based on the constitutive model (Zhang et al., 2007) that can consider the effects of overconsolidation (density), structure (bonding) and stress-induced anisotropy in unified way. This program has been proved (Ye et al., 2007) to be capable of solving repeated static-dynamic loading processes with FD-FE scheme and considering soil-water coupling problems in BVP with infinitesimal and finite deformation algorithms.

4.2 Conditions for numerical simulation of element test as a BVP

Figure 4-1 shows element simulations of Toyoura sand at loose, medium-dense and relatively-dense states subjected to cyclic loading under undrained condition. The purpose of the simulation is just to show the performance of the constitutive model proposed by Zhang et al. (2007). It should be pointed out that, in the figures throughout this paper, deviator stress means stress difference \( q (\sigma_1-\sigma_3) \) and shear strain \( \varepsilon_a \) means axial strain. Material parameters and initial state parameters for these sands with different densities are the same as shown in Table 4-1 and 4-2 (Ye et al., 2007). It is very
clear from the figure that loose sand moves towards zero effective stress state without the process of cyclic mobility; for medium-dense sands, liquefaction occurs with the cyclic mobility, dense sand, however, never liquefies.

In this study, 2D and 3D FD-FE analyses are conducted to simulate the behavior of soil specimen subjected to cyclic loading under undrained condition. In 3D analyses, the soil specimen is a cylinder with a diameter of 5 cm and a height of 10 cm which is modeled with 8-node hexahedral isoparametric elements. The finite element meshes used in the calculations are shown in Figure 4-2(a). In order to check the mesh-size dependency of numerical results, simulations are performed using two different mesh sizes, as shown in Figure 4-2(a), in which the coarse mesh is called as Mesh 1 and the fine mesh is called as Mesh 2. In 2D analyses, the soil specimen is modeled with 4-node isoparametric solid elements called as Mesh 3 with a diameter of 5 cm and a height of 10 cm as shown in Figure 4-2(b). All calculations are carried out under undrained conditions. As to the boundary conditions of the specimen, in the case of ideal element test in which there is no friction between the specimen and loading caps at top and bottom surfaces, the boundary condition at the bottom is assumed with roller condition in which vertical direction at all nodes is fixed while one node is fixed in all directions and another node is fixed in one horizontal direction to prevent rotation of the specimen. When considering friction, an extreme case called as end-fixed case is adopted, in which all the nodes on the top and bottom surfaces are fixed to the rigid loading caps.

![Figure 4-1 Element simulation of sands with different densities ($p_0' = 298$ kPa)](image-url)
Table 4-1 Material parameters of Toyoura Sand

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tr>
<td>Compression index $\lambda$</td>
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<tr>
<td>Swelling index $\kappa$</td>
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<tr>
<td>Critical state parameter $M$</td>
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</tr>
<tr>
<td>Void ratio $N$ ($p'=98$ kPa on N.C.L.)</td>
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</tr>
<tr>
<td>Poisson’s ratio $\nu$</td>
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</tr>
<tr>
<td>Degradation parameter of overconsolidation state $m$</td>
<td>0.10</td>
</tr>
<tr>
<td>Degradation parameter of structure $a$</td>
<td>2.2</td>
</tr>
<tr>
<td>Evolution parameter of anisotropy $b_r$</td>
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</table>

Table 4-2 Initial conditions of sands with different densities

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<th>Condition</th>
<th>Loose sand</th>
<th>Medium-dense sand</th>
<th>Dense sand</th>
</tr>
</thead>
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<tr>
<td>Initial void ratio $e_0$</td>
<td>0.81(Dr=0.44)</td>
<td>0.70(Dr=0.75)</td>
<td>0.63(Dr=0.92)</td>
</tr>
<tr>
<td>Initial mean effective stress $p'$ (kPa)</td>
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<td>294</td>
<td>294</td>
</tr>
<tr>
<td>Initial degree of structure $R_0^*$</td>
<td>0.0095</td>
<td>0.40</td>
<td>0.70</td>
</tr>
<tr>
<td>Initial degree of overconsolidation $1/R_0$</td>
<td>1.19</td>
<td>2.0</td>
<td>5.07</td>
</tr>
<tr>
<td>Initial anisotropy $\zeta_0$</td>
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<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Note: $e_{\max}=0.97$; $e_{\min}=0.61$

Figure 4-2 Finite element meshes and boundary conditions

In simulating cyclic loading, a concentrate vertical load is applied at the center of the top surface, where all the nodes on the top surface are kept to be equal to each other in the vertical direction as a rigid plate as shown in Figure 4-2(c). Cyclic loading with
sine wave is applied with 20 cycles. In the simulation, the following aspects focused on two parts such as the influence of precision on numerical method itself and the influence of nonuniformity on the specimen caused during the test are considered:

- Influence of calculating method, that is, theoretical, static or dynamic method;
- Influence of time interval used in the integration of differential equations;
- Influence of amplitude of cyclic loading;
- Influence of cyclic loading frequency;
- Influence of friction between loading caps and specimen;
- Influence of gravitational stress field;
- Influence of initial imperfection of test specimen;
- Influence of confining stress

Table 4-3 lists the calculating cases under various conditions. For instance, the amplitude of the cyclic loading in shear stress ratio \( \frac{q}{2\sigma_0'} \) varies from 0.15 to 0.40 and the frequency of loading varies from 0.01 Hz to 10.0 Hz. The time interval of each calculating step in Newmark-\( \beta \) integration for solving soil-water coupling problem is 0.002 second both in dynamic and static simulations if without specification.

In the simulation, the soil specimen is Toyoura sand and its material parameters used in the analysis are the same as the parameters in the works by Ye et al. (2007), and their values are listed in Table 4-1. The initial values of the state parameters for the sand are listed in Table 4-4. It should be emphasized that throughout the calculations conducted in this chapter, all the parameters are the same.

<table>
<thead>
<tr>
<th>Table 4-4 Initial conditions of sands in numerical simulation</th>
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<tbody>
<tr>
<td>Initial void ratio ( e_0 )</td>
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<tr>
<td>Initial mean effective stress ( p' ) (kPa)</td>
</tr>
<tr>
<td>Initial degree of structure ( R_0^* )</td>
</tr>
<tr>
<td>Initial degree of overconsolidation ( 1/R_0 )</td>
</tr>
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<td>Initial anisotropy ( \zeta_0 )</td>
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Table 4-3 Calculating cases under various conditions

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<tr>
<th>Figure No.</th>
<th>Dimension</th>
<th>Mesh</th>
<th>Analytical Method</th>
<th>Initial Confining Stress (p’ kPa)</th>
<th>Time Interval (Δt/step (sec))</th>
<th>Cyclic Loading</th>
<th>Frequency (Hz)</th>
<th>Amplitude of Stress Ratio q/2σ</th>
<th>Restraint Condition at Top and Bottom Surface</th>
<th>Gravitational Stress Field</th>
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<tr>
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4.3 Results and discussion

4.3.1 Influence of analytical method

In this section, the influence of analytical method is discussed. The conventional triaxial test with cyclic loading under undrained condition is simulated by three different ways, that is, simulated by theory (element simulation), static and dynamic FEM. In static analysis with FD-FE scheme, inertial force and corresponding damping effect are neglected. Figure 4-3 shows the stress paths, stress-strain relations and time history of excess pore water pressure ratio (EPWPR) at the center of the specimen, obtained from different methods using Mesh 1. In the figure, only the results of the element located at the center of the specimen is considered because all the other elements show the same behavior. By comparing the results from element simulation with those from static analysis, it is known that the overall mechanical behaviors are almost the same. The results from static analysis show approximately the same tendency as those from dynamic analysis. Above results indicate that different methods predict almost the same behavior at low loading frequency (0.1 Hz).

Figure 4-3 Stress paths, stress-strain relations and time history of EPWPR obtained from different analytical methods (free at top and bottom, Mesh 1)
4.3.2 Influence of time interval ($\Delta t$/step) of integration

In order to obtain a reliable calculation result, it is necessary to determine a suitable time interval for each calculation step ($\Delta t$/step) in Newmark-$\beta$ integration. Different time intervals varying from 0.01 sec/step to 0.00002 sec/step are used to check the influence of the time interval for different loading frequency varying form 0.01 Hz to 10 Hz under the condition that numerical parameters are the same as the Table 4-1 and Table 4-4.

Figure 4-4 shows the influence of time interval on the stress paths, stress-strain relations and time history of EPWPR at the center of the specimen under different loading frequencies. In the case of loading frequency of 0.01 Hz, the time interval has little influence on the calculating results when it is less than 0.02 sec/step. In the case of 0.1 Hz, though the stress-strain relation shows bigger loops when the time interval is 0.01 sec/step, the magnitude of shear strain converges to stable value when it is less than 0.002 sec/step, showing that a time interval of 0.002 sec/step is enough to get satisfactory results. In the case of 1.0 Hz, however, the time interval of 0.002 sec/step is not enough to get satisfactory results. As shown in Figure 4-4(c), if the time interval is 0.002 sec/step, then a significant decrease of deviatory stress is observed in the calculation, which is obviously unacceptable. In order to get satisfactory results, a time interval of 0.0002 sec/step is necessary. In the case of 10 Hz, which is not realistic in laboratory, a time interval of 0.0002 sec/step is enough for acquiring stable results, compared with the results from the calculation with 0.00002 sec/step.

It is understood, therefore, that time interval may affect the calculation results greatly under different loading frequencies. Throughout this chapter, suitable time intervals are used in the subsequent calculations according to the loading frequencies and the details are listed in Table 4-3.
Figure 4-4 Stress paths, stress-strain relations and time history of EPWPR calculated with different time intervals (dynamic loading, free at top and bottom, Mesh 1)
4.3.3 Influence of amplitude \((q/2 \sigma_0)\) of cyclic loading

In this section, the influence of amplitude of cyclic loading on the mechanical behaviors of soil is discussed. Three different amplitudes of shear stress ratio \((q/2\sigma_0)\) of 0.15, 0.25 and 0.40 are simulated. Figure 4-5 shows the stress paths, stress-strain relations and time history of EPWPR at the center of the specimen. The higher the shear stress ratio is, more quickly the soil specimen goes into cyclic mobility loop, and the larger the shear strain will be. In reality, such a larger shear stress ratio may cause specimen fail at first loading cycle and cyclic mobility could not be observed. In the calculation, however, the test is assumed as element test, that is, at perfect condition, cyclic mobility may occur even if the specimen liquefies at first loading cycle. It is hard to confirm the correctness of the calculation result because physically it is impossible to conduct an element test at laboratory according to its strict definition.

Figure 4-5 Stress paths, stress-strain relations and time history of EPWPR with different loading ratios of deviatoric stress (dynamic loading, \(f = 0.1\) Hz, free at top and bottom, Mesh 1)
4.3.4 Influence of cyclic loading frequency (f)

In this section, numerical analyses are carried out in 2D and 3D in which the influence of cyclic loading frequency is discussed. Loading frequency ranges from 0.1 Hz to 10.0 Hz. Figure 4-6 shows the stress paths and the stress-strain relations of the element located at the center of specimen obtained from 3D analyses with different cyclic loading frequency using Mesh 1. It can be seen that at high frequency of cyclic loading (10.0 Hz), the deviatory stress decreased dramatically after stress path entered into cyclic mobility, in spite of the fact that external load is still kept constant. It is also known from the figure that cyclic mobility diminished to zero, while at relatively lower frequency of cyclic loading (0.1 & 1.0 Hz), this phenomenon cannot be observed. The reason why this phenomenon happens is that if loading speed is large enough, part of external force will be balanced by the inertia force of soil and pore water, and consequently the effective axial force acting on the specimen will decrease. In reality, however, due to the limitation of cyclic loading device, it is very hard to conduct a conventional triaxial cyclic loading test with loading frequency larger than 1.0 Hz.

The influence of cyclic loading frequency is also discussed with 2D analyses using Mesh 3 under fixed condition. Compared with 3D analyses, similar results are obtained, as shown in Figure 4-7.

Figure 4-8 shows the distributions of vertical strain $\varepsilon_v$ immediately after the cyclic loading finishing obtained from the calculations with loading frequencies ranging from 0.01 to 10.0 Hz in 2D and 3D analyses. It is known from the figure that the lower the frequency is, the more severe of the uneven distribution of the vertical strain will be. At very low frequency of 0.01 Hz, a sharp strain localization is observed while at very high frequency of 10.0 Hz, the specimen deformed uniformly in spite of the strong restraint condition where the top and bottom surfaces are totally fixed in the calculation. Detailed discussion about the influence of restraint condition will be given in next section. It is also known from the figure that both the calculations in 2D and 3D give the same results, implying the fact that the mechanical behavior of soil will be the same under undrained condition irrespectively of the different external loading paths.
Figure 4-6 Stress paths and stress-strain relations at different loading frequencies in 3D analyses (dynamic loading, free at top and bottom, Mesh 1)

(a) \(f = 0.10 \text{ Hz}\)  
(b) \(f = 1.0 \text{ Hz}\)  
(c) \(f = 10.0 \text{ Hz}\)  

Figure 4-7 Stress paths, stress-strain relations and time history of EPWPR at different loading frequencies in 2D analyses (dynamic loading, fixed at top and bottom, Mesh 3)

(a) \(f = 0.01 \text{ Hz}\)  
(b) \(f = 0.1 \text{ Hz}\)  
(c) \(f = 1.0 \text{ Hz}\)  
(d) \(f = 10.0 \text{ Hz}\)
Figure 4.8 Distributions of vertical strain $\varepsilon_z$ at different loading frequencies in 2D and 3D analyses (immediately after dynamic loading finished, dynamic loading, fixed at top and bottom)

4.3.5 Influence of friction between loading caps and specimen

In this section, the influence of friction between loading caps and specimen is discussed. The numerical analyses are carried out in two cases, that is, the end-free case in which the top and the bottom surfaces are free in lateral directions, and the end-fixed case in which the top and the bottom surfaces are fixed in lateral directions. The fixed condition can be regarded as the most serious restraint condition caused by the friction between the specimen and the loading caps. Practical condition in real test is between the end-free and the end-fixed conditions.

Figure 4.9 shows the stress paths and stress-strain relations at different positions obtained from the calculations under free condition using Mesh 1. All the results are the same in all positions showing that the specimen deformed uniformly.
Figure 4-10 shows the stress paths and stress-strain relations at different positions obtained from the calculations under fixed condition using Mesh 1. The mechanical behaviors are different at three positions. The mechanical behavior of soil at the center is similar to those of the soil in the case of free condition as shown in Figure 4-9. Due to the symmetric condition, the behaviors at top and bottom are the same but are totally different from those at the center of the specimen in stress path and stress-strain relation, showing the significance of the restraint condition of the test specimen.

Figure 4-11 shows the distributions of vertical strain \( \varepsilon_z \) and volumetric strain \( \varepsilon_v \) immediately after dynamic loading finished at different restraint conditions. The calculated results obtained from the calculation under free condition are uniform in whole area of the specimen, while the residual volumetric strain obtained from the calculation under fixed condition is not uniform and is much larger near the top and bottom areas than those at other areas. In order to investigate the mesh-size dependency of the nonuniformity, Mesh 1 and Mesh 2 are used in the 3D analyses. It can be seen from the distributions of volumetric strain \( \varepsilon_v \) that the non-uniform deformation of the specimen at fixed condition is very clear. The mesh-size dependency of the numerical results shows that for a fine mesh, the nonuniformity is restricted to a very narrow area, which is usually called as strain-localization area; while for a coarse mesh, the strain-localization area might be much larger than that of fine mesh and with a vague border.

<table>
<thead>
<tr>
<th>Mesh 1</th>
<th>Mesh 2</th>
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<td>( \varepsilon_z )</td>
<td>( \varepsilon_v )</td>
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</table>

(a) Element No.1254      (b) Element No.614        (c) Element No.38

Figure 4-9 Stress paths, stress-strain relations at different places (dynamic loading, \( f = 0.10 \) Hz, free at top and bottom, Mesh 1)
Figure 4-10 Stress paths, stress-strain relations at different places (dynamic loading, $f = 0.10$ Hz, fixed at top and bottom, Mesh 1)

Figure 4-11 Distributions of vertical strain $\varepsilon_z$ and volumetric strain $\varepsilon_v$ at different restraint conditions (immediately after dynamic loading finished, dynamic loading, $f = 0.10$ Hz)

Figure 4-12 shows external force-mean effective stress relations and average stress-strain relations at different places obtained from the dynamic analysis under end-fixed condition, while Figure 4-13 shows external force-mean effective stress relations and average stress-strain relations at different places obtained from the static analysis under end-fixed condition. The average values are calculated from the elements
located at top, middle and bottom layers. The results at the top and bottom layers are the same, showing that the mechanical behaviors of the specimen are symmetric with respect to the center of the specimen in vertical direction. The behavior of soil at middle layer is similar to those of calculated results under free condition shown in Figure 4-9.

The behaviors of soil at top and bottom layers, however, are little different from those of middle layer in the way that the shear strain accumulates more quickly at extensive side after stress path entered into cyclic mobility and that the average stress
paths incline at the range before entering into cyclic region. The restraint condition of the test specimen, however, may have negligible influence on the overall mechanical behavior of the specimen.

4.3.6 Influence of gravitational stress field

The influence of gravity in initial stress field is discussed in this section. Due to small size of the specimen (a height of 10 cm), the influence of gravity in initial stress field is very small compared with the confining stress and is usually neglected in the numerical analysis of soil liquefaction in BVP. In conventional triaxial tests with cyclic loading under undrained condition, it is often observed that after liquefaction, pore water is concentrated at the top of the specimen while the sand settled at the bottom, showing that the gravitational force works during or after the loading. In order to investigate the influence of gravity, two cases of calculation, that is, the case neglecting gravity and the case considering the non-uniform initial stress field due to gravity are conducted.

(a) Element No.614  
(b) Element No.1254  
(c) Element No.614  
(d) Element No.38  
Uniform initial stress field  
Gravitational stress field  
Figure 4-14 Stress paths, stress-strain relations and time history of volumetric strain at different initial stress fields (dynamic loading, $f=0.1$ Hz, free at top and bottom, Mesh 1)
Figure 4-15 Distribution of volumetric strain $\varepsilon_v$ at different initial stress fields (immediately after dynamic loading finished, $f = 0.1$ Hz, free at top and bottom, Mesh 1)

Figure 4-14 presents the stress paths, stress-strain relations and time history of volumetric strain at the top, center and bottom elements of the specimen. At the center, there is no big difference between the calculated results from different numerical cases, shown in Figure 4-14(a) & (c). Different from the uniform distribution of the mechanical behaviors of the specimen in the case of neglecting gravity, under condition of the non-uniform initial stress field, the time histories of the volumetric strain in the specimen show non-uniform responses. This tendency can also be confirmed by the distribution of volumetric strain $\varepsilon_v$ immediately after cyclic loading finished shown in Figure 4-15, in which, the soil at the area near the top surface is less contracted; while the soil at the area near the bottom is more compressive, which means the density of the soil at top is getting less than that of the soil at bottom. This calculated result is consistent with observed results in which after liquefaction of the specimen; pore water is concentrated at the top of the specimen forming a less density area while the sand settled at the bottom of the specimen forming a relatively higher density area. The influence of gravity on the whole stress-strain relations, however, is negligible.

4.3.7 Influence of initial imperfection of test specimen

The influence of initial imperfection of test specimen is discussed in this section with 2D analysis. In order to consider the initial imperfection of test specimen, the lateral coordinates of the specimen are moved towards inside the mesh by 1 mm alternatively at an interval of 1 mm along two sides of the specimen.

Figure 4-16 presents the stress paths and stress-strain relations obtained from the
calculations considering initial imperfection of the test specimen under fixed condition. It is known from the figure that the imperfection has little influence on the behaviors of the specimen because the results from two cases are almost the same.

Figure 4-17 shows the influence of different initial imperfections of the specimen on distribution of vertical strain $\varepsilon_z$ immediately after cyclic loading finished. From the figure, it is known that on the whole, the distribution of volumetric strains shows no much difference except result of the one-point imperfection, implying again that the initial imperfection has little influence on the mechanical behavior of the specimen subjected to cyclic load under undrained condition.

![Influence of initial imperfection of test specimen on stress paths and stress-strain relations](image)

(a) Uniform specimen  (b) Initial imperfection of specimen

Figure 4-16 Influence of initial imperfection of test specimen on stress paths and stress-strain relations (dynamic loading, $f = 0.1$ Hz, fixed at top and bottom, Mesh 3)

![Distribution of vertical strain $\varepsilon_z$](image)

(a) Uniform specimen  (b) Continuous imperfection on two sides  (c) Continuous imperfection on one-side  (d) One-point imperfection

Figure 4-17 Distribution of vertical strain $\varepsilon_z$ at different initial imperfection of test specimen (immediately after dynamic loading finished, $f = 0.1$ Hz, fixed at top and bottom, Mesh 3)
4.3.8 Influence of confining stress

The influence of confining stress is discussed in this section. Four different confining stresses, 294 kPa, 196 kPa, 98 kPa, 49 kPa are considered. Calculations are conducted both in 2D and 3D analyses. The restraint conditions at top and bottom surfaces of the specimen are also considered.

Figure 4-18 Stress paths and stress-strain relations at different confining stresses in 3D analyses (dynamic loading, $f = 0.1$ Hz, free at top and bottom, Mesh 1)

Figure 4-19 Stress paths and stress-strain relations at different confining stresses in 2D analyses (dynamic loading, $f = 0.1$ Hz, free at top and bottom, Mesh 3)

Figure 4-18 shows the stress paths and the stress-strain relations at the center of specimen at different confining stresses in 3D analyses under the condition that the top and bottom surfaces are free. It is known from the figure that though the shear stress ratios are all the same as 0.15 in four different confining stresses, the stress-strain
relations are different from each other. The smaller the confining stress is, the larger the shear strain will be. On the other hand, the stress paths are similar to each other. The same results are obtained in 2D analyses as shown in Figure 4-19.

The influence of confining stress is also discussed under the condition that the top and bottom surfaces are fixed. Figure 4-20 shows the stress paths, the stress-strain relations at the center of the specimen and the distribution of vertical strain $\varepsilon_z$ immediately after dynamic loading finished at different confining stresses in 2D analyses. From the figure, it is known that the smaller the confining stress is, the larger the shear strain will be. Meanwhile, the unevenness of deformation of the specimen at different confining pressure is quite different. The smaller the confining stress is, the more severe the unevenness of deformation will be. In reality, the specimen probably will not deform so much if compared with the calculated results. The reason is that the friction between loading plate and the specimen is simulated with the strongest condition that the both ends are assumed to be fixed in numerical calculations.

Figure 4-20 Stress path, stress-strain relations and distribution of vertical strain $\varepsilon_z$ at different confining stresses in 2D analyses ($f = 0.1$ Hz, fixed at top and bottom, Mesh 3)
4.4 Summary

In this chapter, based on an elastoplastic model that can properly take into consideration the influences of the density, the structure and the stress-induced anisotropy of soils, 2D and 3D FD-FE analyses (code name: DBLEAVES) considering soil-water coupling problem are conducted to simulate the mechanical behavior of sand specimen in undrained triaxial cyclic loading test under different loading conditions. Due to the physical limitations, there exist some inevitable imperfections or nonuniformity of both the test specimen and the test device in common laboratory tests. The main purpose of the paper is to clarify the influence of the imperfection and different loading conditions on the overall mechanical behavior of the soil by the numerical experiments conducted in this paper. The total results obtained from this study are summarized in Table 4-5 & 4-6 that are focused on two parts such as the influence of precision on numerical method itself and the influence of nonuniformity on the specimen caused during the test. The following concluding remarks can be given through the numerical analyses:

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<td>4.3.2</td>
<td>Time interval of integration</td>
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<td>○</td>
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<td>4.3.5</td>
<td>Mesh size</td>
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<th>Element level</th>
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<tr>
<td>4.3.3</td>
<td>Amplitude (q/2σ₀) of cyclic loading</td>
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<td>Cyclic loading frequency (f)</td>
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<td>Friction between loading caps and specimen</td>
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<td>4.3.6</td>
<td>Gravitational stress field</td>
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<td>4.3.7</td>
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<td>4.3.8</td>
<td>Confining stress</td>
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1. Time interval for each calculating step used in Newmark-β integration may affect calculation results of dynamic analysis of soil specimen subjected to cyclic loading greatly. Suitable time intervals should be used to make sure a satisfactory calculation for different loading frequencies. In general, the higher the loading frequency is, the smaller the time interval for each calculating step should be. In laboratory tests, however, due to the limitation of loading device, maximum loading frequency usually less than a fraction 1.0 Hz. Therefore, the time interval of 0.002 sec\/step is enough to get a satisfactory accuracy in the analyses of boundary value problem.

2. The influence of cyclic loading frequency on the mechanical behavior of soil cannot be neglected. In the case of considering the friction between the specimen and loading caps, the lower the loading frequency is, the more severe the unevenness of deformation will be. At very slow loading frequency, clear strain localization is observed.

3. The influence of restraint condition of soil specimen at the top and bottom surfaces is also prominent. Mechanical behaviors of soils distribute unevenly within the specimen when the influence of restraint condition is considered. The influence is also mesh-dependent, that is, for a fine mesh, the nonuniformity is restricted to a very narrow area and consequently strain-localization happens; while for a coarse mesh, the strain-localization area might be much larger and with a vague border. Although the restraint condition has severe influence on the mechanical behavior of the specimen in element level, it may have negligible influence on the overall behavior of the specimen. In reality, the influence due to friction between the loading caps and the specimen may be between the numerical results affected by the restraint condition which varies from complete free to complete fixed in the frictional direction.

4. There exists influence of gravity in initial stress field on the distribution of volumetric strain in soil specimen after liquefaction. In the numerical simulation, soil is less contractive at the area near the top surface and more contractive at the area near the bottom surface after the cyclic loading is finished, which means the density of the soil at top is getting less than that of the soil at bottom. This phenomenon is consistent with the observed phenomenon after liquefaction, that is,
pore water concentrated at the top of the specimen forming a less density area while the sand settled at the bottom of the specimen forming a relatively higher density area. The influence of gravity on the whole stress-strain relations, however, is negligible.

5. From the numerical simulation in which different kinds of initial imperfection are considered, it is known that initial imperfection of test specimen has little influence on the mechanical behavior of soil specimen subjected to cyclic load under undrained condition.

6. Though the inevitable imperfection of test specimen and non-uniform loading conditions of the test device in the so-called element test may affect the mechanical behavior of the soil to some extents, it is concluded that the test results obtained from the cyclic load under undrained condition in conventional triaxial test is still useful in determining the soil parameters of a constitutive model for the reason that the influence of the nonuniformity on the overall mechanical behavior of the soil specimen is negligible from the result evaluated by the average values of finite element analysis.
References


CHAPTER 5 Numerical Simulation of Seismic Behavior of Group-pile Foundation

5.1 Introduction

Group-pile foundations are used extensively for the support of buildings, bridges, and other structures to safely transfer structural loads to the ground and to avoid excess settlement or lateral movement. Horizontal forces may become a major load applied to pile group and may often be cyclic in earthquake. It is known that during a strong earthquake, the dynamic behavior of a group-pile foundation is related not only to the inertial force coming from the superstructures but also to the deformation of the surrounding ground. Furthermore, it should be noted that once a foundation was damaged, its repair is technically difficult and the cost is expensive. Sometimes it is even forced to demolish the entire building even if the upper structure is intact. Thus, the foundations of a building that may be subjected to seismic force should be designed properly so as to maintain its safety. Therefore, it is necessary to understand the mechanical behaviors of the group-pile foundations and superstructures during a major earthquake.

Zhang et al. (2002a) reported the importance of the way to determine the material parameters of soils in FEM analysis. In the research, undisturbed specimens which were collected by a new sampling device were used in drained conventional triaxial compression tests to determine the mechanical behavior of reclaimed gravel ground. Then a real-scale lateral loading test on a 9-pile foundation (Kosa et al. 1998) that was built in the reclaimed gravel ground was simulated with a 3D finite element analysis. The numerical simulation was conducted using the $t_{ij}$ clay and $t_{ij}$ sand models in the total stress analysis condition.

The purpose of this research is to provide an applicable numerical way of evaluating the mechanical behavior of a pile foundation subjected to cyclic lateral loading up to an ultimate state. 3D finite element analyses of a real-scale group-pile foundation (Kosa et al. 1998) subjected to horizontal cyclic loading is conducted using a program named as DBLEAVES (Ye, 2007). Particular attention is paid to the ways to determine the material parameters of soils in FEM analysis, not based on the
comparison of the calculated and measured results of the boundary value problem (BVP), but based on the element tests with undisturbed and remolded specimens. In this research, nonlinear behaviors of ground and piles are described by subloading $t_{ij}$ model proposed by Nakai and Hinokio (2004, Appendix III) and AFD model proposed by Zhang and Kimura (2002b, Appendix IV), respectively. The numerical analyses are conducted by the total stress (non coupling analysis) and effective stress methods (soil-water coupling analysis). By comparing the calculated results with the field measurement, the applicability of the proposed numerical method and the way by which the material parameters of soils can be properly determined are discussed in detail.

In the second half of the chapter, numerical experiment on seismic performance of reinforcement by ground improvement around an existing group-pile foundation is also conducted to determine the optimum pattern in the size and location of ground improvement zone.

5.2 Numerical simulation of real-scale group-pile foundation subjected to horizontal cyclic loading

5.2.1 Brief description of the horizontal cyclic loading test and surrounding ground

Static loading test involves assembling a full-size prototype foundation at the construction site, and then slowly load it to failure. This method is the most accurate way to determine the ultimate compressive and tensile load capacities for deep foundations.

The static horizontal cyclic loading test of 9-pile foundation in real-scale was conducted in Kishiwada Osaka Prefecture in 1994. An elevated highway bridge is supported by a group-pile foundation made of cast-in-place reinforced concrete piles. Picture 5-1 shows the group-pile foundation and devices used in the test. The plan view of the test site is presented in Figure 5-1. The test piles with 1.2m in diameter and 30.4m in length are driven in a 3×3 pattern with a 3m pile spacing between adjacent piles from center to center. Reaction piles have pairs of group piles 3×2 and 3×3. The reinforcement of the piles has two different parts, with a thinner reinforcement at the lower part ($\phi 22\times12$) than the upper part ($\phi 29\times24$), as shown in Figure 5-2. The
surface layer of the ground at the test site is a very young reclaimed layer with a thickness of 13 m having a small N-value (varied N-value is 2~23, the average N-value is 10) of SPT, constructed only three years before the loading test. The reclaimed layer of gravel bed contains big gravels with 30 cm of max particle radius.

The loading procedure consists of nine cycles of applying and releasing unidirectionally with a maximum load 20.5 MN, as shown in Figure 5-3. The loading and unloading frequency was controlled by the capacity of the hydraulic pump. During the nine cycles, loading and unloading were applied in 1 MN/min with a three-minute-long pause between increments. Sustained loads were applied by incrementally increasing the load up to a predetermined level, and maintaining this level at constant value over a 15 minutes period. The time to complete one load cycle varied from approximately 45 minutes to 160 minutes.

![Picture 5-1 Group-pile foundation and horizontal loading test device](image)

![Figure 5-1 Schematic layout of real scale group-pile in plan view](image)
5.2.2 FEM mesh and boundary conditions

Figure 5-4 shows the geologic profile of ground and FEM mesh. The ground is composed of six layers based on the soil property chart. The water table is located 1m below the ground surface in the effective stress analysis. The initial stress of the ground is regarded as stratification bedding without considering the effect of pile driving.

Because of the symmetric condition of geometry and loading, only half of the domain is taken under consideration. The mesh consists of 5653 nodes and 4688 8-node isoparametric solid elements. The boundary conditions of the ground are fixed at the bottom, sliding at the two sides whose normal directions are parallel to y-axis and x-axis. Moreover, the boundary condition of the pile in the calculation is that the head of the
pile is fixedly rotated with free footing and free tip of it.

(a) Enlarged view of footing  
(b) Footing in plan view

(c) Overall view of FEM mesh

Figure 5-4 Geologic profile of ground and FEM mesh

5.2.3 Determination of the material parameters

The parameters in $t_{ij}$ model are determined with conventional triaxial tests and consolidation tests which can be referred to references (Nakai and Matsuoka, 1986; Nakai, 1989). Element simulations of drained triaxial compression tests with undisturbed and remolded specimens are conducted. To prepare a remolded specimen, the particle size distribution of the test specimens were adjusted by removing big grains with a diameter larger than 50 mm. The remolded specimens were collected from 11~12 m below the ground in 1992 and 1994. On the other hand, the undisturbed specimens were collected from the ground with the depth of 1.90~2.60 m, 3.60~4.25 m and 6.65~7.25 m in 2000.

Figure 5-5 and 5-6 show the comparison of experimental and simulated results of triaxial compression test different confining stresses. It is known that the undisturbed
specimens show strain hardening-strain softening while the remolded specimens only show strain hardening along with monotonic contraction. On the whole, the constitutive model is extremely precise in describing the behaviors of the soil while in the volumetric strain at low confining stress of the remolded specimen, a small discrepancy between the test and the theory results exists.

The parameters of ground are determined by the drained triaxial compression tests and tests for physical properties of soil. As to the detailed description of the material parameters of the ground, the stress ratio at critical state $R_f$ is determined by the internal friction angle obtained from the test results under different confining stresses. $R_f$ obtained from the undisturbed specimen is higher than the $R_f$ obtained from the remolded specimen. This difference is thought to be caused by two facts, one is that the grain size of the undisturbed specimens is different from those of remolded one in which big grains with a diameter larger than 50 mm had already been removed, and the other is that the soil structure has been destroyed in the remolded specimens. Compression index $\lambda$, swelling index $\kappa$ and overconsolidation ratio OCR of the reclaimed layer are determined by the element simulations, as shown in Figures 5-5 & 5-6. The values of $\lambda$, $\kappa$ of alluvial clay soil are determined by the relationship between compression index $C_c$ and liquid limit $w_L$ for Osaka alluvial clay soil: $C_c=0.010(w_L-12)$. Here, the value of average liquid limit is 65% from the laboratory test therefore the compression index is determined to be 0.53 ($\lambda=0.434\ C_c$, $\kappa=\lambda/5=0.046$). The coefficient of permeability is determined by Hazen formula: $k=CHD_{10}^2$, where $k$ is permeability (cm/s); $CH$ is Hazen empirical coefficient (usually assumed to be equal to 100) and $D_{10}$ is particle size for which 10% of the soil is finer (cm). $D_{10}$ of B1 layer is 0.081 cm, B2 layer is 0.019 cm and B3 layer is 0.033 cm from the grain size analysis. The material parameters of the ground are listed in Table 5-1. Only the bottom layer of the ground is supposed to be an elastic material in the numerical analysis.

In the calculation, the pile is modeled by the hybrid element proposed by Zhang et al. (2000), which is composed of beam element and solid element, to take into consideration the volume of pile, as shown in Figure 5-7. The beam element bears most of the loading acting on the pile, while the neighboring solid elements bear less loading but occupy the volume of the pile. The best sharing ratio of bending stiffness between the beam element and the solid elements was found to be 9 to 1 (Zhang et al., 2000),
which is also used in the calculation in this paper. Moreover, in order to consider the influence of the axial force on bending stiffness of pile, AFD model is employed in the analysis. The material parameters of the pile are listed in Table 5-2. The concrete footing above the ground is modeled with elastic elements.

![Table 5-1 Parameters of soils](image)

Table 5-1 Parameters of soils

<table>
<thead>
<tr>
<th>Soil</th>
<th>Thickness (m)</th>
<th>E (MPa)</th>
<th>ν</th>
<th>λ</th>
<th>κ</th>
<th>N</th>
<th>σ</th>
<th>β</th>
<th>OCR</th>
<th>ρ (t/m³)</th>
<th>k (cm/s)</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undisturbed</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>3</td>
<td>0.3</td>
<td>0.15</td>
<td>0.0013</td>
<td>0.9</td>
<td>7.59</td>
<td>500</td>
<td>1.5</td>
<td>1.5</td>
<td>1.97</td>
<td>6.56E-01</td>
<td>subloading εₖ model</td>
</tr>
<tr>
<td>B2</td>
<td>3</td>
<td>0.3</td>
<td>0.027</td>
<td>0.005</td>
<td>0.9</td>
<td>7.59</td>
<td>500</td>
<td>1.5</td>
<td>4</td>
<td>1.97</td>
<td>3.61E-02</td>
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</tr>
<tr>
<td>B3</td>
<td>4</td>
<td>0.3</td>
<td>0.08</td>
<td>0.02</td>
<td>0.9</td>
<td>7.59</td>
<td>500</td>
<td>1.5</td>
<td>1</td>
<td>1.97</td>
<td>1.09E-01</td>
<td>subloading εₖ model</td>
</tr>
<tr>
<td>Remolded</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>3</td>
<td>0.3</td>
<td>0.089</td>
<td>0.002</td>
<td>0.9</td>
<td>3.9</td>
<td>500</td>
<td>1.5</td>
<td>3</td>
<td>1.97</td>
<td>6.36E-01</td>
<td>subloading εₖ model</td>
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<tr>
<td>B2</td>
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<td>0.089</td>
<td>0.002</td>
<td>0.9</td>
<td>3.9</td>
<td>500</td>
<td>1.5</td>
<td>3</td>
<td>1.97</td>
<td>3.61E-02</td>
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<tr>
<td>B3</td>
<td>4</td>
<td>0.3</td>
<td>0.089</td>
<td>0.002</td>
<td>0.9</td>
<td>3.9</td>
<td>500</td>
<td>1.5</td>
<td>1</td>
<td>1.97</td>
<td>1.09E-01</td>
<td>subloading εₖ model</td>
</tr>
<tr>
<td>Ac1</td>
<td>11</td>
<td>0.3</td>
<td>0.23</td>
<td>0.046</td>
<td>0.8</td>
<td>3.69</td>
<td>500</td>
<td>1.5</td>
<td>10</td>
<td>1.80</td>
<td>1.00E-02</td>
<td>subloading εₖ model</td>
</tr>
<tr>
<td>Ac1</td>
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<td>0.23</td>
<td>0.046</td>
<td>0.7</td>
<td>4</td>
<td>500</td>
<td>1.5</td>
<td>2</td>
<td>1.70</td>
<td>1.00E-02</td>
<td>subloading εₖ model</td>
</tr>
<tr>
<td>Dc</td>
<td>6</td>
<td>100</td>
<td>0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.80</td>
<td>1.00E-02</td>
<td>elastic model</td>
</tr>
</tbody>
</table>

(1) $\sigma''_3 = 98$ kPa

(2) $\sigma''_3 = 196$ kPa

Figure 5-5 Element simulations and test results of the undisturbed specimens
Figure 5-6 Element simulations and test results of the remolded specimens.
Solid element $(EI)_{\text{solid}}$

Beam element $(EI)_{\text{beam}}$

$$(EI)_{\text{total}} = (EI)_{\text{beam}} + (EI)_{\text{solid}} = 0.9(EI)_{\text{total}} + 0.1(EI)_{\text{total}}$$

Figure 5-7 Hybrid element and its mechanism

Table 5-2 Parameters of piles

<table>
<thead>
<tr>
<th>1. Physical properties of RC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength of concrete : $\sigma_c = 3.8 \times 10^4$ kPa</td>
</tr>
<tr>
<td>Young’s modulus of concrete : $E_c = 2.5 \times 10^7$ kPa</td>
</tr>
<tr>
<td>Young’s modulus of steel : $E_s = 2.1 \times 10^8$ kPa</td>
</tr>
<tr>
<td>Yield stress of steel : $\sigma_y = 3.8 \times 10^5$ kPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>2. Arrangement of the reinforcement :</th>
</tr>
</thead>
<tbody>
<tr>
<td>D29-24 (upper part : 14.5m from the surface of the ground)</td>
</tr>
<tr>
<td>D22-12 (lower part:15.9m)</td>
</tr>
<tr>
<td>Overburden of the reinforcement : 15cm</td>
</tr>
</tbody>
</table>

5.2.4 Numerical results

In the simulation of one-side cyclic lateral loading test, a concentrated lateral load is applied at the center of the side surface of the footing same as the field test, where all the nodes on the side surface placed 0.9 m from the ground surface are kept to be equal to each other in the x, y, z directions as a rigid, as shown in Figure 5-8. The cyclic horizontal load with a maximum value of 20.5 MN adopted in the numerical analysis is the same as the one in the field test shown in Figure 5-3.

Figure 5-9 shows the lateral load and displacement relations at node No.1 shown in Figure 5-8. The results obtained from the analysis based on the undisturbed specimen shows a high strength and experiences a smaller lateral displacement than the results obtained from the analysis based on the remolded specimen. The result of the undisturbed specimen shows a big discrepancy with the field measurement. The reclaimed ground at that time was only a few years since its reclaim. Therefore, the ground at the moment can be regarded as remolded one. The undisturbed soil specimen, however, was sampled 8 years later than the preparation of the remolded one and the in...
situ ground had already experienced extra loads from up-structures of the bridge. Meanwhile, a sedimentary structure was estimated to be developed during the time span (eight years). These two facts might increase the stiffness and the strength of the reclaimed ground. Therefore it is expected that the calculation based on remolded specimen might be closer to the test results than those based on the undisturbed specimen. In Figure 8(b), the result of total stress analysis underestimates the test result at the maximum and residual displacements while the result of effective stress analysis coincides well with the test result until sixth loading cycle and the maximum and residual displacements are close to the test results. In the field test, many cracks occurred on the ground surface around sixth loading cycle. In calculation, however, it is impossible to describe the cracks, which might be the reason why a discrepancy between the test and the calculated results occurred after sixth cycle in effective analysis.

From the above discussion, it can be concluded that in numerical simulation, it is important not only to choose a suitable method, soil-water coupling or non coupling, but also to choose a suitable way of determining the material parameters based on the suitable element tests that can properly represents the in-situ condition.

![Figure 5-8 Loading method](image)

![Figure 5-9 Lateral load-displacement relations (at node No.1)](image)

(a) Results of the undisturbed specimen  (b) Results of the remolded specimen
(a) Arrangement of test piles
(b) Total stress
(c) Effective stress
Figure 5-10 Comparisons of test and calculated results of axial force at 8 MN loading

(a) Test results
(b) Total stress
(c) Effective stress
Figure 5-11 Comparisons of test and calculated results of bending moment at 8 MN loading

(a) At 8 MN loading
(b) At 20.5 MN loading
Figure 5-12 Comparisons of test and calculated results of bending moment in pile No.4

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From now on, only the results of the remolded specimen are discussed in detail. Figure 5-10 (a) shows the arrangement of test piles. Figure 5-10 (b) & (c) present the vertical distribution of axial force at the third loading cycle 8 MN. The axial forces of the effective stress analysis are larger than those of the total stress. Moreover, it is evident that each row carries some proportion of load. The maximum values of axial force for both analyses are front pile, middle pile and rear pile, in that order. It is seen that the front row carries most part of the load, and the rear row carries the least amount.

Figure 5-11 shows the bending moment of the test and analyses results obtained from the remolded specimen at 8 MN loading. Although the shapes of the distributions are similar to the test results for both analyses, the bending moment of the effective stress is much closer to the test result. The maximum bending moments are in the order of front pile, middle pile and rear pile according to their magnitudes. It is seen from the figure that the front pile bears larger bending moment than the rear pile because of the difference of axial forces, that is, compressive axial force in the front pile and tensile axial force in rear pile. The loading sharing ratio of the piles No.4, No.5 and No.6 at the loading stage of 8 MN in the numerical result is 60.0%, 23.7% and 14.4%, respectively, while that in the test result is 63.2%, 36.8% and -5.26%, respectively. It is seen that both the calculated and test results clearly reflect the phenomenon and agree well with each other, showing that the mechanical behavior of pile group can be described properly by using the axial force dependent (AFD) model in the numerical simulations.

The comparisons of test results with analyses results of pile No.4 placed in front row are discussed in detail, as shown in Figure 5-12. In those figures a black dot represents the test result, whereas a sky-blue square and a pink triangle do the same for total stress and effective stress calculations, respectively. It is known from the figure that though test and calculated results agree well with each other, the effective stress analysis performs better than total stress analysis when focused on the maximum value and its occurring depth. The result at 20.5 MN shown in Figure 5-12(b) also shows the same tendency.

The lateral displacement of the ground surface at the end of the test is developed in front of the side of the footing, as shown in Figure 5-13. Figure 5-14 shows the effective stress analysis result of displacement contour on the ground surface at the end of cyclic loading. Residual lateral displacements at the measuring points shown in Figure 5-13,
obtained from field measurement and numerical analyses, are listed in Table 5-3. The calculated lateral displacements on the ground surface show a reasonably good agreement with the field measurements.

![Lateral displacement of the ground surface at the end of the test](image1)

**Figure 5-13 Lateral displacement of the ground surface at the end of the test**

![Displacement contour on the ground surface at the end of cyclic loading](image2)

**Figure 5-14 Displacement contour on the ground surface at the end of cyclic loading (effective stress analysis)**

<table>
<thead>
<tr>
<th>Measuring point</th>
<th>Test result (m)</th>
<th>Total stress result (m)</th>
<th>Effective stress result (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-1</td>
<td>0.13</td>
<td>0.12</td>
<td>0.15</td>
</tr>
<tr>
<td>K-2</td>
<td>0.07</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>P-1</td>
<td>0.11</td>
<td>0.12</td>
<td>0.15</td>
</tr>
<tr>
<td>P-2</td>
<td>0.07</td>
<td>0.07</td>
<td>0.11</td>
</tr>
<tr>
<td>P-3</td>
<td>0.12</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>P-4</td>
<td>0.08</td>
<td>0.05</td>
<td>0.09</td>
</tr>
</tbody>
</table>
Figure 5-15 denotes the distributions of volumetric strain $\varepsilon_v$ at the maximum loading of 20.5 MN. In both analyses, the ground in the area near the front of the footing is compressive in a wide area; while the ground in the area behind the footing is expanded in a narrow area. In the field test, many cracks are observed in the ground near the footing, while in the calculation, large volumetric strain happened at the same place where cracks occurred in the field test.

Figures 5-16 and 17 show stress paths of ground at different positions during cyclic loading in total stress simulation. $J_2$ is the second invariant of the deviatoric stress tensor. The stress path of the element (Element 2, as shown in Figure 5-16(b)) located in front of the footing increases and decreases repeatedly with a relatively large stress range during the cyclic loading. While the effective stress and shear stress of the elements (Element 3 and 4, as shown in Figure 5-16(c) and (d)) located between the piles and the element (Element 5, as shown in Figure 5-16(e)) located behind the footing decrease little by little. The element (Element 1, as shown in Figure 5-16(a)) in a distant place from the group-pile is lightly affected. It is also seen that the elements located in the same column present a close tendency from the comparison between Figure 5-16 and 5-17.

The same phenomenon can be observed in the effective stress analysis as shown in Figures 5-18 and 19, although the maximum shear stress of the elements placed in front of footing (Element 2 and 7) in effective stress analysis is larger than it in total stress analysis. The degree of change in the stress, however, is different in which, effective stress analysis gives a much larger change in the stress path, showing that dilatancy affects directly the stresses due to the consideration of soil-water coupling effect. As to
the behaviors of Elements 8, 9 and 10, effective analysis gives a quite different description about the stress paths, compared to the total stress analysis, about the stress paths in the way that after the stress decreased to some extent, it will turn to increase again due to the dilatant behavior of sand after undergoing some shear strain, which in the total stress analysis, it is impossible to describe this behavior because dilatancy dose not affect the stress directly.

Figure 5-16 Stress paths of the first layer in total stress simulation

Figure 5-17 Stress paths of the third layer in total stress simulation
Figure 5-18 Stress paths of the first layer in effective stress simulation

Figure 5-19 Stress paths of the third layer in effective stress simulation
5.3 Numerical experiment on earthquake resistant reinforcement method based on partial ground solidification of group-pile foundation

5.3.1 Outline of the simulations

The partial ground solidification applicable to existing pile foundation is a very desirable method because it has several benefits such as the cost minimization, time minimization and workable construction. This earthquake resistant reinforcement method on partial ground solidification is useful in increasing the stiffness of the pile foundation through its restricted effect, making the ground solidify in a plate form.

In this verse, numerical experiments of a group-steel pipe-pile foundation subjected to horizontal lateral loading are conducted using the program DBLEAVES (Ye, 2007). Based on the numerical results, optimum size, location and shape of the ground improvement zone are determined properly and the validity of the ground improvement is clarified as well. The general description of this method is shown in Figure 5-20.

Figure 5-21 shows the geologic profile of ground and FEM mesh. The ground is composed of two layers which are homogeneous Toyoura sand layer and bearing layer. Because of the symmetric condition of geometry and loading, only half of the domain is taken under consideration. The boundary conditions of the ground are fixed at the bottom, sliding at the two sides whose normal directions are parallel to y-axis and x-axis. Moreover, the boundary condition of the pile in the calculation is that the head of the pile is fixedly rotated with free footing and free tip of it. The water table is located 1.5 m below the ground surface. The initial stress of the ground is regarded as stratification bedding without considering the effect of pile driving.

In the analysis, the sand layer is supposed to be a subloading $t_{ij}$ model, the bearing layer is supposed to be an elastic material and the reinforcement body is supposed to be a cyclic mobility model (Zhang et al., 2007).

Simulation of drained triaxial compression test with Toyoura sand under constant mean principal stress ($p'=196$ kPa) is conducted to determine the parameters of sand layer. In the test (Nakai and Hinokio, 2004), the initial void ratio of medium dense sand is $e_0=0.666$. The numerical result is reproduced quite well with the test result quantitatively, as shown in Figure 5-22. The material parameters of the ground are listed in Table 5-4.
Figure 5-20 General description of reinforcement method on partial ground solidification

Figure 5-21 Overall view of FEM mesh

Figure 5-22 Experimental and theoretical results of triaxial compression tests

Table 5-4 Parameters of ground

<table>
<thead>
<tr>
<th>Soil</th>
<th>Thickness (m)</th>
<th>E (MPa)</th>
<th>ν</th>
<th>λ</th>
<th>κ</th>
<th>N</th>
<th>R&lt;sub&gt;x&lt;/sub&gt;</th>
<th>a</th>
<th>β</th>
<th>OCR</th>
<th>ρ (t/m&lt;sup&gt;3&lt;/sup&gt;)</th>
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<tbody>
<tr>
<td>Sand</td>
<td>30</td>
<td>-</td>
<td>0.2</td>
<td>0.07</td>
<td>0.0045</td>
<td>1.1</td>
<td>3.2</td>
<td>30</td>
<td>2</td>
<td>500</td>
<td>1.99</td>
</tr>
<tr>
<td>Dc</td>
<td>5</td>
<td>100</td>
<td>0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.8</td>
</tr>
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Table 5-5 Parameters of steel pipe pile

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>outer diameter (mm)</td>
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</tr>
<tr>
<td>plate thickness</td>
<td>14</td>
</tr>
<tr>
<td>material</td>
<td>SKK490</td>
</tr>
<tr>
<td>pile length (mm)</td>
<td>32000</td>
</tr>
</tbody>
</table>

![Steel pipe pile diagram]

Figure 5-23 Theoretical result of man-made soft rock

Table 5-6 Material parameters of man-made soft rock

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression index $\lambda$</td>
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<tr>
<td>Swelling index $\kappa$</td>
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</tr>
<tr>
<td>Stress ratio at critical state $R_f$</td>
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</tr>
<tr>
<td>Void ratio $N$ ($p'=98$ kPa on N.C.L.)</td>
<td>1.05</td>
</tr>
<tr>
<td>Poisson’s ratio $\nu$</td>
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</tr>
<tr>
<td>Degradation parameter of overconsolidation state $m$</td>
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</tr>
<tr>
<td>Degradation parameter of structure $a$</td>
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<tr>
<td>Evolution parameter of anisotropy $b_r$</td>
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</table>

Table 5-7 Initial conditions of man-made soft rock

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial void ratio $e_0$</td>
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<tr>
<td>Initial mean effective stress $p'$ (kPa)</td>
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<tr>
<td>Initial degree of structure $R_\theta$</td>
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<tr>
<td>Initial degree of overconsolidation $1/R_\theta$</td>
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<tr>
<td>Initial anisotropy $\zeta_\theta$</td>
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</tbody>
</table>

The steel pipe piles with 1.0m in diameter, 0.014m in thickness and 33m in length are driven in a 3×3 pattern with 3m pile spacing between adjacent piles from center to center. The pile is modeled by a hybrid element proposed by Zhang et al. (2000), composed of a beam element and a solid element. In order to consider the influence of
axial force on bending stiffness of the pile, AFD model is employed in the analysis. The material parameters of the pile are listed in Table 5-5. The concrete footing above the ground is modeled with an elastic material.

Man-made soft rock is used in the simulation as a reinforcement material. Figure 5-23 shows theoretical result of triaxial compression test simulated with the cyclic mobility model (Zhang et al., 2007). The stress-strain relation shows behavior of the typical man-made soft rock. The peak strength is adjusted to 1.0MPa. The material parameters of the man-made soft rock are listed in Tables 5-6 and 7.

5.3.2 Numerical results of 9-pile foundation under monotonic load

In the simulation of simple lateral loading test, a concentrated lateral load with a maximum value of 50 MN is applied at the center of the side surface of the footing, where all the nodes on the side surface are in the same elevation i.e. 1.0m above the ground surface and moves together with the same amount in the x, y and z directions. The load is applied for one hour in the effective stress analysis.

A series of numerical tests are carried out under different height (H) of reinforcement material, as shown in Figure 5-24. The reinforcement length (L) and the depth (D) from the ground surface to the center of the reinforcement are kept fixed to 9.0 m (same as the footing length) and 7.5 m, respectively. Figure 5-25 shows the relationships of lateral load and displacement in the total stress simulation. Here, NR is the abbreviation of non-reinforcement. It is found from the figure that the larger the height H of reinforcement material is, the smaller the lateral displacement will be. However, the results for the heights of 15.0 m and 12.0 m are almost the same even for the wider reinforcement area. Therefore, it is important to decide the optimum height of the area to be improved where group pile foundation is adopted. Although the results of lateral displacements in the effective stress simulation are much larger than the results in the total stress simulation, the same tendencies are observed in the both cases, as shown in Figure 5-26. A comparison of the maximum lateral displacements is shown in Table 5-8. The hatching cells show the minimum lateral displacement having a major impact. Figure 5-27, represents bending moments at the end of the later loading for several heights of the reinforced area. It is revealed in this figure that the results of the bending moments are not very much dependent on the heights of the reinforced area.
Figure 5-24 Areas of variable reinforcement heights (H)

(a) Nonreinforcement  (b) H=3 m  (c) H=6 m
(d) H=9 m  (e) H=12 m  (f) H=15 m

Figure 5-25 Lateral displacements under variable heights in the total stress simulation

(a) Total  (b) Enlarged

Figure 5-26 Lateral displacements under variable heights in the effective stress simulation

(a) Total  (b) Enlarged
Table 5-8 Maximum values of lateral displacement under variable reinforcement heights

<table>
<thead>
<tr>
<th>Range of reinforcement H (m)</th>
<th>Maximum lateral displacement (cm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonreinforcement</td>
<td>Total stress</td>
<td>Effective stress</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>45.5</td>
<td>63.7</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>41.4</td>
<td>59.7</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>37.5</td>
<td>55.7</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>35</td>
<td>52.5</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>34.9</td>
<td>52.0</td>
<td></td>
</tr>
</tbody>
</table>

A series of numerical simulations are carried out under different depth (D), as shown in Figure 5-28. The reinforcement length (L) and the height (H) are kept fixed to 9.0 m and 6.0 m, respectively. Figure 5-29 shows the relationships of lateral load and displacement in the total stress simulation. The effect of reinforcement decreases for the reinforced area deeper than 4.5 m. Hence, though the group pile foundation is tightly reinforced, the lateral displacement at depth 12.0 m is almost the same as the result of nonreinforcement. Therefore, it can be said that the reinforcement body should be installed in the upper part of the pile where maximum bending moment occurs. Moreover, although the results of lateral displacements in the effective stress simulation are much larger than the results in the total stress simulation, the same tendencies are observed in the both cases, as shown in Figure 5-30. The maximum lateral displacements of the both analyses are listed in Table 5-9. The hatching cells show the minimum lateral displacement having a major impact. From Figure 5-31, it is seen that
the bending moments at the end of loading are totally different depending on the depths of the reinforced area.

(a) D=3.0 m     (b) D=4.5 m       (c) D=6.0 m
(d) D=7.5 m    (e) D=9.0 m       (f) D=10.5 m
(g) D=12.0 m

Figure 5-28 Areas of variable reinforcement depths (D)

(a) Total      (b) Enlarged

Figure 5-29 Lateral displacements under variable depths in the total stress simulation
Figure 5-30 Lateral displacements under variable depths in the effective stress simulation

Table 5-9 Maximum values of lateral displacement under variable reinforcement depths

<table>
<thead>
<tr>
<th>Range of reinforcement D (m)</th>
<th>Maximum lateral displacement (cm)</th>
<th>Total stress</th>
<th>Effective stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonreinforcement</td>
<td>49.5 68.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>38.8 55.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>37.3 54.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.0</td>
<td>38.6 56.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>41.4 59.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0</td>
<td>44.5 62.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.5</td>
<td>47.1 65.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.0</td>
<td>48.5 67.7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-31 Calculated results of bending moment in pile No.5

(a) Arrangement of test piles  (b) Total stress  (c) Effective stress
The effect of length (L) is investigated here, as shown in Figure 5-32. The reinforcement depth (D) and the height (H) are kept fixed to 4.5 m and 6.0 m, respectively. Figure 5-33 shows the relationships of lateral load and displacement in the total stress simulation. The same tendencies are observed in the effective stress simulation, as shown in Figure 5-34. The full reinforcement shown in Figure 5-32(d) has inhibitory effect of approximately 53% on the lateral displacement in the total stress simulation. It is known from the figure that the larger the reinforcement length is, the smaller the lateral displacement will be. However, it is not an effective way to reinforce the ground widely for the reasons of cost, time and workability for construction. Thus reasonable length should be decided in the field based on the reinforcement efficiency obtained from the numerical results. The maximum lateral displacements are listed in Table 5-10. From Figure 5-35, it is found that bending moments at the end of loading are totally different depending on the length of the reinforcement body.

Figure 5-36 shows distribution of the second invariant of the deviatoric strain immediately after loading in the effective stress simulation. Though the deviatoric strain is seen around the group-pile foundation in the non-reinforcement case, the effect of reinforcement is significant in the results of deviatoric strain. Especially the reinforcement body is more effective in the vertical direction shown in Figure 5-36(d), compare to the horizontal direction shown in Figure 5-36(b).
Figure 5-33 Lateral displacements under variable lengths in the total stress simulation

Figure 5-34 Lateral displacements under variable lengths in the effective stress simulation

Table 5-10 Maximum values of lateral displacement under variable reinforcement lengths

<table>
<thead>
<tr>
<th>Range of reinforcement L (m)</th>
<th>Maximum lateral displacement (cm)</th>
<th>Total stress</th>
<th>Effective stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonreinforcement</td>
<td>49.5</td>
<td>68.0</td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>42.1</td>
<td>58.6</td>
<td></td>
</tr>
<tr>
<td>9.0</td>
<td>37.3</td>
<td>54.6</td>
<td></td>
</tr>
<tr>
<td>13.5</td>
<td>28.3</td>
<td>41.0</td>
<td></td>
</tr>
<tr>
<td>18.0</td>
<td>23.4</td>
<td>32.4</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5-35 Calculated results of bending moment in pile No.5

Figure 5-36 Distribution of the deviatoric strain immediately after loading finished in the effective stress simulation
Figure 5 shows the inhibition ratio of lateral displacement and the reinforcement efficiency for the numerical simulations. The Inhibition ratio of lateral displacement is calculated based on the lateral displacement of the non-reinforcement. The reinforcement efficiency is obtained by dividing the reinforcement ratio by the volume of the reinforcement body. Based on the reinforcement efficiency obtained from the numerical simulations, the optimum size and the location of the ground improvement zone are determined. Here, the height (H) of 6m, the depth (D) from 0 to 3m and the length (L) within 9m are found optimum dimensions for this type of soil condition.

From the results mentioned above, the most important point in determining the optimum size and location of the reinforcement for partial ground solidification should be taken into account after careful consideration of the reinforcement efficiency obtained from the reliable numerical simulation in a comprehensive way. The effective stress analysis is
found more accurate numerical method to decide the size of the ground improvement zone.

5.4 Summary

In this chapter, 3D FD-FE soil-water coupling analyses on a real-scale group-pile foundation, which is subjected to horizontal cyclic loading, are conducted to investigate the mechanical behaviors of group-pile foundation. Furthermore, numerical simulations on earthquake resistant reinforcement method based on partial ground solidification of group-pile foundation are also simulated. By comparing the test and the numerical results, the applicability of the proposed numerical method is verified and the following conclusions can be given.

1. Simulations of drained triaxial compression test are conducted to determine the material parameters of the reclaimed layer. The element simulation is extremely precise in describing the behavior of soil.

2. In the lateral load and displacement relations, the total stress analysis underestimates the test result, while the effective stress analysis considering soil-water coupling well predicts the test result at the maximum and residual displacements.

3. The reason why the numerical result obtained from the undisturbed specimen shows a big discrepancy with the measurement value compare to the results of the remolded specimen is, that the reclaimed ground at that time was only a few years since its reclaim. Therefore, the ground at the moment can be regarded as remolded one. The undisturbed soil specimen, however, was sampled 8 years later than the preparation of the remolded one and the in situ ground had already experienced extra loads from up-structures of the bridge. Meanwhile, a sedimentary structure was estimated to be developed during the time span (eight years). These two facts might increase the stiffness and the strength of the reclaimed ground.

4. The calculated results of bending moment and axial force also agree well with the field observation. The difference of the bending moment due to axial force can be properly simulated by the calculation based on AFD model. Although the shapes of the distributions of the bending moment and the axial force are similar to the test
results in both analyses, the results of the effective stress are much closer to the test results.

5. The lateral displacement of the ground surface at the end of the test shows reasonably good agreement with the field measurements.

6. The parameters considered in the numerical simulations contain the size, the location and the shape of the ground improvement zone around the pile group, which is particularly important for the design of seismic enhancement of pile foundation in real engineering problems. Based on the reinforcement efficiency obtained from the numerical simulations, the optimum size and the location of the ground improvement zone are determined. Here, the height (H) of 6m, the depth (D) from 0 to 3m and the length (L) within 9m are found optimum dimensions for this type of soil condition.

7. The most important point in determining the optimum size and location of the reinforcement for partial ground solidification should be taken into account after careful consideration of the reinforcement efficiency obtained from the reliable numerical simulation in a comprehensive way. The effective stress analysis is found more accurate numerical method to decide the size of the ground improvement zone.

8. From the results mentioned above, the applicability of the proposed numerical method is encouraging and therefore it is quite confident to say that the method can be used to predict the mechanical behaviors of group-pile foundation to a satisfactory accuracy, particularly with the effective stress analysis. It should be noticed that when we conduct a numerical simulation of the ground, it is important not only to choose a suitable method, such as soil-water coupling or non coupling, but also to choose a suitable way of determining the material parameters based on suitable element tests that can properly represents the in situ condition.
References


6.1 Conclusions

The main purpose of this research is to study the mechanical behavior of Toyoura sand (TS) subjected to different loadings in variant drainage conditions with laboratory tests, theoretical simulation and numerical calculation based on the modified model. The following concluding remarks can be given:

1. The modified model can describe the overall mechanical behaviors of TS under different drainage conditions and loading conditions;

2. The specimen, usually being regarded as a single element whose mechanical behavior is the same throughout the element, is simulated by FEM analysis, considering the so-called element test as a boundary value problem. The mechanical behaviors of the sand specimen, subjected to cyclic load under undrained condition in conventional triaxial test are carefully investigated. It is confirmed by laboratory tests and finite element analyses that the so-called element test usually shows non-uniform behavior, it is, however, still useful in determining the mechanical behaviors because the influence of the nonuniformity on the overall mechanical behavior of the soil specimen (evaluated by the average values of finite element analysis) is negligible;

3. Mechanical behaviors of group-pile foundation subjected to cyclic lateral loading up to ultimate state are also considered with the verification of real-scale field test. Based on the verification, numerical experiments in determining an optimum pattern for enhancing the seismic resistance of group-pile foundation using ground improvement around the piles are also discussed in detail.

The following are the detailed description of above-mentioned conclusions:

Firstly, the constitutive model proposed by Zhang et al. (2007) is modified in its evolution equation for overconsolidation in order to describe the overall mechanical behaviors of TS in a unified way in which the eight parameters for describing the sand will be constant no matter what the density may be. The performance of the model reveal the fact that the mechanical behaviors of sand subjected to monotonic/cyclic
loading under drained/undrained condition can be uniquely and properly described by
the modified constitutive model. Furthermore, dense sand subjected to rather large
cyclic loading under drained and constant-mean-effective-stress conditions is simulated.
The overall characteristics of the sand are predicted well by the theory. Especially, the
convergence of the volumetric change in the drained cyclic loading test can be
described automatically without changing the values of any parameters, which was
impossible up till now.

Secondly, 2D and 3D soil-water coupling FD-FE analyses are conducted to
simulate the mechanical behavior of sand specimen subjected to cyclic load in
conventional triaxial test under undrained condition using a program named as
DBLEAVES (Ye, 2007). Although the specimen is usually regarded as a single element
whose mechanical behaviors are the same throughout the element, there exists some
inevitable imperfections or nonuniformity caused by physical limitations in common
laboratory test. Among the factors affecting the nonuniformity, the influence of friction
between loading caps and specimen shows the most prominent non-uniform behavior
during the cyclic loading in the element level. The behaviors of the average values are,
however, similar to those of calculated results under free condition. Therefore, it is
concluded that the test results obtained from the cyclic load under undrained condition
in conventional triaxial test is still useful in determining the soil parameters of a
constitutive model for the reason that the influence of the nonuniformity on the overall
mechanical behavior of the soil specimen is negligible from the result evaluated by the
average values of finite element analysis.

Finally, mechanical behaviors of group-pile foundation subjected to cyclic lateral
loading up to ultimate state are simulated with finite element analyses and its
applicability is verified with real-scale field test. It is quite confident to say that the
method can be used to predict the mechanical behaviors of group-pile foundation to a
satisfactory accuracy, particularly with the effective stress analysis. It should be pointed
out that when we conduct a numerical simulation of the ground, it is important not only
to choose a suitable method, such as soil-water coupling or non coupling, but also to
choose a suitable way of determining the material parameters based on the suitable
element tests that can properly represents the in-situ condition. In addition, based on the
verification, numerical experiments in determining an optimum pattern for enhancing
the seismic resistance of group-pile foundation using ground improvement around the piles are also discussed in detail. The parameters considered in the numerical experiments contain the size, the location and the shape of the ground improvement zone around the pile group, which is particularly important for the design of seismic enhancement of pile foundation in real engineering problems.

6.2 Recommendation for future work

1. The precision and the reproducibility of the test method are verified in this dissertation. In the future study, the mechanical behaviors of very dense and loose sand specimens should be discussed in detail. Furthermore, it should be compared between experiments and numerical results focused on the nonuniformity of the specimen, based on the results of numerical simulations about the influence of the nonuniformity on the overall mechanical behavior of the soil specimen.

2. The constitutive model can give a unified description of TS to a rather satisfactory level if considering the fact that only eight material parameters are employed in the model. Under generalized stress paths, however, it can not be said that the model perfectly describes the various behaviors of TS. The fact that sometimes the calculated results do not coincide well with the experimental results quantitatively, implies that the accuracy of the model is still needed to be improved in a further research. Meanwhile, if the influence of intermediate principal stress could be properly taken into consideration as using the $t_{ij}$ concept by Nakai & Mihara (1984), Nakai & Matsuoka (1986), Nakai (1989), and Nakai & Hinokio (2004), precision of the model would be much improved.
References


## APPENDIX I SPECIFICATIONS OF DATA RECORDING DEVICES

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>TDS-302</td>
<td></td>
</tr>
</tbody>
</table>
| (Tokyo Sokki Kenkyujo Co., Ltd.) | • Dimensions: $320(W) \times 150(H) \times 430(D)$mm  
• Weight: 9.5 kg  
• Power supply: AC90-110V, 50/60 Hz, 74V max  
• Number of channels: 500 channels (using external switching boxes)  
• Measuring object: Strain, DC voltage, thermocouples, Pt RTD  
• Measuring speed: 0.08 sec/channel |
| PCI-3165   |               |
| (Interface Corporation) | • Number of channels: Single-ended input 16  
Differential input 8  
• Input mode: Multiplexing  
• Input range: Bipolar $\pm 1V$, $\pm 2.5V$, $\pm 5V$, $\pm 10V$  
• Input protection voltage: $\pm 35V$ (power on)  
$\pm 20V$ (power off)  
• Resolution: 16 bits  
• AD converter: Ad676J or equivalent  
• Relative accuracy: $\pm 2$ LSB(max.)(25°C)  
• Error: $\pm 0.10\%$(max.) (0°C to 50°C): $\pm 10V$  
$\pm 0.15\%$(max.) (0°C to 50°C): $\pm 1V$,  
$\pm 2.5V$, $\pm 5V$  
• Conversion time*: 20 (when single channel fixed)  
110 (when channel scanned) |

Note: "*" Actual throughput depends on the number of samples and other various sources.
APPENDIX II  BRIEF DESCRIPTION OF CONSTITUTIVE MODEL

The original model (Zhang et al., 2007) is proposed based on the concepts of subloading (Hashiguchi and Ueno, 1977) and superloading (Asaoka et al., 1998). Here a brief description of the model is given.

The similarity ratio of the superloading surface to normal yield surface \( R^* \) and the similarity ratio of the superloading surface to subloading surface \( R \) are given as,

\[
R^* = \frac{\bar{p}'}{\bar{q}}, \quad 0 < R^* \leq 1 \quad \text{and} \quad \frac{\bar{q}}{\bar{p}} = \frac{\bar{q}}{\bar{p}}
\]

A II -(1)

\[
R = \frac{\bar{p}'}{\bar{q}}, \quad 0 < R \leq 1, \quad \text{and} \quad \frac{\bar{q}}{\bar{p}} = \frac{\bar{q}}{\bar{p}}
\]

A II -(2)

where, \((\bar{p}', \bar{q}), (\bar{p}', \bar{q})\) and \((\bar{p}, \bar{q})\) represent the present stress state, the corresponding normally consolidated stress state and the structured stress state at \(p-q\) plane, respectively. The normally yield surface is given in the following form as:

\[
f(\bar{p}', \bar{q}, \zeta) + \int_0^t J_t D' d\tau = MD \ln \frac{\bar{p}'}{\bar{p}_0} + MD \ln \frac{M^2 - \zeta^2 + \bar{q}^2}{M^2 - \zeta^2} + \int_0^t J_t D' d\tau = 0
\]

A II -(3)

where, \(\bar{p}' = \bar{p}\), and the other variables involved in Equations, A II - (1), (2) and (3) are defined as:

\[
\eta' = \sqrt{2} \eta \sqrt{\beta}, \quad \eta = \eta - \beta, \quad \eta = S / p', \quad S = T + p' I, \quad p' = -\text{tr} T / 3
\]

A II -(4)

\[
\zeta = \sqrt{2/3} \sqrt{\beta} \sqrt{\beta}, \quad \eta = \sqrt{2/3} \sqrt{\eta \cdot \eta}
\]

A II -(5)

where, \(S\) is the deviatory stress tensor; \(\beta\) is the anisotropic stress tensor, and \(T\) is the Cauchy effective stress tensor and is assumed to be positive in tension. In Equation A II -(3), \(J\) is the Jacobean determination of deformation gradient tensor \(F\) and can be expressed as:

\[
J = \det F = \frac{v}{v_0} = \frac{1+e}{1+e_0}
\]

A II -(6)

where \(v\) and \(v_0\) are the specific volume at the current time (t) and the specific value at the reference time (t=0). \(D\) is the dilatancy parameter which can be expressed by \(\lambda, \kappa\) the compression and the swelling index, respectively, as follows:

\[
D = \frac{\lambda - \kappa}{M(1+e_0)} = \frac{\lambda - \kappa}{Mv_0}
\]

A II -(7)
\( \varepsilon^p = -\int_0^t \mathfrak{J}_t \mathbf{D}^p \, d\tau \) \hspace{1cm} \text{A II -(8)}

By substituting Equations A II -(1) and A II -(2) into Equations A II -(3), subloading yield surface can also be written as:

\[
\begin{align*}
&f(p, \eta', \zeta) + \text{MD} \ln R' - \text{MD} \ln R + \int_0^t \mathfrak{J}_t \mathbf{D}^p \, d\tau \\
&= \text{MD} \ln \left( \frac{\dot{R}}{p_0} \right) + \text{MD} \ln \left( \frac{M^2 - \zeta^2 + \eta^2}{M^2 - \zeta^2} \right) + \text{MD} \ln R' - \text{MD} \ln R + \int_0^t \mathfrak{J}_t \mathbf{D}^p \, d\tau = 0 \\
\end{align*}
\]

\[
\text{A II -(9)}
\]

An associated flow rule is employed in the present model, namely, \( \mathbf{D}^p = \lambda \mathbf{e} / \partial \mathbf{T} \) \hspace{1cm} \text{A II -(10)}

The consistency equation for the subloading yield surface can then be given as:

\[
\frac{\partial f}{\partial \mathbf{T}} + \frac{\partial f}{\partial \mathbf{\beta}} + \frac{\partial \mathbf{R}'}{\partial \mathbf{R}'} - \text{MD} \frac{\mathbf{R}'}{\mathbf{R}} + \mathfrak{J}_t \mathbf{D}^p = 0
\]

\[
\text{A II -(11)}
\]

where, \( \dot{\mathbf{T}} \) and \( \dot{\mathbf{\beta}} \) are the Green-Naghdi rates of stress tensor \( \mathbf{T} \) and anisotropic stress tensor \( \mathbf{\beta} \), respectively. \( \boldsymbol{\Omega} \) is material spin tensor. It is easy to obtain the following relation:

\[
\frac{\partial f}{\partial p} = \text{MD} \left( \frac{1}{p} + \frac{\partial (\eta^2)}{\partial \eta} \right) = \text{MD} \frac{M^2 - \eta^2}{(M^2 - \zeta^2 + \eta^2) \eta} \hspace{1cm} \text{A II -(12)}
\]

From Equation A II -(12), it is clear that the C.S.L., defined by the condition in which \( \partial f / \partial \dot{p} = 0 \), always satisfies the relation \( \eta = M \), implying that the C.S.L., as the threshold between plastic compression and plastic expansion, does not move with the changes in the anisotropy.

Evolution rule for the anisotropic stress tensor is defined as:

\[
\dot{\mathbf{\beta}} = \frac{J}{p} \mathfrak{b} \eta (M - \zeta) \sqrt{2/3} \left\| \mathbf{D}^p \right\| \left\| \frac{\eta}{p} \right\| \hspace{1cm} \text{A II -(13)}
\]

where, \( \zeta < M \) provides a natural physical limitation on the development of anisotropy automatically., it is known from the evolution Equation A II -(13) that development of anisotropy will stop at the state when \( \eta = \mathbf{\beta} \).

Evolution rule for degree of structure \( \mathbf{R}^* \), which is the same as in the work by Asaoka et al. (2002), is adopted:

\[
\dot{R}^* = J U^* \sqrt{2/3} \left\| \mathbf{D}^p \right\|, \quad U^* = a R^* (1 - R^*) / D, \quad (0 < R^* \leq 1) \hspace{1cm} \text{A II -(14)}
\]
The changing rate of overconsolidation is assumed to be controlled by two factors, namely, the plastic component of stretching and the incremental anisotropy as,

\[
\dot{R} = JU\|D^p\| + \frac{R}{MD} \frac{\eta}{M} \frac{\partial f}{\partial \beta} \dot{\beta}
\]

A II -(15)

In which, \( \dot{\beta} \) is proportional to the norm of the plastic component of stretching, \( \|D^p\| \) and \( U \) is given by the following relation as:

\[
U = -\frac{m}{D} \left( \frac{p}{p_0} \right)^2 \ln R
\]

A II -(16)

where \( p_0 = 98.0 \) kPa is reference stress.

If the stretching is divided into elastic and plastic components, and the elastic components follow

\[
\dot{T} = ED^e, \quad D = D^p + D^e, \quad \dot{T} = ED - \Lambda E \frac{\partial f}{\partial T}
\]

A II -(17)

The positive valuable \( \Lambda (\Lambda>\lambda) \) can be rewritten as :

\[
\Lambda = \left. \frac{\partial f}{\partial T} \frac{ED}{E} \right| \frac{\partial f}{\partial T} + \frac{J}{MD} \left( \frac{M^2 - \zeta^2 + \eta^2}{\rho} \right) (\dot{M}^2 - \eta^2)
\]

A II -(18)

where

\[
\dot{M}^2 = M^2 - \frac{mM \ln R}{R} \left\{ \frac{(p/p_0)^2}{(p/p_0)^2 + 1} \right\} \sqrt{6\eta^2 + \frac{1}{3} (M^2 - \eta^2)^2}
\]

A II -(19)

The loading criteria are given below:

\[
\begin{cases}
\Lambda > 0 & \text{loading} \\
\Lambda = 0 & \text{neutral} \\
\Lambda < 0 & \text{unloading}
\end{cases}
\]

A II -(20)

In most cases, the denominator is positive, therefore, \( \Lambda > 0 \) is equivalent to the following relation:

\[
\frac{\partial f}{\partial T} \cdot ED > 0
\]

A II -(21)
APPENDIX III  SUBLOADING TIJ MODEL

An elastoplastic constitutive model for soils, named subloading $t_{ij}$-model (Nakai & Hinokio, 2004), is used in finite element analyses. To take into consideration the influence of the intermediate principal stress, the model is formulated using the modified stress tensor $t_{ij}$ (Nakai and Mihara, 1984), which is defined as

$$t_{ij} = a_{ik} \sigma_{kj}$$  \hspace{1cm} (AIII-1)

in which $a_{ik}$ is the non-dimensional symmetric tensor whose principal values are given by the direction cosines of the normal to the spatially mobilized plane (SMP).

$$a_1 = \sqrt{I_1 / I_3 \sigma_1}, \quad a_2 = \sqrt{I_2 / I_3 \sigma_2}, \quad a_3 = \sqrt{I_3 / I_3 \sigma_3}$$  \hspace{1cm} (AIII-2)

The yield function is given by the following equation as a function of mean stress $t_N$ and stress ratio $X$ based on $t_{ij}$ concept:

$$f = \ln t_{ij} + \zeta(X) - \ln t_{ij1} = \ln \frac{t_N}{t_{N0}} + \zeta(X) - \left( \ln \frac{t_{ij0}}{t_{ij1}} \right) = \ln \frac{t_N}{t_{N0}} + \zeta(X) = \left( \frac{1 + e_0}{\lambda - \kappa} \right) e_{ij}^p - \frac{\rho}{\lambda - \kappa} = 0$$  \hspace{1cm} (AIII-3)

$$\zeta(X) = \frac{1}{\beta} \left( \frac{X}{M^*} \right)^\beta$$  \hspace{1cm} (AIII-4)

where,

As shown in Figure AIII-1, $t_{ij1}$ and $t_{ij1e}$ measure the size of subloading surface and normal yield surface, respectively, and $\rho$ is the state variable which represents the current soil density. Here, the yield surface (subloading surface) not only expands but also shrinks so that the current stress point always lies on this surface. The plastic strain increment is split into two components – a component $d\varepsilon_{ij}^{p(AF)}$ obeying an associated flow in the modified stress $t_{ij}$ space, and an isotropic compression component $d\varepsilon_{ij}^{p(IC)}$ (Nakai and Matsuoka, 1986).

$$d\varepsilon_{ij}^{p} = d\varepsilon_{ij}^{p(AF)} + d\varepsilon_{ij}^{p(IC)}$$  \hspace{1cm} (AIII-5)

where,
\[ d \varepsilon_y^{(AF)} = A \lambda \hat{\sigma}_i \frac{df}{dt} \frac{1}{1 + e_0} \left( \frac{df}{dt} \right) + \frac{\lambda - \kappa}{\frac{1}{1 + e_0} \left( \frac{df}{dt} \right)} \hat{\sigma}_j \]  \hspace{1cm} \text{AIII-(6)}

\[ d \varepsilon_y^{(KE)} = \frac{a_u}{a_u + G(\rho)} \frac{\lambda - \kappa}{1 + e_0} \left( dt \right) \hat{\sigma}_j \]  \hspace{1cm} \text{AIII-(7)}

\( G(\rho) \) is a monotonically increasing function, which satisfies the condition \( G(0) = 0 \) and is given in the form of

\[ G(\rho) = a \cdot \rho^2 \]  \hspace{1cm} \text{AIII-(8)}

This model requires only a few unified material parameters, but can describe properly the following typical characteristics of soils: (a) influence of intermediate principal stress on the deformation and strength of soil is considered by using \( t_i \) concept; (b) influence of stress path on the direction of plastic flow is considered by splitting the plastic strain increment into two components; (c) influence of density and/or confining pressure is considered by adopting the subloading surface concept by Hashiguchi (1980).

![Figure AIII-1 Shape of yield surface and normally yield surface, and definition of \( \rho \)](image-url)
APPENDIX IV  AXIAL-FORCE DEPENDENCY MODEL
(AFD MODEL)

The derivation of a beam model considering the axial-force dependency and taking a weak form of the equilibrium equation for a beam, which satisfies the compatibility of the deformation, is given in detail. For abbreviation, the model is called as axial-force dependent model (AFD model, Zhang and Kimura, 2002a). In the model, the plane-section assumption is still kept valid and the stress-strain relations of reinforcement and concrete are shown in Figure IV-1. As shown in Figure, $\varepsilon_{a}$, the strain at an arbitrary point $P(x,y)$ at the sectional plane of a beam, can be divided into three parts, that is, the bending strain $\varepsilon_{m1}$ due to $M_x$, the bending strain $\varepsilon_{m2}$ due to $M_y$ and axial strain $\varepsilon_0$ due to axial force, as shown in following equation,

$$\varepsilon_{a} = \varepsilon_{m1} + \varepsilon_{m2} + \varepsilon_0 = \left(x\left[H_u''(z)\right] + y\left[H_v''(z)\right] - \left[H_w'(z)\right]\right) \cdot [A][\delta] = \left[F(z)\right]^T \cdot [A][\delta] \quad \text{AIV - 1}$$

where,

$${\{\delta\}} = \left\{u_i \ v_i \ w_i \ \theta_{xi} \ \theta_{yi} \ u_j \ v_j \ w_j \ \theta_{xj} \ \theta_{yj}\right\}^T$$

is the nodal displacement vector.

$$\{F(z)\}^T = \left(x\left[H_u''(z)\right] + y\left[H_v''(z)\right] - \left[H_w'(z)\right]\right) \quad \text{AIV - 2}$$

\[\begin{align*}
\sigma_c &= \sigma_c + \sigma_r \left(\varepsilon_{tc} - \varepsilon_e\right) \\
\sigma_r &= \left(\frac{\sigma_c + \sigma_t}{\varepsilon_c + \varepsilon_t}\right)
\end{align*}\]

\[\begin{align*}
\nu &= 0.5, \lambda = 0.5, \mu = 4, \tau = 4 \\
\varepsilon_c &= 0.5 \varepsilon_e, \varepsilon_r = 0.5 \varepsilon_e
\end{align*}\]

Figure IV-1 Nonlinear properties of reinforcement and concrete
The virtual energy stored in the beam element due to a virtual strain can be expressed as,

$$U = \iiint \sigma \varepsilon \, dv = \{d \varepsilon\}^T \int \int \int E \left[ A \right]^T \left\{ F(z) \right\} \left\{ A \right\} dv \cdot \{ \varepsilon \}$$  \hspace{1cm} \text{(5)}$$

On the other hand, virtual energy $W$ brought about by the external force due to a virtual displacement, is $W=\{d \varepsilon\}^T \{F\}$. Therefore, the virtual energy theory ($W=U$) can be obtained as

$$\{F\} = \iiint E \left[ A \right]^T \left\{ F(z) \right\} \left\{ A \right\} dv \cdot \{ \varepsilon \} = [K] \cdot \{ \varepsilon \}$$  \hspace{1cm} \text{(6)}$$

where, $[K]$is the stiffness matrix of the beam element and can be rewritten as

$$[K] = \iiint E \left[ A \right]^T [I] [A] dv$$  \hspace{1cm} \text{(7)}$$

$$[I] = \{ F(z) \} \cdot \{ F(z) \}^T = \begin{bmatrix} x \{ H_u''(z) \}^T + y \{ H_v''(z) \}^T \cdot \{ H_w'(z) \} \\ x \{ H_u''(z) \}^T + y \{ H_v''(z) \}^T \cdot \{ H_w'(z) \} \end{bmatrix}$$  \hspace{1cm} \text{(8)}$$

In the classical beam theory, $[I]$ is defined by Equation AIV- (9) in which the influence on the $M$-$\Phi$ relation due to the axial force is not considered, namely,

$$[I] = \left[ I_1 \right] = \begin{bmatrix} 2 \{ H_u''(z) \} \cdot \{ H_u''(z) \}^T + 2 \{ H_v''(z) \} \cdot \{ H_v''(z) \}^T + \{ H_w'(z) \} \cdot \{ H_w'(z) \} \end{bmatrix}^T$$  \hspace{1cm} \text{(9)}$$
Based on Equations AIV-(8) and (9), $[I]$ can be rewritten as

$$[I] = [I_1] + [I_2]$$  \hspace{1cm} \text{AIV-(10)}

where, $[I_2]$ is evaluated by the following equation:

$$[I_2] = \frac{AIV}{x} \left( \begin{array}{c}
\{H_u''(x)\} \{H_v''(x)\} + \{H_v''(x)\} \{H_u''(x)\} \\
- \alpha \{H_u''(x)\} \{H_w''(x)\} + \{H_w''(x)\} \{H_u''(x)\} \\
\end{array} \right)$$  \hspace{1cm} \text{AIV-(11)}

$[I_2]$ is the newly added item which takes into consideration the influence on the $M-\Phi$ relation due to the axial force. Based on the above equations, $[K]$ can be expressed as

$$[K] = [A]^T \cdot [D] \cdot [A], \quad [D] = \begin{bmatrix} 0_{5 \times 5} & 0_{5 \times 5} \\ 0_{5 \times 5} & D_1 \end{bmatrix}$$  \hspace{1cm} \text{AIV-(12)}

where,

$$D_1 = \begin{bmatrix}
4EI_y & 4EI_{xy} & -2IE_x & 6I^2EI_{xy} & 6I^2EI_y \\
4EI_{xy} & 4EI_x & -2IE_y & 6I^2EI_x & 6I^2EI_{xy} \\
-2IE_x & -2IE_y & IEA & -3I^2E_y & -3I^2E_x \\
6I^2EI_{xy} & 6I^2EI_x & -3I^2E_y & 12I^3EI_x & 12I^3EI_{xy} \\
6I^2EI_y & 6I^2EI_{xy} & -3I^2E_x & 12I^3EI_{xy} & 12I^3EI_y
\end{bmatrix}$$  \hspace{1cm} \text{AIV-(13)}

$$EA = \int E \cdot dA, \quad EI_y = \int E \cdot x^2 \cdot dA$$

$$EI_x = \int E \cdot y^2 \cdot dA, \quad EI_{xy} = \int E \cdot x \cdot y \cdot dA$$  \hspace{1cm} \text{AIV-(14)}

$$Ex = \int E \cdot x \cdot dA, \quad Ey = \int E \cdot y \cdot dA$$

The integration in Equation 14 can be evaluated by discretizing area $A$ into $N$ small areas $A_i$ and taking the sum as

$$EI_y = \int E \cdot x^2 \cdot dA = \sum_{i=1}^{N} E_i x_i^2 A_i.$$  \hspace{1cm} \text{AIV-(15)}